Unit	**1. INTRODUCTION TO STRUCTURAL STEEL AND DESIGN OF CONNECTIONS**	**Lecture No**	**01**
Topic	Steel: General Introduction, Types and Properties		
Learning Outcome (LO) At the end of this lecture, students will be able to		**Bloom's Knowledge Level**	
LO1	Define steel	K1	
LO2	Describe the types of steel available in the market	K1	
LO3	State the various strength and serviceability properties of steel members	K1	

In civil engineering, steel refers to a versatile and widely used construction material known for its strength, durability, and ductility. Steel is primarily an alloy of iron and carbon, with other elements such as manganese, silicon, and sometimes small amounts of other metals added to enhance its properties.

Types of Steel:

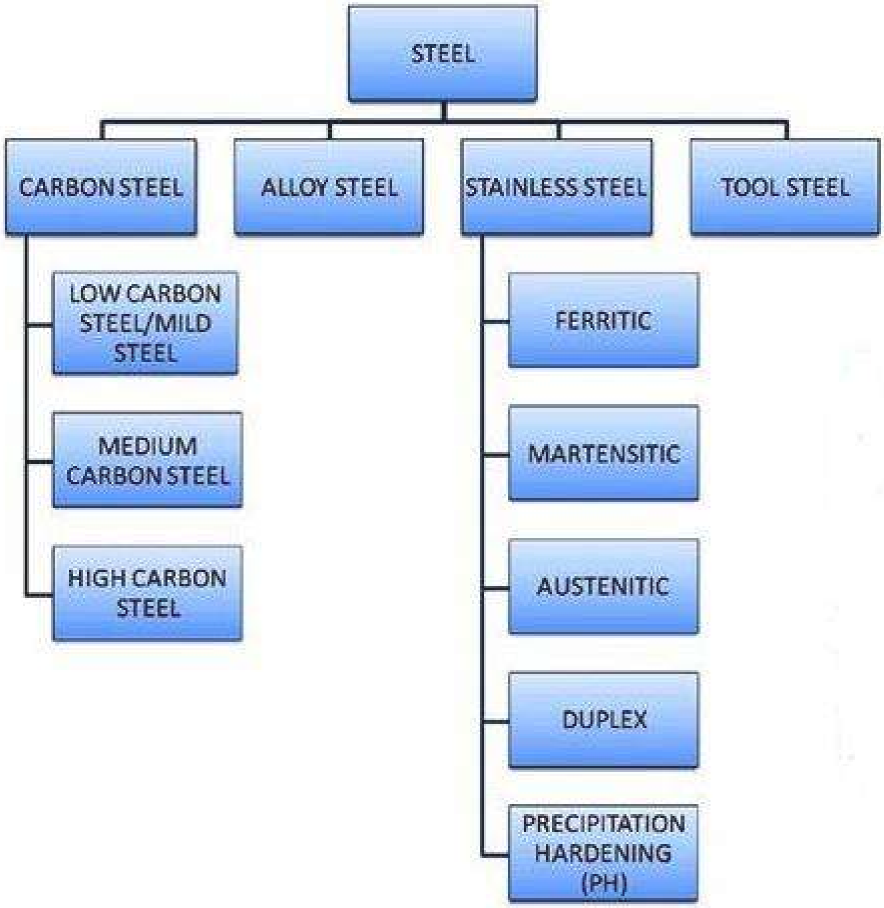
Structural steel: Used in the construction of buildings, bridges, and other large structures due to its high strength-to-weight ratio. Structural steel members such as beams, columns, and trusses provide the framework for buildings and bridges, supporting loads and resisting forces such as gravity, wind, and seismic loads.

Reinforcing steel (rebar): Used to reinforce concrete structures such as foundations, columns, slabs, and walls. Reinforcing steel bars are embedded within concrete to increase its tensile strength and prevent cracking under tensile loads.

Steel pipes and tubes: Utilized for various purposes in civil engineering, including the conveyance of fluids (such as water, gas, and sewage), as well as structural applications like piling and underground utility installations.

Steel plates and sheets: Employed in the construction of tanks, vessels, industrial equipment, and structural components requiring flat or curved surfaces.

Pre-engineered steel buildings: Prefabricated steel structures designed and manufactured off-site, then assembled on-site, offering cost-effective and efficient solutions for various building types, including warehouses, factories, and agricultural buildings.



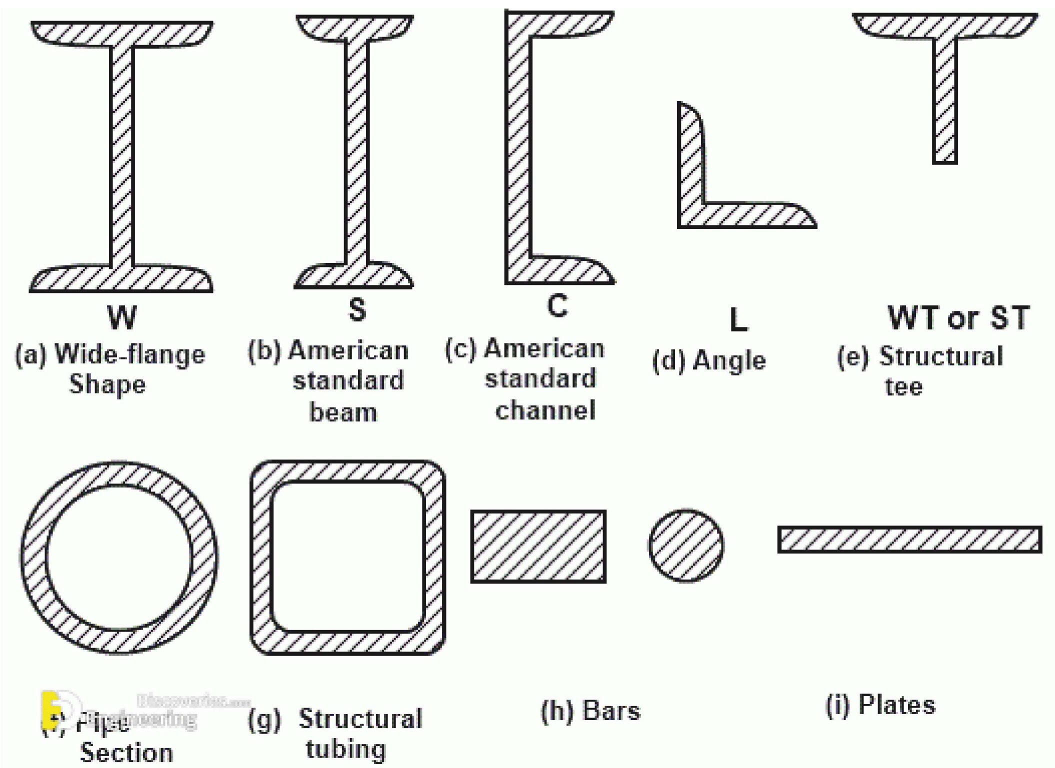


Fig., Various types of steel members

Codal Norms for types of steel:

IS 808: 1989 - Dimensions for Hot Rolled Steel Beam, Column, Channel, and Angle Sections: This standard specifies the dimensions of various hot-rolled structural steel sections such as beams (I-sections, H-sections), columns (UC-sections, parallel flange channels), angles, and tees.

IS 12778: 2004 - Hot Rolled Parallel Flange Steel Sections for Beams, Columns, and Bearing Piles - Dimensions and Sectional Properties: This standard specifies the dimensions and sectional properties of hot-rolled parallel flange steel sections used for beams, columns, and bearing piles.

IS 811: 1987 - Cold Formed Light Gauge Structural Steel Sections: This standard covers the dimensions and sectional properties of cold-formed light gauge structural steel sections such as Z-sections, C-sections, and lipped channels, used in lightweight construction.

IS 2062: 2011 - Hot Rolled Medium and High Tensile Structural Steel: This standard specifies the requirements for hot-rolled medium and high tensile structural steel, including chemical composition, mechanical properties, and dimensions of various grades of steel plates, sheets, and wide flats used in structural applications.

IS 1148: 1981 - Dimensions for Steel Plates, Sheets, Strips, and Flats for General Engineering Purposes: This standard specifies the dimensions for steel plates, sheets, strips, and flats used in general engineering purposes, including construction, manufacturing, and fabrication.

IS 1852: 1985 - Rolling and Cutting Tolerances for Hot Rolled Steel Products: This standard specifies the permissible variations in dimensions and mass for hot-rolled steel products, including beams, channels, angles, and flats.

Properties of Steel members:

Steel members used in civil engineering applications possess various strength and serviceability properties that are crucial for ensuring structural integrity and performance.

1. **Strength Properties**:

    - **Yield Strength**: The stress at which steel begins to deform plastically, exhibiting permanent deformation. It is a critical parameter for determining the limit of elastic behavior.

    - **Ultimate Tensile Strength (UTS)**: The maximum stress a material can withstand before failure occurs under tension. It indicates the maximum load-carrying capacity of the steel member.

    - **Modulus of Elasticity (Young's Modulus)**: The measure of a material's stiffness, representing the ratio of stress to strain within the proportional limit. It influences the deflection behavior of steel members under loading.

2. **Ductility Properties**:

    - **Elongation**: The percentage increase in gauge length of a tensile specimen before rupture occurs. It reflects the material's ability to deform before failure and is an important indicator of ductility.

- **Reduction in Area**: The percentage reduction in cross-sectional area of a tensile specimen at the point of rupture. It provides additional information about the material's ductility.

Mechanical properties	Material	ASTM A533	AISI 1020	AISI 4340
Yield strength MPa		670	295	731
Ultimate strength MPa		720	395	855
Modulus of Elasticity GPa		205	200	205
Hardness, Brinell		230	111	255
Elongation at Break		28	36.5	21.7

3. **Serviceability Properties**:

- **Deflection**: The degree of bending or deformation experienced by a steel member under load. Excessive deflection can lead to serviceability issues such as excessive vibrations, discomfort, or functional problems in a structure.

- **Stability**: The ability of a steel member or structural system to maintain its equilibrium and resist buckling under compressive loads. Stability considerations are crucial for ensuring the safety and performance of steel structures.

- **Fatigue Resistance**: The ability of steel members to withstand repeated loading and unloading cycles over time without experiencing failure. Fatigue resistance is essential for structures subjected to fluctuating or cyclic loads, such as bridges and crane structures.

4. **Corrosion Resistance**:

- Steel members may be protected against corrosion through various methods such as coatings, galvanization, or the use of corrosion-resistant alloys. Corrosion resistance ensures the long-term durability and service life of steel structures, especially in aggressive environments.

Assessment questions to the lecture

Qn No	Question	Answer	Bloom's Knowledge Level
1	Which IS code specifies the dimensions of various hot-rolled structural steel sections such as beams, columns, channels, and angles? a) IS 12778: 2004 b) IS 808: 1989 c) IS 2062: 2011 d) IS 1148: 1981	b	K1
2	What does the term "yield strength" refer to in the context of steel members? a) The maximum stress a material can withstand before	b	K1

	failure occurs under tension b) The stress at which steel begins to deform plastically, exhibiting permanent deformation c) The measure of a material's stiffness, representing the ratio of stress to strain within the proportional limit d) The ability of a steel member to maintain its equilibrium and resist buckling under compressive loads		
3	Which property of steel members is crucial for ensuring resistance to repeated loading and unloading cycles over time without experiencing failure? a) Ultimate Tensile Strength (UTS) b) Elongation c) Modulus of Elasticity (Young's Modulus) d) Fatigue Resistance	d	K1

Students must prepare answers for the following questions at the end of the lecture.

Qn No	Question	Marks	CO	Bloom's Knowledge Level
1	Define Steel. State its applications in civil engineering	7	1	K1
2	Describe the various types of steel available in the Indian Market	8	1	K1
3	Explain in detail the various strength and serviceability properties of steel members.	8	1	K1

Reference Book:

- N Subramaniam , Design of Steel Structures, Oxford University press, New Delhi, 2011

Unit	1. INTRODUCTION TO STRUCTURAL STEEL AND DESIGN OF CONNECTIONS	Lecture No	02
Topic	Indian Standard Rolled Sections		
Learning Outcome (LO) At the end of this lecture, students will be able to		**Bloom's Knowledge Level**	
LO1	List the various types of I beams as per BIS	K2	
LO2	List the various types of I beams as per BIS	K1	
LO3	List the various types of Angles as per BIS	K2	
LO4	List the various types of Cold Formed Sections s as per BIS	K2	

In the Indian Standards (IS) code system, hot rolled sections and cold formed sections refer to two different methods of manufacturing structural steel sections, each with its own set of specifications and applications.

1. **Hot Rolled Sections:**

    - Hot rolled sections are manufactured through a process in which steel billets or ingots are heated above their recrystallization temperature and then passed through a series of rollers to achieve the desired shape and dimensions.
    - This process results in sections with a characteristic scale on their surface due to the heating process. Hot rolled sections typically have larger dimensions and are more suitable for heavy-duty structural applications.
    - Examples of hot rolled sections include I-beams (ISMB, ISHB, ISWB), channels (ISMC, ISJC), angles (ISA), and flats (ISFL).
    - Hot rolled sections are covered under various IS codes such as IS 808:1989, "Dimensions for Hot Rolled Steel Beam, Column, Channel, and Angle **Sections."**

2. **Cold Formed Sections:**

    - Cold formed sections are manufactured by shaping steel coils or sheets at room temperature through a process of bending or rolling. This process results in sections with precise dimensions and a smooth surface finish.
    - Cold formed sections are typically produced using thin gauge steel and are characterized by their uniformity, accuracy, and high strength-to-weight ratio. They are suitable for lightweight structural applications.
    - Examples of cold formed sections include Z-sections (ISZ), C-sections (ISMC), lipped channels (ISLC), and cold formed angles (ISCA).
    - Cold formed sections are covered under various IS codes such as IS 811:1987, "Cold Formed Light Gauge Structural Steel Sections."

Types of I Beams:

The various types of beams specified in the Indian Standards (IS) code system:

1. **ISJB (Indian Standard Junior Beams)**:

- ISJB sections are junior I-beams with relatively small dimensions and lighter weight compared to other types of beams.
- These beams are designated with a prefix "ISJB" followed by the nominal depth of the section in millimeters. For example, ISJB 100, ISJB 125, ISJB 150, etc.
- ISJB sections are commonly used in construction for lighter structural applications such as small building frames, roof trusses, and lightweight supports.

2. **ISLB (Indian Standard Light Weight Beams)**:

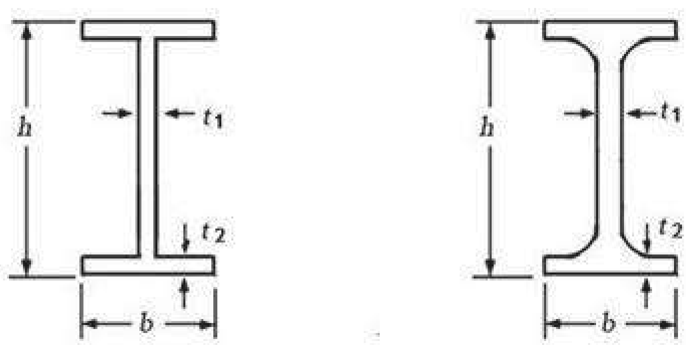
- ISLB sections are light weight I-beams with symmetrical cross-sections, featuring relatively narrow flanges and a vertical web.
- These beams are designated with a prefix "ISLB" followed by the nominal depth of the section in millimeters. For example, ISLB 100, ISLB 125, ISLB 150, etc.
- ISLB sections are commonly used in construction for lighter structural applications such as roof trusses, purlins, and small building frames.

3. **ISMB (Indian Standard Medium Weight Beams)**:

- ISMB sections are medium weight I-beams with symmetrical cross-sections, consisting of relatively narrow flanges connected by a vertical web.
- These beams are designated with a prefix "ISMB" followed by the nominal depth of the section in millimeters. For example, ISMB 200, ISMB 250, ISMB 300, etc.
- ISMB sections are widely used in construction for carrying bending loads in building frames, bridges, and other structural applications.

4. **ISHB (Indian Standard Heavy Weight Beams)**:

- ISHB sections are heavy weight I-beams with symmetrical cross-sections, featuring wider flanges and a thicker web compared to ISMBs.
- These beams are designated with a prefix "ISHB" followed by the nominal depth of the section in millimeters. For example, ISHB 450, ISHB 500, ISHB 600, etc.
- ISHB sections are suitable for applications requiring higher load-bearing capacity and stiffness, such as heavy-duty industrial structures, bridges, and large building frames.



5. **ISWB (Indian Standard Wide Flange Beams)**:

    - ISWB sections are wide flange I-beams with symmetrical cross-sections, featuring wider flanges compared to ISMBs and ISHBs.

    - These beams are designated with a prefix "ISWB" followed by the nominal depth of the section in millimeters. For example, ISWB 400, ISWB 450, ISWB 500, etc.

    - ISWB sections are suitable for applications requiring higher load-bearing capacity and stiffness, such as heavy-duty industrial structures, bridges, and large building frames.

These beams are designed and manufactured in accordance with the relevant Indian Standards to ensure structural integrity, safety, and performance in construction projects. Engineers and designers select the appropriate type and size of beam based on structural requirements, load calculations, and design specifications.

Types of Channels:

1. **ISJC (Indian Standard Junior Channel)**:

    - ISJC sections are junior channel sections with a C-shaped cross-section and parallel flanges.

    - These sections are designated with a prefix "ISJC" followed by the nominal depth of the section in millimeters. For example, ISJC 75, ISJC 100, ISJC 125, etc.

    - ISJC sections are commonly used for light structural applications such as framing for windows, doors, partitions, and lightweight roof trusses.

2. **ISLC (Indian Standard Light Channel)**:

    - ISLC sections are light channel sections with a C-shaped cross-section and parallel flanges.

    - These sections are designated with a prefix "ISLC" followed by the nominal depth of the section in millimeters. For example, ISLC 75, ISLC 100, ISLC 125, etc.

    - ISLC sections are typically used for light structural framing, including supporting secondary loads, bracing, and small-scale construction projects.

3. **ISMC (Indian Standard Medium Channel)**:

    - ISMC sections are medium channel sections with a C-shaped cross-section and parallel flanges.

- These sections are designated with a prefix "ISMC" followed by the nominal depth of the section in millimeters. For example, ISMC 75, ISMC 100, ISMC 125, etc.
- ISMC sections find applications in structural framing, such as columns, beams, and purlins, where moderate load-bearing capacity and stiffness are required.

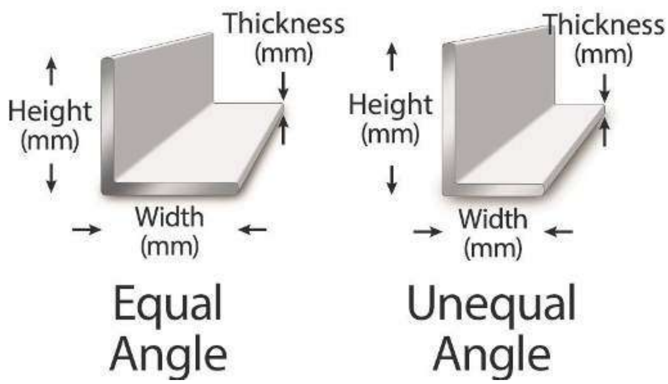
4. **ISSC (Indian Standard Special Channel)**:

- ISSC sections are special channel sections with a C-shaped cross-section and varying dimensions, typically designed for specific applications.
- These sections are designated with a prefix "ISSC" followed by the nominal depth of the section in millimeters. For example, ISSC 100, ISSC 150, ISSC 200, etc.
- ISSC sections are used in specialized structural configurations, such as crane runways, conveyor systems, and equipment supports, where standard channel sections may not be suitable.

Types of Angles:

1. **Equal Angles (ISA)**:

- Equal angles, also known as ISA sections, have legs of equal lengths, forming a right angle.
- These angles are designated with a prefix "ISA" followed by the dimensions representing the length of each leg in millimeters. For example, ISA 50 x 50, ISA 75 x 75, ISA 100 x 100, etc.
- IS 808 provides standard dimensions and sectional properties for equal angles, ensuring uniformity and compatibility in construction applications.
- Equal angles are commonly used for framing, bracing, supports, and connections in various structural systems.



2. **Unequal Angles (ISA)**:

- Unequal angles, also specified under ISA sections, have legs of unequal lengths, forming a non-right angle.
- These angles are designated with a prefix "ISA" followed by the dimensions representing the lengths of both legs in millimeters, as well as the thickness of the angle. For example, ISA 75 x 50 x 6, ISA 100 x 75 x 8, ISA 125 x 75 x 10, etc.

- IS 808 provides standard dimensions and sectional properties for unequal angles, ensuring consistency and reliability in construction applications.
- Unequal angles are used in situations where one leg needs to bear a greater load than the other or where specific structural configurations require varying dimensions.

Types of Cold Formed Sections:

1. **Z-Sections**:
   - Z-sections have a profile resembling the letter "Z" and are formed by bending thin gauge steel sheets or strips.
   - These sections are used for various applications in lightweight construction, including purlins, framing members, and secondary structural components.
2. **C-Sections**:
   - C-sections have a profile resembling the letter "C" and are formed by bending thin gauge steel sheets or strips with outward-facing flanges.
   - These sections are commonly used for framing, studs, joists, and other structural elements in lightweight construction.
3. **Lipped Channels**:
   - Lipped channels, also known as lip channels or hat channels, have a C-shaped cross-section with one or more lips or flanges extending outward from one or both sides.
   - These sections are widely used for framing, supports, and bracing in lightweight construction, particularly in roof and wall systems.
4. **Sigma Sections**:
   - Sigma sections have a profile resembling the Greek letter "Σ" and are formed by cold-forming thin gauge steel sheets or strips.
   - These sections are used in various structural applications, including framing, trusses, and secondary members in lightweight construction.
5. **Angles**:
   - Cold-formed angles have an L-shaped cross-section and are formed by bending thin gauge steel sheets or strips.
   - These sections are used for various structural purposes, including bracing, supports, and connections in lightweight construction.

