# DARBHANGA COLLEGE OF ENGINEERING

## DEPARTMENT OF CIVIL ENGINEERING

**Subject Code: 011620 Subject Name: Design of Steel Structures**

**Subject teacher: Ahsan Rabbani**

1. **Unit mass of Steel is**
   a) 785 $kg/m^3$
   b) 450 $kg/m^3$
   c) 450 $kg/cm^3$
   d) 7850 $kg/m^3$

Answer: d

2. **Which of the following is a correct criterion to be considered while designing?**
   a) Structure should be aesthetically pleasing but structurally unsafe
   b) Structure should be cheap in cost even though it may be structurally unsafe
   c) Structure should be structurally safe but less durable
   d) Structure should be adequately safe, should have adequate serviceability

Answer: d

3. **The structure is statically indeterminate when**
   a) static equilibrium equations are insufficient for determining internal forces and reactions on that structure
   b) static equilibrium equations are sufficient for determining internal forces and reactions on that structure
   c) structure is economically viable
   d) structure is environment friendly

Answer: a

4. **Which of the following relation is correct?**
   a) Permissible Stress = Yield Stress x Factor of Safety
   b) Permissible Stress = Yield Stress / Factor of Safety
   c) Yield Stress = Permissible Stress / Factor of Safety
   d) Permissible Stress = Yield Stress – Factor of Safety

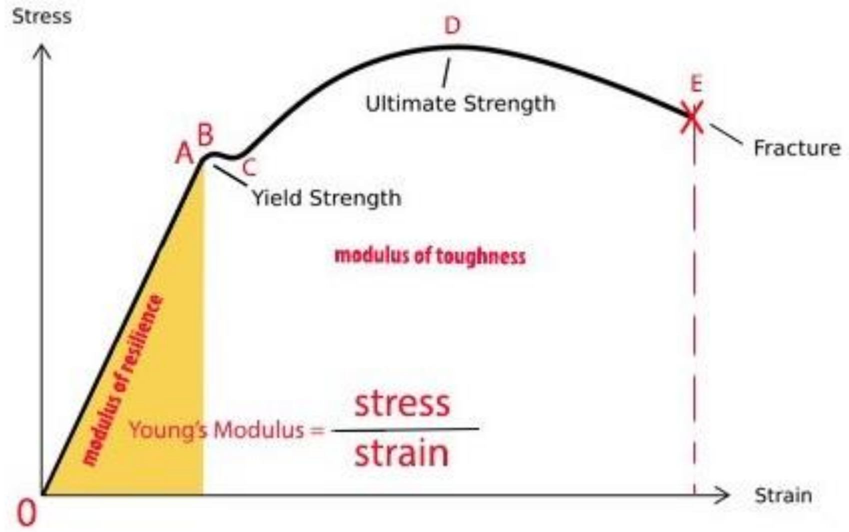
Answer: b

5. **Describe stress strain curve for the Mild Steel with neat sketch.**

Answer:

When steel is curved, it is important to keep the stress-strain curve ratio for mild steel in mind. Below is a stress-strain graph that reviews the properties of steel in detail.

If tensile force is applied to a steel bar, it will have some elongation. If the force is small enough, the ratio of the stress and strain will remain proportional. This can be seen in the graph as a straight line between zero and point A – also called the **limit of proportionality**. If the force is greater, the material will experience elastic deformation, but the ratio of stress and strain will not be proportional. This is between points A and B, known as the elastic limit.




Beyond the elastic limit, the mild steel will experience plastic deformation. This starts the yield point – or the rolling point – which is point B, or the upper yield point. As seen in the graph, from this point on the correlation between the stress and strain is no longer on a straight trajectory. It curves from point C (lower yield point), to D (maximum ultimate stress), ending at E (fracture stress).

Now, we'll look at each individual measure on the graph above and explain how each is derived.

- **Stress**: If an applied force causes a change in the dimension of the material, then the material is in the state of stress. If we divide the applied force (F) by the cross-sectional area (A), we get the stress.

The symbol of stress is σ (Greek letter sigma). For tensile (+) and compressive (-) forces. The standard international unit of stress is the pascal (Pa), where 1 Pa = 1 $N/m^2$. The formula to derive the stress number is $\sigma = F/A$.

For tensile and compressive forces, the area taken is perpendicular to the applied force. For sheer force, the area is taken parallel to the applied force. The symbol for shear stress is tau (τ).

- **Strain:** Strain is the change in the dimension ($L-L_0$) with respect to the original. It is denoted by the symbol epsilon (ε). The formula is $\varepsilon = (L-L_0) / L_0$. For a shear force, strain is expressed by γ (gamma)

- **Elasticity:** Elasticity is the property of the material which enables the material to return to its original form after the external force is removed.

- **Plasticity:** This is a property that allows the material to remain deformed without fracture even after the force is removed.

The definitions below are important for understanding the Stress-Strain interactions as seen in the graph.

- **Hooke's Law:** Within the proportional limit (straight line between zero and A), strain is proportionate to stress.

- **Young's modulus of elasticity:** Within the proportional limit, stress = E × strain. E is a proportionality constant known as the modulus of elasticity or Young's modulus of elasticity. Young's modulusis a measure of the ability of a material to withstand changes in length when under lengthwise tension or compression. E has the same unit as the unit of stress because the strain is dimensionless. The formula is $E = \sigma / \varepsilon$ Pa.

- **Modulus of Resilience**: The area under the curve which is marked by the yellow area. It is the energy absorbed per volume unit up to the elastic limit. The formula for the modulus of resilience is $1/2 \times \sigma \times \varepsilon = 0.5 \times (FL/AE)$.

- **Modulus of toughness**: This is the area of the whole curve (point zero to E). Energy absorbed at unit volume up to breaking point.

Chicago Metal Rolled Products sets the industry standard for adherence to the stress strain curve for mild steel and other materials.

6. **What are the Advantages and disadvantages of steel as structural materials?**

Answer:
Steel is one of the most generally utilized materials of construction time. Without the use of steel, the structure doesn't make a solid while seismic tremors like earthquakes etc. happen. Steel structures are susceptible to various ecological conditions. There are a few properties wherein solid structures are preferred over steel and the utilization of steel is consistently expanding everywhere throughout the world in development projects and also in civil engineering-related fields. According to 'Lorraine Farrelly', before the utilization of steel in development building, became a common practice, the weight of the structure material and the forces of gravity and pressure defined the endurance, chance of stability in structure, and its architectural possibilities. Each steel structure has some advantages as well as disadvantages. And now we are going to elaborate on the complete description regarding steel here.

**ADVANTAGES OF STEEL BUILDINGS**

a. Steel is moderately cheap when compared with other structure materials
b. Steel structures are highly fire-resistant when contrasted to a wooden structure as wood is a combustible material and less fire-resistant when contrasted with RCC structure.
c. One of the advantages of using a steel structure in development is the ability of steel to span greater distances with steel ceiling joists. This enables architects to grow their choices, enabling them to make new/huge space utilizing steel items that simply weren't accessible with different materials.
d. Steel can be easily & effectively manufactured and delivered greatly. Steel structures can be delivered off-site at shop floors and after that gathered nearby. This spares time and increases the efficiency of the general development process.
e. Steel structures can withstand outside weights, for example, earthquakes, thunder storms, and cyclones. A well-fabricated steel structure can last more than30 years whenever looked after well.

f. Flexibility is one of the great advantages of steel structure, which means that it tends to be planned according to the design requirements. This plans a steel structure so that it can withstand heavy winds or earthquakes, especially in the case of the bridges or tall towers.
g. Because of simple-to-make portions of a steel structure, it is hassle-free to install and assemble them on-site, and furthermore, there is no need of estimating and cutting of parts nearby.
h. Some of the common advantages of using steel buildings are Design, Strength and Durability, Light in Weight, Easy Installation and Speed in Construction, Versatile, Flexibility, Ductility, Easy Fabrication in Different Sizes, Fire Resistance, Pest and Insect Resistant, Moisture and Weather Resistance, Adaptability, Cost-effective, Environment Friendly, Energy Efficiency, Improved Construction Quality, Temporary Structures, Safe and Resistant and Risk Index.

**DISADVANTAGES OF STEEL BUILDINGS**

a. Buckling is an issue with steel structures. As the length of the steel segment builds the chances of buckling also increases.
b. Steel is available only at the steel plants where it is produced and should be transported for long distances to the site of construction, not at all like concrete or different materials that might be accessible right at the site of development.
c. Due to the activity of rust in steel, costly paints are required to re-establish from time to time. So that resistance against serious conditions increments.
d. Despite the fact that steel is a flexible material, it is difficult to make field corrections if one or more components do not fit appropriately. Large portions of the metal structure makes perform adhere to strict quality assurance procedure guarantee all pieces of a structure fit accurately. But in actual it is not possible. One can't form it or cut it in the ideal shape on-site once it is fabricated.
e. Steel can't mold in any path you required. It must be utilized in structures in which areas initially exist.
f. Steel is agood conductor of heat, touches off materials in contact and often causes fires, which quickly spread to different segments of a structure. Hence, steel structures may require extra fireproofing treatment.
g. If steel loses its great property of ductility and then there are more chances to increase the fractures.
h. Some of the common disadvantages of using steel buildings are High Maintenance & Capital Cost, Susceptibility to Buckling, Fatigue and Fracture, Fireproof Treatment, Fire Damage and Fabrication Error.

7. **Describe the various types of loads and Load Combinations as per IS code.**

Answer:

Clause 3.2 of IS 800:2007 specifies the various loads and forces that has to be considered while performing the design of steel structures. As per Cl. 3.2.1 of IS 800:2007, for the purpose of designing any element, member or a structure, the following loads (actions) and their effects shall be taken into account, where applicable, with partial safety factors and combinations (Cl. 5.3.3 of IS 800:2007). (a) Dead loads; (b) Imposed loads (live load, crane load, snow load, dust load, wave load, earth pressures, etc); (c) Wind loads; (d) Earthquake loads; (e) Erection loads; (f) Accidental loads such as those due to blast, impact of vehicles, etc; and (g) Secondary ffects due to contraction or expansion resulting from temperature changes, differential settlements of the structure as a whole or of its components,eccentric connections,rigidity of joints differing from design assumptions.

**1. Dead loads (Cl. 3.2.1.1 of IS 800:2007)**

Dead loads should be assumed in design as specified in IS 875 (Part 1).

**2. Imposed Loads (Cl. 3.2.1.2 of IS 800:2007)**

IS 800:2007 specifies in Cl.3.2.1.2 that imposed loads for different types of occupancy and function of structures shall be taken as recommended in IS 875 (Part 2). Imposed loads arising from equipment, such as cranes and machines should be assumed in design as per manufacturers/suppliers data (Cl. 3.5.4 of IS 800:2007). Snow load shall be taken as per IS 875 (Part 4).

**3. Wind loads (Cl. 3.2.1.3 of IS 800:2007)**

Wind loads on structures shall be taken as per the recommendations of IS 875 (Part 3).

**4. Earthquake loads (Cl. 3.2.1.4 of IS 800:2007)**

Earthquake loads shall be assumed as per the recommendations of IS 1893 (Part 1).

**5. Erection Loads (Cl. 3.3 of IS 800:2007)**

All loads required to be carried by the structure or any part of it due to storage or positioning of construction material and erection equipment, including all loads due to operation of such equipment shall be considered as erection loads. The structure as a whole and all parts of the structure in conjunction with the temporary bracings shall be capable of sustaining these loads during erection.

**6. Temperature Effects (Cl. 3.4 of IS 800:2007)**

Expansion and contraction due to changes in temperature of the members and elements of a structure shall be considered and adequate provision made for such effect. The co-efficient of thermal expansion for steel is as given in Cl. 2.2.4.l of IS 800:2007.

**7. Load Combinations**

All structures must be designed to support their own weight along with any superimposed forces, such as the dead loads from other materials, live loads, wind pressures, seismic forces, snow and ice loads, and earth pressures (if buried underground). Because various loads may act on a structure simultaneously, load combinations should be evaluated to determine the most severe conditions for design (worst case scenario). These load combinations vary from one document to another, depending upon the jurisdiction. There are a set of combinations for the allowable stress design and another set that incorporates load factors for strength design.

Load combinations for design purposes shall be those that produce maximum forces and effects and consequently maximum stresses and deformations. The following combination of loads with appropriate partial safety factors as given in Table 4 of IS 800:2007 may be considered. The table is reproduced here as Table 2 for ready reference. a) Dead load + imposed load, b) Dead load + imposed load + wind or earthquake load, c) Dead load + wind or earthquake load, and d) Dead load+ erection load. The effect of wind load and earthquake loads shall not be considered to act simultaneously. The load combinations are outlined in detail in Cl. 3.5 of IS 800:2007.

**Table 4 Partial Safety Factors for Loads, $\gamma_f$, for Limit States**

(*Clauses* 3.5.1 *and* 5.3.3)

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

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

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

*Abbreviations*:

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

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

8. **Explain limit state of serviceability and limit state of collapse briefly.**

Answer:

The most important limit states which are considered in design as are follows:

(i) Limit state of collapse.

(ii) Limit state of serviceability

**Limit State of Collapse**

This limit state is also called as strength limit state as it corresponds to the maximum load carrying capacity i.e., the safety requirements of the structure. The limit state of collapse is assessed from collapse of the whole or part of the structure. As per this limit state, the resistance to bending, shear, torsion and axial loads at every section shall not be less than that produced by the most unfavorable combination of loads on that structure. The following limit states of collapse are considered in design:

(i) Limit state of collapse in flexure (bending)

(ii) Limit state of collapse in compression

(iii) Limit state of collapse in shear

(iv) Limit state of collapse in torsion.

**Limit State of Serviceability**
A structure is of no use if it is not serviceable. Thus, this limit state is introduced to prevent excessive deflection and cracking. It ensure the satisfactory performance of the structure at working loads. It is estimated on the basis of elastic theory or working stress method because deformation is of significance under working load and not at collapse. Limit state of serviceability of following limit states:

(i) Limit state of deflection

(ii) Limit state of cracking

(iii) Limit state of vibration

The structure should be designed which considering all the appropriate limit state of safety and serviceability and on the basis of most critical limit state and then checked for all other limit states.

9. **What are the factors to be considered in mechanical properties of structural steel?**

Answer:

Steel derives its mechanical properties from a combination of chemical composition, heat treatment and manufacturing processes. While the major constituent of steel is iron, the addition of very small quantities of other elements can have a marked effect upon the properties of the steel. The strength of steel can be increased by the addition of alloys such as manganese, niobium and vanadium. However, these alloy additions can also adversely affect other properties, such as ductility, toughness and weldability .

Minimizing the sulphur level can enhance ductility , and toughness can be improved by the addition of nickel. The chemical composition for each steel specification is therefore carefully balanced and tested during its production to ensure that the appropriate properties are achieved.

The alloying elements also produce a different response when the material is subjected to heat treatments involving cooling at a prescribed rate from a particular peak temperature. The manufacturing process may involve combinations of heat treatment and mechanical working that are of critical importance to the performance of the steel.

Mechanical working takes place as the steel is being rolled or formed. The more steel is rolled, the stronger it becomes. This effect is apparent in the material standards, which tend to specify reducing levels of yield strength with increasing material thickness.

The effect of heat treatment is best explained by reference to the various production process routes that can be used in steel manufacturing, the principal ones being:

- As-rolled steel
- Normalized steel
- Normalized-rolled steel
- Thermomechanically rolled (TMR) steel
- Quenched and tempered (Q&T) steel.

Steel cools as it is rolled, with a typical rolling finish temperature of around 750°C. Steel that is then allowed to cool naturally is termed 'as-rolled' material. Normalizing takes place when as-rolled material is heated back up to approximately 900°C, and held at that temperature for a specific time, before being allowed to cool naturally. This process refines the grain size and improves the mechanical properties, specifically toughness. Normalized-rolled is a process where the temperature is above 900°C after rolling is completed. This has a similar effect on the properties as normalizing, but it eliminates the extra process of reheating the material. Normalized and normalized-rolled steels have an 'N' designation.

The use of high tensile steel can reduce the volume of steel needed but the steel needs to be tough at operating temperatures, and it should also exhibit sufficient ductility to withstand any ductile crack propagation. Therefore, higher strength steels require improved toughness and ductility, which can be achieved only with low carbon clean steels and by maximizing grain refinement. The implementation of the thermomechanical rolling process (TMR) is an efficient way to achieve this.

Thermomechanically rolled steel utilises a particular chemistry of the steel to permit a lower rolling finish temperature of around 700°C. Greater force is required to roll the steel at these lower temperatures, and the properties are retained unless reheated above 650°C. Thermomechanically rolled steel has an 'M' designation.

The process for Quenched and Tempered steel starts with a normalized material at 900°C. It is rapidly cooled or 'quenched' to produce steel with high strength and hardness, but low toughness. The toughness is restored by reheating it to 600°C, maintaining the temperature for a specific time, and then allowing it to cool naturally (Tempering). Quenched and tempered steels have a 'Q' designation.

Quenching involves cooling a product rapidly by immersion directly into water or oil. It is frequently used in conjunction with tempering which is a second stage heat treatment to temperatures below the austenitizing range. The effect of tempering is to soften previously hardened structures and make them tougher and more ductile.