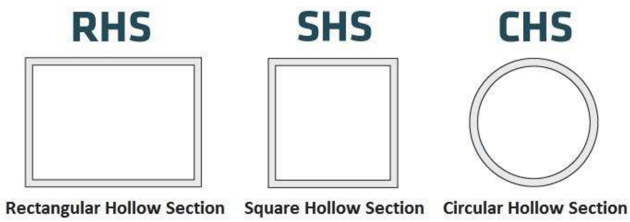
Types of Hollow Sections:

1. **Rectangular Hollow Sections (RHS)**:
   - RHS are rectangular-shaped hollow structural sections with uniform wall thickness along their length.

- These sections are specified under IS 4923:1997, "Hollow Steel Sections for Structural Use - Specification."
- IS 4923 provides requirements for the dimensions, sectional properties, tolerances, and material properties of RHS, ensuring consistency and reliability in construction applications.
- RHS are commonly used in construction for structural framing, columns, beams, trusses, and other load-bearing elements where rectangular cross-sections are required.

2. **Square Hollow Sections (SHS)**:

- SHS are square-shaped hollow structural sections with uniform wall thickness along their length.
- These sections are also specified under IS 4923:1997, "Hollow Steel Sections for Structural Use - Specification."
- IS 4923 outlines the requirements for the dimensions, sectional properties, tolerances, and material properties of SHS to ensure conformity and performance in construction applications.
- SHS find applications in structural framing, supports, bracing, and architectural features where square cross-sections are desired.



3. **Circular Hollow Sections (CHS)**:

- CHS are circular-shaped hollow structural sections with uniform wall thickness along their circumference.
- These sections are specified under IS 1161:2014, "Steel Tubes for Structural Purposes - Specifications."
- IS 1161 provides requirements for the dimensions, sectional properties, tolerances, and material properties of CHS for structural use in construction.
- CHS are commonly used in construction for columns, poles, piling, and structural elements requiring circular cross-sections.

Assessment questions to the lecture

Qn No	Question	Answer	Bloom's Knowledge Level
1	What is the manufacturing process used for hot rolled sections in structural steel? A) Bending or rolling thin gauge steel sheets at room temperature B) Heating steel billets or ingots above their recrystallization temperature and passing them through rollers C) Casting molten steel into molds D) Welding together steel plates to form the desired shapes	b	K1
2	Which Indian Standard code specifies the dimensions for hot rolled steel beam, column, channel, and angle sections? A) IS 4923 B) IS 811 C) IS 808 D) IS 1161	c	K1
3	Which type of cold formed section has a profile resembling the letter "Z"? A) Z-Sections B) C-Sections C) Lipped Channels D) Sigma Sections	C	K1

Students must prepare answers for the following questions at the end of the lecture.

Qn No	Question	Marks	CO	Bloom's Knowledge Level
1	Explain in detail the various types of Hot Rolled Sections as per IS code.	13	1	K1
2	Explain in detail the various types of Cold Formed Sections as per IS code.	13	1	K1

Reference Book:

- N Subramaniam , Design of Steel Structures, Oxford University press, New Delhi, 2011

Unit	1. INTRODUCTION TO STRUCTURAL STEEL AND DESIGN OF CONNECTIONS	Lecture No	03
Topic	Concept of Limit state design for steel		
Learning Outcome (LO) At the end of this lecture, students will be able to		**Bloom's Knowledge Level**	
LO1	State the various strength Limit states in steel	K1	
LO2	Identify the stress strain parameters of steel	K1	
LO3	Describe the Serviceability limit states of steel	K1	

Limit state design (LSD) is a fundamental approach in structural engineering that ensures the safety and serviceability of structures under various loading conditions. It is widely adopted in the design of steel structures to account for uncertainties, variability in material properties, and the complexity of real-world structural behavior. The concept of limit state design is crucial for complying with safety standards and optimizing the performance of steel structures.

In India, the principles of limit state design for steel structures are outlined in the Indian Standard (IS) code IS 800:2007, "General Construction in Steel – Code of Practice." This code provides guidelines and recommendations for the design, fabrication, and erection of steel structures using the limit state design approach. Let's delve into the concept of limit state design for steel structures as per IS 800:2007 in detail:

1. **Introduction to Limit State Design**:

    - Limit state design is based on the concept of defining critical limit states, which encompass both strength and serviceability criteria that a structure must satisfy throughout its design life.
    - The two primary limit states considered in structural design are the ultimate limit state (ULS) and the serviceability limit state (SLS).
    - The ultimate limit state refers to the state at which the structure or any of its components reaches the limit of its load-carrying capacity, leading to failure or collapse.
    - The serviceability limit state focuses on ensuring that the structure performs adequately under service loads without experiencing excessive deflections, vibrations, or other forms of distress that may affect its functionality or durability.

2. **Design Philosophy**:

    - IS 800:2007 adopts the partial safety factor method for limit state design, which involves applying appropriate factors to account for uncertainties in material properties, loadings, and modeling assumptions.
    - The design philosophy emphasizes the importance of ensuring an adequate margin of safety against failure by considering the ultimate and serviceability limit states simultaneously.

- The code specifies partial safety factors for various loadings, such as dead loads, live loads, wind loads, and seismic loads, as well as for material properties and other factors affecting structural behaviour

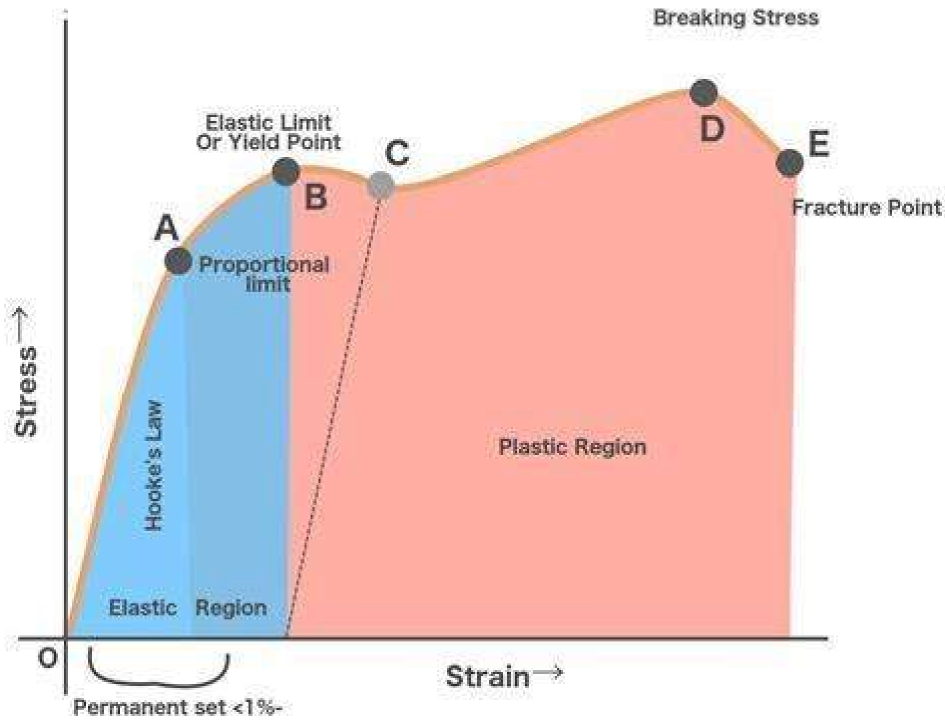


Fig., Stress strain curve for Steel

3. **Limit States Considered in Steel Design**:

- Ultimate Limit States (ULS):
    - Strength Limit State: Ensures that the structure has sufficient strength to resist applied loads without reaching a state of collapse or failure. This includes considerations for axial compression, bending, shear, and torsion.
    - Stability Limit State: Ensures that the structure maintains stability against buckling and lateral-torsional instability under various loading conditions.
    - Fatigue Limit State: Addresses the potential for fatigue failure due to repeated loading and unloading cycles, particularly in members subjected to fluctuating or cyclic loads.
- Serviceability Limit States (SLS):
    - Deflection Limit State: Limits the deflection of structural members under service loads to prevent excessive deformations that may affect functionality or aesthetics.
    - Vibration Limit State: Ensures that the structure remains free from excessive vibrations under service loads to prevent discomfort, damage to sensitive equipment, or functional issues.

4. **Material Properties and Design Considerations**:

    - IS 800:2007 provides specifications for the material properties of structural steel, including yield strength, tensile strength, ductility, and toughness.
    - Design considerations include factors such as member slenderness, end conditions, connections, and detailing to ensure proper load transfer, stability, and resistance to various limit states.
    - The code also addresses issues such as fire protection, corrosion protection, and durability to enhance the performance and longevity of steel structures.

5. **Design Process**:

    - The design process involves determining the loads and load combinations based on the structural configuration, occupancy, and intended use of the building or structure.
    - Structural analysis is performed to assess the response of the structure under different loading conditions, considering the appropriate limit states and design criteria.
    - Structural members are proportioned and detailed to ensure that they meet the requirements for strength, stability, and serviceability as per the limit state design philosophy.

6. **Verification and Documentation**:

    - Verification involves checking the adequacy of the proposed structural design against the specified limit states and design criteria.
    - This includes verifying member capacities, deflections, stability, and other performance indicators using analytical methods, software simulations, or empirical equations.
    - Documentation of the design process, including calculations, drawings, specifications, and compliance with relevant codes and standards, is essential for ensuring accountability, transparency, and regulatory compliance.

Assessment questions to the lecture

Qn No	Question	Answer	Bloom's Knowledge Level
1	What is the primary focus of the ultimate limit state (ULS) in structural design according to IS 800:2007? A) Ensuring minimal deflection of structural members under service loads B) Preventing excessive vibrations in the structure under service loads C) Ensuring that the structure has sufficient strength	c	K1

	to resist applied loads without reaching a state of collapse or failure D) Addressing potential fatigue failure due to repeated loading and unloading cycles		
2	Which limit state ensures that the structure performs adequately under service loads without experiencing excessive deflections or vibrations? A) Strength Limit State B) Stability Limit State C) Deflection Limit State D) Fatigue Limit State	c	K1
3	What approach does IS 800:2007 adopt for limit state design? A) Elastic design method B) Load and resistance factor design (LRFD) C) Ultimate strength design (USD) D) Partial safety factor method	d	K1

<u>Students must prepare answers for the following questions at the end of the lecture.</u>

Qn No	Question	Marks	CO	Bloom's Knowledge Level
1	Explain the significance of the partial safety factor method in limit state design as per IS 800:2007.	7	1	K1
2	Discuss the role of the serviceability limit state (SLS) in structural design according to IS 800:2007.	7	1	K1
3	Explain in detail the stress strain curve of steel with proper sketch.	13	1	K1

Reference Book:

- N Subramaniam , Design of Steel Structures, Oxford University press, New Delhi, 2011

Unit	1. INTRODUCTION TO STRUCTURAL STEEL AND DESIGN OF CONNECTIONS	Lecture No	04
Topic	Design of Simple Bolted Connections		
Learning Outcome (LO) At the end of this lecture, students will be able to		**Bloom's Knowledge Level**	
LO1	Design Simple bolted connections	K3	

The design of connections is done as per Section 10 of IS 800:2007.

SECTION 10
CONNECTIONS

10.1 General

10.1.1 This section deals with the design and detailing requirements for joints between members. Connection elements consist of components such as cleats, gusset plates, brackets, connecting plates and connectors such as rivets, bolts, pins, and welds. The connections in a structure shall be designed so as to be consistent with the assumptions made in the analysis of the structure and comply with the requirements specified in this section. Connections shall be capable of transmitting the calculated design actions.

10.1.2 Where members are connected to the surface of a web or the flange of a section, the ability of the web or the flange to transfer the applied forces locally should be checked and where necessary, local stiffening provided.

10.1.3 Ease of fabrication and erection should be considered in the design of connections. Attention should be paid to clearances necessary for field erection, tolerances, tightening of fasteners, welding procedures, subsequent inspection, surface treatment and maintenance.

10.1.4 The ductility of steel assists the distribution of forces generated within a joint. Effects of residual stresses and stresses due to tightening of fasteners and normal tolerances of fit-up need not therefore be considered in connection design, provided ductile behaviour is ensured.

10.1.5 In general, use of different forms of fasteners to transfer the same force shall be avoided. However, when different forms of fasteners are used to carry a shear load or when welding and fasteners are combined, then one form of fastener shall be normally designed to carry the total load. Nevertheless, fully tensioned friction grip bolts may be designed to share the load with welding, provided the bolts are fully tightened to develop necessary pretension after welding.

10.1.6 The partial safety factor in the evaluation of design strength of connections shall be taken as given in Table 5.

10.2 Location Details of Fasteners

10.2.1 *Clearances for Holes for Fasteners*

Bolts may be located in standard size, over size, short slotted or long slotted hole.

a) *Standard clearance hole* — Except where fitted bolts, bolts in low-clearance or oversize holes are specified, the diameter of standard clearance holes for fasteners shall be as given in Table 19.

b) *Over size hole* — Holes of size larger than the standard clearance holes, as given in Table 19 may be used in slip resistant connections and hold down bolted connections; only where specified, provided the over size holes in the outer ply is covered by a cover plate of sufficiently large size and thickness and having a hole not larger than the standard clearance hole (and hardened washer in slip resistant connections).

c) *Short and long slots* — Slotted holes of size larger than the standard clearance hole, as given in Table 19 may be used in slip resistant connections and hold down bolted connections, only where specified, provided the over size holes in the outer ply is covered by a cover plate of sufficiently large size and thickness and having a hole of size not larger than the standard clearance hole (and hardened washer in slip resistant connection).

10.2.2 *Minimum Spacing*

The distance between centre of fasteners shall not be less than 2.5 times the nominal diameter of the fastener.

Table 19 Clearances for Fastener Holes

(*Clause* 10.2.1)

Sl No.	Nominal Size of Fastener, d mm	Size of the Hole = Nominal Diameter of the Fastener + Clearances (mm)			
		Standard Clearance in Diameter and Width of Slot	Over Size Clearance in Diameter	Clearance in the Length of the Slot	
				Short Slot	Long Slot
(1)	(2)	(3)	(4)	(5)	(6)
i)	12 – 14	1.0	3.0	4.0	2.5 d
ii)	16 – 22	2.0	4.0	6.0	2.5 d
iii)	24	2.0	6.0	8.0	2.5 d
iv)	Larger than 24	3.0	8.0	10.0	2.5 d

Example 5.1 *Design a lap joint between two plates as shown in Fig. 5.55 so as to transmit a factored load of 70 kN using M16 bolts of grade 4.6 and grade 410 plates.*

Solution

Strength calculation

Nominal diameter of the bolt $d = 16$ mm

Hole diameter = 16 + 2 = 18 mm

Bolts are in single shear and hence shear capacity of the bolt

$= (f_u/\sqrt{3})\,(n_n A_{nb} + n_s A_{sb})/\gamma_{mb} = (400/\sqrt{3})\,(1 \times 157)/1.25 = 29.0$ kN

Since the top plate is only 12 mm, it is assumed that the shear plane is through the threaded portion and hence $n_s = 0$

Bearing capacity of the thinner plate

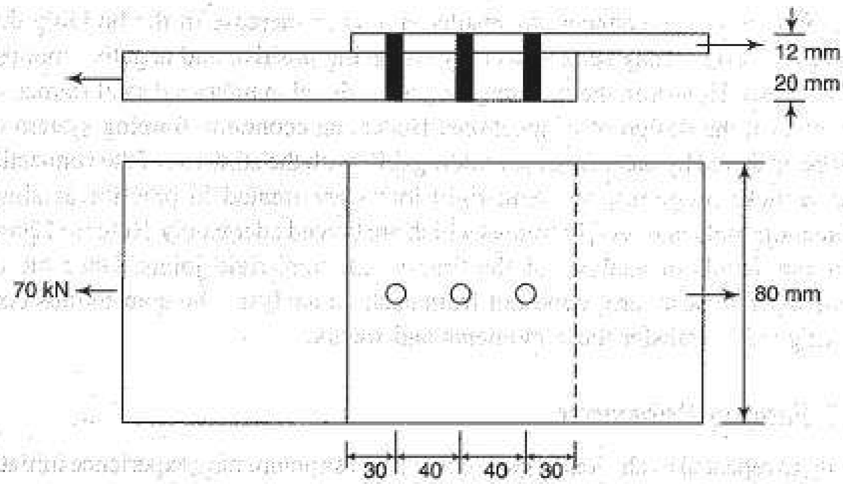


Fig. 5.55

$= 2.5 k_b d t f_u/\gamma_{mb} = (2.5 \times 0.49 \times 410/1.25) \times 16 \times 12 = 77.15$ N

k_b is smaller of 30/(3 × 18), 40/(3 × 18) – 0.25, 400/410, 1.0.

Hence, $k_b = 0.49$,

Bolt value = 29 kN

Required number of bolts = 70/29 = 2.41 = 3 bolts

Detailing

Minimum pitch = 2.5 × d = 40 mm

Minimum edge distance = 29 mm (as per Table 5.6)

Provide three bolts as shown in Fig. 5.55.

Example 5.4 *A member of a truss consists of two angles ISA 75 × 75 × 6 placed back to back. It carries an ultimate tensile load of 150 kN and is connected to a gusset plate 8-mm thick placed in between the two connected legs. Determine the number of 16-mm-diameter 4.6 grade ordinary bolts required for the joint. Assume f_u of plate as 410 MPa.*

Solution

The arrangement of joint is as shown in Fig. 5.58. The bolts are in double shear. They bear against 8-mm gusset and two 6-mm angles, the former controlling the value in bearing as shown below. From Table 5.9,

Strength in double shear for 16-mm-diameter 4.6 grade bolts = 2 × 29 = 58 kN

Strength in bearing on 8-mm plate = (2.5 × 0.49 × 16 × 8 × 410)/(1.25 × 1000)
= 51.4 kN

(Note: with e = 30 and p = 40, k_b = 0.49 from Example 5.1)

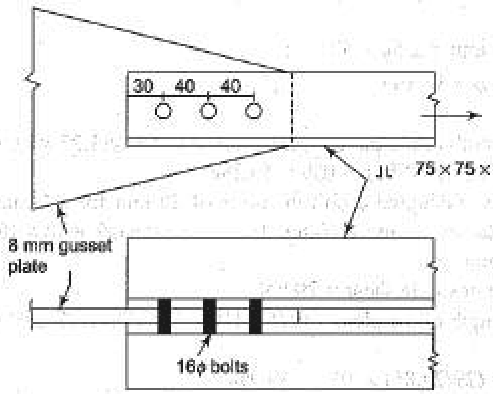


Fig. 5.58

Strength in bearing on two 6-mm thick angles
= [2.5 × 0.49 × (2 × 6) × 16 × 410]/(1.25 × 1000) = 77.0 kN

Thus,

Strength of the bolt = 51.4 kN

Required number of bolts = 150/51.4 = 2.92

Therefore, provide three bolts as shown in Fig. 5.58.

Example 5.13 *Design a connection of a truss joint as shown in Fig. 5.65, using M16 black bolts of property class 4.6 and grade 410 steel. Assume that the members shown are capable of resisting the loads.*

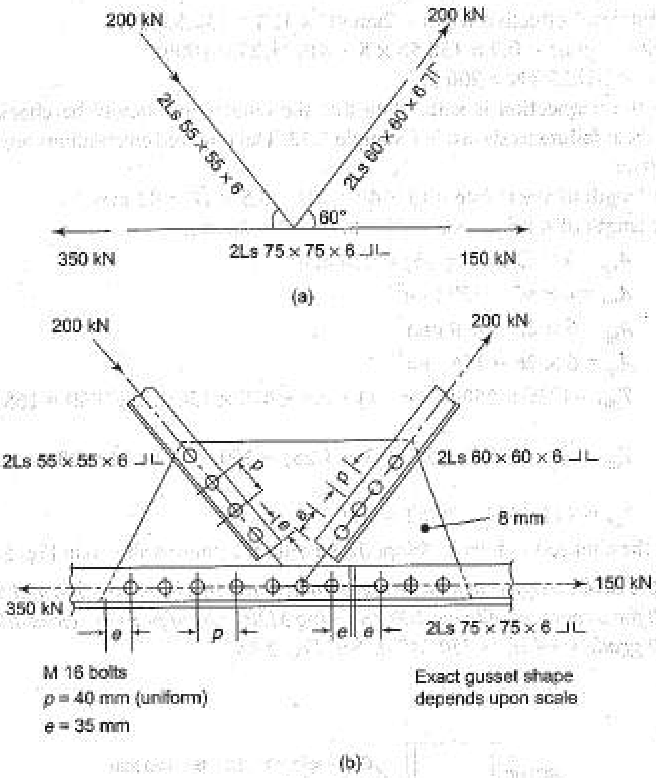


Fig. 5.65

Solution

Assume 8-mm thick gusset. The gusset plate is sandwiched between the angles and hence the bolts will be in double shear.

For 16-mm diameter property class 4.6 bolt, from Table 5.9

Strength in double shear = 29.0 × 2 = 58 kN; k_b = 40/(3 × 18) – 0.25 = 0.49

Strength in bearing = 2.5 × 0.49 × 16 × 8 × 410/(1,25 × 1000) = 51.4 kN

Hence,

Strength of bolt = 51.4 kN

Bolts for 200 kN = 200/51.4 = 3.9 (hence provide four bolts)

Bolts for 150 kN = 150/51.4 = 2.9 (hence provide three bolts)

Bolts for 350 kN = 350/51.4 = 6.8 (hence provide seven bolts)
Provide edge distance = 2 × 16 = 32, say 35 mm
Pitch = 2.5 × 16 = 40 mm

Check for gusset plate

Distance from first bolt to last bolt in member carrying 200 kN
$= 3p = 3 \times 40 = 120$ mm
'Whitmore' effective width = 2tan30° × 120 = 138.56 mm
Capacity of plate = 0.9 × 138.56 × 8 × 410/(1.25 × 1000)
= 327.22 kN > 200 kN

Hence the connection is safe. Note that the connection should be checked for block shear failure as shown in Example 5.12. The required calculations are shown as follows:

Net length of shear face = (3 × 40 + 35) − 3.5 × 18 = 92 mm
Net length of tension face = 35 − 0.5 × 18 = 26 mm

$$A_{vg} = 6 \times (3 \times 40 + 35) = 930 \text{ mm}^2$$
$$A_{vn} = 6 \times 92 = 552 \text{ mm}^2$$
$$A_{tg} = 6 \times 35 = 210 \text{ mm}^2$$
$$A_{tn} = 6 \times 26 = 156 \text{ mm}^2$$
$$T_{db1} = [930 \times 250/(\sqrt{3} \times 1.1) + 0.9 \times 410 \times 156/1.25]/1000 = 168.08 \text{ kN}$$

and

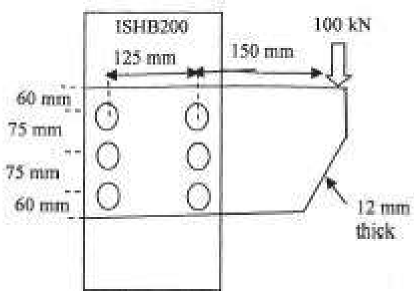
$$T_{db2} = [0.9 \times 410 \times 552/(\sqrt{3} \times 1.25) + 250 \times 210/1.1]/1000 = 141.8 \text{ kN}$$

Thus,

$$T_{db} = 141.8 \text{ kN} > 200/2 = 100 \text{ kN}$$

Hence the connection is safe. Adopt the structural details as shown in Fig. 5.65(b).

Students must prepare answers for the following questions at the end of the lecture.

Qn No	Question	Marks	CO	Bloom's Knowledge Level
1	Design a bolted connection for two plates that are 10 mm thick and 150 mm wide. The connection will be subjected to an axial load of 50 kN and will use grade 8.8 bolts. Assume that the bearing strength of the bolts is 200 MPa and the allowable stress on the plates is 165 MPa. (April /May 2023)	13	1	K3
2	Design the bracket connection with 24mm Gr 4.6 bearing bolts. Is it sufficient to bear the applied factored load? (Apr/May 2021) 	13	1	K3

Reference Book:

- N Subramaniam , Design of Steel Structures, Oxford University press, New Delhi, 2011

Unit	1. INTRODUCTION TO STRUCTURAL STEEL AND DESIGN OF CONNECTIONS	Lecture No	05
Topic	Design of Eccentric Bolted Connections		
Learning Outcome (LO) At the end of this lecture, students will be able to		Bloom's Knowledge Level	
LO1	Design Eccentric Bolted Connections	K3	

10.3.1.3 In the calculation of thread length, allowance should be made for tolerance and thread run off.

10.3.2 A bolt subjected to a factored shear force (V_{sb}) shall satisfy the condition

$$V_{sb} = V_{db}$$

where V_{db} is the design strength of the bolt taken as the smaller of the value as governed by shear, V_{dsb} (*see* **10.3.3**) and bearing, V_{dpb} (*see* **10.3.4**).

10.3.3 *Shear Capacity of Bolt*

The design strength of the bolt, V_{dsb} as governed shear strength is given by:

$$V_{dsb} = V_{nsb} / \gamma_{mb}$$

where

V_{nsb} = nominal shear capacity of a bolt, calculated as follows:

$$V_{nsb} = \frac{f_u}{\sqrt{3}}\left(n_n A_{nb} + n_s A_{sb}\right)$$

where

f_u = ultimate tensile strength of a bolt;

n_n = number of shear planes with threads intercepting the shear plane;

n_s = number of shear planes without threads intercepting the shear plane;

A_{sb} = nominal plain shank area of the bolt; and

A_{nb} = net shear area of the bolt at threads, may be taken as the area corresponding to root diameter at the thread.

10.3.3.1 *Long joints*

When the length of the joint, l_j of a splice or end connection in a compression or tension element containing more than two bolts (that is the distance between the first and last rows of bolts in the joint, measured in the direction of the load transfer) exceeds $15d$ in the direction of load, the nominal shear capacity (*see* **10.3.2**), V_{db} shall be reduced by the factor β_{lj}, given by:

$$\beta_{lj} = 1.075 - l_j / (200\ d) \text{ but } 0.75 \le \beta_{lj} \le 1.0$$
$$= 1.075 - 0.005(l_j / d)$$

where

d = Nominal diameter of the fastener.

NOTE — This provision does not apply when the distribution of shear over the length of joint is uniform, as in the connection of web of a section to the flanges.

10.3.3.2 *Large grip lengths*

When the grip length, l_g (equal to the total thickness of the connected plates) exceeds 5 times the diameter, d of the bolts, the design shear capacity shall be reduced by a factor β_{lg}, given by:

$$\beta_{lg} = 8\ d / (3\ d + l_g) = 8 / (3 + l_g / d)$$

β_{lg} shall not be more than β_{lj} given in **10.3.3.1**. The grip length, l_g shall in no case be greater than $8d$.

10.3.3.3 *Packing plates*

The design shear capacity of bolts carrying shear through a packing plate in excess of 6 mm shall be decreased by a factor, β_{pk} given by:

$$\beta_{pk} = (1 - 0.0125\ t_{pk})$$

where

t_{pk} = thickness of the thicker packing, in mm.

10.3.4 *Bearing Capacity of the Bolt*

The design bearing strength of a bolt on any plate, V_{dpb} as governed by bearing is given by:

$$V_{dpb} = V_{npb} / \gamma_{mb}$$

where

V_{npb} = nominal bearing strength of a bolt
$= 2.5\ k_b\ d\ t\ f_u$

where

k_b is smaller of $\dfrac{e}{3d_0}$, $\dfrac{p}{3d_0} - 0.25$, $\dfrac{f_{ub}}{f_u}$, 1.0;

e, p = end and pitch distances of the fastener along bearing direction;

d_0 = diameter of the hole;

f_{ub}, f_u = ultimate tensile stress of the bolt and the ultimate tensile stress of the plate, respectively;

d = nominal diameter of the bolt; and

t = summation of the thicknesses of the connected plates experiencing bearing stress in the same direction, or if the bolts are countersunk, the thickness of the plate minus one half of the depth of countersinking.

The bearing resistance (in the direction normal to the slots in slotted holes) of bolts in holes other than standard clearance holes may be reduced by multiplying the bearing resistance obtained as above, V_{npb}, by the factors given below:

a) Over size and short slotted holes – 0.7, and
b) Long slotted holes – 0.5.

NOTE — The block shear of the edge distance due to bearing force may be checked as given in **6.4**.

10.3.5 *Tension Capacity*

A bolt subjected to a factored tensile force, T_b shall satisfy:

$$T_b \leq T_{db}$$

where

$T_{db} = T_{nb} / \gamma_{mb}$

T_{nb} = nominal tensile capacity of the bolt, calculated as:

$0.90 f_{ub} A_n < f_{yb} A_{sb} (\gamma_{mb} / \gamma_{m0})$

where

f_{ub} = ultimate tensile stress of the bolt,

f_{yb} = yield stress of the bolt,

A_n = net tensile stress area as specified in the appropriate Indian Standard (for bolts where the tensile stress area is not defined, A_n shall be taken as the area at the bottom of the threads), and

A_{sb} = shank area of the bolt.

10.3.6 *Bolt Subjected to Combined Shear and Tension*

A bolt required to resist both design shear force (V_{sd}) and design tensile force (T_b) at the same time shall satisfy:

$$\left(\frac{V_{sb}}{V_{db}}\right)^2 + \left(\frac{T_b}{T_{db}}\right)^2 \leq 1.0$$

where

V_{sb} = factored shear force acting on the bolt,

V_{db} = design shear capacity (*see* **10.3.2**),

T_b = factored tensile force acting on the bolt, and

T_{db} = design tension capacity (*see* **10.3.5**).

10.4 Friction Grip Type Bolting

10.4.1 In friction grip type bolting, initial pretension in bolt (usually high strength) develops clamping force at the interfaces of elements being joined. The frictional resistance to slip between the plate surfaces subjected to clamping force opposes slip due to externally applied shear. Friction grip type bolts and nuts shall conform to IS 3757. Their installation procedures shall conform to IS 4000.

10.4.2 Where slip between bolted plates cannot be tolerated at working loads (slip critical connections), the requirements of **10.4.3** shall be satisfied. However, at ultimate loads, the requirements of **10.4.4** shall be satisfied by all connections.

10.4.3 *Slip Resistance*

Design for friction type bolting in which slip is required to be limited, a bolt subjected only to a factored design shear force, V_{sf} in the interface of connections at which slip cannot be tolerated, shall satisfy the following:

$$V_{sf} \leq V_{dsf}$$

where

$V_{dsf} = V_{nsf} / \gamma_{mf}$

V_{nsf} = nominal shear capacity of a bolt as governed by slip for friction type connection, calculated as follows:

$$V_{nsf} = \mu_f \, n_e \, K_h \, F_o$$

where

μ_f = coefficient of friction (slip factor) as specified in Table 20 ($\mu_f = 0.55$),

n_e = number of effective interfaces offering frictional resistance to slip,

K_h = 1.0 for fasteners in clearance holes,

= 0.85 for fasteners in oversized and short slotted holes and for fasteners in long slotted holes loaded perpendicular to the slot,

= 0.7 for fasteners in long slotted holes loaded parallel to the slot,

γ_{mf} = 1.10 (if slip resistance is designed at service load),

= 1.25 (if slip resistance is designed at ultimate load),

F_o = minimum bolt tension (proof load) at installation and may be taken as $A_{nb} f_o$,

A_{nb} = net area of the bolt at threads, and

f_o = proof stress (= $0.70 f_{ub}$).

NOTE — V_{sf} may be evaluated at a service load or ultimate load using appropriate partial safety factors, depending upon whether slip resistance is required at service load or ultimate load.

10.4.3.1 *Long joints*

The provision for the long joints in **10.3.3.1** shall apply to friction grip connections also.

10.4.4 *Capacity after slipping*

When friction type bolts are designed not to slip only under service loads, the design capacity at ultimate load may be calculated as per bearing type connection (*see* **10.3.2** and **10.3.3**).

NOTE — The block shear resistance of the edge distance due to bearing force may be checked as given in **6.4**.

10.4.5 *Tension Resistance*

A friction bolt subjected to a factored tension force (T_f) shall satisfy:

Example 5.26 *Design a bolted web cleat connection for an ISMB 600 and two coped beams of size ISMB 400 (300 kN reaction due to factored loads) and ISMB 250 (75 kN reaction due to factored loads) using grade 8.8 bolts of 20 mm diameter (see Fig. 5.78).*

Solution

For M20 grade 8.8 bolts:

Shear capacity in single shear = $245 \times 800/(\sqrt{3} \times 1.25 \times 1000) = 90.5$ kN

Shear capacity in double shear = $2 \times 90.5 = 181$ kN

Left hand side beam ISMB 400

Web thickness = 8.9 mm

Bearing capacity of bolt against the web; with $k_b = 0.6$

$= 2.5 \times 0.6 \times 20 \times 8.9 \times 410/(1.25 \times 1000) = 87.6$ kN

Try five bolts at 60 mm spacing and 40-mm edge distance. Horizontal shear force on bolt due to eccentricity

$= 300 \times (50 + 0.5 \times 12) \times 120/[2(60^2 + 120^2)]$

$= 56$ kN

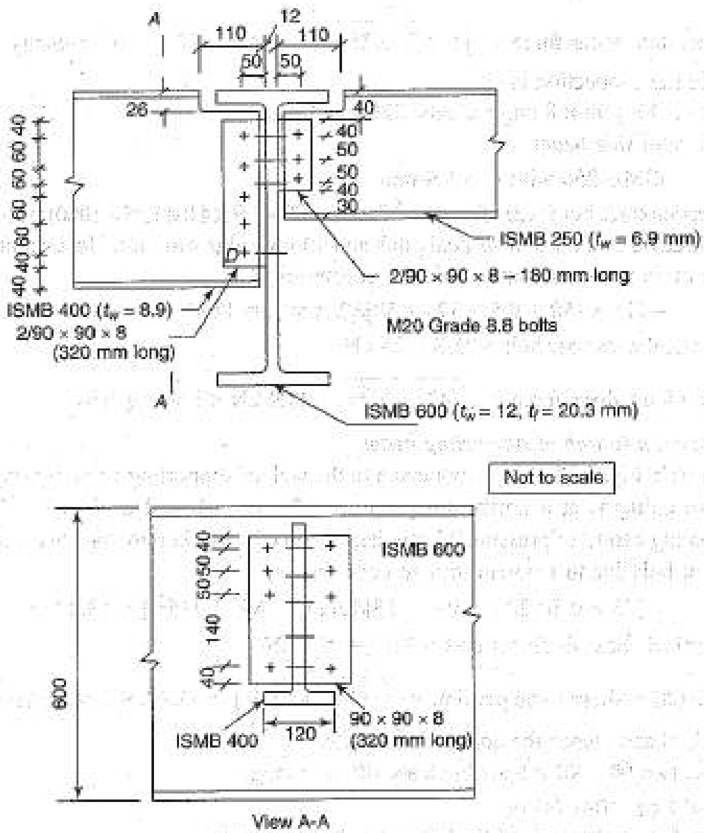


Fig. 5.78

Vertical shear force per bolt = 300/5 = 60 kN

Resultant shear force = $\sqrt{(56^2 + 60^2)}$ = 82.07 kN > bolt capacity. Hence the connection is safe.

Connection to web of supporting beam

Assuming a cleat of 2L ISA 90 × 90 × 8 mm

Bearing capacity on 8 mm cleat = 2.5 × 0.6 × 20 × 8 × 410/(1.25 × 1000) = 78.72 kN

Single shear capacity of 78.72 kN governs

Try eight bolts, two columns at a horizontal gauge length of 120 mm with a vertical pitch of 50 mm for the first three bolts and 140-mm for the last bolt with an edge distance of 40 mm in the cleat. Assuming centre of pressure at 25 mm below top of cleat,

Horizontal shear force on bottom bolt due to moment due to eccentricity

$$= [300 \times 0.5(120 - 8.9) \times 255]/[2(15^2 + 65^2 + 115^2 + 255^2)] = 25.69 \text{ kN}$$

Vertical shear force per bolt = 300/8 = 37.5 kN

Resultant shear force = $\sqrt{(37.5^2 + 25.69^2)}$ = 45.46 kN < bolt capacity

Hence the connection is ok.

Use two 90 × 90 × 8 angle cleats, 320 mm long.

Right hand side beam

ISMB 250 with t_w = 6.9 mm

Bearing capacity of web of beam = 2.5 × 0.5 × 20 × 6.9 × 410/(1.25 × 1000) = 56.58 kN

Try three bolts at 50-mm vertical pitch and 40-mm edge distance. Horizontal shear force on bolt due to moment (due to eccentricity)

$$= [75 \times (50 + 0.5 \times 12) \times 50]/(2 \times 50^2) = 42 \text{ kN}$$

Vertical shear per bolt = 75/3 = 25 kN

Resultant shear force = $\sqrt{(42^2 + 25^2)}$ = 48.9 kN < bolt capacity

Connection to web of supporting beam

As seen in Fig. 5.78, this is connected to the web of supporting beam by six bolts, in two columns at a horizontal gauge of 120 mm c/c and a pitch of 50 mm. Assuming centre of pressure 25 mm below top of cleat, horizontal shear force on bottom bolt due to moment (due to eccentricity)

$$= [75 \times 0.5(120 - 6.9) \times 115]/[2(15^2 + 65^2 + 115^2)] = 13.8 \text{ kN}$$

Vertical shear force per bolt = 75/6 = 12.5 kN

Resultant shear force per bolt = $\sqrt{(13.8^2 + 12.5^2)}$ = 18.6 kN < bolt capacity in double shear. Hence the connection is safe.

Use two 90 × 90 × 8 angle cleats 180 mm long.

Web of supporting beam

Check for combined load for left and right hand beam.

Bearing capacity on 12 mm web $= 2.5 \times 0.5 \times 20 \times 12 \times 410/(1.25 \times 1000)$
$= 98.4$ kN

Total horizontal shear force due to moment due to eccentricity (on third row)
$= [(25.69 \times 115/255] + 13.8 = 25.38$ kN

Total vertical shear force $= 37.5 + 12.5 = 50$ kN

Resultant shear force $= \sqrt{(50^2 + 25.38^2)} = 56.1$ kN < bearing capacity.

Hence, the connection is safe.

Left hand beam

At the end of the notch, depth of section $= 400 - 26 = 374$ mm

Shear capacity $= (374 \times 8.9) \times (250/\sqrt{3})/(1.1 \times 1000)$
$= 436.76$ kN > shear force $= 300$ kN

Shear capacity through bolt holes

Block shear capacity:

$$T_{db1} = [250/(\sqrt{3} \times 1.1) \times 8.9 \times (54 + 4 \times 60) + (0.9 \times 410/1.25) \times 8.9 \times (50 - 22/2)]/1000 = 445.8 \text{ kN}$$

$$T_{db2} = [(0.9 \times 410/(\sqrt{3} \times 1.25) \times 8.9 \times (54 + 4 \times 60 - 4.5 \times 22) + 50 \times 8.9 \times (250/1.10)]/1000 = 396.92 \text{ kN}$$

Hence,

$$T_{db} = 396.92 \text{ kN}$$

Shear force = 300 kN < shear capacity. Hence the connection is safe.

Right hand side beam

At the end of the notch, depth of section $= 250 - 26 = 224$

Shear capacity $= (224) \times 6.9 \times (250/\sqrt{3})/(1.1 \times 1000)$
$= 202.80$ kN > shear force $= 75$ kN

Block shear capacity of web

$$T_{db1} = [250/(\sqrt{3} \times 1.1) \times 6.9 \times (54 + 2 \times 50) + (0.9 \times 410/1.25) \times 6.9\,(50 - 22/2)]/1000 = 218.86 \text{ kN}$$

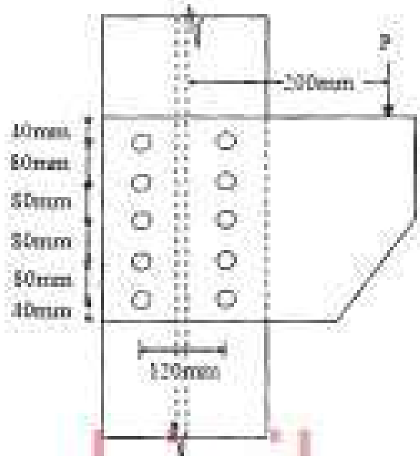
$$T_{db2} = [0.9 \times 410/(\sqrt{3} \times 1.25) \times 6.9 \times (54 + 2 \times 50 - 2.5 \times 22) + 50 \times 6.9 \times (250/1.1)]/1000 = 194.83 \text{ kN}$$

$$T_{db} = 194.83 \text{ kN}$$

Shear force = 75 kN < Shear capacity

Hence, the connection is safe.

<u>Students must prepare answers for the following questions at the end of the lecture.</u>

Qn No	Question	Marks	CO	Bloom's Knowledge Level
1	Determine the safe load P that can be carried by the joint shown in fig. Consider 20mm bolt with Gr 4.6  (Nov/Dec 2021)	13	5	K2

Reference Book:

- N Subramaniam , Design of Steel Structures, Oxford University press, New Delhi, 2011

Unit	1. INTRODUCTION TO STRUCTURAL STEEL AND DESIGN OF CONNECTIONS	Lecture No	06
Topic	Design of Welded Connections		
Learning Outcome (LO) At the end of this lecture, students will be able to		**Bloom's Knowledge Level**	
LO1	Design welded Connection between steel members	K3	

IS 800 : 2007

10.5 Welds and Welding

10.5.1 *General*

Requirements of welds and welding shall conform to IS 816 and IS 9595, as appropriate.

10.5.1.1 *End returns*

Fillet welds terminating at the ends or sides of parts should be returned continuously around the corners for a distance of not less than twice the size of the weld, unless it is impractical to do so. This is particularly important on the tension end of parts carrying bending loads.

10.5.1.2 *Lap joint*

In the case of lap joints, the minimum lap should not be less than four times the thickness of the thinner part joined or 40 mm, whichever is more. Single end fillet should be used only when lapped parts are restrained from openings. When end of an element is connected only by parallel longitudinal fillet welds, the length of the weld along either edge should not be less than the transverse spacing between longitudinal welds.