10. **What are special features of limit state design method compare to other methods of design of steel structures?**

Answer:

Limit state design has advancement over the traditional design philosophies. It considers the safety at the ultimate load and serviceability at the working load, sort of extension of the WSM and ULM.

*“Limit state is the state of impending failure, beyond which a structure ceases to perform its intended function satisfactorily, in terms of either safety or serviceability.”*

Unlike WSM which based calculations on service load conditions alone, and unlike ULM, which based calculations on ultimate load conditions alone, LSM aims for a comprehensive and rational solution to the design problem, by considering safety at ultimate loads and serviceability at working loads.

The LSM philosophy uses a multiple safety factor format which attempts to provide adequate safety at ultimate loads as well as adequate serviceability at service loads, by considering all possible ‘Limit State’.

A limit state is a state of impending failure, beyond which a structure ceases to perform its intended function satisfactorily, in terms of either safety of serviceability i.e. it either collapses or becomes unserviceable.There are two types of limit states:

Ultimate limit states (limit states of collapse):- which deal with strength, overturning, sliding, buckling, fatigue fracture etc.

Serviceability limit states: – which deals with discomfort to occupancy and/ or malfunction, caused by excessive deflection, crack width, vibration leakage etc., and also loss of durability etc.

11. **Which of the following is advantage of HSFG bolts over bearing type bolts?**
    a) joints are not rigid
    b) bolts are subjected to shearing and bearing stresses
    c) high strength fatigue
    d) low static strength

Answer: c

12. **Tacking fasteners are used when**
    a) minimum distance between centre of two adjacent fasteners is exceeded
    b) maximum distance between centre of two adjacent fasteners is exceeded
    c) maximum distance between centre of two adjacent fasteners is not exceeded
    d) for aesthetic appearance

Answer: b

13. **Strength of bolt is**
    a) minimum of shear strength and bearing capacity of bolt
    b) maximum of shear strength and bearing capacity of bolt
    c) shear strength of bolt
    d) bearing capacity of bolt

Answer: a

14. **The types of welded joints does not depend on**
    a) size of members connected at joint
    b) type of loading
    c) area available for welding
    d) size of weld

Answer: d

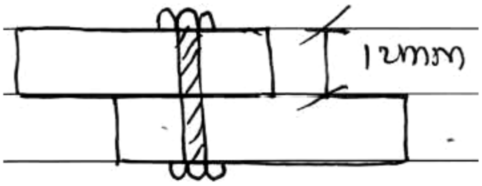
15. **The design nominal strength of fillet weld is given by**
    a) $f_u$
    b) $\sqrt{3}\ f_u$
    c) $f_u/\sqrt{3}$
    d) $f_u/(1.25 \times \sqrt{3})$

Answer: c

16. **Which of the following is not true regarding effective throat thickness of weld?**
    a) Effective throat thickness should not be less than 3mm
    b) It should not exceed 0.7t or 1t, where t is thickness of thinner plate of elements being welded
    c) Effective throat thickness = K x size of weld, where K is a constant
    d) Effective throat thickness = K x (size of weld)$^2$ , where K is a constant

Answer: d

17. **A lap joint consists of two plates 200 x 12 mm connected by means of 20 mm diameter bolts of grade 4.6. All bolts are in one line. Calculate strength of single bolt and no. of bolts to be provided in the joint.**




Solution:
Given

Nominal diameter of bolt = 20 mm

$\therefore$ Net area of bolt at thread ($A_{nb}$) $= 0.78 \times \frac{\pi}{4} \times d^2$

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

$A_{nb} = 245.04\ mm^2$

For fe 410 grade steel plate (assumed)
Ultimate stress for plate $f_y = 410\ N/mm^2$
For 4.6 grade of bolt
Ultimate stress for bolt ($f_{ub}$) = $4 \times 100 = 400\ N/mm^2$
Yield stress for bolt ($f_{yb}$) = $400 \times 0.6 = 240\ N/mm^2$

Now find design shearing strength of bolt (Vdsb)
$\therefore$ we know that

$$\therefore \quad Vdsb = \frac{fub}{\sqrt{3} \times Y_{mb}} [n_n \times A_{nb} + n_s + A_{ns}]$$

Here number of shear plane with threat intercepting the shear plane $n_n = 1$
Number of shear plane without thread intercepting the shear plane $n_s = 0$

$$\therefore \quad V_{dsb} = \frac{400}{\sqrt{3} \times 1.25} \times [1 \times 243.04 + 0]$$

$\gamma_{mb}$ = partial factor of safety for bolt material = 1.25
$V_{dsb} = 45.27 \times 10^3 N$

Now find design bearing strength of bolt ($V_{dsb}$)

$$V_{dph} = 25 \times kb \times (d \times t) \times \frac{fy}{\gamma_{mb}}$$

Here coefficient $k_b$ is minimum of

(1) $$\left[\frac{e}{3dh}, \frac{p}{3dh} - 0.25, \frac{f_{ub}}{f_u}, 1\right]$$

(a) Diameter of hole (dh) = Nominal diameter + 2
= 20 + 2 = 22 mm

(b) End distance (e) = 2d = 2 $\times$ 20 = 40 mm

(c) Pitch (p) = 2.5 d
= 2.5 $\times$ 20 = 50 mm

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

(ii) $\frac{p}{3dh} - 0.25 = \frac{50}{3 \times 22} - 0.25 = 0.507$

(iii) $\frac{fub}{fy} = \frac{400}{410} = 0.975$ &

(iv) 1

Hence Kb = 0.507 mm ... take minimum value

Now find design bearing strength of bolt ($V_{dpb}$)

$$= 2.5 \times Kb \times (d \times t) \times \frac{f_y}{r_{mb}}$$

$$= 2.5 \times 0.507 \times (20 \times 12) \times \frac{410}{1.25}$$

$$V_{dph} = 99.77 \times 10^3 \text{ N}$$

Now find bolt value i.e. strength of bolt

$\therefore$ Bolt value = minimum strength between shearing & bearing strength of bolt i.e. minimum between $V_{dsb}$ & $V_{dpb}$

$= 45.27 \times 10^3$ N

Full strength of member $= 0.9 \times \dfrac{fu}{r_m} \times$ Area of plan

$$= \frac{0.9 \times 410}{1.25}(250 - 1 \times 22) \times 12$$

$= 630.54 \times 10^3$ N

Full strength of plan

$$\therefore \text{ No of bolts} = \frac{\text{full strength of plate}}{\text{Bolt value}}$$

$$= \frac{630.54 \times 10^3}{45.27 \times 10^3}$$

= 13.92 Say 14 Nos

18. **Design the Lap joint for the plates of sizes 100 × 16 mm and 100 × 10 mm thick connected so as to transmit a factored load of 100 kN using single row of 16 mm diameter bolts of grade 4.6 and plate of 410 grade.**

Solution: Given

$f_u = 410$ N/mm$^2$ $f_{ub} = 400$ N/mm$^2$

d = 16 mm do = 18mm

$\gamma_{mb} = 1.25$ $P_u = 100$ kN

Strength of bolt:

Since it is lap joint bolt is in single shear, the critical section being at the root of bolt.

$$A_{nb} = 0.78 \times \frac{\pi}{4} \times d^2$$

$$= 0.78 \times \frac{\pi}{4} \times 16^2 = 156.82$$

**Design**

strength of bolt in shear

$$\text{i.e. } Vdsb = \frac{Fub}{\sqrt{3}} \frac{(n_n\, Anb + n_s\, A_{sb})}{rmb}$$

$$= \frac{400}{\sqrt{3}} \frac{1 \times 157}{1.25} = 29.006 \times 10^3 \text{ N}$$

$\therefore$ Vdsh = 29 N

$$\therefore \text{ No. of bolts required} = \frac{Pu}{Vdsb} = \frac{100}{29}$$

$= 3.4 \cong 4$ No.

No. of bolts required = 4 no.

Arranging bolts in single rows

Equating tensile capacity per pitch length

$$Tdn = 0.9 \frac{F_u}{rm_1}(P - do) \cdot t$$

$$29 \times 10^3 = 0.9 \times \frac{410}{125}(P-18) \times 10$$

$$P = \left(\frac{29 \times 10^3 \times 125}{0.9 \times 410 \times 10}\right) + 18$$

$$= 27.82 < 2.5 \times d = 2.5 \times 16$$

$$= 40$$

$\because$ Provide pitch P = 40 mm

and edge distance = $17 \times$ do [for rough edge]

$= 17 \times 18$

$= 30.6 \cong 30$

kb is smallest of

i) $\frac{e}{3do} = \frac{30}{3 \times 18} = 0.56$

ii) $\frac{P}{3d_0} - 0.25 = \frac{40}{3 \times 18} - 0.25$

$= 0.49$

iii) $\frac{Fub}{Fu} = \frac{400}{410} = 0975$

(iv) 1

Min. Value = 0.49

Hence Kb = 0.49

$\therefore$ Design bearing strength

$$Vdsh = \frac{Vnpb}{rmb} = \frac{2.5 \times kb.d.t\ F_u}{rmb}$$

$$= \frac{2.5 \times 0.49 \times 16 \times 10 \times 410}{1.25}$$

$= 64288$ N = 64.29 kN

Vdsb = 64.29 kN > 29 kN

$\because$ Ok no revision is required

Check for the strength of plate

$$Tdn = \frac{0.9\,An \cdot Fu}{rm} = \frac{0.9 \times (100 - 2 \times 18) \times 10 \times 410}{1.25}$$

$= 188.93$ kN $>$ 110kN

safe

Provide 4–16 mm $\phi$ bolts of 40mm

Pitch with edge distance of 30 mm as shown in fig.

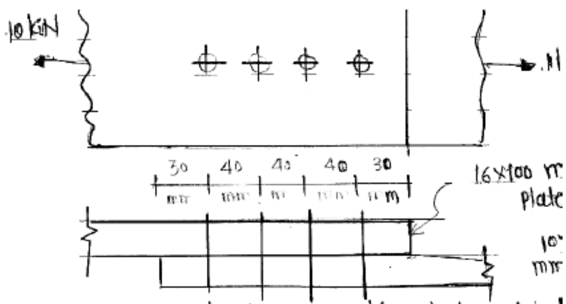



Fig: showing connection of Lap joint

19. **State types of bolted joints and types of failure in case of bolted joints.**

Answer:

**i) Types of bolted joint**

(a) Lap Joint
- Single line bolting
- Double line bolting

(b) Butt Joint
- Single cover Butt joint
- Double cover Butt joint

**ii) Failure of Bolted joint**

(a) Failure of plate
- By tearing of plate (shear failure)
- By tensile failure of plate
- By bearing of plate

(b) Failure of bolt
- By shear failure of bolt
- By tensile failure of bolt
- By bearing failure of bolt

20. **State various advantages of welded joints and disadvantages of bolted joints.**

**Advantages of Welded Joints**

1) The welded structures are usually lighter than riveted structures. This is due to the reason, that in welding, gussets or other connecting components are not used.
2) The welded joints provide maximum efficiency (may be 100%) which is not possible in case of riveted joint.
3) Alterations and additions can be easily made in the existing structures.
4) As the welded structure is smooth in appearance, therefore it looks pleasing.
5) In welded connections, the tension members are not weakened as in the case of riveted joints.
6) A welded joint has a great strength. Often a welded joint has the strength of the parent metal itself.
7) Sometimes, the members are of such a shape (i.e. circular steel pipes) that they afford difficulty for riveting. But they can be easily welded.
8) The welding provides very rigid joints. This is in line with the modern trend of providing rigid frames.
9) It is possible to weld any part of a structure at any point. But riveting requires enough clearance.
10) The process of welding takes less time than the riveting.

**Disadvantages of bolted joints :**

1) Due to holes made in members to be connected, tensile strength of the members is reduced.
2) Rigidity of joint is affected due to loose fit.
3) Deflection may increase due to affected Rigidity of joint
4) Nuts are likely to loose due to moving load vibration.
5) Bolted structures are heavier than welded structure due to use of connecting angles.
6) Circular section can not be bolted.
7) It is not possible to get 100% efficiency in case of bolted connection
8) Problem may arise in case of mismatching of holes.

21. **List the values of partial safety factor for material strength in case of resistance by yielding, buckling and ultimate stress in bolted connection.**

Answer:

| | Descriptions | Partial safety Factor |
|---|---|---|
| 1 | Resistance governed by yielding $r_{mo}$ | 1.10 |
| 2 | Resistance of member to buckling $r_{mo}$ | 1.10 |
| 3 | Resistance governed by ultimate stress $r_{m1}$ | 1.25 |
| 4 | Bolted connection in friction and Bearing $r_{mf}$ and $r_{mb}$ | 1.25 [shop and field fabrication] |

22. **Explain what do you mean by shear lag?**

Answer:

While transferring the tensile force from gussel plate to tension member through one leg by bolts or welds, the connected leg of section (such as angle, channel) may be subjected to more stress than the outstanding leg and finally the stress distribution becomes uniform over the section away from the connection. Thus one port behind the other is called as shear lag.

The tearing strength of an angel section connected through one leg is affected by shear lag also. Thus, the design strength, $Td_n$ governed by tearing at net section is given by

$$Td_n = 0.9\frac{Anc\,fu}{vm_j} + \beta\frac{Ago\,fy}{vm_o}$$

Where $\beta = 1.4 = 0.076\frac{w}{t}\times\frac{fy}{fu}\times\frac{bs}{Lc}$

$b_3$ = Shear log width as shown in fig

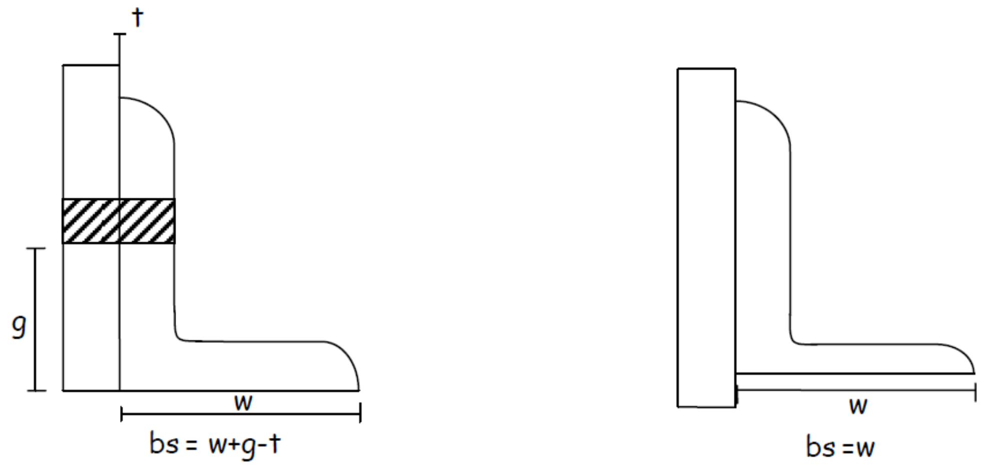



**Fig:** Shear leg width

23. **A discontinuous compression member consists of 2 ISA 90 × 90 × 10 mm connected back to back on opposite sides of 12 mm thick gusset plate and connected by welding. The length of strut is 3 m. It is welded on either side. Calculate design compressive strength of strut.**
**For ISA 90 × 90 × 10, Cxx = Cyy = 25.9 mm Ixx = Iyy = 126.7 × 104 mm4, rzz = 27.3 mm values of $f_{cd}$ are**

| **KL/r** | **90** | **100** | **110** | **120** |
|---|---|---|---|---|
| **$f_{cd}$ ($N/mm^2$)** | **121** | **107** | **94.6** | **83.7** |

Solution:

(i) $r_{zz}$ = 27.3 mm (Due to symmetry @ zz axis)

(ii) $I_{yy} = 2[Iy + A \cdot h^2]$

$= 2[126.7 \times 10^4 + 1703\,(25.9 + 12/2)^2]$

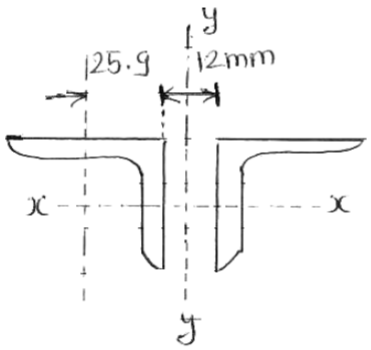
(A is calculated by calculating Area of both leg separately and then adding them)

$\therefore I_{yy} = 5999979\ mm^4$

(iii) $\therefore r_{yy} = \sqrt{\dfrac{I_{yy}}{A_g}} = \sqrt{\dfrac{5999979}{2 \times 1703}} = 41.97$ mm