10.5.1.3 A single fillet weld should not be subjected to moment about the longitudinal axis of the weld.

10.5.2 *Size of Weld*

10.5.2.1 The size of normal fillets shall be taken as the minimum weld leg size. For deep penetration welds, where the depth of penetration beyond the root run is a minimum of 2.4 mm, the size of the fillet should be taken as the minimum leg size plus 2.4 mm.

10.5.2.2 For fillet welds made by semi-automatic or automatic processes, where the depth of penetration is considerably in excess of 2.4 mm, the size shall be taken considering actual depth of penetration subject to agreement between the purchaser and the contractor.

10.5.2.3 The size of fillet welds shall not be less than 3 mm. The minimum size of the first run or of a single run fillet weld shall be as given in Table 21, to avoid the risk of cracking in the absence of preheating.

10.5.2.4 The size of butt weld shall be specified by the effective throat thickness.

10.5.3 *Effective Throat Thickness*

10.5.3.1 The effective throat thickness of a fillet weld shall not be less than 3 mm and shall generally not exceed $0.7t$, or $1.0t$ under special circumstances, where t is the thickness of the thinner plate of elements being welded.

Table 21 Minimum Size of First Run or of a Single Run Fillet Weld

(*Clause* 10.5.2.3)

Sl No.	Thickness of Thicker Part mm		Minimum Size mm
	Over	Up to and Including	
(1)	(2)	(3)	(4)
i)	–	10	3
ii)	10	20	5
iii)	20	32	6
iv)	32	50	8 of first run 10 for minimum size of weld

NOTES

1 When the minimum size of the fillet weld given in the table is greater than the thickness of the thinner part, the minimum size of the weld should be equal to the thickness of the thinner part. The thicker part shall be adequately preheated to prevent cracking of the weld.

2 Where the thicker part is more than 50 mm thick, special precautions like pre-heating should be taken.

10.5.3.2 For the purpose of stress calculation in fillet welds joining faces inclined to each other, the effective throat thickness shall be taken as K times the fillet size, where K is a constant, depending upon the angle between fusion faces, as given in Table 22.

10.5.3.3 The effective throat thickness of a complete penetration butt weld shall be taken as the thickness of the thinner part joined, and that of an incomplete penetration butt weld shall be taken as the minimum thickness of the weld metal common to the parts joined, excluding reinforcements.

10.5.4 *Effective Length or Area of Weld*

10.5.4.1 The effective length of fillet weld shall be taken as only that length which is of the specified size and required throat thickness. In practice the actual length of weld is made of the effective length shown in drawing plus two times the weld size, but not less than four times the size of the weld.

10.5.4.2 The effective length of butt weld shall be taken as the length of the continuous full size weld, but not less than four times the size of the weld.

Table 22 Values of K for Different Angles Between Fusion Faces

(*Clause* 10.5.3.2)

Angle Between Fusion Faces	60°–90°	91°–100°	101°–106°	107°–113°	114°–120°
Constant, K	0.70	0.65	0.60	0.55	0.50

Example 6.1 *Two plates of thickness 14 mm and 12 mm are to be jointed by a groove weld as shown in Fig. 6.86. The joint is subjected to a factored tensile force of 350 kN. Assuming an effective length of 150 mm, check the safety of the joint for*

Case (i) Single-V groove weld joint

Case (ii) double-V groove weld joint

Assume that Fe 410 grade steel plates are used and that the welds are shop welded.

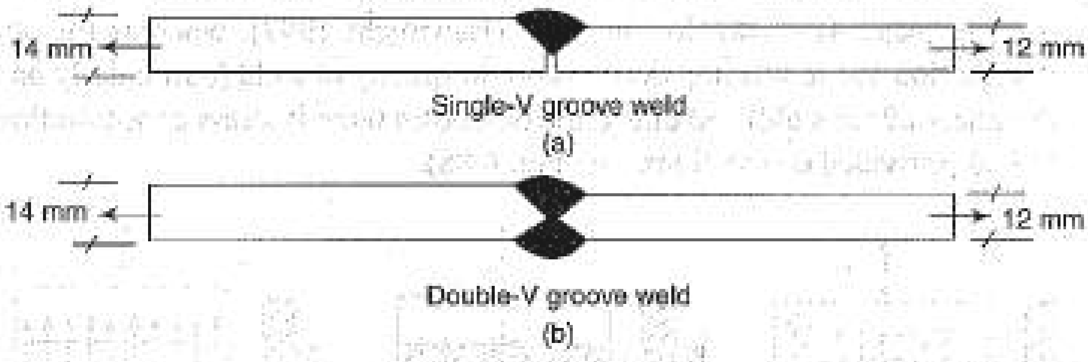


Fig. 6.86

Solution

Case (i)

Single-V groove weld: In this case, incomplete penetration results due to single-V groove weld.

Hence, throat thickness

$$t_e = 5/8t = 5/8 \times 12 = 7.5 \text{ mm}$$

Effective length of weld

$$L_w = 150 \text{ mm}$$

Strength of weld $= L_w t_e f_y / \gamma_{mw} = 7.5 \times 150 \times (250/1.25)/1000$

$= 225 \text{ kN} < 350 \text{ kN}$

Hence the joint is not safe.

Case (ii)

In the case of double-V groove weld, complete penetration takes place

Throat thickness = thickness of thinner plate = 12 mm

Strength of weld $= 12 \times 150 \times (250/1.25)/1000$

$= 360 \text{ kN} > 350 \text{ kN}$

Hence, the joint is safe.

Example 6.7 *Determine the size and length of the fillet weld for the lap joint to transmit a factored load of 120 kN shown in Fig. 6.88, assuming site welds, Fe 410 steel, and E 41 electrode. Assume width of the plate as 75 mm.*

Solution

Minimum size of weld for a 8-mm thick section = 3 mm
Maximum size of weld = 8 – 1.5 = 6.5 mm
Choose the size of weld as 6 mm.
Effective throat thickness = 0.7 × 6 = 4.2 mm

Strength of weld = $4.2 \times 410/(\sqrt{3} \times 1.5) = 662.7$ N/mm
Assuming that there are only two longitudinal (side) welds,
Required length of weld = $120 \times 10^3/662.7$
= 181 mm
Length to be provided on each side = 181/2 = 90.5 mm > 75 mm
Hence, provide 90.5-mm weld on each side with an end return of 2 × 6 = 12 mm.
Therefore, the overall length of the weld provided = 2 × (90.5 + 2 × 6) = 205 mm

Example 6.8 *Rework Example 6.7 using 3 mm welds.*

Solution

Effective throat thickness = 0.7 × 3 = 2.1 mm

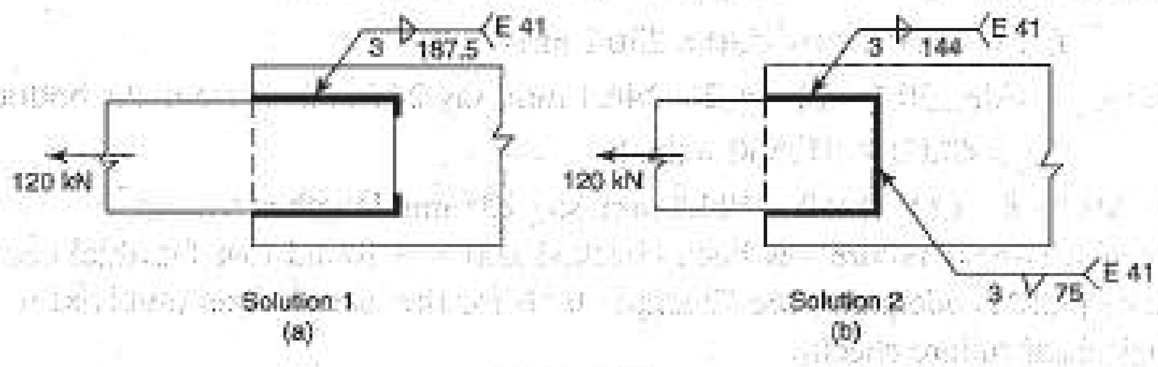
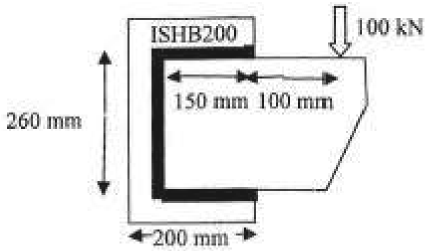
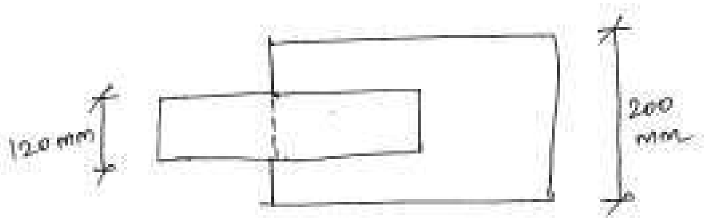


Fig. 6.89

(i) If only longitudinal welds are provided,
Length of each side weld = 363/2 = 181.5 mm
Total length on each side including end return = 181.5 + 2 × 3 = 187.5 mm
(ii) If welds are provided on three sides
Length of each side weld = (363 – 75)/2 = 144 mm > 75 mm (width of the plate)
The solution as shown in Fig. 6.89(b) is preferred since it is more compact, reduces the overall length of the connection, and provides better stress distribution.

<u>**Students must prepare answers for the following questions at the end of the lecture.**</u>

Qn No	Question	Marks	CO	Bloom's Knowledge

				Level
1	Design this welded connection to support the factored load of 100kN (April /May 2021)  	13	1	K3
2	Design the fillet weld for these 10mm plates of Grade Fe410 to transfer load equal to the bearing capacity of the smaller plate. (Nov/Dec 2021)  	13	1	K3

Reference Book:

- N Subramaniam , Design of Steel Structures, Oxford University press, New Delhi, 2011

Unit	1. INTRODUCTION TO STRUCTURAL STEEL AND DESIGN OF CONNECTIONS	Lecture No	07
Topic	Design of Welded Connections		
Learning Outcome (LO) At the end of this lecture, students will be able to		**Bloom's Knowledge Level**	
LO1	Design welded connections for steel members	K3	

Example 6.9 *A tie member of a truss consisting of an angle section ISA 65 × 65 × 6 of Fe 410 grade, is welded to an 8-mm gusset plate. Design a weld to transmit a load equal to the full strength of the member. Assume shop welding. See Fig. 6.90.*

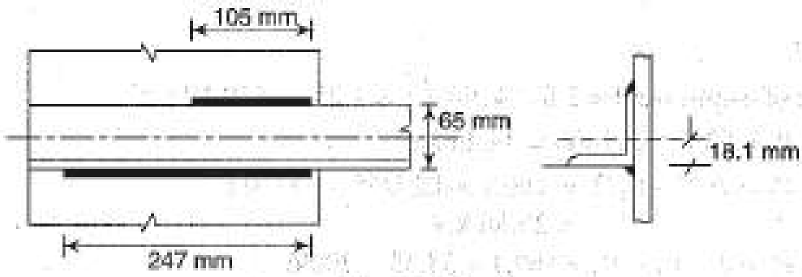


Fig. 6.90

Solution

From IS handbook no.1 or IS 808, the properties of ISA 65 × 65 × 6 are

$A = 744\ \text{mm}^2$

$C_z = 18.1$ mm

Tensile capacity of the member = 744 × 250/1.1 = 169.1 kN

The force resisted by the weld at the lower side of the angle

$P_1 = 169.1 \times (65 - 18.1)/65 = 122.01$ kN

Force to be resisted by the upper side of the angle

$P_2 = 169.1 \times 18.1/65 = 47.09$ kN

Assuming a weld size of 4 mm (> 3 mm).

Effective throat thickness of the weld = 0.7 × 4 = 2.8 mm

Strength of the weld = $2.8 \times 410/(\sqrt{3} \times 1.25) = 530$ N/mm

$L_{w1} = 122.01 \times 10^3/530 = 230.1$ mm

Hence, provide 230.1 + (2 × 4)2 = 246.1 mm, say 247 mm length at the bottom

$L_{w2} = 47.09 \times 10^3/530 = 88.8$

Provide 88.8 + (2 × 4) × 2 = 104.8 mm, say 105 mm length at the top.

The block shear failure has been checked and it is found that the thickness of gusset plate is adequate (see Example 6.10 for the calculations involved in the block shear failure check).

Example 6.10 *Design a joint according to the instructions given in Example 6.9, if the welding is done on three sides of the angle as shown in Fig. 6.91.*

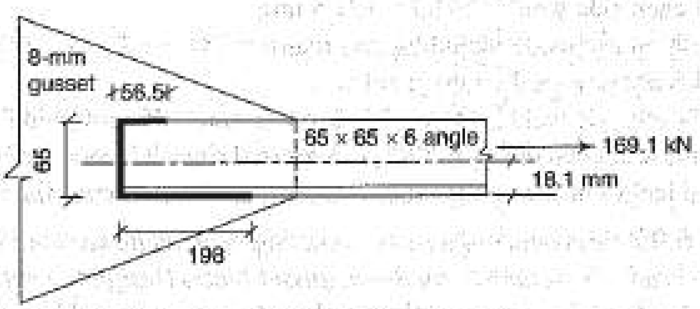


Fig. 6.91

Solution

Strength of 4-mm weld = $2.8 \times 410/(\sqrt{3} \times 1.25) = 530$ N/mm

$$P_2 = 530 \times 65/1000 = 34.45 \text{ kN}$$

$$P_1 = Ty/d - P_2/2 = 169.1 \times 18.1/65 - 34.45/2$$
$$= 29.86 \text{ kN}$$

$$P_3 = T - P_1 - P_2 = 169.1 - 34.45 - 29.86$$
$$= 104.79 \text{ kN}$$

$$L_{w1} = 29.86 \times 1000/530 = 56.3 \text{ mm, say } 56.5 \text{ mm}$$
$$L_{w3} = 104.79 \times 1000/530 = 197.7 \text{ mm, say } 198 \text{ mm}$$

Total length of weld = 65 + 56.5 + 198 = 319.5 mm

Check for block-shear failure

Since the member is welded to the gusset plate, no net areas are involved and hence A_{vn} and A_{tn} in the equation for T_{db} (Section 6.4.1 of the code) should be taken to be the corresponding gross areas. Using the weldment with L_1 = 198 mm, L_2 = 56.5 mm and 65 mm at the end of the angle yields

$$T_{db1} = [8 \times (198 \times 2) \times 250/(\sqrt{3} \times 1.1) + 0.9 \times 410 \times 8 \times 65/1.25]/1000$$
$$= 569.2 \text{ kN}$$

$$T_{db2} = [0.9 \times 410 \times (198 \times 2)/(\sqrt{3} \times 1.25) + 250 \times 8 \times 65/1.1]/1000$$
$$= 658.1 \text{ kN}$$

Hence,

$$T_{db} = 569.2 \text{ kN} > 169.1 \text{ kN}$$

Hence, the thickness of gusset plate is adequate.

Note L_2 does not enter into this calculation because a shear rupture of the gusset plate along the toe of the angle runs for the full length of the contact with the toe, 198 mm, instead of only the length L_2.

Example 6.11 *Design a suitable fillet weld to connect the web plate to the flange plate and the flange plate to the cover plate of the compression flange of a plate girder as shown in Fig. 6.92. The size of the web plate is 1000 × 10 mm. The flange and cover plate are of the dimensions 300 × 16 mm and 275 × 12 mm, respectively. The maximum factored shear force V = 800 kN. Assume shop welding.*

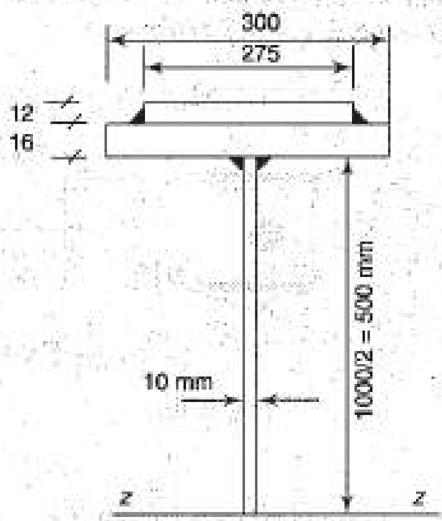


Fig. 6.92

Solution

Connection of web and flange plate

Minimum size of weld = 3 mm

Maximum size of weld = 10 – 1.5 = 8.5 mm

Adopt a 6-mm size weld

$$\tau_{vf,\,cal} = VA_f\bar{Y}_f/(I_{zz}\,\Sigma t)$$

$$t = 0.7 \times 6 = 4.2 \text{ mm}$$

$$\Sigma t = 2 \times 4.2 = 8.4 \text{ mm}$$

$$A_f\bar{Y}_f = 300 \times 16(500 + 8) + 275 \times 12(500 + 16 + 6)$$

$$= 243.84 \times 10^4 + 172.26 \times 10^4$$

$$= 416.1 \times 10^4$$

$$I_{zz} = 2 \times [275 \times 12^3/12 + 275 \times 12 \times 522^2 + 300 \times 16^3/12 + 300 \times 16 \times 508^2] + 10 \times 1000^3/12$$

$$= 5.109 \times 10^9 \text{ mm}^4$$

$$\tau_{vf,\,cal} = 800 \times 10^3 \times 416.1 \times 10^4/(5.109 \times 10^9 \times 8.4)$$

$$= 77.56 \text{ N/mm}^2 < 410/(\sqrt{3} \times 1.25) = 189 \text{ N/mm}^2$$

Hence, the weld is safe.

Connection of flange plate to cover plate

Adopt a 6-mm fillet weld.

$$t_e = 6 \times 0.7 = 4.2 \text{ mm}$$

$$\Sigma t = 2 \times 4.2 = 8.4 \text{ mm}$$

$$A\bar{Y} = 275 \times 12 \times (500 + 16 + 6) = 172.26 \times 10^4$$

$$\tau_{vf,\,cal} = 800 \times 10^3 \times 172.26 \times 10^4/(5.109 \times 10^9 \times 8.4)$$

$$= 32.11 \text{ N/mm}^2 < 189 \text{ N/mm}^2$$

Hence the weld is safe.

Students must prepare answers for the following questions at the end of the lecture.

Qn No	Question	Marks	CO	Bloom's Knowledge Level
1	A tie member of roof consists of ISA 100x75x8mm. The angles are connected to either side of a 10mm gusset plates and the member is subjected to a working pull of 300kN. Design the welded connection. Assume connections are made in the workshop (April /May 2022)	13	1	K3

Reference Book:

- N Subramaniam , Design of Steel Structures, Oxford University press, New Delhi, 2011

Unit	1. INTRODUCTION TO STRUCTURAL STEEL AND DESIGN OF CONNECTIONS	Lecture No	08
Topic	Types of Failure and Efficiency of Joint in Steel		
Learning Outcome (LO) At the end of this lecture, students will be able to		**Bloom's Knowledge Level**	
LO1	Understand the concept of EDM	K2	
LO2	Know the components and usage of a Total Station	K1	
LO3	Differentiate the abilities of a total station from a theodolite	K2	
LO4	Understand the applications of a Total Station	K2	

According to IS (Indian Standard) code specifications, particularly IS 800:2007, several types of failures can occur in steel members. These failures are classified based on the limit states considered in structural design. The primary types of failures in steel members as per IS code include:

1. **Ultimate Limit State (ULS) Failures:**
   - **Strength Failure:** Occurs when the applied loads exceed the ultimate capacity of the member, leading to collapse or failure. This can include yielding, buckling, or rupture of the steel material.
   - **Stability Failure:** Occurs when the member loses stability due to buckling or lateral-torsional instability under compressive or bending loads. It includes Euler buckling, flexural-torsional buckling, and local buckling.
2. **Serviceability Limit State (SLS) Failures:**
   - **Deflection Failure:** Occurs when the deflection of the member under service loads exceeds allowable limits, resulting in excessive deformations that may affect functionality or aesthetics.
   - **Vibration Failure:** Occurs when the member experiences excessive vibrations under service loads, leading to discomfort, damage to sensitive equipment, or functional issues.
3. **Fatigue Failure:**
   - Occurs due to repeated cyclic loading and unloading, leading to progressive damage and eventual failure of the member. Fatigue failure is a concern in members subjected to fluctuating or cyclic loads, such as bridges, crane structures, and machine components.
4. **Connection Failures:**
   - **Bolt Shear Failure:** Occurs when the shear force acting on bolts exceeds their capacity, causing them to shear off.

- **Bolt Bearing Failure:** Occurs when the bearing stress between the bolt and the connected plates exceeds the bearing strength of the material, leading to crushing or deformation.
- **Weld Failure:** Occurs when the welded connections between steel members fail due to inadequate welding, material defects, or excessive loads.

5. **Corrosion-Related Failures:**

- Occurs due to the degradation of steel material caused by corrosion in aggressive environments. Corrosion can weaken the structural integrity of steel members over time, leading to premature failure.

Failures in Steel Members in **Tension:**

1. **Yield Failure:**

- **Definition:** Yield failure occurs when a steel member reaches its yield point, the stress at which it undergoes significant plastic deformation without an increase in load. At this point, the material has yielded or deformed permanently.
- **Mechanism:** Steel exhibits elastic behavior up to its yield point, where it transitions to plastic deformation. Beyond the yield point, additional applied loads cause the material to deform plastically until it reaches its ultimate strength.
- **Significance:** Yield failure is a critical consideration in structural design because it marks the onset of permanent deformation in the member. Designers must ensure that the applied loads do not exceed the yield strength of the steel to prevent excessive plastic deformation and structural instability.
- **Consequences:** Excessive plastic deformation due to yield failure can lead to structural deformations, deflections, or even collapse if not properly accounted for in the design.