$r_{min}$ = minimum of $r_{zz}$ and $r_{yy}$

$r_{min}$ = 27.3 mm




(iv) For discontinuous double angle, effective length

KL = 0.85L = 0.85 × 3 = 2.10 m = 2100mm

$S.R. = \dfrac{KL}{r_{min}} = \dfrac{2100}{27.3} = 76.92$

| KL/r (SR) | fcd |
|---|---|
| 70 | 152 |
| 80 | 136 |

Hence,

$$fcd = fcd_1 - \frac{fcd_1 - fcd_2}{SR_2 - SR_1}$$

$$fcd = fs_2 - \frac{152 - 136}{80 - 70}(76.92 - 70)$$

$$fcd = 140.928\ N/mm^2$$

(v) Design compressive Strength

$Pd = fcd \times Ag$

$Pd = 140.928 \times (2 \times 1703)$

$Pd = 480 \times 10^3\ N$

$Pd = 480\ kN$

24. **Check whether ISMB250@37.4 kg/m is suitable or not as a simply supported beam over an effective span of 6 m. The compression flange of beam is laterally supported throughout the span. It carries udl of 15 kN/m (including self wt.). Properties of ISMB 250 are bf = 125 mm, tf = 12.5 mm, tw = 6.9 mm, Ixx = 5131.6×104 mm4, Zxx = 410×103 mm3, r1 = 13.0 mm, $Z_{px}$ = 465.71 × $10^3$ $mm^3$, $y_{m0}$ = 1.1, $\beta_b$ = 1 and fy = 250 MPa.**

Solution:

(i) Loads and factored BMS

$w = 15 kN/m$

Factored udl, $wd = 15 \times 1.5 = 22.5\ kN/m$

Factored BM, $Md = \dfrac{wd.le^2}{8} = \dfrac{22.5 \times 6^2}{8}$

$= 101.25\ kN/m$

Factored S.F. $Vd = \dfrac{wd.le}{2} = \dfrac{22.5 \times 6}{2} = 67.5kN$

(ii) Plastic modulus of section required

$Z_p$ reqd. $= \dfrac{Md.\gamma_{mo}}{fy} = \dfrac{101.25 \times 10^6 \times 1.1}{250}$

$= 445.5 \times 10^3\ mm^3$

$Z_p$ reqd. $<$ $Z_P$ avil $(=465.71 \times 10^3\ mm^3)$

(iii) Classification of beam section

$d = h - 2(f_t + \gamma_1) = 250 - 2(12.5 + 13)$

$= 199\ mm$

$$\frac{bh}{tf} = \frac{\frac{125}{2}}{12.5} = 5.0 < 9.4$$

$$\frac{d}{tw} = \frac{199}{6.9} = 28.84 < 67$$

As $\dfrac{bh}{tf} < 9.4$ and $\dfrac{d}{tw} < 67$ $\quad\therefore$ Section classification is plastic

(iv) Check for shear

$$Vdr = \frac{f_y \times tw \times h}{\gamma_{mo}\sqrt{3}} \quad OR \quad 0.525 \quad fy.tw.h$$

$$= \frac{250 \times 6.9 \times 250}{1.1 \times \sqrt{3}} = 226348 \text{ N}$$

$$= 226.35 \text{ KN} > Vd\ (=67.5\text{kN})$$

Also, $\frac{Vd}{Vdr} = \frac{67.5}{226.35} = 0.298 < 0.6$

$\therefore$ Check for shear is satisfied.

(v) Check for deflection

$$\delta_{allowable} = \frac{L}{300}$$

$$= \frac{6000}{300}$$

$$= 20 \text{ mm}$$

$$d_{max} = \frac{5}{384} \frac{wL^4}{FI}$$

$$= \frac{5}{384} \times \frac{15 \times 6000^4}{2 \times 10^5 \times 5131.6 \times 10^4}$$

As $\delta_{max} > \delta_{allowable}$

$\therefore$ Deflection check is not O.K.

Hence, ISMB 250 is not a suitable section for given loading and span

25. **State types of bolted joints and types of failure in case of bolted joints.**

Answer:

**i) Types of bolted joints**

(a) Lap Joint

- Single line bolting
- Double line bolting

(b) Butt Joint

- Single cover Butt joint
- Double cover Butt joint

**ii) Failure of Bolted joint**

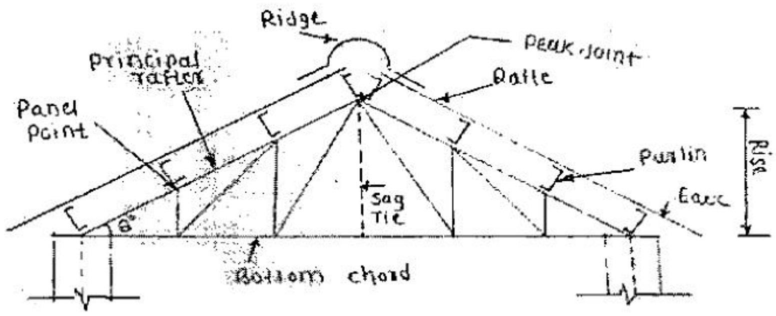
(a) Failure of plate

- By tearing of plate (shear failure)
- By tensile failure of plate
- By bearing of plate

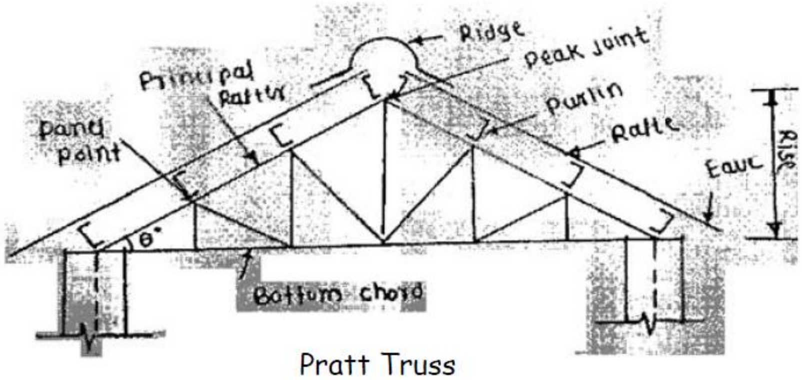
(b) Failure of bolt
- By shear failure of bolt
- By tensile failure of bolt
- By bearing failure of bolt

26. **Draw sketches of Howe type and Pratt type truss showing pitch, rise, panel point, panel, principal rafters and all members in one of the above types.**

Answer:




Home Truss

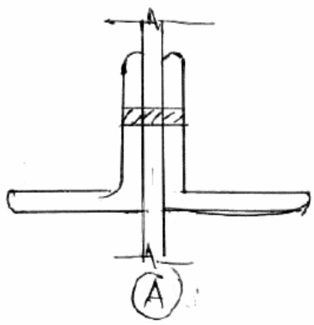


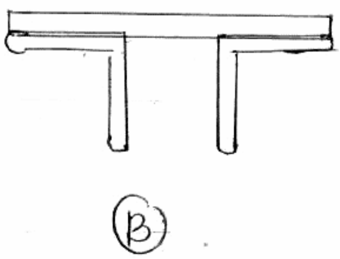

Pratt Truss

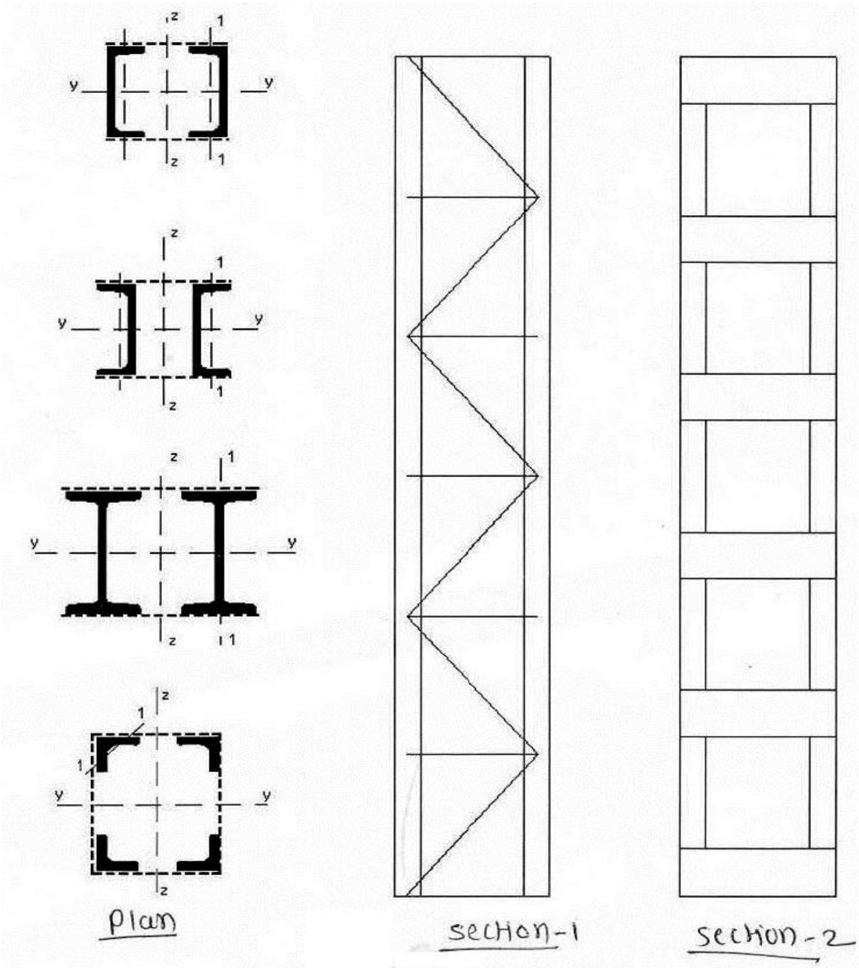
27. **Sketch different sections used as built-up strut and built-up column.**

Answer:

**<u>Built-up strut</u>**



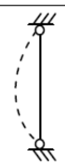



Built-up columns

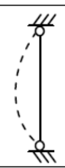
28. **State with a sketch the effective length for a compression member as per IS 800 – 2007 having end conditions as**
    **(i) Translation restrained at both ends and rotation free at both ends**
    **(ii) Translation and rotation restrained at both ends**

Answer:

(i) Translation restrained at both ends and rotation free at both ends

| | | | | | |
|---|---|---|---|---|---|
| Restrained | Free | Restrained | Free |  | 1.0L |

(ii) Translation and rotation restrained at both ends

| | | | | | |
|---|---|---|---|---|---|
| Restrained | Restrained | Restrained | Restrained |  | 0.65 L |

29. **State the function of lacing and battening.**

Answer:

1. **Function of lacing**
   - To connect the different components of built up column together so that they will act as one unit
   - To keep the distance between two components of built up column uniform and constant.
   - To keep the distance between two components of built up column uniform and constant.

2. **Function of battening**
   - The batten is placed opposite to each other at each end of the member and at points where the member is proportioned uniform throughout.
   - When battens are used effective length of column should be increased by 10%
   - Battens shall be designed to carry the bending moments and shear forces arising from transverse shear force equal to 2.5 percent of the total axial force on the whole compression member, at any point in the length of the member, divided equally between parallel planes of battens.

30. **Limiting width to thickness ratio for single beam section of plastic class is 9.4 and d/tw = 84. State whether ISMB 500 @ 852 N/m is of plastic class or not. For ISMB 500; h = 500 mm, bf = 180 mm, tf = 17.2 mm, tw = 10.2 mm, r1 = 17.0 mm, fy = 250 MPa.**

Solution:

$$\frac{h}{bf} < 8.4\,\varepsilon \;\ldots\; \text{For class } -1 \text{ (plastic)}$$

Given section is ISMB500

$\therefore$ h = 500

& bf = 180

$$\frac{h}{bf} = \frac{500}{180} = 2.78 < 8.4\,\varepsilon$$

$$\text{but } \varepsilon = \sqrt{\frac{250}{f_y}}$$

$$\therefore \quad \varepsilon = \sqrt{\frac{250}{250}}$$

$$\varepsilon = 1$$

$$\therefore \frac{h}{bf} = 2.78 < 8.4 \times 1$$

$$= 2.78 < 8.4 \quad \ldots \quad \text{hence the class is plastic}$$

31. **Which of the following statement is correct?**
    a) angles placed on same side of gusset plate produce eccentricity about one plane only
    b) angles placed on same side of gusset plate produce eccentricity about two planes
    c) angles placed on opposite side of gusset plate produce eccentricity about one plane only
    d) angles placed on opposite side of gusset plate produce eccentricity about two planes

Answer: a

32. **Which of the following is true about built up section?**
    a) Built up members are less rigid than single rolled section
    b) Single rolled section are formed to meet required area which cannot be provided by built up members
    c) Built up members can be made sufficiently stiff
    d) Built up sections are not desirable when stress reversal occurs

Answer: c

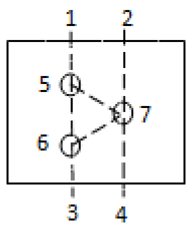
33. **What is the maximum effective slenderness ratio for members always in tension?**
    a) 400
    b) 200
    c) 350
    d) 150

Answer: a

34. **The design tensile strength of tensile member is**
    a) minimum of strength due to gross yielding, net section rupture, block shear
    b) maximum of strength due to gross yielding, net section rupture, block shear
    c) strength due to gross yielding
    d) strength due to block shear

Answer: a

35. **Which section to be considered in the design for the net area of flat?**




    a) 1-5-6-3
    b) 2-7-4
    c) 1-5-7-4
    d) 1-5-7-6-3

    Answer: d

36. **What is the net section area of steel plate 40cm wide and 10mm thick with one bolt if diameter of bolt hole is 18mm?**

a) 38.2 $cm^2$
b) 20 $cm^2$
c) 240 $mm^2$
d) 480 $mm^2$

Answer: a

37. **Which of the following is property of compression member?**
a) member must be sufficiently rigid to prevent general buckling
b) member must not be sufficiently rigid to prevent local buckling
c) elements of member should be thin to prevent local buckling
d) elements of member need not prevent local buckling

Answer: a

38. **Which of the following is true about tubular section?**
a) tubes have low buckling strength
b) tubes have same radius of gyration in all direction
c) tubes do not have torsional resistance
d) weight of tubular section is more than the weight required for open profile sections

Answer: b

39. **Effective length of compression member is**
a) distance between ends of members
b) distance between end point and midpoint of member
c) distance between points of contraflexure
d) distance between end point and centroid of member

Answer: c

40. **What is slenderness ratio of compression member?**
a) ratio of effective length to radius of gyration
b) ratio of radius of gyration to effective length
c) difference of radius of gyration and effective length
d) product of radius of gyration and effective length

Answer: a

41. **Which of the following is true?**
a) built up column lacings or battens are uneconomical if load carrying members permit greater reduction in weight than what is added by lacing or batten

b) built up column lacings or battens are economical if load carrying members permit greater reduction in weight than what is added by lacing or batten
c) no related shear stress force in plane of cross section
d) built up column designed as axially loaded column can never be eccentrically loaded

Answer: b

42. **Which of the following is true?**
a) in case of rolled section, less thickness of plate is adopted to prevent local buckling
b) for built-up section and cold formed section, longitudinal stiffeners are not provided to reduce width to smaller sizes
c) local buckling cannot be prevented by limiting width-thickness ratio
d) in case of rolled section, high thickness of plate is adopted to prevent local buckling

Answer: d

43. **What are laterally restrained beams?**
a) adequate restraints are provided to beam
b) adequate restraints are not provided to beam
c) economically not viable
d) unstable beams

Answer: a

44. **Critical bending moment capacity of a beam undergoing lateral torsional buckling is a function of**
a) does not depend on anything
b) pure torsional resistance only
c) warping torsional resistance only
d) pure torsional resistance and warping torsional resistance

Answer: d

45. **Which of the following statement is not correct?**
a) Hollow circular tube has more efficiency as flexural member
b) Hollow circular tube has lesser efficiency as flexural member
c) It is the most efficient shape for torsional resistance
d) It is rarely used as a beam element

Answer: a

46. **Which of following statement is correct?**
a) elastic buckling stress may be decreased by using longitudinal stiffeners
b) elastic buckling stress may be decreased by using intermediate stiffeners
c) elastic buckling stress may be increased by using intermediate transverse

stiffeners
d) elastic buckling stress is not affected by intermediate or longitudinal stiffeners

47. **Structural members subjected to bending and large axial compressive loads are known as**
    a) Strut
    b) Purlin
    c) beam-column
    d) lintel

Answer: c

48. **Which of the following assumptions were not made while deriving expression for elastic critical moment?**
    a) beam is initially undisturbed and without imperfections
    b) behaviour of beam is elastic
    c) load acts in plane of web only
    d) ends of beam are fixed support

Answer: d

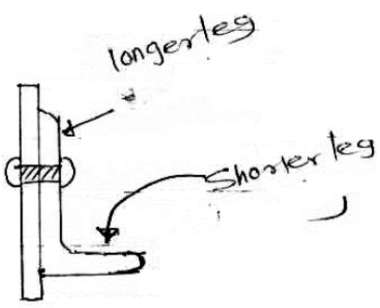
49. **The web is susceptible to shear buckling when $d/t_w$**
    a) <67ε
    b) < 2×67ε
    c) >67ε
    d) < 70ε

Answer: c

50. **Find the value of permissible stress in axial tension ($\sigma_{at}$) for fy = 250 MPa. State why unequal angles with long legs connected are more efficient?**

(i) 6at = 0.60 × fy
= 0.60 × 250
6at = 150 N/mm$^2$

(ii) Generally longer legs are connected in case of unequal angle section because of the following reason.
Consider angle is connected in the following manner as shown in fig




In the shorter leg is connected to gusset plate, then the bending stress induced in the section is large due to outstanding longer kg, because of which the stress distribution in the section is no–uniform and hence it may lead to fracture of the member prematurity

51. **Design a tension member consisting of single unequal angle section to carry a tensile load of 340 kN. Assume single row 20 mm bolted connection. The length of member is 2.4 m. Take fu = 410 MPa, α = 0.80**

| Section available (mm) | Area ($mm^2$) |
|---|---|
| ISA 100 × 75 × 8 | 1336 |
| ISA 125 × 75 × 8 | 1538 |
| ISA 150 × 75 × 8 | 1748 |

## Solution:

**(A)** Appropriate gross area required

$$\text{Reqd } Ag = \frac{1.1 \times Tag}{fy}$$

$$= \frac{1.1 \times 340 \times 10^3}{250}$$

$$= 1496 \text{ mm}^2$$

Try 15A 125 × 75 × 8 mm giving Ag = 1538 $mm^2$ $r_{min}$ = 16.1 mm. Assuming longer leg connected, check the strength of the section

i) Design strength due to yielding of gross section

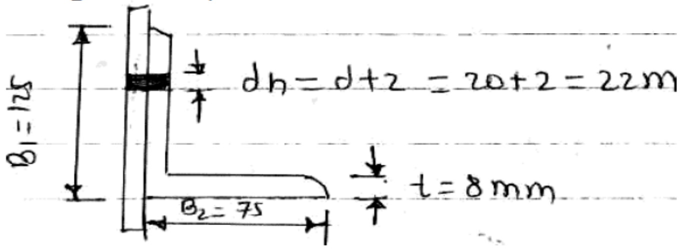
$$Tag = \frac{Ag \times fy}{r_{mo}}$$

$$= \frac{1538 \times 250}{1.10}$$

$$= 349545.4 N$$

$$Tag = 349.54 \text{ kN}$$

ii) Design strength due rupture of critical section




$$Tdn = \alpha An \frac{lu}{r_{m1}}$$

$$An = Anc + Ago$$

$$Anc = (B_1 - d_h - t/2) \times t$$

$$= (125 - 22 - 8/2) \times 8$$

$$Anc = 792 \text{ mm}^2$$

$$Ago = (B_2 - t/2)t$$

$$= (75 - 8/2)8$$

$$= 568 \text{mm}^2$$

An = Anc + Ago

An = 792 + 568

An = 13602 $mm^2$

Considering more than four bolt's in a raw $\alpha$ = 0.8

$$Tdn = \frac{0.8 \times 1360 \times 410}{1.25}$$

Tdn = 356.864 kN

Design of bolts

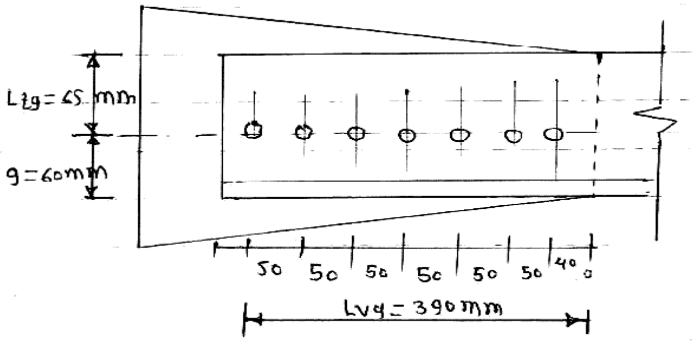
Capacity of bolts in single shear = 45. 3 KN

Capacity of bolt in bearing = $20 \times 8 \times 410 \times 10^{-3}$

= 65.6 kN

least bolt value = 45.3 kN (min of two above)

Number of bolts required = $\frac{340}{45.3}$ = 75 say 8




Assuming edge dist$^n$ = 40mm

9 = 60mm

Spacing of bolts = 50mm

Avg = Lvg $\times$ t = 390 $\times$ 8 = 3120mm$^2$

Avg = 3120 mm$^2$

$$Avn = \{Lug - [No.\ of\ bolts - 0.5\,dh]\} xt$$

$$Avn = \{390 - [(8 - 0.5)22]\} \times 8 = 1800mm^2$$

Avn = 1800mm$^2$

Atg = Ltg $\times$ t = 65 $\times$ 8 = 500 mm$^2$

Atg = 520 mm$^2$

$$Atn = (65 - (0.5 \times 22)) \times 8$$

Atn = 432 mm$^2$

$$Tdb_1 = Avg\,fy / \left(\sqrt{3} \times \gamma_{mc}\right) + 0.9Atn\,fu / \gamma_{m1}$$

$$= 3120 \times 250 / \left(\sqrt{3} \times 1.10\right) + 0.9 \times 432 \times \frac{410}{1.25}$$

= 409393.8 + 127526.4

= 5369202 N

$Tdb_1$ = 536.92 kN

$Tdb_2 = 424.962$ kN

$Tdb$ = lesser than $Tdb_1$ and $Tdb_2$ = 424.96 kN

$\therefore$ The tensile strength of angle = lesser of Tag, Tdn and Tdb
(349.54, 356.86 and 426.96)
= 349.54 kN

This is greater than required 340 kN

Check for slenderness ratio $\lambda = \frac{L}{r_{min}} = \frac{2400}{16.1}$

$149.06 < 250$

52. **A hall of size 12m x 18m is provided with Fink type trusses at 3 m c/c. Calculate panel point load in case of Dead load and live load from following data.**

**a. Unit weight of roofing = 150 N/m$^2$**
**b. Self-weight of purlin = 220 N/m$^2$**
**c. Weight of bracing = 80 N/m$^2$**
**d. Rise to span ratio = 1/5**
**e. No. of panels = 6**

Solution:

**(A)** Span of truss = 12 m
Spacing = 3m /c/c
Types of truss = sink
No. of panel point = 6

$$\text{Rise} = \frac{\text{span}}{5}$$

$$= \frac{12}{5} = 24\text{m}$$

$$\theta = \tan^{-1}\left(\frac{\text{Risc}}{L/2}\right) = \tan^{-1}\left(\frac{2.4}{6}\right) = 21.80^{\circ}$$

Calculation of dead load
(i) Weight of roofing = 150 N/m$^2$
(ii) Weight of Purlin = 220 N/m$^2$
(iii) Weight of truss = $\left(\frac{L}{3}+5\right)\times 10$
= 90 N/m$^2$
(iv) Weight of bracing = 80 N/m$^2$

Total dead load = 540 N/m$^2$
Total dead load on one truss = 540 N/m$^2$
$= 540 \times 12 \times 3$
= 19.44 kN

Dead load on each panel point = $\frac{19.44}{2} = 3.2H$ kN

D on end panel point = $\frac{324}{2}$

= 1.62 kN

Live load calculation

L.L. on purlin = 750 – (0 – 10) × 20)

= 750 – [2180 – 10) × 20]

= 514 N/m$^2$ > 400 N/m$^2$

L.L of truss = 2/3 × 514 = 342.67 N/m$^2$

∴ Total L. L. = L.L. of truss × span × spacing

= 342.67 × 12 × 3

= 12336 N

L.L. m each panel = $\frac{12336}{6}$ = 2056 N

L.L. m end panel = $\frac{2056}{2}$ = 1028 N

53. **An industrial building has trusses for 14 m span. Trusses are spaced at 4m c/c and rise of truss in 3.6m. Calculate panel point load in case of live load and wind load using following data :**

**a. Coefficient of external wind pressure = - 0.7**

**b. Coefficient of internal wind pressure = ± 0.2**

**c. Design wind pressure = 1.5 kPa**

**d. Number of panels = 08**

**(A)** Span of trus = 14m

Spacing of truss = 3.6 m

No. of panels = 8

Design wind pressure = 1.5 kpa

= 1.5 × 10$^3$ N/m$^2$

$$\theta = \tan^{-1}\left(\frac{\text{Rise}}{\text{Span}/2}\right) = \frac{3.6}{14/2} = 27.22^\circ$$

∴ θ = 27.22°

Wind load calculation

Coefficient of external wind pressure

Cpe = -0.7

Coefficient of internal wind pressure

Cpi = ±02

Total wind press = [Cpe – Cpi] × $P_2$

Wind load combination

i) w.c = [–0.7 – (0.2)] × 1500 = 750 N/ m$^2$

ii) w.c = [–0.7 – (+0.2)] × 1500 = 1350 N/m$^2$

Max. intensity = –1350 N/m$^2$

Length of principle dafter = $\frac{L/2}{\cos\theta}$

= [1412]

cos 27.22

Length of principle dafter = 7.87 m

$\because$ Sloping area = $2 \times 7.87 \times 4$

= $62.96\ m^2$

$\because$ Total wind load = Max. intensity $\times$ sloping area

= $1350 \times 62.96$

= 84996 N

Wind

$\because$ load an each panel = $\dfrac{84996}{8}$

$\therefore$ wind load on end panel = -10624.5 N

$\therefore$ wind load on end panel = $\dfrac{-10624.5}{2}$

= 5312.25 N

Live load calculation

Live load on purlin = $750 - [(\theta - 10) \times 20]$

= $750 - [(27.22 - 10) \times 20]$

= $405.6 \times 4v\ N/m^2$

Hence ok

L.L. on truss

= $2/3 \times 405.6$

= $270.4\ N/m^2$

$\because$ Total L.L = L.L. intensity $\times$ Span $\times$ spacing

= $270.4 \times 14 \times 4$

= 15142.4 N

$\because$ load on each panel = $\dfrac{T.L}{\text{No. of Panel}}$

= $\dfrac{15142.4}{8}$

= 1892.8 N = 1.892 kN

and load on end panel = $\dfrac{1892.8}{2}$

= 946.4 N = 0.926 kN

54. **Design a slab base for column ISHB 400 @ 82.2 kg/m to carry factored axial compressive load of 2000 kN. The base rests on concrete pedestal of grade M20. For ISHB 400, bf = 250 mm, fy = 250 MPa, fu = 410 MPa, $\gamma_{mo}$ = 1.1, tf = 12.7 mm.**

**Solution:**

**(A)** Given

Factored load pu = 2000 kN

= $2000 \times 10^3$ N

Fck = 20

D = 400

B = 250 i.e bf

$v_{mo}$ = 1.1

tf = 12.7

fy = 250 $N/mm^2$

Bearing Strength of conc

$= 0.6\ fek$

$= 0.6 \times 20 = 12\ N/m^2m$

Bearing area of base plate

$$A = \frac{Pu}{\text{Bearing strength of conc}}$$

$$A = \frac{2000 \times 10^3}{12} = 166.67 \times 16^3$$

Size of base plate

length of plate

$$Lp = \frac{D-B}{2} + \sqrt{\left(\frac{D-B}{2}\right)^2 + A}$$

$$= \frac{400-250}{2} + \sqrt{\left(\frac{400-250}{2}\right)^2 + 1666.67 \times 10^3}$$

$$= 490.08 \cong 500$$

$$Bp = \frac{A}{Lp} = \frac{166.67 \times 10^3}{500} = 333.34 \cong 350$$

Larger Projection

$$a = \left(\frac{Lp - p}{2}\right) = \frac{500 - 400}{2} = 50\ mm$$

Smaller Projection

$$b = \left(\frac{Bp - B}{2}\right) = \frac{350 - 250}{2} = 50\ mm$$

Area of base plate

$Ap = 500 \times 350 = 175 \times 10^3$

Ultimate Pressure from below m the
Slab base

$$w = \frac{Pu}{A} = \frac{2000 \times 10^3}{175 \times 10^3} = 11.42\ N/mm^2$$

Thickness of slab base

$$f_s = \sqrt{\frac{2.5\,w\left(a^2 - 0.3b^2\right)vm_0}{fy}}$$

$$= \sqrt{\frac{2.5 \times 11.42\left(50^2 - 0.3 \times 50^2\right) \times 1.10}{250}}$$

$= 14.82\ mm \quad > \quad tf \quad$ I.e. $k - 7$

$\cong 15\ mm$

Hence provide slob base plate having dimension
$500 \times 350 \times 15$

55. **Write steps to calculate the thickness of base plate used in slab base. Why anchor bolts are used in slab base?**

**Answer:**

**Design steps to find thickness**

1) To calculate area (A) of base plate

   A = Column load/Bearing strength

   Bearing strength of concrete = $0.6\ f_{ck}$

2) Select the size of base plate.

   $L_p$ & $B_p$ be the sizes of plate

   D = length or longer length

   B = width or shorter side of the column

   Consider square plate

$$L_P = \frac{(D-B)}{2} + \sqrt{\left[\left\{\frac{(D-B)}{2}\right\}^2 + A\right]}$$

$$B_P = \frac{A}{L_p}$$

   Large projection $a = \frac{(L_p - D)}{2}$

   Shorter projection $b = \frac{(B_p - B)}{2}$

   Area of base plate provided = $L_P \times B_P = (D + 2a) \times (B + 2b)$

3) Calculate ultimate bearing pressure

$$w = \frac{P}{(L_p \times B_p)}$$

[1 mark]

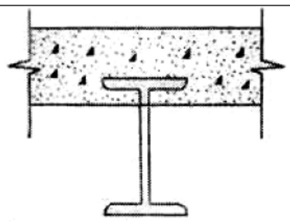
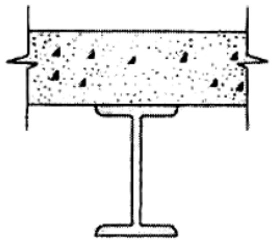
4) Calculate thickness of base plate

$$t_s = \left[\left(\frac{2.5 \times w(a^2 - 0.3 \times b^2) r_{mo}}{f_y}\right)\right]^{0.5}$$

[1 mark]

**Function of anchor bolt** : To connect concrete pedestal and base plate anchor bolts are used.

56. **Differentiate between Laterally supported and unsupported beams with a neat sketch.**

| | Laterally supported beam | Laterally unsupported beam |
|---|---|---|
| 1) | In laterally supported beam, compression flanges are embedded in concrete. | In laterally unsupported beam, compression flanges are not embedded in concrete. |
| 2) | Compression flange of Beam is restrained against rotation | Compression flange of Beam is free for rotation. |
| 3) | Lateral deflection of compression flange is not occur. | Lateral deflection of compression flange is occur. |
| 4) |  Laterally supported.(it means compression flange is restrained) |  Laterally unsupported. |

57. **Define Gusseted base. Also draw its labelled sketch showing all details.**

**Answer:**

**Definition**

When the load on column is large or column subjected to moment along with axial load, base is provided called gusseted base.

It consists of base plate, gusset angle, connecting angle on either side of column.

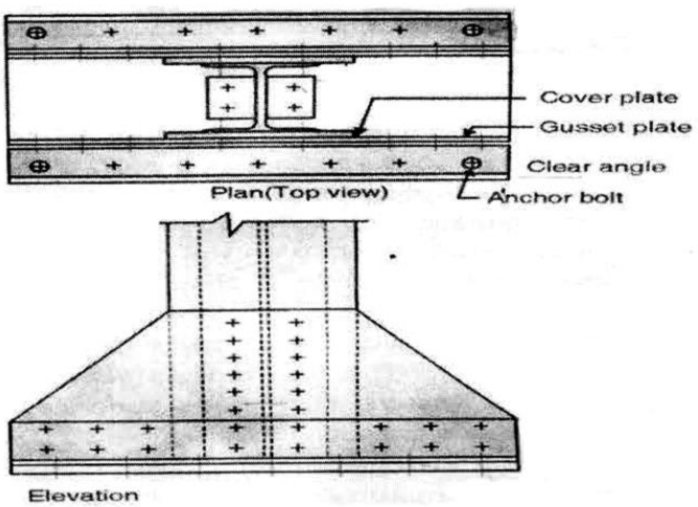



**Fig.** : Gusseted Base

58. **How beam sections are classified for bending as per IS : 800- 2007. Describe any two of them.**

Answer:

**Classification beam:**

1) Plaster or class – I
2) Compact or class – II
3) Semi compact or class – III
4) Slender or class – IV

**Explain in detail**

**1) Plastic or class – I**

Cross section which can develop plastic hinge, sustain large rotation capacity required to develop plastic mechanism are called as plastic section. These sections are unaffected by local buckling and are able to develop their full plastic moment capacities until a collapse mechanism is formed.

**2) Compact or class – II**

In compact section, the full cross section forms first plastic hinge but local buckling prevents subsequent moment redistribution. These sections develop full plastic moment capacities $M_P$ but fails by local buckling due to inadequate plastic hinge rotation capacity.

**3) Semi compact or class – III**

In semi plastic section the extreme fibres reach the yield stress but local buckling prevents the development of plastic moment resistance.

**4) Slender or class – IV**

The slender section cannot attain even the first yield moment because of premature local buckling of web or flange.