2. **Rupture Failure:**

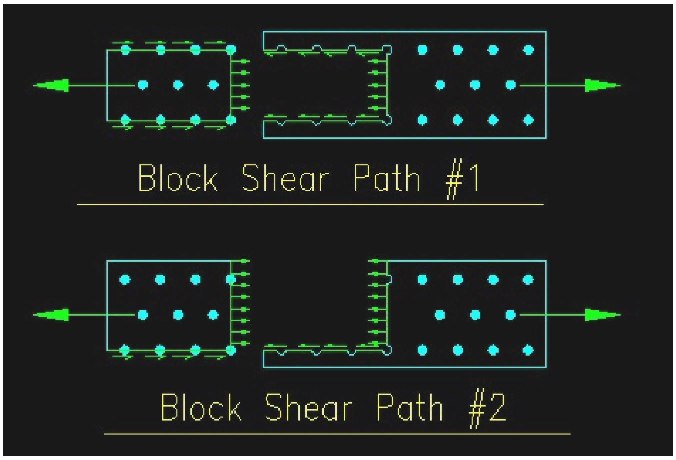
- **Definition:** Rupture failure, also known as ultimate failure, occurs when a steel member fails catastrophically under excessive loads, resulting in the material fracturing or tearing apart.

- **Mechanism:** Rupture failure occurs after the steel has undergone significant plastic deformation and has reached its ultimate strength. At this point, the applied loads exceed the material's capacity to resist further deformation, leading to fracture along the weakest section of the member.

- **Significance:** Rupture failure represents the ultimate limit state in structural design, indicating the maximum load-carrying capacity of the member. Designers must ensure that structures are capable of withstanding loads without experiencing rupture failure under normal service conditions.

- **Consequences:** Rupture failure can result in sudden and catastrophic structural collapse, posing significant safety risks to occupants and nearby structures. Therefore, it is essential to design structures with sufficient strength and redundancy to prevent rupture failures.

3. **Block Shear Failure:**



- **Definition:** Block shear failure occurs in steel members subjected to combined tension and shear stresses, typically near bolted or welded connections.
- **Mechanism:** Block shear failure involves the simultaneous failure of the connected plates due to tension along one plane and shear along another plane. This failure mode typically occurs at the gross or net section of the member, where the forces are transferred through the connection.
- **Significance:** Block shear failure is a critical consideration in the design of connections, as it can compromise the integrity of the entire structure. Designers must ensure that connections are adequately designed to resist block shear failure under the applied loads.
- **Consequences:** Block shear failure can lead to partial or complete detachment of the connected plates, compromising the structural stability and load-carrying capacity of the connection. This can result in localized or widespread structural failure, depending on the severity of the failure mode.

Efficiency of Steel Joint

1. **Load Transfer Efficiency:**
   - The primary function of a steel joint is to transfer loads between connected members without compromising structural integrity. IS 800 provides guidelines for designing joints to ensure effective load transfer while considering factors such as applied loads, member geometry, and connection details.
   - The efficiency of load transfer in a joint is assessed based on the ability of the connection to resist various loading conditions, including axial forces, bending moments, shear forces, and torsional loads. Designers must ensure that joints are capable of transferring these loads safely and efficiently to prevent structural failure.
2. **Connection Type and Configuration:**
   - IS 800 specifies different types of steel connections, including bolted, welded, and riveted joints, each with its own efficiency considerations.
   - Bolted connections are commonly used in steel structures due to their ease of installation, adjustability, and versatility. IS 800 provides guidelines for selecting appropriate bolt sizes, spacings, and configurations to ensure efficient load transfer and structural performance.
   - Welded connections offer high efficiency in load transfer and are often preferred for their strength and durability. However, proper welding procedures and inspection practices must be followed to achieve reliable and efficient welded joints as per IS code requirements.
3. **Efficiency Factors:**
   - IS 800 specifies factors that influence the efficiency of steel joints, including material properties, connection detailing, fabrication quality, and installation practices.

- Material properties, such as the yield strength and ductility of the steel, directly affect the capacity and behavior of joints under load. Designers must select appropriate materials and grades to ensure efficient joint performance.
- Connection detailing, including geometry, dimensions, and clearances, plays a critical role in the efficiency of joints. Proper detailing ensures that loads are distributed evenly and that stress concentrations and potential failure modes are minimized.
- Fabrication quality, including the accuracy of hole drilling, bolt tightening, welding procedures, and surface preparation, is essential for achieving efficient and reliable joints. IS 800 provides guidelines for quality control measures to ensure that fabricated connections meet specified standards.
- Installation practices, such as proper alignment, tightening torque, and inspection procedures, are crucial for ensuring the integrity and efficiency of steel joints in service. Adherence to recommended installation practices outlined in IS 800 helps prevent joint failure due to installation errors or deficiencies.

4. **Limit State Design Approach:**

- IS 800 adopts the limit state design approach to ensure the efficiency and safety of steel joints under various limit states, including ultimate and serviceability limit states.
- The design of steel joints considers factors such as ultimate strength, stability, fatigue resistance, and serviceability requirements to ensure that connections perform satisfactorily throughout their design life.
- Designers use limit state design principles, partial safety factors, and appropriate load combinations specified in IS 800 to assess the efficiency of steel joints and ensure compliance with code requirements.

<u>Assessment questions to the lecture</u>

Qn No	Question	Answer	Bloom's Knowledge Level
1	What type of failure occurs when the applied loads exceed the ultimate capacity of a steel member, leading to collapse or failure? a) Yield failure b) Rupture failure c) Block shear failure d) Stability failure	B	K1
2	Which factor does NOT influence the efficiency of steel joints according to IS 800? a) Material properties b) Connection detailing c) Concrete strength d) Fabrication quality	C	K1

3	Which limit state is primarily concerned with ensuring that the deflection of a steel member under service loads does not exceed allowable limits? a) Strength limit state b) Stability limit state c) Serviceability limit state d) Fatigue limit state	C	K1

Students must prepare answers for the following questions at the end of the lecture.

Qn No	Question	Marks	CO	Bloom's Knowledge Level
1	Explain the various failure patterns in steel sections.	13	1	K1
2	What Yield, Rupture and Block shear failure in steel? Explain with neat sketches.	13	5	K1

Reference Book:

- N Subramaniam , Design of Steel Structures, Oxford University press, New Delhi, 2011

Unit	1. INTRODUCTION TO STRUCTURAL STEEL AND DESIGN OF CONNECTIONS	Lecture No	09
Topic	Prying Action in Steel and HSFG Bolts		
Learning Outcome (LO) At the end of this lecture, students will be able to		**Bloom's Knowledge Level**	
LO1	Define the prying action in steel members	K1	
LO2	State the uses and properties of HSFG bolts.	K1	

Prying Action in Steel Elements:

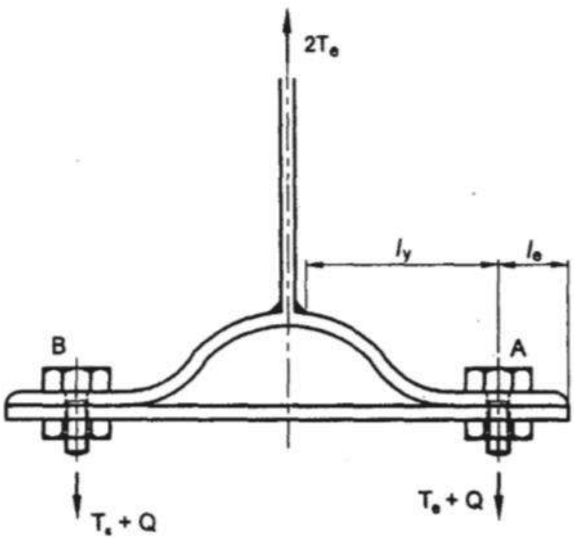


FIG. 16 COMBINED PRYING FORCE AND TENSION

Prying action is an important consideration in the design of bolted connections in steel structures, particularly in situations where the connected plates are subject to tension loads. It refers to the tendency of the connected plates to act as a lever, exerting additional force on the bolts and potentially causing them to fail due to shear or bearing. The concept of prying action is addressed in the Indian Standard (IS) code IS 800:2007, which provides guidelines for the design of steel structures in India.

Prying action typically occurs in bolted connections where the applied load induces tension in the connected plates, causing them to pull away from each other. As the plates separate, they create a moment arm, or lever, around the bolt line. This moment arm exerts an additional force on the bolts, which must be accounted for in the design of the connection to ensure its structural integrity.

The IS 800:2007 code outlines specific provisions for addressing prying action in bolted connections. These provisions are based on empirical formulas and design guidelines derived from experimental testing and analytical studies. The key aspects of prying action considered in the code include:

1. **Calculation of Prying Force**: The code provides formulas for calculating the prying force exerted on the bolts due to the tension in the connected plates. This force is influenced by

factors such as the applied load, bolt diameter, plate thickness, and geometry of the connection.

2. **Resistance to Prying**: To ensure the structural adequacy of the connection, the resistance to prying action must be evaluated. This involves assessing the capacity of the bolts to resist the additional force induced by prying and determining whether the connection meets the required safety factors.

3. **Bolt Strength and Capacity**: The code specifies minimum requirements for the strength and capacity of bolts used in bolted connections subject to prying action. This includes considerations for the material properties of the bolts, such as their yield strength, ultimate strength, and ductility.

4. **Connection Detailing and Geometry**: Proper detailing and geometry of the bolted connection are crucial for mitigating prying action and ensuring efficient load transfer. The code provides guidelines for the arrangement of bolts, spacing, edge distances, and other connection details to optimize resistance to prying forces.

5. **Factor of Safety**: Designing bolted connections to resist prying action requires applying appropriate safety factors to account for uncertainties in material properties, loading conditions, and other factors. The code specifies partial safety factors to be used in the design process to ensure adequate resistance to prying forces.

HSFG Bolts:

1. **Material Properties**: HSFG bolts are manufactured from high-strength alloy steels with specific mechanical properties to withstand tensile loads encountered in structural applications. According to IS 3757:1985, HSFG bolts must conform to the material requirements specified in relevant Indian Standards, such as IS 1367 for high-strength structural bolts.

The material used for HSFG bolts typically has a minimum tensile strength ranging from 800 MPa to 1000 MPa, ensuring sufficient capacity to resist applied loads without yielding or failure. Additionally, the material composition and heat treatment processes are controlled to achieve the desired mechanical properties, including yield strength, ultimate tensile strength, and elongation.

2. **Design Considerations**: Designing HSFG bolted connections involves determining the appropriate bolt size, grade, and spacing to accommodate the applied loads and ensure structural integrity. IS 3757:1985 provides guidelines for calculating the required number and size of HSFG bolts based on factors such as:

    - Applied loads: The magnitude and distribution of the loads acting on the connection, including tension, shear, and bending.
    - Connection geometry: The configuration and arrangement of the connected members, including thickness, material, and edge distances.
    - Load transfer mechanisms: The mechanisms by which loads are transferred between the connected members, considering factors such as friction, bearing, and prying action.
    - Serviceability requirements: Criteria related to deflection, vibration, and fatigue performance under service loads.

The design process typically involves performing structural analysis and load calculations to determine the optimal configuration and size of HSFG bolts to meet the specified design criteria and safety factors.

3. **Installation Procedures**: Proper installation of HSFG bolts is essential for ensuring the integrity and performance of bolted connections. IS 3757:1985 provides detailed requirements for the installation procedures, including:

    - Bolt tightening: HSFG bolts are tightened to a specified torque or tension level using calibrated torque wrenches or tensioning equipment. The tightening procedure must be carried out sequentially and uniformly to achieve the desired preload in each bolt.
    - Surface preparation: The mating surfaces of the connected members must be clean, flat, and free from contaminants to ensure optimal frictional resistance and load transfer.
    - Joint assembly: The bolts, nuts, and washers are assembled with proper alignment and orientation to facilitate uniform loading and prevent eccentricities or misalignments.
    - Inspection and testing: Visual and dimensional inspections are performed to verify the quality and condition of the bolts, including dimensions, thread condition, and surface finish. Additionally, non-destructive testing methods such as ultrasonic or magnetic particle inspection may be employed to detect defects or anomalies.

4. **Quality Control Measures**: IS 3757:1985 specifies quality control measures to ensure the reliability and performance of HSFG bolts in service. These measures include:

- Material testing: Incoming raw materials are subjected to chemical composition analysis, mechanical testing, and heat treatment to verify compliance with specified standards and requirements.

- Manufacturing processes: Bolt manufacturing processes, including forging, machining, and heat treatment, are monitored and controlled to maintain dimensional accuracy, surface finish, and mechanical properties.

- Batch testing: Random samples of HSFG bolts are subjected to batch testing, including tensile testing, hardness testing, and dimensional inspection, to validate conformance with the specified standards and performance criteria.

- Certification and documentation: Manufacturers provide certification and documentation for HSFG bolts, including material test certificates, inspection reports, and compliance statements, to verify product quality and traceability.

ED

Torque for HT / HSFG Bolts

Torque values are in "Nm"

Bolt Size	8.8	10.9	12.9	Bolt Size	8.8	10.9	12.9
M2	0.373	0.520	0.628	M30	1422.0	2010.0	2403.0
M2.3	0.598	0.843	1.010	M33	1932.0	2716.0	3266.0
M2.6	0.863	1.206	1.451	M36	2481.0	3491.0	4197.0
M3	1.344	1.883	2.256	M39	3226.0	4531.0	5443.0
M3.5	2.060	2.893	3.481	M42	3991.0	5609.0	6727.0
M4	3.040	4.315	5.148	M45	4992.0	7012.0	8414.0
M5	6.031	8.483	10.200	M48	6021.0	8473.0	10150
M6	10.300	14.710	17.625	M52	7747.0	10885	13092
M7	17.162	24.517	28.439	M56	9650.0	13582	16279
M8	25.497	35.304	42.168	M60	11964	16867	20202
M10	50.014	70.608	85.317	M64	14416	20300	24320
M12	87.279	122.60	147.10	M68	17615	24771	29725
M14	138.30	194.20	235.40	M72	21081	29645	35575
M16	210.80	299.10	357.90	M76	24973	35118	42141
M18	289.30	411.90	490.30	M80	29314	41222	49467
M20	411.90	578.60	696.30	M90	42525	59801	71761
M22	559.00	784.50	941.40	M100	59200	83250	99900
M24	700.00	1000.0	1196.0				
M27	1049.0	1481.0	1775.0				

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Assessment questions to the lecture

Qn No	Question	Answer	Bloom's Knowledge Level
1	What is prying action in bolted connections? a) The tendency of connected plates to act as a lever, exerting additional force on the bolts b) The resistance of bolts to shear and bearing stresses c) The ability of bolts to withstand tension loads only d) The process of tightening bolts to a specified torque level	a	K1
2	According to IS 800:2007, what factors influence the prying force exerted on bolts in bolted connections? a) Applied load, bolt diameter, plate thickness, and connection geometry b) Material composition and heat treatment processes c) Surface preparation and joint assembly d) Material testing and manufacturing processes	a	K1
3	What is the purpose of HSFG bolts in bolted connections? a) To provide resistance to prying action b) To increase the efficiency of load transfer c) To facilitate easy installation and maintenance d) To withstand tensile loads encountered in structural applications	D	K1

Students must prepare answers for the following questions at the end of the lecture.

Qn No	Question	Marks	CO	Bloom's Knowledge Level
1	How does prying action impact the design of bolted connections in steel structures, and what specific provisions does IS 800:2007 provide to address this phenomenon?	7	1	K1
2	Explain the significance of material properties, design considerations, installation procedures, and quality control measures associated with HSFG (High Strength Friction Grip) bolts in bolted connections, as per the guidelines outlined in IS 3757:1985.	7	1	K1

Reference Book:

- N Subramaniam , Design of Steel Structures, Oxford University press, New Delhi, 2011

Unit	2. DESIGN OF TENSION AND COMPRESSION MEMBERS	Lecture No	10
Topic	Design of Simple members in Tension		
Learning Outcome (LO) At the end of this lecture, students will be able to		**Bloom's Knowledge Level**	
LO1	Explain the behaviour of members subjected to Tension load	K2	
LO2	Design a Simple member subjected to tension	K3	

Tension Members: (Sec 6 IS 800:2007)

Tension members are linear members in which axial forces act to cause elongation (stretch). Such members can sustain loads up to the ultimate load, at which stage they may fail by rupture at a critical section. However, if the gross area of the member yields over a major portion of its length before the rupture load is reached, the member may become non-functional due to excessive elongation. Plates and other rolled sections in tension may also fail by block shear of end bolted regions

The factored design tension T, in the members shall satisfy the following requirement:

$$T < T_d$$

where

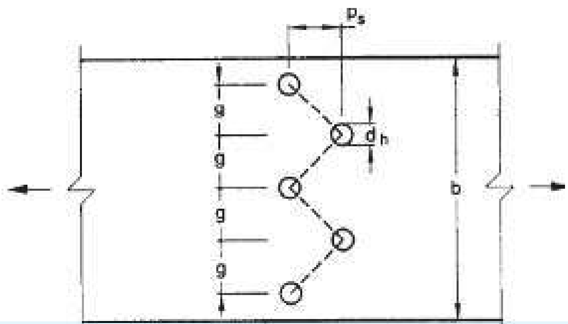
T_d = design strength of the member.

The design strength of a member under axial tension, T_d, is the lowest of the design strength due to yielding of gross section, T_{dg}; rupture strength of critical section, T_{dn}; and block shear T_{db}, given in 6.2, 6.3 and 6.4, respectively.

The tension members are checked for three major load conditions, viz.,

- Yielding
- Rupture
- Block Shear

The IS800:2007 code provisions for the above-mentioned conditions are:



6.2 Design Strength Due to Yielding of Gross Section

The design strength of members under axial tension, T_{dg}, as governed by yielding of gross section, is given by

$$T_{dg} = A_g f_y / \gamma_{m0}$$

where

f_y = yield stress of the material,

A_g = gross area of cross-section, and

γ_{m0} = partial safety factor for failure in tension by yielding (*see* Table 5).

6.3 Design Strength Due to Rupture of Critical Section

6.3.1 *Plates*

The design strength in tension of a plate, T_{dn}, as governed by rupture of net cross-sectional area, A_n, at the holes is given by

$$T_{dn} = 0.9 A_n f_u / \gamma_{m1}$$

where

γ_{m1} = partial safety factor for failure at ultimate stress (*see* Table 5),

f_u = ultimate stress of the material, and

A_n = net effective area of the member given by,

$$A_n = \left[b - n d_h + \sum_i \frac{p_{si}^2}{4 g_i} \right] t$$

where

b, t = width and thickness of the plate, respectively,

d_h = diameter of the bolt hole (2 mm in addition to the diameter of the hole, in case the directly punched holes),

g = gauge length between the bolt holes, as shown in Fig. 5,

p_s = staggered pitch length between line of bolt holes, as shown in Fig. 5,

n = number of bolt holes in the critical section, and

i = subscript for summation of all the inclined legs.

6.4 Design Strength Due to Block Shear

The strength as governed by block shear at an end connection of plates and angles is calculated as given in **6.4.1**.

6.4.1 *Bolted Connections*

The block shear strength, T_{db} of connection shall be taken as the smaller of,

$$T_{db} = [A_{vg} f_y / (\sqrt{3}\,\gamma_{m0}) + 0.9 A_{tn} f_u / \gamma_{m1}]$$

or

$$T_{db} = (0.9 A_{vn} f_u / (\sqrt{3}\,\gamma_{m1}) + A_{tg} f_y / \gamma_{m0})$$

where

A_{vg}, A_{vn} = minimum gross and net area in shear along bolt line parallel to external force, respectively (1-2 and 3-4 as shown in Fig. 7A and 1-2 as shown in Fig. 7B),

A_{tg}, A_{tn} = minimum gross and net area in tension from the bolt hole to the toe of the angle, end bolt line, perpendicular to the line of force, respectively (2-3 as shown in Fig. 7B), and

f_u, f_y = ultimate and yield stress of the material, respectively.

Example 7.4 *Determine the design tensile strength of plate (160 × 8 mm) connected to 10-mm thick gusset using 16-mm bolts, as shown in Fig. 7.27, if the yield and the ultimate stress of the steel used are 250 MPa and 410 MPa, respectively.*

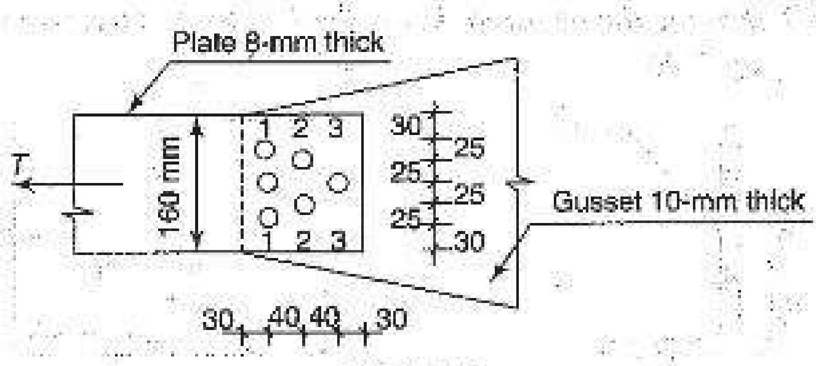


Fig. 7.27

Solution

$f_y = 250$ MPa

$f_u = 410$ MPa

Calculation of net area

A_n (Path 11) = $(160 - 3 \times 18) \times 8 = 848$ mm^2

A_n (Path 1221) = $[(160 - 4 \times 18) + (2 \times 40^2)/(4 \times 25)] \times 8 = 960$ mm^2

A_n (Path 12321) = $[(160 - 5 \times 18) + (4 \times 40^2)/(4 \times 25)] \times 8 = 1072$ mm^2

Factored design tension in member by

(i) Yielding of gross section,

$$T_{dg} = [f_y \times A_g/\gamma_{m0}]$$

$$= [250 \times (160 \times 8)/1.10] \times 10^{-3} = 290.9 \text{ kN}$$

(ii) Rupture of net section

$$T_{dn} = (0.9 \times f_u \times A_n/\gamma_{m1})$$

$$= (0.9 \times 410 \times 848/1.25) \times 10^{-3} = 250.33 \text{ kN}$$

Therefore, the design tensile strength of the plate = 250.33 kN

Check for minimum edge distance

Provided edge and end distance = 30 mm > 1.5 × 18 = 27 mm.

Hence, the edge distance is as required.

Example 7.6 *Determine the tensile strength of a roof truss diagonal 100 × 75 × 6 mm (f_y = 250 MPa) connected to the gusset plate by 4-mm welds as shown in Fig. 7.29.*

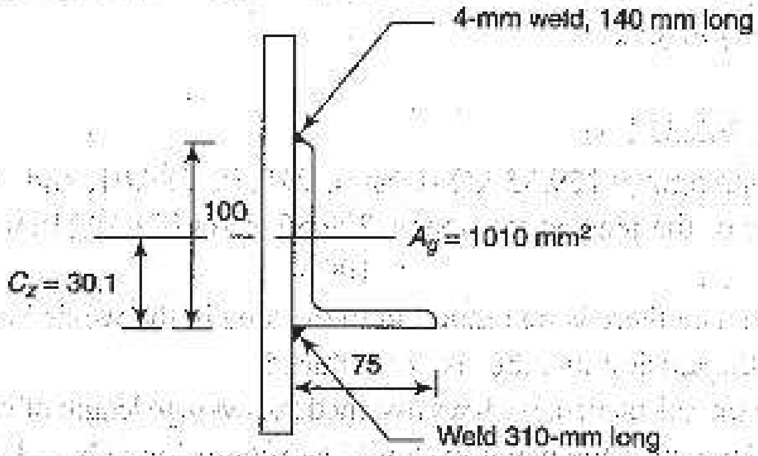


Fig. 7.29

Solution

Area of the connected leg = (100 – 6/2) × 6 = 582 mm²

Area of the outstanding leg = (75 – 6/2) × 6 = 432 mm²

$$A_g = 1010 \text{ mm}^2$$

(a) Strength governed by yielding of cross section

$$T_{dg} = A_g f_y/\gamma_{m0} = (1010 \times 250/1.10) \times 10^{-3} = 229.55 \text{ kN}$$

(b) Strength governed by rupture of critical section

$$T_{dn} = 0.9 f_u A_{nc}/\gamma_{m1} + \beta A_{go} f_y/\gamma_{m0}$$

Assuming average length of weld L_w = 225 mm

$$\beta = 1.4 - 0.076(w/t)(f_y/f_u)(b_s/L_w)$$
$$= 1.4 - 0.076[(75 - 3)/6](250/410)(75/225)$$
$$= 1.215$$

Hence,

$$T_{dn} = [0.9 \times 410 \times 582/1.25 + 1.215 \times 432 \times 250/1.10] \times 10^{-3}$$
$$= 291.1 \text{ kN}$$

Alternatively,

$$T_{dn} = \alpha A_n f_u/\gamma_{m1} = [0.8(1010) \times 410/1.25] \times 10^{-3}$$
$$= 265.02 \text{ kN}$$

Hence, T_{dn} = 291.1 kN

(c) Strength governed by block shear

Since the member is welded to the gusset plate, no net areas are involved and hence A_{vn} and A_{tn} in the equation for T_{db} (Section 6.3.1 of the code) should be taken as the corresponding gross areas (Gaylord et al. 1992). Assuming average length of the weld on each side as 225 mm and the gusset plate thickness as 8 mm,

$$T_{db1} = [8 \times (225 \times 2) \times 250/(\sqrt{3} \times 1.1) + 0.9 \times 410 \times 8 \times 100/1.25] \times 10^{-3}$$
$$= 708.53 \text{ kN}$$
$$T_{db2} = [0.9 \times 410 \times 8 \times 225 \times 2/(\sqrt{3} \times 1.25) + 250 \times 8 \times 100/1.1] \times 10^{-3}$$
$$= 798.38 \text{ kN}$$

Hence,

$$T_{db} = 708.53 \text{ kN}$$

Thus, tensile strength = 229.55 kN (least of 229.55, 285.01, and 708.53)

The efficiency of the tension member = 229.55 × 1000 × 10/(1010 × 250/1.10)
= 100%

It is clear that since there is no reduction in the area in the welded connection, the efficiency of the tension member is not reduced.

Note that in the calculation, we have assumed the average length of weld as 225 mm on each side. However, the welding should be proportioned based on the position of the neutral axis.

Thus, for the tensile capacity = 229.55 kN, with capacity of 4-mm weld = 0.530 kN/mm

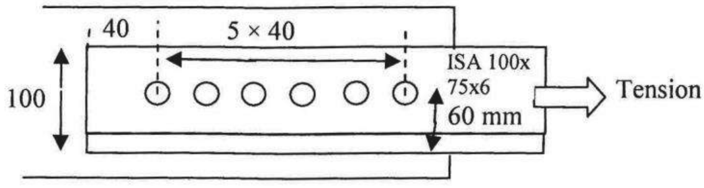
Length of the weld at the upper side of the angle

= (229.55 × 30.1/100)/0.530 = 130 mm, say 140 mm

Length of the weld at the bottom side of the angle

= [229.55 × (100 − 30.1)/100]/0.530 = 302 mm, say 310 mm

Students must prepare answers for the following questions at the end of the lecture.

Qn No	Question	Marks	CO	Bloom's Knowledge Level
1	An ISA 50 × 50 × 6 is to be used for tension member 4 m long. The member will always be in tension. Check whether the slenderness ratio satisfies the limit specified in code.(April May 2021)	2	2	K3
2	A single unequal angle isa 100 × 75 × 6 is connected to a 10 mm thick gusset plate at the ends with six 18 mm diameter holes (for bolts) to transfer tension. Determine the block shear strength of the angle specimen assuming 100 mm leg is connected to gusset plate. .(April May 2021) 	13	2	K3

Reference Book:

- N Subramaniam , Design of Steel Structures, Oxford University press, New Delhi, 2011

Unit	2. DESIGN OF TENSION AND COMPRESSION MEMBERS	Lecture No	11
Topic	Design of Built Up members in Tension		
Learning Outcome (LO) At the end of this lecture, students will be able to		**Bloom's Knowledge Level**	
LO1	Design a Built-up member subjected to tension	K3	

Example 7.8 *A tie member in a bracing system consists of two angles 75 × 75 × 6 bolted to a 10-mm gusset, one on each side using a single row of bolts [See Fig. 7.30(a)] and tack bolted. Determine the tensile capacity of the member and*

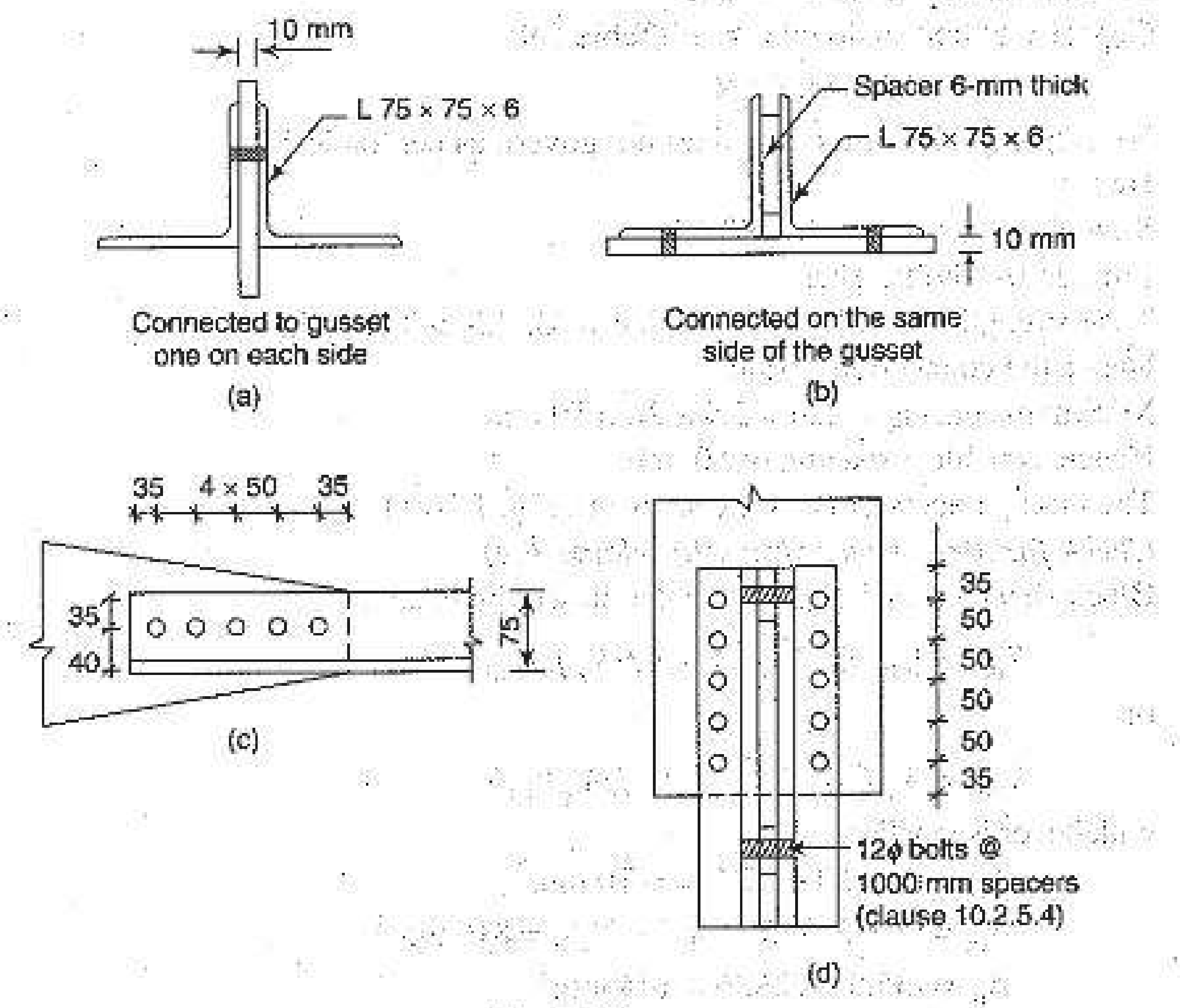


Fig. 7.30

the number of bolts required to develop full capacity of the member. What will be the capacity if the angles are connected on the same side of the gusset plate and tack bolted [Fig. 7.30(b)]? What is the effect on tensile strength if the members are not tack bolted?

Solution

(a) Two angles connected to opposite side of the gusset as in Fig. 7.30(a)

(i) Design strength due to yielding of gross section $T_{dg} = f_y(A_g/\gamma_{m0})$

$A_g = 866$ mm^2 (for a single angle)

$T_{dg} = 250 \times 2 \times (866/1.10) \times 10^{-3}$

$T_{dg} = 393.64$ kN

(ii) The design strength governed by tearing at net section

$T_{dn} = \alpha A_n(f_u/\gamma_{m1})$

Assume a single line of four numbers of 20-mm-diameter bolts ($\alpha = 0.8$)

$A_n = [(75 - 6/2 - 22)6 + (75 - 6/2)6]2$

$A_n = (300 + 432)2 = 1464$ mm^2

$T_{dn} = (0.8 \times 1464 \times 410/1.25) = 384.15$ kN

Therefore,

Tensile capacity = 384.15 kN

Design of bolts

Choose edge distance = 35 mm

Capacity of bolt in double shear (Table 5.9)

$= 2 \times 45.3 = 90.6$ kN

Bearing capacity of the bolt does not govern as per Table 5.9.

Hence,

Strength of a single bolt = 90.6 kN

Provide five bolts. Then,

Total strength of the bolts = $5 \times 90.6 = 453$ kN > 384.15 kN

Hence the connection is safe.

Minimum spacing = $2.5t = 2.5 \times 20 = 50$ mm

Hence, provide a spacing of 50 mm.

The arrangements of bolts are shown in Fig. 7.30(c).

Check for block shear strength: (clause 6.4)

Block shear strength T_{db} of connection will be taken as

$$T_{db1} = [A_{vg}f_y/\sqrt{3}\gamma_{m0}) + (0.9A_{tn}f_u/\gamma_{m1})]$$

or

$$T_{db2} = 0.9f_u A_{vn}/\sqrt{3}\gamma_{m1} + (f_y A_{tg}/\gamma_{m0})$$

whichever is smaller.

$A_{vg} = (4 \times 50 + 35)6 = 1410$ mm^2

$A_{vn} = (4 \times 50 + 35 - 4.5 \times 22)6 = 816$ mm^2

$$T_{db1} = \{[(1410 \times 250)/(\sqrt{3} \times 1.10)] + [(0.9 \times 144 \times 410)/1.25]\} \times 10^{-3}$$
$$= 227.5 \text{ kN}$$

$$T_{db2} = \{[(0.9 \times 410 \times 816)/(\sqrt{3} \times 1.25)] + [(250 \times 210)/1.10]\} \times 10^{-3}$$
$$= 186.8 \text{ kN}$$

For double angle,
block shear strength = 2 × 186.8 = 373.6 kN
Therefore,
Tensile capacity = 373.6 kN (least of 393.64 kN, 384.14 kN, and 373.6 kN)

(b) Two angles connected to the same side of the gusset plate [Fig. 7.30(b)]

(i) Design strength due to yielding of the gross section = 393.64 kN
(ii) Design strength governed by tearing at the net section = 384.14 kN
Assuming ten bolts of 20 mm diameter, five bolts in each connected leg
Capacity of an M20 bolt in single shear = 45.3 kN
Total strength of bolts = 10 × 45.3 = 453 kN > 393.64 kN
Hence the connection is safe.

The arrangement of bolts is shown in Fig. 7.30(d). Since it is similar to the arrangement in Fig. 7.30(c), the block shear strength will be the same, i.e., 373.6 kN.

Hence, the tensile capacity = 373.6 kN

The tensile capacity of both the arrangements (angles connected on the same side and connected to the opposite side of gusset) are same, as per the code though the load application is eccentric in this case. Moreover, the number of bolts are ten whereas in case (a) we used only five bolts since the bolts were in double shear.

(c) If the angles are not tack bolted, they behave as single angles connected to gusset plate.

In this case also the tensile capacity will be the same and we have to use ten M20 bolts. This fact is confirmed by the test and FEM results of Usha (2003), stating that 'The net section strength of double angles on opposite sides of the gusset and tack connected adequately over the length is nearly the same as that of two single angles acting individually. Current design provisions indicating greater efficiency of such double angles are not supported by the test and FEM results.'

Example 7.10 *A tension member in a bridge structure 10-m long is subjected to an axial tensile (factored) load of 1800 kN. Design the section with channels facing each other (see Fig. 7.32). Assume f_u = 410 MPa and f_y = 250 MPa.*

Solution

Required area = 1.1 × 1800 × 1000/250 = 7200 mm²

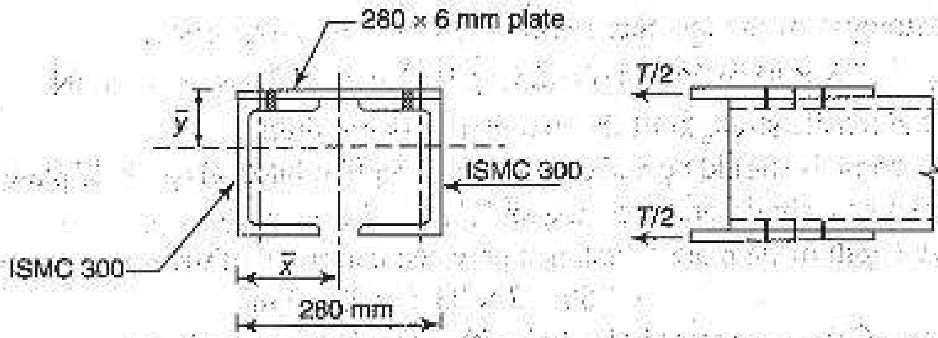


Fig. 7.32

Select two channels ISMC 300 (36.3 kg/m) each with the following properties

A = 4630 mm²; B = 90 mm, g = 50 mm

I_{zz} = 6420 × 10⁴ mm⁴

I_{yy} = 313 × 10⁴ mm⁴

t_f = 13.6 mm; t_w = 7.8 mm

r_{zz} = 118 mm

C_y = 23.5 mm

Assuming 16-mm-diameter bolts for the connection,

Diameter of hole = 16 + 2 = 18 mm

$A_g = 2 \times 4630 = 9260 \text{ mm}^2$

Assuming staggered bolts, deduction of two holes will be made in the calculation of the net area. Net area is calculated as per Section 7.9.

Net area provided = gross area – area of bolt holes – (0.5 × web area)

$= 9260 - 2 \times 18 \times 13.6 - 0.5\,[2 \times (300 - 2 \times 13.6) \times 7.8]$

$= 8770.4 - 2127.84 = 6642.56 \text{ mm}^2 < 7200 \text{ mm}^2$

As the net area provided by the two channels is less than the net area required, we have to provide additional cover plates.

Required net area of cover plates = 7200 – 6642.56 = 557.44 mm²

Let us provide an additional plate of size 280 × 6 mm with area 1680 mm² and arrange two channels as shown in Fig. 7.32.

$$\bar{y} = \frac{2 \times 4630(150 + 6) + 280 \times 6 \times 3}{2 \times 4630 + 280 \times 6} = 132.5 \text{ mm}$$

$$I_{zz} = 2 \times 6420 \times 10^4 + 280 \times 6^3/12 + 280 \times 6(132.5 - 6/2)^2 + 2 \times 4630 \times (156 - 132.5)^2$$

$$= 161.7 \times 10^6 \text{ mm}^4$$

$$A = 2 \times 4630 + 280 \times 6 = 10940 \text{ mm}^2$$
$$I_{yy} = 2[313 \times 10^4 + 4630(280/2 - 23.5)^2] + 6 \times 280^3/12$$
$$= 142.91 \times 10^6 \text{ mm}^4$$
$$r_{min} = \sqrt{(142.91 \times 10^6/10940)} = 114.3 \text{ m}$$
$$\lambda = l/r = 10 \times 1000/114.3 = 87.49 < 180$$

Hence λ is within allowable limit.

$$\text{Net area} = 2 \times 4630 + 280 \times 6 - 2[18 \times 13.6 + 18 \times 6]$$
$$- 0.5[2 \times (300 - 2 \times 13.6) \times 7.8]$$
$$= 8106.56 \text{ mm}^2$$

Tensile strength of the member (assuming more than four bolts)

$$= (0.8 \times 8106.56 \times 410/1.25) \times 10^{-3} = 2127.2 \text{ kN} > 1800 \text{ kN}$$

Hence, the tensile strength of the member is as required.

The two channels should be tied effectively at regular intervals with tie plates so as to function as a single unit (see clause 7.7.2.3 of the code)

Effective depth of tie plate = distance between centroids of the main components

$$= 280 - 2 \times 23.5 = 233 \text{ mm}$$

Assuming 16-mm-diameter bolts and with edge distance = 30 mm

Overall depth = $233 + 2 \times 30 = 293 \text{ mm} \approx 300 \text{ mm}$

Length of the plate = width of the member = 280 mm

Thickness of tie plate = (1/50)(distance between innermost connecting line of bolts)

$$= (1/50)(280 - 2 \times 50) = 3.6 \text{ mm}$$

Provide (300 × 280 × 6)-mm tie plates.

Students must prepare answers for the following questions at the end of the lecture.