59. **A simply supported beam of 6 m span supports on R. C. C. slab where in compression flange is embedded. The beam is subjected to a dead load of 25 kN/m and super imposed load of 20 kN/m, over entire span. Calculate plastic and elastic modulus required.**

**Assume $r_f$ = 1.5, $\gamma_m$ = 1.1 $f_y$ = 250 N/mm$^2$.**

**Solution:**

**1) Calculation of factored load**

Dead load = $1.5 \times 25 = 37.5$ KN/m

Live load = $1.5 \times 20 = 30$ KN/m

**2) Calculate Maximum bending moment and shear force.**

$$\text{B.M.} = \frac{WL^2}{8} + \frac{WL^2}{8} = \frac{37.5 \times 6^2}{8} + \frac{30 \times 6^2}{8} = 303.75 \text{ KN.m}$$

$$\text{S.F.} = \frac{WL}{2} + \frac{WL}{2} = \frac{37.5 \times 6}{2} + \frac{30 \times 6}{2} = 202.5 \text{ KN.m}$$

**3) Plastic modulus**

$$Z_P = \frac{M \times r_{mo}}{f_y} = \frac{303.75 \times 10^6 \times 1.1}{250} = 1.3365 \times 10^6 \text{ mm}^3$$

**4) Elastic modulus**

$$Z_e = \frac{Z_P}{1.14} = \frac{1.3365 \times 10^6}{1.14} = 1.17236 \times 10^6 \text{ mm}^3$$

60. **Which of the following are true about roof trusses?**

a) principal rafter are compression members used in buildings
b) principal rafter is bottom chord member of roof truss
c) struts are compression members used in roof trusses
d) struts are tension members used in roof trusses

Answer: c

61. **Which of the following is not a load on columns in buildings?**
a) load from floors
b) load from foundation
c) load from roofs
d) load from walls

Answer: b

62. **What are loads on columns in industrial buildings?**
a) wind load only
b) crane load only
c) wind and crane load
d) load from foundation

Answer: c

63. **Which of the following assumptions is correct for plastic design?**
a) material obeys Hooke's law before the stress reaches $f_y$
b) yield stress and modulus of elasticity does not have same value in compression and tension
c) material is homogenous and isotropic in both elastic and plastic states.
d) material is not sufficiently ductile to permit large rotations

Answer: c

64. **What is plastic hinge?**
a) zone of bending due to flexure in a structural member
b) zone of yielding due to flexure in a structural member
c) zone of non-yielding due to flexure in a structural member
d) zone of yielding due to twisting in a structural member

Answer: b

65. **What is plastic-collapse load?**
a) load at which sufficient number of elastic hinges are formed
b) load at which sufficient number of plastic hinges are not formed
c) load at which sufficient number of plastic hinges are formed
d) load at which structure fails

Answer: c

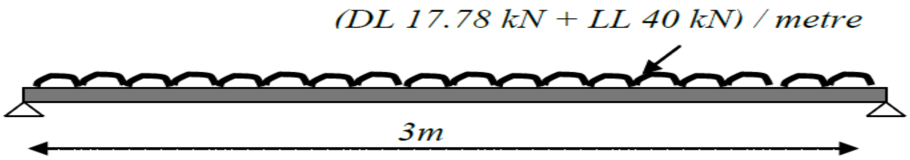
66. **Which of the following is true?**
   a) ultimate load is reached when a mechanism is formed
   b) ultimate load is not reached when a mechanism is formed
   c) plastic hinges are not required for beam to form a mechanism
   d) frictionless hinges are not required for beam to form a mechanism

Answer: a

67. **Which of the following condition is true for kinematic theorem?**
   a) load must be greater than collapse load
   b) load must be less than collapse load
   c) load must be not equal to collapse load
   d) load cannot be related to collapse load

Answer: a

68. **Design a suitable 'I' beam for a simply supported span of 3 m and carrying a dead or permanent load of 17.78 kN/m and an imposed load of 40 kN/m. Assume full lateral restraint and stiff support bearing of 100 mm.**




Solution:

***Design load calculation:***

$$factored\ load = \gamma_{LD} \times 17.78 + \gamma_{LL} \times 40\ kN$$

*in this example the following load factors are chosen.*

$\gamma_{LD}$ *and* $\gamma_{LL}$ *are taken as 1.35 and 1.50 respectively.*

$\gamma_{LD}$ – *partial safety factor for dead or permanent loads*
$\gamma_{LL}$ – *partial safety factor for live or imposed loads*

$$Total\ factored\ load = 1.35 \times 17.78 + 1.5 \times 40.0 = 84\ kN/m$$

$$Factored\ bending\ moment = 84 \times 3^2/8 = 94.50\ kN-m$$

*Z—value required for* $f_y = 250\ MPa$ ; $\gamma_m = 1.15$

$$Z_{reqd} = \frac{94.5 \times 1000 \times 1000 \times \gamma_m}{250}$$

$$Z_{reqd} = 434.7\ cm^3$$

**Try ISMB 250**

$$\varepsilon = \sqrt{\frac{250}{250}} = 1.0 \qquad D = 250\ mm$$

$B = 125\ mm$

$t = 6.9\ mm$

$T = 12.5\ mm$

$I_{xx} = 5131.6\ cm^4$

$I_{yy} = 334.5\ cm^4$

**Section classification:**

Flange criterion $= B/2T \qquad = 5.$

Web criterion $= (D - 2T)/t = 32.61$

Since $B/2T < 8.92\ \boldsymbol{\varepsilon}$ & $(D\text{-}2T)/t < 82.95\ \boldsymbol{\varepsilon}$

The section is classified as ' **PLASTIC** '

**Moment of resistance of the cross section:**

Since the section considered is 'PLASTIC'

$$M_C = \frac{S \times f_y}{\gamma_m}$$

Where S is the plastic modulus

'S' for ISMB 250 $= 459.76\ cm^3$

$M_c = 459.76 \times 1000 \times 250 / 1.15$

$= 99.95\ kN\text{-}m > 94.5\ kN\text{-}m$

Hence **ISMB-250** is adequate.in flexure.

**Shear resistance of the cross section:**

This check needs to be considered more importantly in beams where the maximum bending moment and maximum shear force may occur at the same section simultaneously, such as the supports of continuous beams. For the present example this checking is not required. However for completeness this check is presented.

Shear capacity $P_v = \dfrac{0.6 f_y A_v}{\gamma_m}$

$A_v = 250 \times 6.9 = 1725\ mm^2$

$P_v = 0.6 \times 250 \times 1725 / 1.15 = 225\ kN$

$F_v$ = factored max shear $= 84 \times 3 / 2 = 126.\ kN$

$F_v / P_v = 126/225.0 = 0.56 < 0.6$

Hence the effect of shear need not be considered in the moment capacity calculation.

**Check for Web Buckling:**

The slenderness ratio of the web $= L_E/r_y = 2.5\ d/t = 2.5 \times 194.1/6.9$

$= 70.33$

The corresponding design compressive stress $f_c$ is found to be

$f_c = 203\ MPa$ (Design stress for web as fixed ended column)

Stiff bearing length $= 100\ mm$

$45^0$ dispersion length $n_1 = 125.0\ mm$

$P_w \quad (100 + 125.0) \times 6.9 \times 203.0$

$= 315.16\ kN$

$315.16 > 126$ Hence web is safe against shear buckling

***Check for web crippling at support***

*Root radius of ISMB 250* $=$ *13 mm*

*Thickness of flange + root radius* $=$ *25.5 mm*

*Dispersion length (1:2.5)* $n_2 = 2.5 \times 25.5 = 63.75$ *mm*

$P_{crip} = (100+63.75) \times 6.9 \times 250 / 1.15$

$= 245.63 \text{ kN} > 126 \text{kN}$

*Hence ISMB 250 has adequate web crippling resistance*

***Check for serviceability – Deflection:***

*Load factors for working loads* $\gamma_{LD}$ *and* $\gamma_{LL} = 1.0$

*design load* $= 57.78$ *kN/m.*

$$\delta = \frac{5 \times 57.78 \times 3000^4}{384 \times 2.1 \times 10^5 \times 5131.6 \times 10^4}$$

*Max deflection* $= 5.65\, mm$

$$= \frac{L}{531}$$

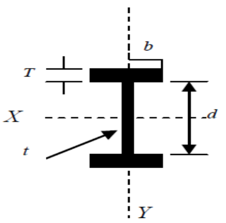
$$\frac{L}{531} < \frac{L}{200}$$

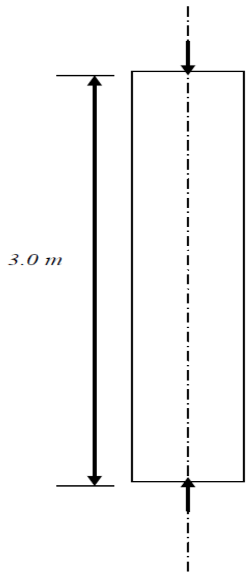
*Hence serviceability is satisfied*

***Result** :-- Use ISMB – 250.*

69. **Obtain factored axial load on the column section ISHB400. The height of the column is 3.0 m and it is pin-ended. Use fy = 250 N/mm$^2$, E = 2 x 10$^5$ N/mm2, ym= 1.15.**

*CROSS-SECTION PROPERTIES:*




Solution:

*Flange thickness* = $T$ = 12.7 mm

*Clear depth between flanges* = $d$ = 400 – (12.7 * 2) = 374.6 mm

*Thickness of web* = $t$ = 10.6 mm

*Flange width* = $2b$ = 250 mm

$b$ = 125 mm

*Self –weight* = $w$ = 0.822 kN/m

*Area of cross-section* = $A$ = 10466 $mm^2$

$r_x$ = 166.1 mm

$r_y$ = 51.6 mm

**(i) Type of section:**

$$\frac{b}{T} = \frac{125}{12.7} = 9.8 < 10 \in$$

$$\frac{d}{t} = \frac{374.6}{10.6} = 35.3 < 41 \in$$

$$where, \in = \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{250}} = 1.0$$

*Hence, cross- section is "COMPACT"*

**(ii) Effective Length:**

*As, both ends are pin-jointed effective length* = $\ell_x = \ell_y = 3.0$ m

**(iii) Slenderness ratios:**

$$\lambda_x = \frac{\ell_x}{r_x} = \frac{3000}{166.1} = 18.1$$

$$\lambda_y = \frac{\ell_y}{r_y} = \frac{3000}{51.6} = 58.1$$

**(iv) Values of ( $\alpha$ ) :**

*For rolled I-sections,*

*In x – direction* $\alpha_x = 0.0020$
*In y – direction* $\alpha_y = 0.0035$

$$\lambda_o = 0.2\pi \sqrt{E/f_y} = 0.2 * \frac{22}{7} \sqrt{\frac{200000}{250}}$$

$$= 17.8$$

**(v) values of $\eta$:**

$$\eta = \alpha \ (\lambda - \lambda_0)$$

$$\eta_x = \alpha_x (\lambda_x - \lambda_0) = 0.002 * (18.1 - 17.8) = 0.001$$

$$\eta_y = \alpha_y (\lambda_y - \lambda_0) = 0.0035 * (58.1 - 17.8) = 0.141$$

**(vi) Calculation of maximum compressive stress at failure ($\sigma_c$):**

We have,

$$\sigma_e = \frac{\pi^2 E}{\lambda^2}$$

$$\sigma_c = \phi \pm \sqrt{\phi^2 - f_y \sigma_e} \le f_y$$

$$\text{where, } \phi = \frac{f_y + (\eta+1)\,\sigma_e}{2}$$

In x-direction,

$$\sigma_{ex} = \frac{\pi^2 E}{\lambda_x^2} = \frac{\pi^2 * 200000}{(18.1)^2} = 6025\ N/mm^2$$

$$\varphi_x = \frac{250 + (0.001+1)*6025}{2} = 3140\ N/mm^2$$

$$\sigma_{cx} = 3140 \pm \sqrt{(3140)^2 - 250*6025} \le 250$$

$$= 250\ N/mm^2$$

In y-direction,

$$\sigma_{ey} = \frac{\pi^2 E}{\lambda_y^2} = \frac{\pi^2 * 200000}{(58.1)^2} = 585\ N/mm^2$$

$$\varphi_y = \frac{250 + (0.141+1)585.0}{2} = 459 N/mm^2$$

$$\sigma_{cy} = 459 \pm \sqrt{(459)^2 - 250*585} \le 250$$

$$= 205\ N/mm^2$$

Hence, Allowable axial compressive stress, $\sigma_c = 205\ N/mm^2$

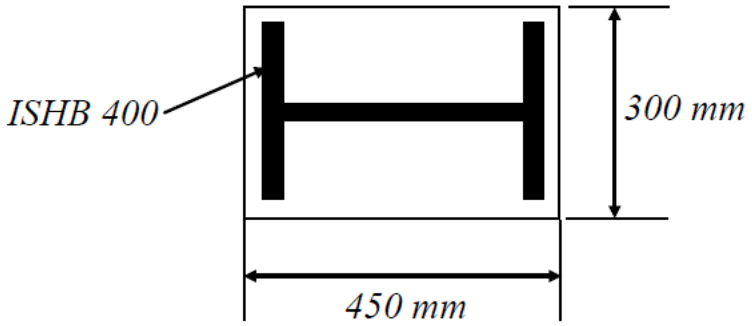
Safe axial compressive stress $= \sigma_c/\gamma_m = 205/1.15 = 178\ N/mm^2$

**(vii) Factored Load:**

$$\text{Factored Load} = \sigma_c A/\gamma_m = 178 * 10466/1000$$

$$= 1863\ kN$$

70. **Design a simple base plate for a ISHB400 @ 0.822 kN/m column to carry a factored load of 1800 kN. Use fcu = 40 N/mm2 ; fy = 250 N/mm2 ; γm = 1.15**




Solution:

*Bearing strength of concrete* $= 0.4f_{cu} = 0.4 * 40 = 16\ N/mm^2$

*Area required* $= 1800*10^3/16 = 112500\ mm^2$

*Use plate of 450 X 300 mm (135000 mm²)*

*Assuming projection of 25 mm on each side*

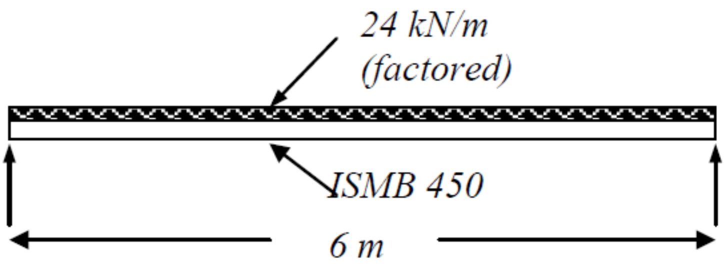
$w = (1800 * 10^3) / (450 * 300) = 13.33\ N/mm^2$

$f_{yp} = 250/1.15 = 217.4\ N/mm^2$

$$t_p = \sqrt{\frac{2.5w\left(a^2 - 0.3b^2\right)}{f_{yp}}} = \sqrt{\frac{2.5\times 13.33\left(25^2 - 0.3\times 25^2\right)}{217.4}} = 8.2\ mm$$

*Hence, use 450 X 300 X 10 mm plate.*

71. **Check the adequacy of ISMB 450 to carry a uniformly distributed load of 24 kN / m over a span of 6 m. Both ends of the beam are attached to the flanges of columns by double web cleat.**

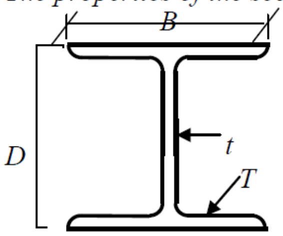


*Design check:*

*For the end conditions given, it is assumed that the beam is simply supported in a vertical plane, and at the ends the beam is fully restrained against lateral deflection and twist with, no rotational restraint in plan at its ends.*

*Section classification of ISMB 450*

*The properties of the section are:*




*Depth, D = 450 mm*

*Width, B = 150 mm*

*Web thickness, t = 9.4 mm*

*Flange thickness, T = 17.4 mm*

*Depth between fillets, d = 379.2 mm*

*Radius of gyration about minor axis,* $r_y = 30.1$ *mm*

*Plastic modulus about major axis,* $S_x = 1512.8 * 10^{-3}\ mm^3$

*Assume* $f_y = 250\ N/mm^2$, $E=200000\ N/mm^2$, $\gamma_m = 1.15$,

$p_y = f_y / \gamma_m = 250 / 1.15 = 217.4\ N/mm^2$

**(I) Type of section**

**Flange criterion:**

$$b = \frac{B}{2} = \frac{150}{2} = 75\ mm$$

$$\frac{b}{T} = \frac{75.0}{17.4} = 4.31$$

$$\frac{b}{T} < 8.92\varepsilon \quad \text{where } \varepsilon = \sqrt{\frac{250}{f_y}}$$

*Hence O.K.*

**Web criterion:**

$$\frac{d}{t} = \frac{379.2}{9.4} = 40.3$$

$$\frac{d}{t} < 82.95\,\varepsilon$$

*Hence O.K.*

*Since* $\frac{b}{T} < 8.92\,\varepsilon$ *and* $\frac{d}{t} < 82.95\,\varepsilon$, *the section is classified as 'plastic'*

***(II)Check for lateral torsional buckling:***

*Equivalent slenderness of the beam,* $\lambda_{LT} = n\,u\,v\,\lambda$

*where,* $n$ *= slenderness correction factor (assumed value of 1.0)*

$u$ *= buckling parameter (assumed as 0.9)*

$\lambda$ *= slenderness of the beam along minor axis*

$$= \frac{6000}{30.1} = 199.33$$

$v$ *= slenderness factor (which is dependent on the proportion of the flanges and the torsional index [D / T])*