Qn No	Question	Marks	CO	Bloom's Knowledge Level
1	A built-up tension member consists of two L-shaped angles connected by a gusset plate. The member is required to carry a tensile load of 120 kN. Determine the minimum size of the angles and gusset plate required if the allowable tensile stress is 150 MPa. Assume a bolt diameter of 16 mm and a bolt spacing of 50 mm. (April May 2023)	13	2	K3
2	Discuss the design of built-up tension members. (April May 2023)	7	2	K2

Reference Book:

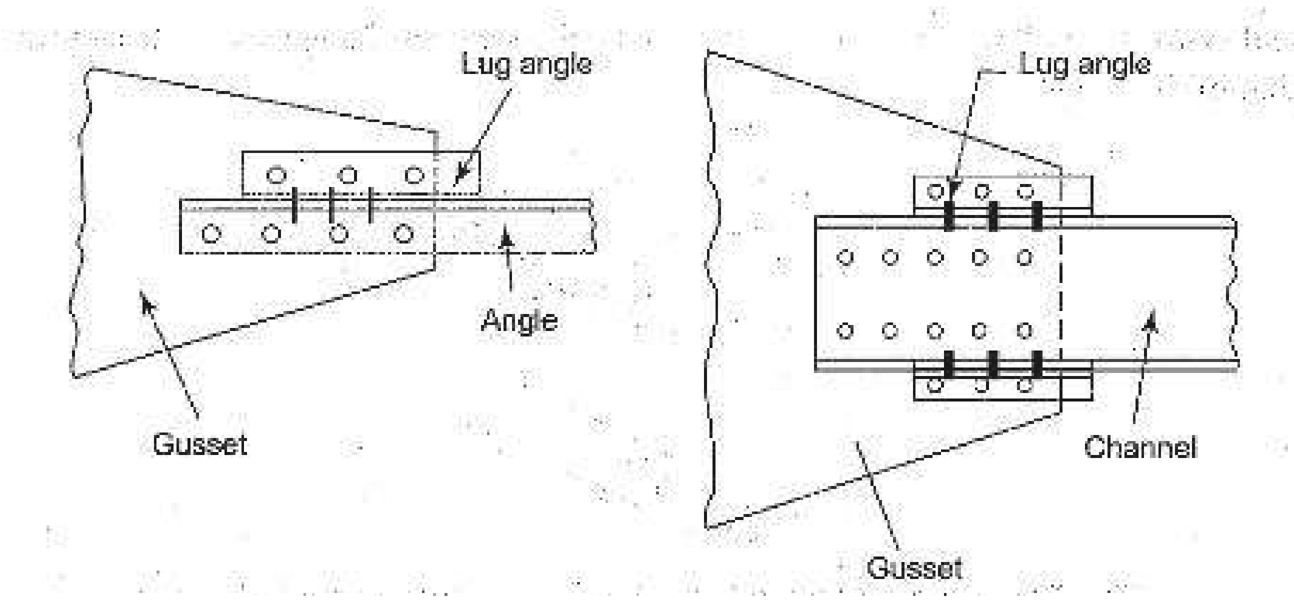
- N Subramaniam , Design of Steel Structures, Oxford University press, New Delhi, 2011

Unit	2. DESIGN OF TENSION AND COMPRESSION MEMBERS	Lecture No	12
Topic	Design of Lug Angles		
Learning Outcome (LO) At the end of this lecture, students will be able to		**Bloom's Knowledge Level**	
LO1	Design Lug Angles for Tension members	K3	

Lug Angles:

When a tension member is subjected to heavy load, the number of bolts or the length of weld required for making a connection with other members becomes large; resulting in uneconomical size of the gusset plates. In such situations, an additional short angle may be used to reduce the joint length and shear lag as shown in Fig. 7.20. Such an angle is called the lug angle.

The location of the lug angle is of some importance; it is more effective at the beginning of the connection, as in Fig. 7.20, rather than at the end. The use of lug angles with angles or channels reduces the net area of the main members due to the additional bolt holes in projected members. This reduction in the net area of the member should not be excessive.



In the connections of the lug angles to the member or the gusset plate more than two bolts are used. Since both legs of the angles or channels are connected to the lug angles, the net area of the members should be calculated simply as gross area minus the area of the holes.

Lug angles may be eliminated by providing unequal angle sections with the wider leg as the connected leg and using two rows of staggered bolts. In many cases, the cost of providing the lug angles (including their connection and the extra fabrication required to make the holes) may be found to be expensive than providing extra length and thickness of gusset plate. Hence, they are not used in practice.

Example 7.12 *A diagonal member of a roof carries an axial tension of 450 kN. Design the section and its connection with a gusset plate and lug angle. Use f_y = 250 MPa and f_u = 410 MPa.*

Solution

Factored tensile load = 450 kN

Required net area of section $= T_u\gamma_{m1}/(0.9f_u)$

$= 450 \times 1000 \times 1.25/(0.9 \times 410)$

$= 1524\ \text{mm}^2$

Choose ISA 150 × 75 × 10 with A = 2160 mm², r_{vv} = 16.1 mm

Providing 20-mm-diameter bolts; strength of a bolt in single shear = 45.3 kN

(Strength in bearing will not govern.)

Required number of bolts = 450/45.3 ≈ 10

Using a pitch of 2.5 × 20 = 50 mm and an edge distance of 30 mm

Length of gusset plate = 9 × 50 + 2 × 30 = 510 mm

Area of connected leg $A_{nc} = [150 - 22 - (10/2)] \times 10 = 1230\ \text{mm}^2$

Area of outstanding leg $A_{go} = [75 - (10/2)] \times 10 = 700\ \text{mm}^2$

$$A_n = 1230 + 700 = 1930\ \text{mm}^2 > 1524\ \text{mm}^2$$

Tearing strength of the net section

$$T_{dn} = \alpha A_n f_u/\gamma_{m1} = 0.8 \times 1930 \times 410/1.25$$
$$= 506.4\ \text{kN} > 450\ \text{kN}$$

Hence safe.

Without lug angle, the length of the gusset plate is 480 mm. If the bolts are staggered and arranged in two rows, the length of the gusset plate may be reduced. We will now provide a lug angle (see Fig. 7.34).

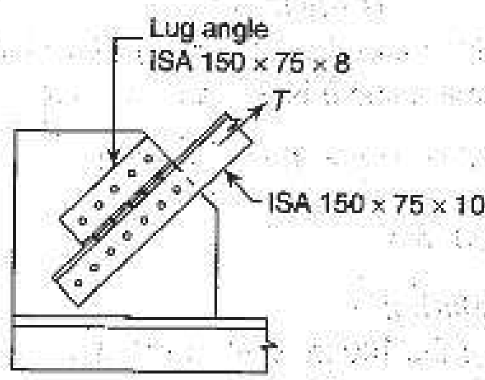


Fig. 7.34

Design of lug angle

Total factored tensile load = 450 kN

Gross area of the connected leg = $[150 - (10/2)] \times 10 = 1450\ \text{mm}^2$

Gross area of outstanding leg = $[75 - (10/2)] \times 10 = 700\ \text{mm}^2$

In an unequal angle, the load gets distributed in the ratio of the gross area of connected and outstanding legs.

Load shared by outstanding leg of main angle

$$= 450 \times 700/(1450 + 700) = 146.5\ \text{kN}$$

Load on lug angle = $1.2 \times 146.5 = 175.8$ (clause 10.12.2)

Required net area for lug angle $= 175.8 \times 10^3 \times 1.25/(0.9 \times 410)$

$$= 596\ \text{mm}^2$$

Use ISA $150 \times 75 \times 8$ angle with $A = 1750\ \text{mm}^2$

Assuming that the section is weakened by one row of 20-mm-diameter bolt

Net area = $1750 - 22 \times 8 = 1574\ \text{mm}^2 > 596\ \text{mm}^2$ (For simplicity the effect of $\Sigma p^2/4g$ is not considered here but should ideally be considered as explained in previous examples.)

The lug angle is also kept with its 75-mm long leg as outstanding leg

Number of bolts to connect 150-mm leg of lug angle with gusset plate

$$= 175.8/45.3 = 4$$

Provide five bolts of 20 mm diameter to connect lug angle with gusset plate.

Check

Load on connected leg = $450 \times 1450/(1450 + 700) = 303.5$ kN

Required number of bolts = $303.5/45.3 = 7$

Hence provide seven 20-mm-diameter bolts to connect the diagonal tension member with the gusset.

Required number of bolts to connect outstanding legs of the two angles (clause 10.12.2)

$$= 1.4 \times 146.5/45.3 = 5$$

Hence, provide five bolts of 20 mm diameter.

Required length of gusset plate = $6 \times 50 + 2 \times 30 = 360$ mm (compared with 510 mm without lug angle).

the number of bolts required to develop full capacity of the member. What will be the capacity if the angles are connected on the same side of the gusset plate and tack bolted [Fig. 7.30(b)]? What is the effect on tensile strength if the members are not tack bolted?

Solution

(a) Two angles connected to opposite side of the gusset as in Fig. 7.30(a)

(i) Design strength due to yielding of gross section $T_{dg} = f_y(A_g/\gamma_{m0})$

$$A_g = 866 \text{ mm}^2 \text{ (for a single angle)}$$
$$T_{dg} = 250 \times 2 \times (866/1.10) \times 10^{-3}$$
$$T_{dg} = 393.64 \text{ kN}$$

(ii) The design strength governed by tearing at net section

$$T_{dn} = \alpha A_n(f_u/\gamma_{m1})$$

Assume a single line of four numbers of 20-mm-diameter bolts ($\alpha = 0.8$)

$$A_n = [(75 - 6/2 - 22)6 + (75 - 6/2)6]2$$
$$A_n = (300 + 432)2 = 1464 \text{ mm}^2$$
$$T_{dn} = (0.8 \times 1464 \times 410/1.25) = 384.15 \text{ kN}$$

Therefore,

Tensile capacity = 384.15 kN

Design of bolts

Choose edge distance = 35 mm

Capacity of bolt in double shear (Table 5.9)

$$= 2 \times 45.3 = 90.6 \text{ kN}$$

Bearing capacity of the bolt does not govern as per Table 5.9.

Hence,

Strength of a single bolt = 90.6 kN

Provide five bolts. Then,

Total strength of the bolts = $5 \times 90.6 = 453$ kN > 384.15 kN

Hence the connection is safe.

Minimum spacing = $2.5t = 2.5 \times 20 = 50$ mm

Hence, provide a spacing of 50 mm.

The arrangements of bolts are shown in Fig. 7.30(c).

Check for block shear strength: (clause 6.4)

Block shear strength T_{db} of connection will be taken as

$$T_{db1} = [A_{vg}f_y/\sqrt{3}\gamma_{m0}) + (0.9A_{tn}f_u/\gamma_{m1})]$$

or

$$T_{db2} = 0.9f_uA_{vn}/\sqrt{3}\gamma_{m1} + (f_yA_{tg}/\gamma_{m0})$$

whichever is smaller.

$$A_{vg} = (4 \times 50 + 35)6 = 1410 \text{ mm}^2$$
$$A_{vn} = (4 \times 50 + 35 - 4.5 \times 22)6 = 816 \text{ mm}^2$$

<u>Students must prepare answers for the following questions at the end of the lecture.</u>

Qn No	Question	Marks	CO	Bloom's Knowledge Level
1	Discuss the provisions does the lug angle provide in tension member.(Nov Dec 2023)	7	2	K2
2	A built-up tension member consists of two L-shaped angles connected by a gusset plate. The member is required to carry a tensile load of 120 kN. Determine the minimum size of the angles and gusset plate required if the allowable tensile stress is 150 MPa. Assume a bolt diameter of 16 mm and a bolt spacing of 50 mm. (Apr May 2023)	13	2	K3

Reference Book:

- N Subramaniam , Design of Steel Structures, Oxford University press, New Delhi, 2011

Unit	2. DESIGN OF TENSION AND COMPRESSION MEMBERS	Lecture No	13
Topic	Design of Tension Splices		
Learning Outcome (LO) At the end of this lecture, students will be able to		**Bloom's Knowledge Level**	
LO1	Design splices for tension members	K3	

Tension Splices:

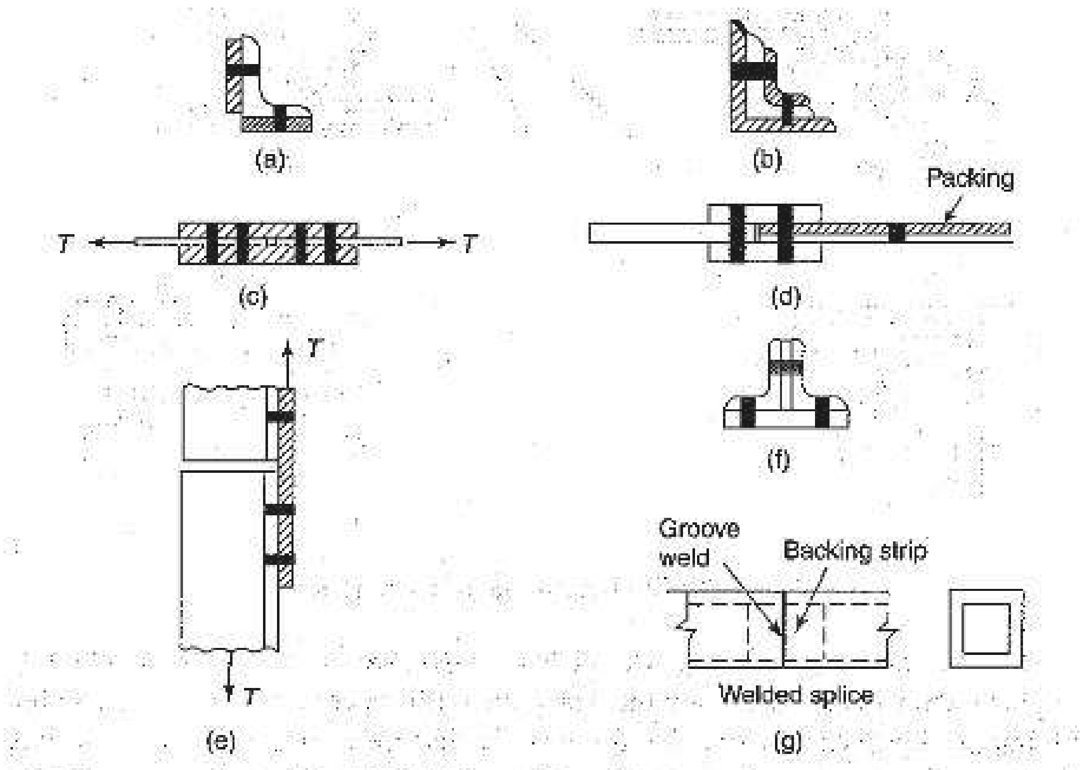


Fig. 7.22 Splices in tension members

When the available length is less than the required length of a tension member, splices are provided. The various types of splices that can be provided are shown in Figs 7.22(a) to (c). If the sections are not of the same thickness, packings are introduced, as shown in Fig. 7.22(d).

In the design of a tension splice, the effect of eccentricity is neglected; as far as possible it should be avoided. Thus Fig. 7.22(c) shows an angle section spliced on one leg of the angle only by a plate. Such an arrangement causes eccentricity and introduces bending moments. To overome this, both the legs of the angle should be spliced, as shown in Fig. 7.22(a). The splice as shown in Fig. 7.22(b) is used in the legs of transmission line or communication towers and aids transfer of tensile loads, without any eccentricity,

The splice cover plates or angles and its connections should be designed to develop the net tensile strength of the main member. The forces in the main member are transferred to the cover plate angle sections through the bolts/welding and carried through these covers across the joint and is transferred to the other portion of the section through the fasteners.

Example 7.2 *Determine the minimum net area of the plates as shown in Figs 7.25(a) and (b) with a plate of size of 210 × 8 mm and 16-mm bolts.*

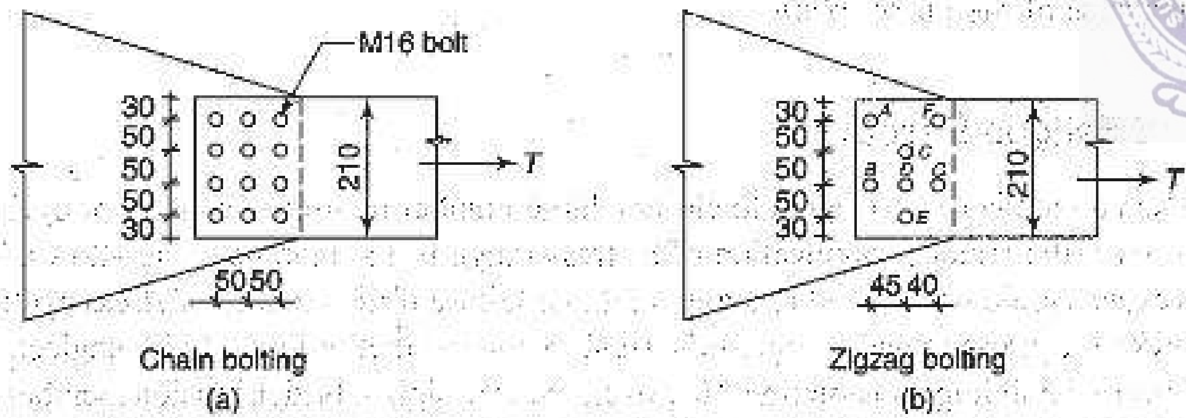


Fig. 7.25

Solution

(a) Chain bolting

For a 16-mm bolt, hole diameter = 18 mm

Net area = $(b - nd)t$

$= (210 - 4 \times 18) \times 8$

$= 1104 \text{ mm}^2$

(b) Zigzag bolting

Staggered length correction = $p_i^2/4g_i$

Path AB and FG (two holes):

Net area = $(210 - 2 \times 18) \times 8 = 1392 \text{ mm}^2$

Path CDE (three holes):

Net area = $(210 - 3 \times 18) \times 8 = 1248 \text{ mm}^2$

Path ACDE (four holes and one stagger):

Net area = $[210 - 4 \times 18 + 45^2/(4 \times 50)]8 = 1185 \text{ mm}^2$

Path FCDE (four holes and one stagger):

Net area = $[210 - 4 \times 18 + 40^2/(4 \times 50)]8 = 1168 \text{ mm}^2$

Path ACG or FCB (three holes and two staggers):

Net area = $[210 - 3 \times 18 + 45^2/(4 \times 50) + 40^2/(4 \times 50)]8 = 1393 \text{ mm}^2$

Path FCG (three holes and two staggers):

Net area = $210 - 3 \times 18 + 2 \times 40^2/(4 \times 50)]8 = 1376 \text{ mm}^2$

The minimum net area is for path FCDE = 1168 mm^2. Note that the minimum net area occurs at a path which has the maximum number of holes and minimum number of staggers.

Students must prepare answers for the following questions at the end of the lecture.

Qn No	Question	Marks	CO	Bloom's Knowledge Level
1	Design a tension splice to connect a 300mm x 20mm plate with a 300mm x 10mm plate. The design load is 500kN. Use 20mm diameter bolts. (Nov Dec 2021)	13	2	K3

Reference Book:

- N Subramaniam , Design of Steel Structures, Oxford University press, New Delhi, 2011

Unit	2. DESIGN OF TENSION AND COMPRESSION MEMBERS	Lecture No	14
Topic	Design of Simple Compression Members		
Learning Outcome (LO) At the end of this lecture, students will be able to		**Bloom's Knowledge Level**	
LO1	Design a compression member using simple sections	K3	

Compression Members: (IS800:2007 provision)

7.1 Design Strength

7.1.1 Common hot rolled and built-up steel members used for carrying axial compression, usually fail by flexural buckling. The buckling strength of these members is affected by residual stresses, initial bow and accidental eccentricities of load. To account for all these factors, the strength of members subjected to axial compression is defined by buckling class a, b, c, or d as given Table 7.

7.1.2 The design compressive strength P_d, of a member is given by:

$$P < P_d$$

where

$$P_d = A_e f_{cd}$$

where

A_e = effective sectional area as defined in **7.3.2**, and

f_{cd} = design compressive stress, obtained as per **7.1.2.1**.

7.1.2.1 The design compressive stress, $f_{cd,}$ of axially loaded compression members shall be calculated using the following equation:

$$f_{cd} = \frac{f_y / \gamma_{m0}}{\phi + \left[\phi^2 - \lambda^2\right]^{0.5}} = \chi f_y / \gamma_{m0} \leq f_y / \gamma_{m0}$$

where

ϕ = $0.5 [1 + \alpha (\lambda - 0.2) + \lambda^2]$

λ = non-dimensional effective slenderness ratio

$$= \sqrt{f_y / f_{cc}} = \sqrt{f_y \left(\frac{KL}{r}\right)^2 / \pi^2 E}$$

$$f_{cc} = \text{Euler buckling stress} = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}$$

where

KL/r = effective slenderness ratio or ratio of effective length, KL to appropriate radius of gyration, r;

α = imperfection factor given in Table 7;

χ = stress reduction factor (*see* Table 8) for different buckling class, slenderness ratio and yield stress

$$= \frac{1}{\left[\phi + \left(\phi^2 - \lambda^2\right)^{0.5}\right]}$$

λ_{m0} = partial safety factor for material strength.

Example 9.1 *An ISHB 300 is to be used as a short column carrying axial load. Is its compressive strength likely to be affected by local buckling assuming (a) Fe 410 steel with f_y = 250 MPa (b) Fe 540 steel with a design strength of f_y = 410 MPa.*

Solution

From Section tables, b_f = 250 mm, t_f = 10.6 mm, D = 300 mm, R = 11.0 mm, t_w = 7.6 mm, and A = 7480 mm^2.

(a) For f_y = 250 MPa, from Table 2 of the code, IS 800 : 2007, (b/t) allowable = 15.7

Actual (b/t) for flange = (250/2)10.6 = 5.9 < 15.7

d/t for web = [300 – 2 (10.6 + 11)]/7.6 = 33.79

From Table 2 of code, (d/t) max = 42

Thus full cross section is effective and design strength = 250 × 7480/(1.1 × 1000) = 1700 kN

(b) For f_y = 410 MPa,

$$\text{Flange limit is } 15.7\sqrt{(250/410)} = 12.26 > 5.9$$

$$\text{Web limit} = 42\sqrt{(250/410)} = 32.79 < 33.79$$

Therefore, cross section is slender on account of proportions of the web.