*= 0.71 (for equal flanges and* $\lambda = 199.33$*)*

*Now,* $\lambda_{LT} = 1.0 * 0.9 * 0.71 * 199.33$

$= 127.37$

*Bending strength,* $p_b$ *= 84 Mpa (for* $\lambda_{LT} = 127.37$*) (from Table 11 of BS 5950 Part I)*

*Buckling resistance moment* $M_b = S_x * p_b$

$= (1512.78 * 84)/1000$

$= 127.07$ *kN m*

*For the simply supported beam of 6.0 m span with a factored load of 24.0 KN/m*

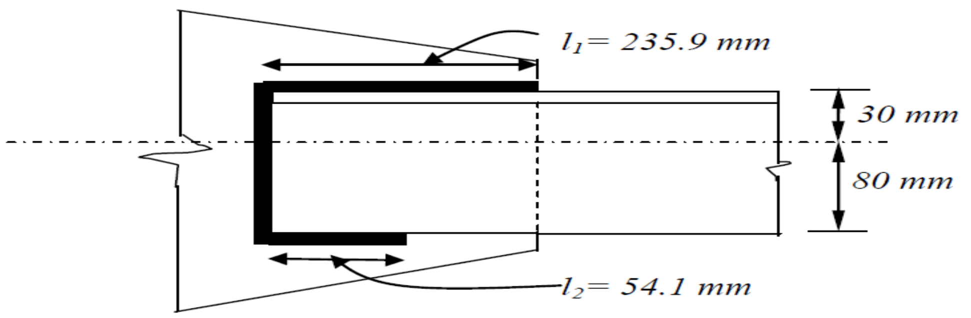
$$M_{max} = \frac{w\ell^2}{8} = \frac{24*6^2}{8}$$

$$= 108.0 \text{ KN m} < 127.07 \text{ kN m}$$

*Hence* $M_b > M_{max}$

*ISMB 450 is adequate against lateral torsional buckling.*

72. **In a roof truss, a tie member ISA 110 mm X 110mm X 8 mm carries a factored tension of value 210 kN. The tie is connected to a gusset plate 8 mm thick. Design the welded joint. Factored yield strength of steel is 217.4 N/mm2 and shear strength of weld is 125 N/mm2.**




*Fillet weld for tie member of a roof truss*

Solution:

*For this problem we would provide a weld group consisting of transverse and longitudinal welds and ensure that the CG of the weld group coincides with the line of action of the externally applied load.*

*First we would decide about the weld size. This is decided by the thickness of the rolled section and the plating. Weld which are applied to rounded toe of rolled section should not be more than ¾ of its thickness or plating and hence we get a weld size of 6mm (3/4* 8). The maximum size of the end weld is also limited by the thickness of the plating, which is 8-1.5=6.5 mm. Hence 6 mm fillet welds are O.K.*

*Transverse weld is provided equal to the size of the leg = 110 mm.*

*Force transmitted by transverse weld = (125* 0.7* 6* 110) /1000*
*=57.75 kN*

*Remaining force to be transmitted by the longitudinal welds = 210 – 57.75*
*=152.25 kN*

*We must ensure that the CG of the welds coincides with line of action of the external force. This could be ensured by providing longitudinal welds along the near and far side of the angle and also by ensuring that the moment of the all the forces about any of the line of the weld vanishes.*

*Let us assume that the lengths of the welds in the heel and toe sides are* $l_1$ *and* $l_2$ *respectively.*

*Total weld length required for 152.25 kN*
*=152.25 * 1000 / ((125* 0.7*6))=290 mm*

*Taking moment of all forces about the heel side longitudinal weld, we get*

$57.75 * 1000 * 55 + l_1 * 0 + l_2 * (125. * 0.7 * 6) * 110$
$= 210 * 1000 * 30.$

*Therefore* $l_2$ $= 54.09$ *mm*

*Hence we get the weld length* $l_2$ *as say 54.09 mm. The bracketed term in the above expression represents the strength of the weld for 1 mm.*

*Now we get the length* $l_1$ *as 290 – 54.09 =235.91 mm*

*Alternatively the longitudinal weld length* $l_1$ *is obtained by taking moment of all the forces about the toe side weld line. Hence we have demonstrated as to how a weld group could be designed to have a CG coinciding with the externally applied load.*

*It is also to be noted that in case it is desired to reduce the length of the joint then the heel side weld size can be increased.*

73. **Design a bolted connection between a bracket 8 mm thick and the flange of an ISHB 400 column using HSFG bolts, so as to carry a vertical load of 100 kN at a distance of 200 mm from the face of the column as shown in Fig. E1.**

Solution:

*1) Bolt force:*

$P_x = 0; P_y = 100\ kN;$

*Total eccentricity* $x' = 200 + 250/2 = 325\ mm$

$M = P_y x' = 100 x 325 = 32500\ kN\text{-}mm$

*Try the arrangement shown in Fig. E1*

*Note: minimum pitch = 60 mm and*

*minimum edge dist. = 60 mm*

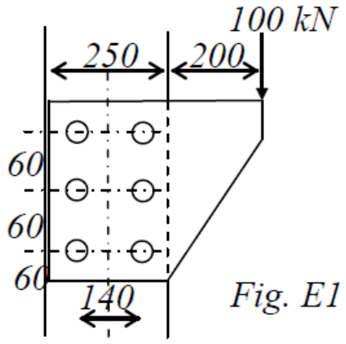



*Fig. E1*

$n = 6$

$\Sigma r_i^2 = \Sigma x_i^2 + \Sigma y_i^2 = 6(70)^2 + 4(60)^2 = 43800\ mm^2$

*Shear force on the farthest bolts (corner bolts)*

$$R_i = \sqrt{\left\{\left[\frac{32500 \times 60}{43800}\right]^2 + \left[\frac{100}{6} + \frac{32500 \times 70}{43800}\right]^2\right\}} = 81.79\ kN$$

*2) Bolt capacity*

*Try M20 HSFG bolts*

*Bolt capacity in single shear* $= 1.1\ K\ \mu\ P_o = 1.1 \times 0.45 \times 177 = 87.6\ kN$

*ISHB 400 flange is thicker than the bracket plate and so bearing on the bracket plate will govern.*

*Bolt capacity in bearing* $= d\ t\ p_{bg} = 20 \times 8 \times 650 \times 10^{-3} = 104\ kN$

$\therefore$ *Bolt value* $= 87.6\ kN > 81.79$ *safe.*

74. **Design a bolted web cleat beam-to-column connection between an ISMB 400 beam and an ISHB 200 @ 40 kg/m column. The connection has to transfer a factored shear of 150 kN. Use bolts of diameter 20 mm and grade 4.6.**

**Solution:**

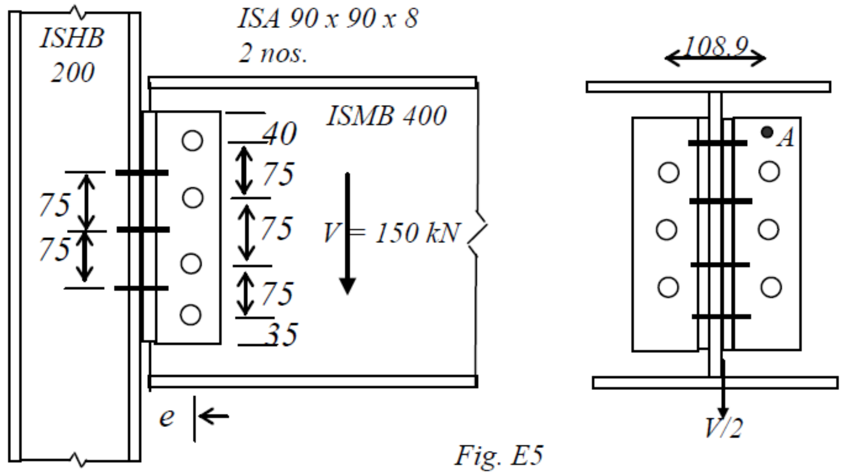



Fig. E5

1) The recommended gauge distance for column flange is 100 mm.
   Therefore required angle back mark is 50 mm.
   Use web cleats of ISA 90x90x8 giving gauge g = 50+50+8.9=108.9 mm

2) Connection to web of beam- Bolt capacity
   shear capacity of bolt in double shear = $2\times160\times245\times10^{-3}=78.4$ kN
   bearing capacity of bolt on the beam web = $418\times20\times9.0\times10^{-3}= 75.24$ kN
   bolt value = 75.24 kN

   Try 4 bolts as shown in the Figure with vertical pitch of 75 mm

   Assuming the shear to be acting on the face of the column, its eccentricity with the centre of the bolt group will produce horizontal shear forces in the bolts in addition to the vertical shear.

   horizontal shear force on top bolt due to moment due to eccentricity e
   $= 150\times50\times112.5/2(37.5^2+112.5^2) = 30.0$ kN

   vertical shear force per bolt = 150/4 = 37.5 kN

   resultant shear = $\sqrt{(30.0^2+37.5^2)} = 48.0$ kN < bolt value Safe !

*3) Connection to column flange: Bolt capacity*

*shear capacity of bolt in single shear* $= 160\times245\times10^{-3} = 39.2$ *kN*
*bearing capacity of bolt on column flange* $= 418\times20\times9.0\times10^{-3} = 75.24$ *kN*
*bolt value* $= 39.2$ *kN*

*Try 6 bolts as shown in the Fig.E5 with vertical pitch of 75 mm*

*4) Check bolt force*
*Similar to the previous case, the shear transfer between the beam web and the angle cleats can be assumed to take place on the face of the beam web. However, unlike the previous case, no relative rotation is possible between the angle and the beam web.*

*Assuming centre of pressure 25 mm below top of cleat (point A), horizontal shear force on bolt due to moment due to eccentricity e*
$= (150\times50/2)\times 200/(50^2+125^2+200^2) = 12.9$ *kN*

*vertical shear force per bolt* $= 150/6 = 25.0$ *kN*

*resultant shear* $= \sqrt{(12.9^2+25.0^2)} = 28.13$ *kN* $<$ *bolt value OK*

*Use 2 Nos ISA 90x90x8 of length 375 mm as angle cleats*

75. **Design a double web cleat connection for an ISMB 400 coped beam to an ISMB 600 main beam so as to transfer a factored load of 300 kN using HSFG bolts of 20mm diameter and grade 8.8.**

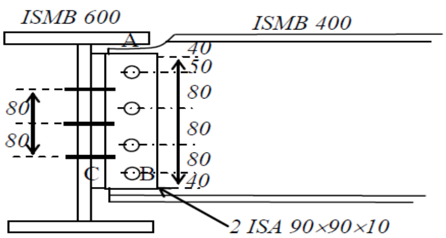



*Fig. E7*

***Solution:***
*1) Connection to web of ISMB 400*

*For M20 Gr.8.8 HSFG bolts in double shear*
*Slip resistance per bolt* $= 2\times1.1\times0.45\times144 = 142.6$ *kN*
*Bearing capacity of web per bolt* $= 20\times8.9\times650\times10^{-3} = 115.7$ *kN*
*Bolt value* $= 115.7$ *kN*

*Try 4 bolts as shown in the Figure with vertical pitch of 80 mm*

*Assuming the shear to be acting on the face of the ISMB 600 web, its eccentricity with the centre of the bolt group will produce horizontal shear forces in the bolts in addition to the vertical shear.*

*horizontal shear force on top bolt due to moment due to eccentricity e*
$= (300/2)\times50\times112.5/(37.5^2+112.5^2) = 60.0$ *kN*

*vertical shear force per bolt* $= 300/4 = 75.0$ *kN*

*resultant shear* $= \sqrt{(60^2+75^2)} = 96.0$ *kN* $<$ *bolt value Safe !*

2) *Connection to web of ISMB 600*

*Try 6 bolts as shown in the Figure with vertical pitch of 80 mm*

*For M20 Gr.8.8 HSFG bolts in single shear*
*Slip resistance per bolt* $= 1.1 \times 0.45 \times 144 = 71.28$ *kN*
*Bearing capacity of web per bolt* $= 20 \times 12 \times 650 \times 10^{-3} = 156$ *kN*
*Bolt value* $= 71.28$ *kN*

*Assuming center of pressure 27.5 mm below the top of the angle*

*horizontal shear force on bottom bolt due to moment due to eccentricity e*
$= (300/2)\times 50\times 200/(50^2+125^2+200^2) = 25.82$ *kN*

*vertical shear force per bolt* $= 300/6 = 50.0$ *kN*

*resultant shear* $= \sqrt{(25.82^2+50^2)} = 56.27$ *kN < bolt value Safe !*

*3) Check web of ISMB 400 for block shear*

*Block shear capacity = shear capacity of AB + 0.5×tensile capacity of BC*
$= 0.6\times 250\times 0.9\times 1.1(3\times 80+50-3.5\times 22)\times 8.9\times 10^{-3}$
$+ 0.5\times 250\times 1.1(45-0.5\times 22)\times 8.9\times 10^{-3} = 323.12 > 300$ *kN Safe!*

76. **The shear lag width for ISA 75X75X10 is (Assume gauge distance = 40 mm).**

Solution:

The length of outstanding leg will be w = 75 mm and w1 = 40 mm.

So the shear lag width, bs = w + w1 – t = 75 + 40 – 10 = 105 mm.

77. Explain various types of standard rolled steel sections.

Answer:

**Various types of standard rolled steel sections**
i) Rolled steel I-sections (Beam sections)
ii) Rolled steel channel sections
iii) Rolled steel Tee sections
iv) Rolled steel angle sections

v) Rolled steel bars
vi) Rolled steel flats
vii) Rolled steel plates
viii) Rolled steel sheets
ix) Rolled steel strips
x) Rolled steel tubular sections

**(a) Rolled steel I – sections (Beam sections)**

• Indian Standard Junior Beam (ISJB)
• Indian Standard Light Beam (ISLB)
• Indian Standard Medium weight Beam (ISMB)
• Indian Standard Wide flange Beam (ISWB)
• Indian Standard Heavy Beam (ISHB)
• An I – Section is designated by its depth and weight

**Eg:** An ISLB 500 @ 735.8 N/m means, An I – section is 500 mm deep and self weight is 735.8 N per meter length.
• Special beam section available from Indian rolling mill is Indian Column Section (ISC)

**(b) Rolled Steel Channel Sections**
• Indian Standard Junior Channel (ISJC)
• Indian Standard Light Channel (ISLC)
• Indian Standard Medium Weight Channel with Sloping Flange (ISMC)
• Indian Standard Medium Weight Channel with parallel flange (ISMCP)
• Indian Standard Gate Channel (ISGC)
• Designated by its depth and weight

**Ex:** ISLC 350 @ 380.63 N/m

**(c) Rolled Steel T – Sections**
• Indian Standard rolled Normal T – section (ISNT)
• Indian Standard rolled Deep legged T – (ISDT)
• Indian Standard rolled silt Light weight T – bars (ISLT)
• Indian Standard rolled silt Medium weight T – bars (ISMT)
• Indian Standard rolled silt T – bars from H – section (ISHT)
• Designated by its depth and weight

**Ex** : ISNT 125 @ 274 N/m

**(d) Rolled Steel Angle sections**
• Indian standard equal angles, Indian standard unequal angles and Indian standard bulb angles
• Designated by abbreviation ISA along with widths of both legs and thickness.
• Indian equal angles are designated as ISA or ISEA (**Ex.** ISEA 100 x 100 x 10 mm), Indian standard unequal angles are designated as ISA (**Ex.** ISA 125 x 75 x 10 mm) and Indian standard bulb angles are designated as ISBA.

78. **Which of the following is correct in case of angle members?**
   a) connection of lug angle to angle member should be capable of developing a strength of 10% of excess of force of outstanding leg of angle
   b) connection of lug angle to angle member should be capable of developing a strength of 20% of excess of force of outstanding leg of angle
   c) lug angles and their connection to gusset should be capable of developing a strength of less than 20% of excess of force of outstanding leg of angle
   d) lug angles and their connection to gusset should be capable of developing a strength of not less than 20% of excess of force of outstanding leg of angle

Answer: d

79. **The effective length of compression flange of simply supported beam not restrained against torsion at ends is**
   a) 1.2 L
   b) 1.0 L
   c) 0.8 L
   d) 0.5 L

Answer: a

80. **Which of the following assumptions were not made while deriving expression for elastic critical moment?**
   a) beam is initially undisturbed and without imperfections
   b) behaviour of beam is elastic
   c) load acts in plane of web only
   d) ends of beam are fixed support

Answer: d

81. **A single angle section 90X60X10 is connected with gusset plate with 7 bolts of 20 mm diameter in one line at pitch of 50 mm and edge distance of 30 mm. What is the design tensile strength of the section for rupture of net section? (Assume the section is connected with longer leg and gauge distance = 50 mm)**

Solution:

Anc = (90 - 10/2 - 22) × 10 = 630 mm2
Ago = (60 - 10/2) × 10 = 550 mm2
An = 630 + 550 = 1180 mm2

The length of outstanding leg will be w = 60 mm and w1 = 50 mm. So the shear lag width, bs = w + w1 – t = 60 + 50 – 10 = 100 mm.

Distance between end bolts , Lc = 6 × 50 = 300 mm.

$$\beta = 1.4 - 0.076\frac{b_s}{L_c} \times \frac{w}{t} \times \frac{f_y}{f_u} = 1.4 - 0.076 \times \frac{100}{300} \times \frac{60}{10} \times \frac{250}{410} = 1.307$$

$$\text{Thus, } T_{dn} = \frac{0.9 f_u A_{nc}}{\gamma_{m1}} + \frac{\beta f_y A_{go}}{\gamma_{mo}} = \frac{0.9 \times 410 \times 630}{1.25} + \frac{1.307 \times 250 \times 550}{1.1}$$

$$= 349.35 \times 10^3 \text{ N} = 349.35 \text{ kN.}$$

82. **A single ISA 75 × 50 × 8 is connected (longer leg) with gusset plate using use 4 bolts of 20 mm diameter in one line at pitch of 50 mm and edge distance of 30 mm. What is the Design tensile strength due to block shear failure? (Assume gauge distance = 35 mm)**

Solution:
Avg = 8 × (3 × 50 + 30) = 1440 mm2
Avn = 8 × (3 × 50 + 30 – 3.5 × 22) = 824 mm2
Atg = 8 × 40 = 320 mm2 [assuming gauge *g* = 35 for 75 mm leg]
Atn = 8 × (40 – 0.5 × 22) = 232 mm2

$$T_{db1} = \frac{0.9A_{vn}f_u}{\sqrt{3}\gamma_{m1}} + \frac{f_y A_{tg}}{\gamma_{m0}} = \frac{0.9\times410\times824}{\sqrt{3}\times1.25} + \frac{250\times320}{1.1} = 213.16 \times 10^3 \text{ N} = 213.16 \text{ kN}$$

$$T_{db2} = \frac{A_{vg}f_y}{\sqrt{3}\gamma_{m0}} + \frac{0.9\, f_u A_{tn}}{\gamma_{m1}} = \frac{1440\times250}{\sqrt{3}\times1.1} + \frac{0.9\times410\times232}{1.25} = 257.44 \times 10^3 \text{ N} = 257.44 \text{ kN}$$

So, $T_{db}$ = 213.16 kN.

83. **An ISA 90 x 90 x 8 used as tension member is connected to a 10 mm gusset plate by fillet weld of size 5 mm. The design strength of the member is 300 kN. Calculate the length of the weld.**

Solution: Force resisted by weld at lower side of angle $P_1 = 300 \times \frac{90-25.1}{90} = 216.33$ kN

Force resisted by weld at upper side of angle $P_2 = 300 \times \frac{25.1}{90} = 83.67$ kN

Assuming size of weld as 5mm, the throat thickness $t_e$ will be 0.707 × 5 = 3.535 mm

Length required at lower side $L_{w1} = \frac{P_1}{\frac{t_e f_u}{\sqrt{3}\gamma_{mw}}} = \frac{216.33\times10^3}{\frac{3.535\times410}{\sqrt{3}\times1.25}} = 323.15$ mm ≈ 324 mm

Length required at upper side $L_{w2} = \frac{P_2}{\frac{t_e f_u}{\sqrt{3}\gamma_{mw}}} = \frac{83.67\times10^3}{\frac{3.535\times410}{\sqrt{3}\times1.25}} = 124.9$ mm ≈ 125 mm

84. What are the various assumptions used in plastic analysis theory?

Answer:

The following are the assumptions are made in plastic design to simplify computations:

1) The material obeys Hooke, Law till the stress reaches fy.

2) The yield stress and modulus of elasticity have the same value in compression and tension.

3) The material is homogeneous and isotropic in both the elastic and plastic states.

4) The material is assumed to be sufficiently ductile to permit large rotation of the section to take place.

5) Plastic hinge rotation is large compare with the elastic deformations so that all the rotations are concentrated at the plastic hinges. The segments between the plastic hinges are rigid.

6) The magnitude of bending moment caused by the external loads will at the most be equal to the plastic moment reached the capacity of the section.

7) The influence of normal and shear forces on plastic moments is not considered.

8) Plane sections remain plane even after bending and the effect of shear is neglected.

9) The equilibrium of forces at the time of collapse is considered for the undeformed state of the structure.

10) No instability occurs in any member of the structure upto collapse.

85. As per IS: 800 what are the various conditions satisfied in order to use plastic method of analysis?

Answer:

IS: 800 stipulates that the following conditions should be satisfied in order to use the plastic method of analysis:

1) The yield stress of steel used should not be greater than 450MPa.

2) The stress-strain characteristic of the steel used should obey the following conditions, in order to ensure plastic moment redistribution. a) The yield plateau (horizontal portion of the stress-strain curve) should be greater than 6 times the yield strain. b) The ratio of the ultimate tensile stress to the yield stress should be more than 1.2. c) The elongation on the standard gauge length should be more than 15%. d) The steel should exhibit strain-hardening capacity.

3) The members shall be hot-rolled or fabricated using hot-rolled plates.

4) The cross section of the members not containing plastic hinges should be ‘compact’ and those of member containing plastic hinges should be ‘plastic’. 5) The cross-section should be symmetrical about its axis perpendicular to the axis of the plastic hinge rotation.

These limitations are intended to ensure that there is a sufficiently long plastic plateau to enabling a hinge to form and that the steel will not experience premature strain hardening.

86. Write down various advantages and disadvantages of plastic design.

Answer:

Advantages of Plastic Design

Plastic design methods offer the following advantages:
1) Realization of uniform and realistic F.O.S for all parts of the structures (in contrast to elastic methods, where the safely factor varies)
2) Simplified analytical procedure and readily of obtaining design moments, since there is no need to satisfy elastic strain compatibility conditions.
3) Saving of material over elastic methods resulting in lighter structures.
4) No effect due to temperature changes, settlement of supports, imperfection, erection method, etc. (because their only effect is to change the amount of rotation required). This is in contrast to the elastic method, where extra calculation are required. However, calculation for instability and elastic deflection required careful considerations in plastic method. The

plastic design method is very popular for design of some structure, e.g, beams and portal frames.
5) Gives some idea of collapse mode and strength of structure.
6) In the elastic method of design, the design process is repeated several times to obtain an optimum solution, where the plastic method of design produces a balanced section in a single attempt.

Dis-advantages of plastic design:

The disadvantages of plastic design method are the following:
1) Obtaining collapse load is difficult if the structure is reasonably complicated.
2) There is little saving in column design.
3) Difficult to design for fatigue.
4) Lateral bracing requirements are more than stringent than elastic design.
5) Calculations for elastic deformations require careful considerations.
6) When more than one loading condition occurs, it is necessary to perform separate calculations, one for each loading condition; the section requiring the largest plastic moment is selected. Unlike the elastic method of design, wherein the moment produced by different loading condition can be added together, the plastic moment obtained by different loading conditions cannot be combined(i.e, the plastic moment calculated for a given set of loads is valid only for that loading condition). This is because the 'principle of super position' becomes invalid when certain parts of the structure have yielded.

87. Write a short note on plastic hinge and hinge length.

Answer:

Plastic Hinge:
A plastic hinge is a zone of yielding due to flexure in a structural member. Although hinges do not actually form, it cal be seen that large changes of slope occurs over small length of the member at position of maximum moments. A strain hardening action usually occurs at these hinges so that large deflections are accompanied by a slight increase in load.

A structure can support the computed ultimate load due to the formation of plastic hinges at certain critical sections. The member remain elastic until the moment reaches a value Mp, the maximum moment of resistance of a fully yielded cross section or fully plastic moment of a section(Mp = fy Zp). Any additional moment will cause the beam to rotate with little increase in stress. The rotation occurs at a constant moment (Mp). The zone acts as if it was hinges except with a constant restraining moment (Mp). The plastic hinge, therefore, can be defined as a yielded zone due to flexure in a structure in which infinite rotation can take place at a constant restraining moment (Mp) of the section. It is represented normally by a black dot. The value of the moment at the adjacent sections of the yield zone for a certain length is more than the yield moment. This length is known as hinge length, depends upon the loading and geometry of the section. To simplify the analysis, this small length is neglected and the plastic hinge is assumed to be formed at discrete points of zero length. But, it cannot be neglected for the calculation of deflections and the design of bracings as the length over which yielding extends is quite important.

The plastic hinges are formed first at the sections subjected to the greatest deformation ( curvature). The possible places for plastic hinges in a structure with prismatic members are points of concentrated loads, at the ends of member meeting at a connection involving a change in geometry and at the point of zero shear in a span under distributed load.

Hinge Length:

Consider a simply supported rectangular beam subjected to a gradually increasing concentrated load P, at the centre. A plastic hinge will be formed at the centre.
Mp = PL/4; My = fy x Ze = fy x bd^2/6 = fy (1/6) x {4x(1/4)} bd^2 = (2/3) x fy xbd^2/4 = (2/3) x fy x Zp = (2/3)Mp, i.e, Mp is 1.5 times more than My.

From the BM diagram, Mp / (L/2) = My / (L/2-x/2) => x=L/3. Therefore, the hinge length of the plasticity zone is equal to 1/3rd of the span.

Similarly, the hinge length of the plasticity zone for a simple beam subjected to uniformly distributed load is L/sqrt(3).

88. Explain in detail shape factor and load factor.

Answer:

Shape Factor (v):

The ratio Mp / My is a property of a cross sectional shape and is independent of the material properties. This ration is known as the shape factor v and is given by v = Mp / My = = fy Zp / fy Ze.= Zp / Ze For wide-flange I-section in flexure about the strong axis, the shape factor ranges from 1.09 to about 1.18 with the average value being 1.14. One may conservatively take the plastic moment strength Mp of I-section bent about their strong axis to be at least 15% greater than the strength My. On the other hand, the shape factor for I-section bent about their minor axis is about the same as for a rectangular section, i.e, about 1.5.

Load Factor:

Load factor is defined as the ratio of the collapse load to the working load(service load) and is represented by F, i.e, F = Pc / Pw
If a collection of beams having different end conditions (free or fixed) and the working load W were first design elastically and then plastically, the ratio Pc / Pw will not be identical. Only the beams that are simply supported will produce a constant ratio of Pc / Pw and for these cases the values of Pc / Pw will be the lowest.

From a practical point of view, a minimum acceptable and constant load factor is required, and that found for a simply supported beam may be regarded satisfactory. For a simple beam the variation of the bending moment with the load is linear.

In actual practice a load factor varying from 1.7 to 2.0 is assumed depending upon the designer's judgment. When the structures are subjected to wind the corresponding load factor for plastic design is reduced by 25%. The prime function of the load factor is to ensure that the structure will be safe under the collapse load. Therefore, it may be regarded as a factor of safety based upon the collapse load. It depends upon the nature of loading, the support conditions, and the geometrical shape of the structural members. Uncertainty of the loads, imperfection in workmanship and error in fabrication are some of the other factors which influence the choice of the load factor.

89. Explain various mechanism of plastic analysis.

Answer:
When a structure is subjected to a system of loads, it is stable and hence functional until a sufficient number of plastic hinges have been formed to render the structure unstable. As soon as the structure reaches an unstable condition, it is considered to have been failed. The segments of the beams between the plastic hinges are able to move without an increase of load. This condition in a member is called mechanism. The concept of mechanism formation in a structure due to loading beyond the elastic limit and of virtual work is used in the plastic analysis and design of steel structures. If an indeterminate structure has the redundancy r, the insertion of r plastic hinges makes it statically determinate. Any further hinge converts this statically determinate structure into mechanism. Hence, for collapse, the numbers of plastic hinges required are (r+1).

Types of Mechanism: Various possible mechanism are listed below:
a) Beam mechanism
b) Panel / sway mechanism
c) Joint mechanism
d) Gable mechanism
e) Composite (combined) mechanism

Number of Independent Mechanism: Let,

N = number of possible plastic hinges
r = number of redundancies
n = possible independent mechanism.

Then, n = N – r

After finding out the number of independent mechanism all the possible combinations are made in such a way so as to make the external works maximum or the internal work a minimum. This is done to obtain the lowest load.

90. What are the different theorems of plastic analysis? Explain

Answer:
The plastic analysis of a structure is govern by three theorem, which are as follows:
1) The static or lower bound theorem: states that a load ($P<P_c$)computed on the basis of an assumed equilibrium moment diagram, in which the moments(M) are nowhere greater than the plastic moment($M<M_p$), is less than, at the best equal to, the correct collapse load. Hence the static method represents the lower limit to the true ultimate load and has a maximum factor of safety. The static theorem satisfies the equilibrium and yield conditions.

2) Kinematic or Upper bound theorem: states that a load computed on the basis of an assumed mechanism will always be greater than or at the best equal to , the true collapse load ($P>P_c$). Hence the kinematic method represents an upper limit to the true ultimate load and has a smaller factor of safety. The kinematic theorem satisfies the equilibrium and continuity conditions.

3) Uniqueness theorem: The lower and upper bound theorems can be combined to produce the uniqueness theorem, which states that the load that satisfies both the theorems at the same time is the correct collapse load. When both the theorems are satisfied in a given problem then the solution is said to be the correct (unique) one.

Using the principle of virtual work and the upper and lower bound theorems, a structure can be analysed for its ultimate load by any of the following methods:

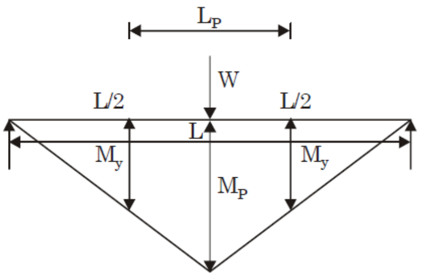
1) Static method: This consists of selecting the redundant forces, The free and redundant bending moment diagram is drawn for the structure. A combined bending moment diagram is drawn in such a way that a mechanism is formed. The collapse load is found by working out the equilibrium equation. It is checked that the bending moment is not more than the fully plastic moment at any section.

2) Kinematic method: This consists of locating the possible places of plastic hinges. The possible independent and combined mechanism are ascertained. The collapse load is found by applying the principle of virtual work. A bending moment diagram corresponding to the collapse mechanism is drawn and it is checked that the bending moment is not more than the fully plastic moment at any section.

For complicated frames, the static method of analysis is more difficult, and finding the correct equilibrium equation becomes illusive. In these cases, the kinematic method is more practical.

91. **A simply supported beam of span L supports a concentrated load W at its midspan. If the cross-section of the beam is circular, then the length of elastic-plastic zone of the plastic hinge will be**

Solution:




$$\frac{M_P}{M_y} = \frac{\frac{L}{2}}{\frac{L-L_P}{2}}$$

$$\frac{Mp}{My} = \frac{1}{1-\frac{L_P}{L}}$$

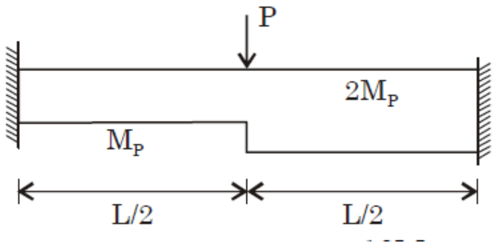
$$1-\frac{Lp}{L} = \frac{1}{\frac{M_P}{M_y}}$$

$$L_P = L\left[1-\frac{1}{\frac{M_P}{M_y}}\right]$$

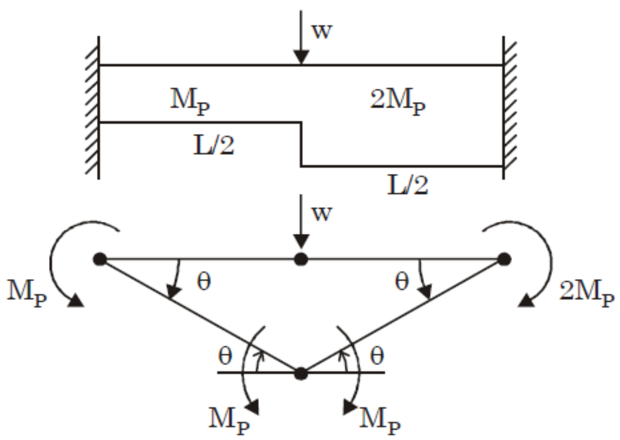
$$= L\left[1-\frac{1}{\text{Shape factor}}\right]$$

$$= L\left[1-\frac{1}{1.7}\right] = \frac{7L}{17}$$

92. **A fixed beam made of steel is shown in the figure below. At collapse, the value of load P will be equal to**




Solution:




$$5M_P\theta = \frac{WL}{2}\theta$$

$$\Rightarrow \qquad W = \frac{10M_P}{l}$$

93. **Explain various types of bolt briefly.**

Answer:

<u>Types of bolts</u>
There are several types of bolts used to connect structural members. Some of them are listed below

(a) Black bolts or unfinished bolts

• Black bolts are referred to as ordinary, rough or common bolts. They are least expensive bolts and are made of low carbon steels (mild steel) with square or hexagonal head. The diameter of the hole is about 1.0 to 2.0 mm larger than the bolt diameter for ease in fitting. They are designated as Md× l, ‘d’ – shank diameter of bolt and , l – length of the bolt.

They are primarily used in light structures under static loads such as small trusses, purlins, bracings. They are also used as temporary fasteners during erection where HSFG bolts or welding are used as permanent fasteners.

• These bolts are not recommended for connections, which are subjected to impact load, vibration and fatigue.

• For bolt of a grade or property class 4.6 represents the ultimate tensile strength is 400 N/mm2 and yield strength is 0.6 times 400 which is 240N/mm2.

• Ordinary bolted joints, the force transfer through interlocking and bearing of bolts and joint is called bearing type joint.

(b) High Strength Friction Grip (HSFG) bolts

• High strength friction grip bolts are made from bars of medium carbon heat treated steel (high tensile steel). The bolt property class 10.9S and 12.9S are commonly used in steel connections.

• The HSFG bolts are available in sizes from 16mm to 36mm and are designated as M16, M20, M24 and M30.

• These bolts tightened (by torque wrenches) until they have very high tensile stresses, so that connected parts are clamped tightly together between the bolt head and nut, this permits load to be transferred primarily by friction not by shear.

• These bolts are most suitable for bridges where the stress reversal may occur or slippage is undesirable also for seismic loading and for fatigue load.

• High strength bolts have replaced rivets and black bolts are being used in structures, high raised building, bridges etc.

94. **Explain various steps involved in the design of laterally unsupported beam.**

Answer:

Steps involved are:

**STEP 1: FIND OUT ULTIMATE LOAD ON BEAM.**
Factored Ultimate Load (Factored Load) w = 1.5 × Working Load

**STEP 2: FIND OUT MAXIMUM BENDING MOMENT (M) AND SHEAR FORCE (V) ON BEAM.**

**STEP 3: CALCULATE PLASTIC SECTION MODULUS REQURIED FOR TRIAL SECTION.**

$$Z_{P(required)} = 1.3\frac{M\gamma_{mO}}{f_y}$$

**STEP 4: SELECT SUITABLE SECTION BASED ON $Z_p$ FROM IS: 800: 2007, PAGE NO. 138, 139. WRITE DOWN SE TIONAL PROPERTIES.**

**STEP 5: SECTION CLASSIFICATION.**
a. Find out value of $b/t_f$ and $d/t_w$. (refer Figure. 2, Page no. 19, IS 800: 2007 to find b and d)
$t_f$ = thickness of flange $t_w$ = thickness of web.
b. Refer Table 2, Page no. 18, IS 800: 2007 and classify the section semi-compact, compact, plastic or slender.

**STEP 6: CHECK FOR SHEAR.** (Clause no. 8.4.1., Page no. 59, IS 800: 2007)
**a. Find out design shear strength $V_d$.**

$$V_d = \frac{f_y}{\sqrt{3}\gamma_{mo}} h t_w$$

**b. Beam is checked for high / low shear case**
$V \le 0.6\ V_d$ low shear case
$V > 0.6\ V_d$ high shear case

**STEP 7: CHECK FOR BENDING.**

**a. For low shear Case** (Clause no. 8.2.2, Page no. 54, IS 800: 2007)

$M_d > M$

$M_d$ = Design Bending Strength and M = Bending Moment

$$M_d = \beta_b Z_p f_{bd}$$

$$f_{bd} = \frac{\chi_{LT} f_y}{\gamma_{mo}}$$

$$\chi_{LT} = \frac{1}{\varphi_{LT} + (\varphi^2{}_{LT} - \lambda^2{}_{LT})^{0.5}}$$

$\beta_b$ = 1 for plastic and compact sections.

= $Z_e/Z_p$ for semi compact sections.

$Z_e$ = Elastic section Modulus

$Z_p$ = Plastic section Modulus

$$\lambda_{LT} = \sqrt{\frac{\beta_b Z_p f_y}{M_{cr}}}$$

$$M_{cr} = \sqrt{\frac{\pi^2 EI}{L_{LT}{}^2}\left(GI_t + \frac{\pi^2 EI_w}{L_{LT}{}^2}\right)}$$

Where,

$$I_w = (1 - B_f)B_f I_y h_y{}^2$$

$$I_t = \Sigma \frac{b_i t_i{}^3}{3}$$

$$G = \frac{E}{2(1+\mu)}$$

$I_w$ = warping constant (page no. 129 , IS 800)

$B_f = \frac{I_{fc}}{I_{fc}+I_{ft}} = 0.5$ (for symmetrical section $I_{fc} = I_{ft}$).

$h_y$ = distance between shear center of flanges.

$L_{LT}$ =eff. len. for lateral torsional buckling (table15,pg58)

$\phi = 0.5[1+\alpha_{LT}(\lambda_{LT} - 0.2)+ \lambda^2{}_{LT}]$

$\alpha_{LT}$ = 0.21 for rolled section.

**b. For High shear Case** (Clause no. 8.2.1.3, Page no. 53, IS 800: 2007)

Refer Clause no. 8.2.1.3, Page no. 53, IS 800: 2007. Generally low shear case is preferred.

**STEP 8: CHECK FOR WEB BUCKLING AT SUPPORT** (Clause no. 8.7.3.1, Page no. 67, IS 800: 2007)

a. Capacity of section = $A_b f_{cd} > V$

b. $A_b = (b_1 + n_1)\, t_w$ when load is at support

$A_b = (b_1 + 2n_1)\, t_w$ when load is not at support

Where, $b_1$= stiff bearing length of load = assume between 0 to 100mm

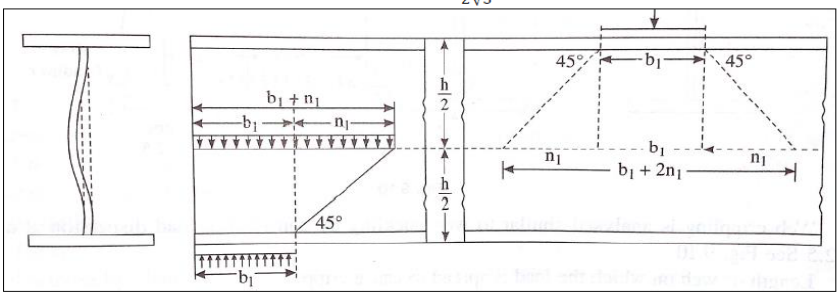
$n_1$ = for 45˚ dispersion consider h/2

d. Find out $F_{cd}$ = Design Compressive Stress considering class c and $f_y$ = 250 MPa.

Slenderness ratio $= \frac{kl}{r} = \frac{0.7d}{r}$

D = depth of the web between the flanges

r = least radius of gyration of the section = $\frac{t_w}{2\sqrt{3}}$



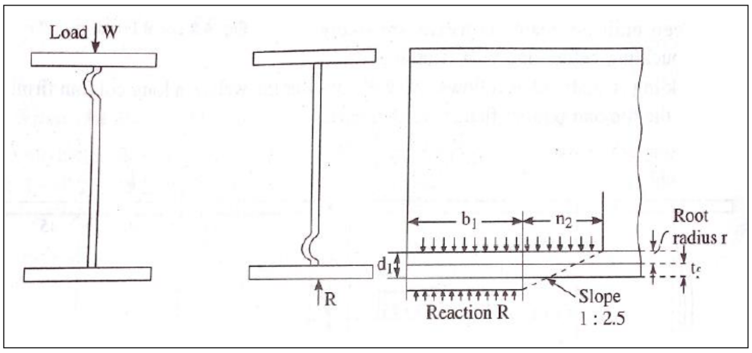
**STEP 9: CHECK FOR WEB CRIPPLING** (Clause no. 8.7.4, Page no. 67, IS 800: 2007)

Design crippling strength $F_w = \frac{(b_1+n_2)t_w f_{yw}}{\gamma_{mo}} > V$

Where, $b_1$ = stiff bearing length = 0 to 100 mm

$n_2 = 2.5\ (t_f + r_1)$

$f_{yw}$ = yield stress of web




**STEP 10: CHECK FOR DEFLECTION**

a. Actual deflection for simply supported

$$\delta_{max} = \frac{5}{384}\frac{wl^4}{EI}$$

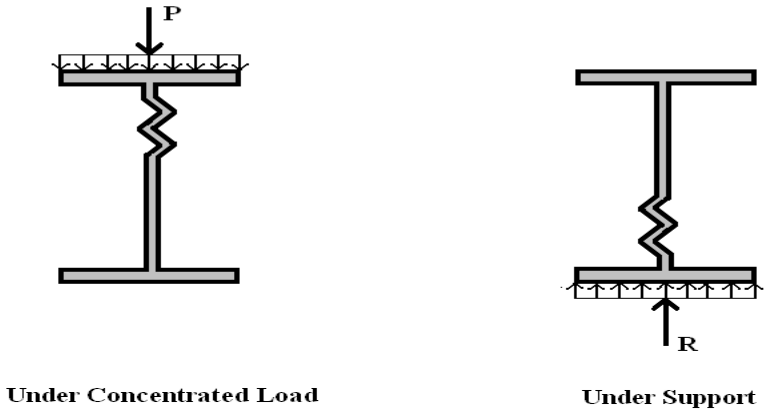
b. Permissible deflection = Span/300 (table 6, Page no. 31, IS 800: 2007)

95. **Explain web crippling and web buckling with the help of sketch.**

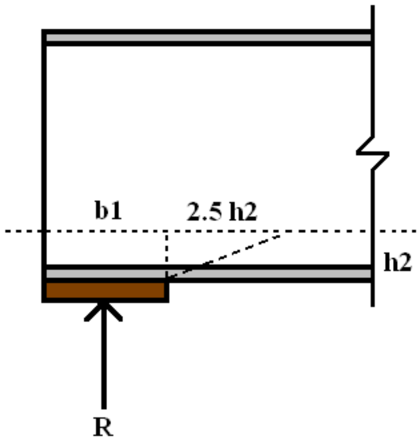
Answer:

<u>Web crippling:</u>

Web crippling causes local crushing failure of web due to large bearing stresses under reactions at supports or concentrated loads. This occurs due to stress concentration because of the bottle neck condition at the junction between flanges and web. It is due to the large localized bearing stress caused by the transfer of compression from relatively wide flange to narrow and thin web. Web crippling is the crushing failure of the metal at the junction of flange and web. Web crippling causes local buckling of web at the junction of web and flange.




WEB CRIPPLING OF BEAMS




Web crippling calculations

For safety against web crippling, the resisting force shall be greater than the reaction or the concentrated load. It will be assumed that the reaction or concentrated load is dispersed into the web with a slope of 1 in 2.5 as shown in the figure

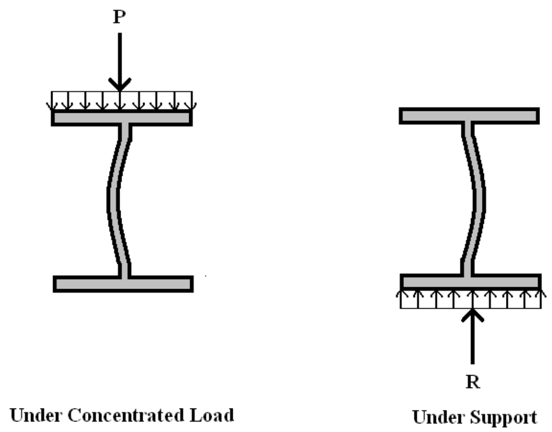
Let Resisting force = Fwc
Thickness of web = tw
Yield stress in web = fyw
Width of bearing plate = b1
Width of dispersion = n2 = 2.5 h2
Depth of fillet = h2 (from SP [6])
Fwc = [(b1 + n2) tw fyw ] / $y_{mo}$ ≥ Reaction, $R_u$

For concentrated loads, the dispersion is on both sides and the resisting force can be expressed as

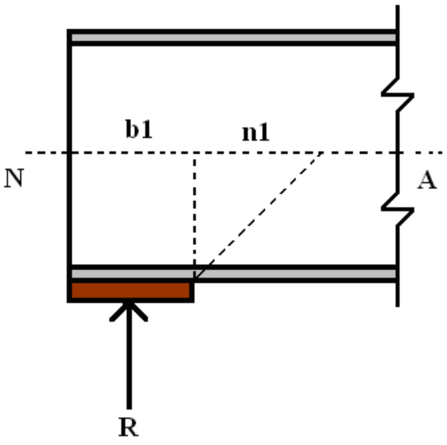
Fwc = [(b1 + 2 n2) tw fyw ] / $y_{mo}$ ≥Concentrated load, $W_u$

**Web Buckling:**

The web of the beam is thin and can buckle under reactions and concentrated loads with the web behaving like a short column fixed at the flanges. The unsupported length between the fillet lines for I sections and the vertical distance between the flanges or flange angles in built up sections can buckle due to reactions or concentrated loads. This is called web buckling.




WEB BUCKLING OF BEAMS




**Web Buckling Calculations**

For safety against web buckling, the resisting force shall be greater than the reaction or the concentrated load. It will be assumed that the reaction or concentrated load is dispersed into the web at 45° as shown in the figure.

Let Resisting force = Fwb
Thickness of web = tw
Design compressive stress in web = fcd
Width of bearing plate = b1
Width of dispersion = n1
Fwb = (b1 + n1) tw fcd ≥ Reaction, $R_u$

For concentrated loads, the dispersion is on both sides and the resisting force can be expressed as

Fwc = [(b1 + 2 n1) tw fcd ] ≥Concentrated load, $W_u$

The design compressive stress fcd is calculated based on a effective slenderness ratio of 0.7 d / $r_y$, where d = clear depth of web between the flanges.
$r_y$ = radius of gyration about y-y axis and is expressed as
= sqrt (Iyy / area) = sqrt [$(t_w)^3$ / 12/ t] = sqrt [$(t_w)^2$ / 12]

kl / $r_y$ = (0.7 d) / sqrt [$(t_w)^2$ / 12] = 2.425 * d / tw

96. **Explain various steps involved in the design of laterally supported beam as per IS 800 (2007).**

Answer:

Various steps involved are:

1. Calculate the factored load and the maximum bending moment and shear force
2. Obtain the plastic section modulus required

$$Z_{req} = \frac{(M \times \gamma_{mo})}{fy}$$

Select a suitable section for the beam-ISLB, ISMB, ISWB or suitable built up sections (doubly symmetric only). (Doubly symmetric, singly symmetric and asymmetric- procedures are different)
3. Check for section classification such as plastic, compact, semi-compact or slender. Most of the sections are either plastic or compact. Flange and web criteria.

$$\frac{d}{t_w}\ ,\ \frac{b}{t_f},\ \varepsilon = \sqrt{\frac{250}{fy}} = 1$$

4. Calculate the design shear for the web and is given by

$$V_{dp} = \frac{(Av \times fy)}{\sqrt{3} \times \gamma_{mo}} > V_d \ and, V < 0.6V_d$$

5. Calculate the design bending moment or moment resisted by the section (for plastic and compact)

$M_d = \beta_p \times Z_p \times f_y / \gamma_{mo}$

6. Check for buckling
7. Check for crippling or bearing
8. Check for deflection

97. **At the location of plastic hinge**
(a) Radius of curvature is infinite
(b) Curvature is infinite
(c) Moment is infinite
(d) Flexural stress is infinite

Answer: b

98. **A ductile structure is defined as one for which the plastic deformation before fracture**
(a) is smaller than the elastic deformation
(b) vanishes
(c) is equal to the elastic deformation
(d) is much larger than the elastic deformation

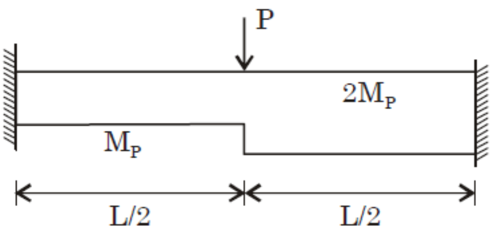
Answer: d

99. **Assertion (A):** The shape factor of a circular section is less than that of a rectangular section.
**Reason (R):** Compared to rectangular section, a circular section has more area near the neutral axis than at the extreme fibre.
Of these statements:
(a) both A and R are true and R is the correct explanation of A
(b) both A and R are true but R is not a correct explantion of A

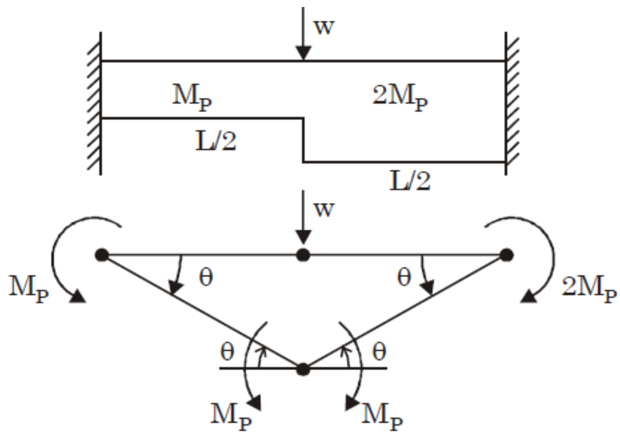
(c) A is true but R is false
(d) A is false but R is true

Answer: d

100. A fixed beam made of steel is shown in the figure below. At collapse, the value of load P will be equal to




Solution:




$$5M_p\theta = \frac{WL}{2}\theta$$

$$\Rightarrow \qquad W = \frac{10M_p}{l}$$