Effective cross section

$$d = 300 - 2(10.6 + 11) = 256.8$$

$$A_{eff} = 7480 - \{256.8 - 7.6 \times 32.79\}\ 7.6$$

$$= 7480 - (256.8 - 249.2)\ 7.6$$

$$= 7480 - 57.76 = 7422.24\ \text{mm}^2$$

Design strength = $7422.24 \times 410/(1.1 \times 1000) = 2766.5$ kN

Note that in this case the local buckling reduces the compression strength by about 1%. Since the web proportions control and most of the section's area is concentrated in the (semi-compact) flanges, the reduced effectiveness of the web results in much smaller loss of design capacity.

Example 9.2 *Determine the design axial load on the column section ISMB 350, given that the height of column is 3.0 m and that it is pin-ended. Also assume the following: $f_y = 250$ N/mm², $f_u = 410$ N/mm²; $E = 2 \times 10^5$ N/mm².*

Solution

Unless the axis about which buckling will occur is obvious, all possibilities must be checked. For ISMB section, r_y is normally between one fourth to one fifth of r_z and hence the likely mode of failure is by buckling about the minor axis. However, in case where different effective lengths apply for the two planes, both possibilities should normally be checked. Here both the possibilities are shown just for illustration.

Cross-section properties

Flange thickness, $t_f = 14.2$ mm

Thickness of web, $t_w = 8.1$ mm

Flange width, $b = 140$ mm

Self weight, $w = 524$ N/m

Cross-sectional area, $A = 6670$ mm²

$r_z = 143$ mm

$r_y = 28.4$ mm

$$\phi = 0.5\,[1 + \alpha(\lambda - 0.2) + \lambda^2]$$
$$= 0.5[1 + 0.21\,(0.2361 - 0.2) + 0.2361^2] = 0.532$$
$$f_{cd} = (f_y/\gamma_{mo})/\{\phi + [\phi^2 - \lambda^2]^{0.5}\}$$
$$= (250/1.10)/\{0.532 + [0.532^2 - 0.2361^2]^{0.5}\}$$
$$= 225.3\ \text{N/mm}^2$$

About y-y axis: $\alpha = 0.34$

$$\lambda_y = \sqrt{[f_y(KL/r)^2/(\pi^2 E)]}$$
$$= \sqrt{[250 \times (3000/28.4)^2/(\pi^2 \times 2 \times 10^5)]} = 1.189$$
$$\phi = 0.5[1 + \alpha\,(\lambda - 0.2) + \lambda^2]$$
$$= 0.5[1 + 0.34(1.189 - 0.2) + 1.189^2] = 1.375$$
$$f_{cd} = (f_y/\gamma_{mo})/\{\phi + [\phi^2 - \lambda^2]^{0.5}\}$$
$$= (250/1.10)/\{1.375 + [1.375^2 - 1.189^2]^{0.5}\} = 110\ \text{N/mm}^2$$

The same result may be obtained by using Table 9b of IS 800 : 2007. Thus, for $KL/r = 105.63$ and $f_y = 250$ MPa, from Table 9b, we get

$$f_{cd} = 110\ \text{N/mm}^2$$

(iv) *Design stresses*

In z-direction,

$$f_{cd} = 225.3\ \text{MPa}$$

In y-direction,

$$f_{cd} = 110.0\ \text{MPa}$$

Hence design axial compressive stress,

$$f_{cd} = 110\ \text{MPa}$$

The design strength,

$$P_d = 6670 \times 110/1000$$
$$= 733.7\ \text{KN}$$

Students must prepare answers for the following questions at the end of the lecture.

Qn No	Question	Marks	CO	Bloom's Knowledge Level
1	Calculate the compressive resistance of a isa 200 × 200 × 20 angle loaded through only one leg when it is connected by two bolts at the ends considering as fixed. (Apr-May 2021)	13	2	K3
2	(i) Explain the design of axially loaded solid section columns. (7) (ii) Discuss the advantages and limitations of using solid section columns. (6) (Apr May 2023)	13	2	K2

Reference Book:

- N Subramaniam , Design of Steel Structures, Oxford University press, New Delhi, 2011

Unit	2. DESIGN OF TENSION AND COMPRESSION MEMBERS	Lecture No	15
Topic	Design of Built-up Compression Members		
Learning Outcome (LO) At the end of this lecture, students will be able to		**Bloom's Knowledge Level**	
LO1	Design a compression member using Built-up sections	K3	

Example 9.4 *Calculate the compressive resistance of a compound column consisting of ISHB 300 with one cover plate of 350 × 20 mm on each flange (see Fig. 9.56) and having a length of 5 m. Assume that the bottom of the column is fixed and top is rotation fixed, translation free and f_y = 250 MPa.*

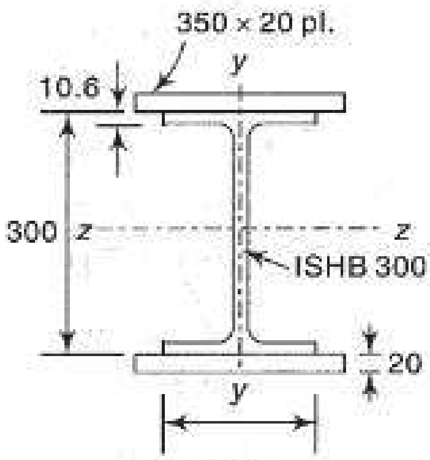


Fig. 9.56

Solution

From section tables for ISHB 300

Area = 7480 mm^2; I_{zz} = 12600 × 10^4 mm^4; I_{yy} = 2200 × 10^4 mm^4

(i) Determining the radii of gyration for the compound section

Area = 7480 + 350 × 20 × 2 = 21,480 mm^2

From Appendix A (Table A.15), approximate values of

$r_z = 0.40h = 0.40 \times 300 = 123$ mm

$r_y = 0.21b = 0.21 \times 350 = 73.5$ mm

Let us do the calculation to verify the above result

I_{zz} for plates $= 2[I + Ay_1^2]$

$= 2 \times [350 \times 20^3/12 + 350 \times 20 \times 160^2]$

$= 35887 \times 10^4$ mm^3

Total $I_{zz} = 12600 \times 10^4 + 35887 \times 10^4 = 484867 \times 10^4$ mm^4

Hence, $r_z = \sqrt{(484867 \times 10^4/21480)} = 150.24$ mm

I_{yy} of ISHB 300 = 2200 x 10^4 mm^4

I_{yy} of plates $= 2 \times 20 \times 350^3/12 = 14291.7 \times 10^4$ mm^4

Total $I_{yy} = 2200 \times 10^4 + 14291.7 \times 10^4 = 16491.7 \times 10^4$ mm^4

$$r_y = \sqrt{(16491.7 \times 10^4/21480)} = 87.62 \text{ mm}$$

(ii) Buckling curve classification

From Table 10 of the code for buckling about any axis use curve c.

(iii) Since r_y is small, the buckling will be about the *y-y* axis

Effective length = 2 × 5000 = 10,000 mm

$\lambda = KL/r_y = 10{,}000/87.62 = 114.13 < 180$

From Table 9c of the code, for

$f_y = 250$ MPa and $\lambda = 114.13$,

$f_{cd} = 90.1$ N/mm^2

Hence design strength = 21480 × 90.1/1000 = 1935.35 kN

Example 9.5 *A heavy column is required to support a gantry girder and a special H-section is to be fabricated. The trial section is shown in Fig. 9.57(a). Check its suitability to support a factored load of 11,000 kN, assuming both ends are pinned and a length of 8 m. Steel of design strength 250 N/mm² is to be used. Could a rolled section be suitably reinforced (by welding cover plates to its flanges) so as to provide an alternate solution?*

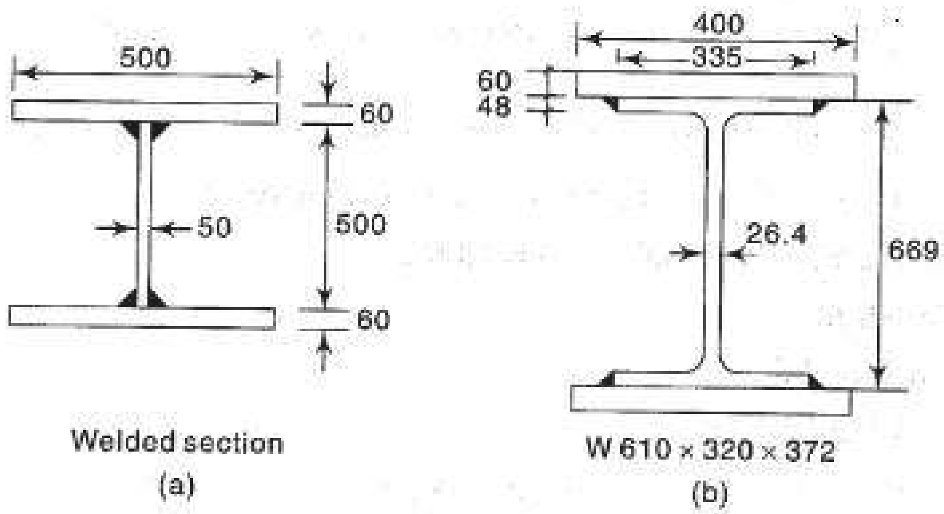


Fig. 9.57

Solution

(a) For the welded section shown in Fig. 9.57(a)

$A = (500 \times 60) \times 2 + 500 \times 50 = 85{,}000$ mm^2

$I_y = 2(60 \times 500^3)/12 + 500 \times 50^3/12 = 125520.8 \times 10^4$ mm^4

$$r_y = \sqrt{(I_y/A)} = \sqrt{125520.8 \times 10^4/85{,}000)} = 121.5 \text{ mm}$$

$$\lambda = KL/r_y = 1 \times 8000/121.5 = 65.83$$

(i) *Section classification* (Table 2 of IS 800)

Flange $b/t_f = (500/2)/60 = 4.1 < 13.6$

Web $d/t_w = 500/50 < 42$

Hence, the section is not slender.

(ii) *Buckling curve classification*

From Table 10 of the code, $t_f > 40$ mm

For *z-z* axis use curve '*c*'

For *y-y* axis use curve '*d*'

(iii) *Design strength*

From Table 9d, for $KL/r = 65.83$, and $f_y = 230$ N/mm², (As per Table 1 of the code)

$$f_{cd} = 133.25 \text{ N/mm}^2$$

Hence design strength $= f_{cd} \times A = 133.25 \times 85{,}000/1000 = 11{,}326$ kN > 11,000 kN

Hence, the section is suitable.

(b) The heaviest rolled section is wide flange W 610 × 320 with the following properties:

$$A = 47630 \text{ mm}^2,\ I_y = 30200 \times 10^4 \text{ mm}^4,\ r_y = 70.96 \text{ mm}$$

Hence we have to provide substantial cover plates to make the area about 85,000 mm². As a first trial use 400 × 60 mm plates on both flanges as shown in Fig. 9.57(b).

$$A = 47630 + 2\ (400 \times 60) = 95630 \text{ mm}^2$$

$$I_y = 30200 \times 10^4 + 2\ (60 \times 400^3)/12 = 94200 \times 10^4 \text{ mm}^4$$

$$r_y = \sqrt{(94200 \times 10^4/95{,}630)} = 99.25 \text{ mm}$$

From Table 10 of the code, since $t_f > 40$ mm use curve 'd'.

$$\lambda = KL/r_y = 1 \times 8000/99.25 = 80.60$$

From Table 9d, for

$$f_y = 230 \text{ N/mm}^2 \text{ and } \lambda = 80.6,$$

$$f_{cd} = 112.28 \text{ N/mm}^2$$

Capacity of the section $= 112.28 \times 95630/1000$

$= 10737 \text{ kN} \approx 11{,}000$ kN

Hence the section is suitable.

Students must prepare answers for the following questions at the end of the lecture.

Qn No	Question	Marks	CO	Bloom's Knowledge Level
1	A built-up column of effective length 4m is made of two ISMC 350 channels placed face to face, separated by a distance of 250 mm (between web ends) and carries a load of 800 kN. Design a suitable lacing system with connections. (Nov Dec 2022)	13	2	K3
2	(b) A column 4 m long has to support factored load af 6000 kN. The column is effectively hold at both ends and restrained in direction at One of the ends. Design the column using Beam sections and plates. (Apr May 2022)	13	2	K2

Reference Book:

- N Subramaniam , Design of Steel Structures, Oxford University press, New Delhi, 2011

Unit	2. DESIGN OF TENSION AND COMPRESSION MEMBERS	Lecture No	16
Topic	Design of Built-up Laced Column		
Learning Outcome (LO) At the end of this lecture, students will be able to		**Bloom's Knowledge Level**	
LO1	Design a compression member using Built-up Laced sections	K3	

Example 9.20 *Design a built-up laced column with four angles to support an axial load of 900 kN. The column is 12-m long and both the ends are held in position and restrained against rotation. Assume Fe 410 grade steel.*

Solution

Required area of the column = $900 \times 10^3/(0.6 \times 250) = 6000\ \text{mm}^2$

Provide four angles ISA $100 \times 100 \times 8$ mm. In this case we do not have any restriction of size (except for any architectural constraint). Hence, we will work backwards on the spacing from the assumed area and design compressive stress. The relevant properties are

$A = 1540\ \text{mm}^2$

$c_{zz} = c_{yy} = 27.6$ mm

$r_{zz} = r_{yy} = 30.7$ mm

$I_{zz} = I_{yy} = 145 \times 10^4\ \text{mm}^4$

Area provided = $4 \times 1540 = 6160\ \text{mm}^2$

For $6280\ \text{mm}^2$, the required $f_{cd} = 900 \times 10^3/6160 = 146.1$ MPa

From Table 9c, allowable L/r (for $f_{cd} = 146.1$ MPa) ≈ 75

From Table 11 of the code, for the fixed condition, $K = 0.65$

$L = 0.65 \times 12 \times 10^3 = 7800$ mm

Required $r = 7800/75 = 104$ mm

Moment of inertia of required section $I = Ar^2$

$= 6160 \times 104^2 = 66.626 \times 10^6\ \text{mm}^4$

Equating required and provided moment of inertia,

$66.626 \times 10^6 = 4 \times 145 \times 10^4 + 6160\bar{y}^2$

$\bar{y} = 99.37$ mm

Spacing of angle $S = 2 \times (99.37 + 27.6) = 253.94$ mm

Therefore, provide $S = 255$ mm

Now, $I_{zz} = I_{yy} = 4 \times 145 \times 10^4 + 6160(255/2 - 27.6)^2 = 67.277 \times 10^6$ mm^4

$$r = \sqrt{(67.277 \times 10^6/6160)} = 104.5$$

$$L/r = 7800/104.5 = 74.64$$

From Table 9c, for $L/r = 74.64$ and $f_y = 250$ MPa,

$$f_{cd} = 144.58 \text{ MPa}$$

Capacity of the built up column = 6160 × 144.58/1000 = 890.6 kN ≈ 900 kN

Hence the column is safe. If necessary, the spacing may be increased to 260 mm

Connecting system

Let us provide a double lacing system with the lacing flats inclined at 45°. Both are provided at the centre of the leg of angle.

Spacing of lacing bar, $L_o = (255 - 50 - 50)\cot 45°$

$$= 155 \text{ mm}$$

$L_o/r_{yy} = 155/30.7 = 5.05 < 50$

It should also be less than $0.7 \times 74.7 = 52.29 > 5.05$

Shear force, $V = (2.5/100) \times 900 \times 10^3 = 22{,}500$ N

Transverse shear in each panel = $V/N = 22{,}500/2 = 11250$ N

As double lacing is provided,

Compressive force in lacing bar = $(V/2N)\operatorname{cosec}\theta$

$$= 11250/2 \times \operatorname{cosec} 45°$$

$$= 7955 \text{ N}$$

Section of lacing flat

Assuming 20 mm bolts, width of the flat (clause 7.6.2) = 60 mm

Length of lacing = (255 – 50 – 50) cosec 45° = 219.2

Minimum thickness of the lacing flat = (1/60) × 219.2

$$= 3.65 \text{ mm}$$

Provide a flat of size 60 × 6 mm.

Minimum radius of gyration,

$$r = t/\sqrt{12} = 6/\sqrt{12} = 1.73 \text{ mm}$$

$$L_1/r = 0.7(219.2)/1.73 = 88.69 < 145$$

Hence the flat is safe.

For $L_1/r = 88.69$ and $f_y = 250$ MPa, from Table 9c,

$$f_{cd} = 122.97 \text{ N/mm}^2$$

Capacity of lacing bar = 122.67 × 60 × 6

$$= 44{,}267 \text{ N} > 7955 \text{ N}$$

Hence the lacing bar is safe.

Connections

Strength of a 20 mm diameter bolt in double shear (Table 5.9) = 2 × 45.3 = 90.6 kN

Strength of the bolt in bearing $= 2.5k_b dt \times f_u/\gamma_{mb}$

$= 2.5 \times 0.6 \times 120 \times 6 \times 410/(1.25 \times 1000) = 59$ kN

Hence strength of bolt = 59 kN

Number of bolts $= 2 \times 7955 \times \cot 45°/(59 \times 10^3) = 0.27$

Provide one 20-mm diameter bolt.

Note The size of the lacing plate may be reduced to 50 × 6 m with a 16-mm diameter bolt.

Tie plate

Tie plates are to be provided at each end of the built-up column.

Effective depth $= 255 - 2 \times 27.6 = 199.8 \approx 200 > 2 \times 100$ mm

Overall depth of the tie plate $= 200 + 2 \times 25 = 250$ mm

(Minimum edge distance of a 16-mm diameter bolt = 25 mm)

Thickness of tie plate $= 1/50(255 - 50 - 50) = 3.1$ mm

Therefore, provide a 255 × 250 × 6 mm tie plate and connect it with three 16-mm diameter bolts as shown in Fig. 9.66.

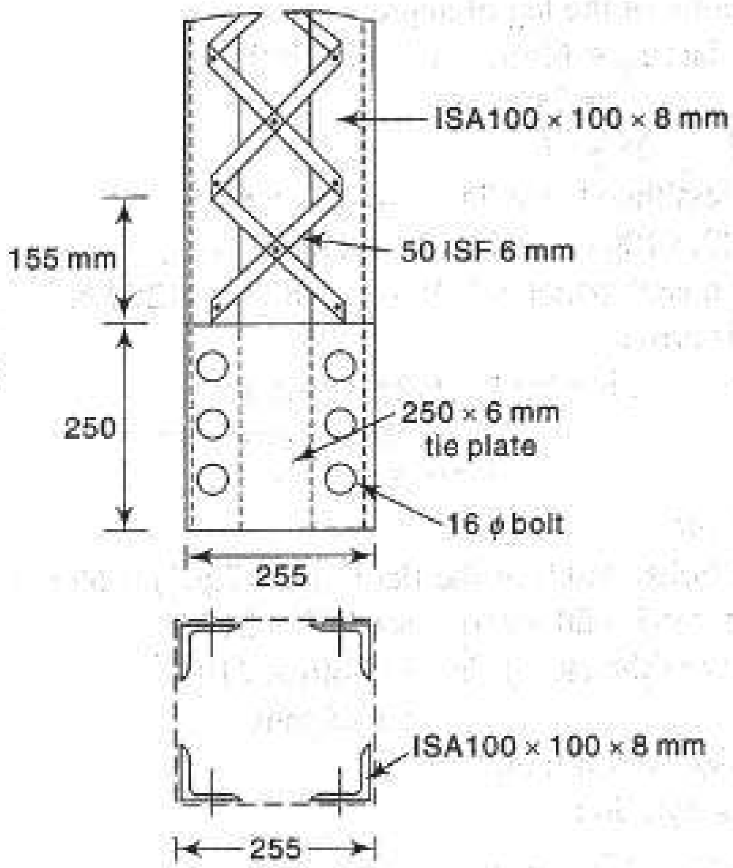


Fig. 9.66

Students must prepare answers for the following questions at the end of the lecture.

Qn No	Question	Marks	CO	Bloom's Knowledge Level
1	A built-up column consists of four ISA 100 x 75 x 8 angles and two 10 mm think gusset plates. The length of the column is 3.5 m and it is required to carry a compressive load of 200 kN. The allowable compressive stress is 150 MPa. Determine the minimum size of the gusset plates required (Apr May 2023)	13	2	K3

Reference Book:

- N Subramaniam , Design of Steel Structures, Oxford University press, New Delhi, 2011

Unit	2. DESIGN OF TENSION AND COMPRESSION MEMBERS	Lecture No	17
Topic	Design of Built-up Batten Column		
Learning Outcome (LO) At the end of this lecture, students will be able to		**Bloom's Knowledge Level**	
LO1	Design a compression member using Built-up Battened sections	K3	

Example 9.19 *Design a column having an effective length of 6 m and subjected to a factored axial load of 2400 kN. Provide the channels back-to-back connected by welded battens. Assume Fe 410 grade steel.*

Solution

Required area of cross section = $2400 \times 10^3/(0.6 \times 250) = 16{,}000\ \text{mm}^2$

Let us assume two channels ISMC 400 at 501 N/m. Relevant properties of ISMC 400 as per IS 808 – 1989 are

$A = 6380\ \text{mm}^2$; $b_f = 100$ mm; $t_f = 15.3$ mm; $r_z = 154$ mm, $r_y = 28.2$ mm;

$I_{zz} = 15{,}200 \times 10^4\ \text{mm}^4$; $I_{yy} = 508 \times 10^4\ \text{mm}^4$; $c_{yy} = 24.2$ mm

Area provided = $2 \times 6380 = 12{,}760\ \text{mm}^2$

$$\lambda = L/r = 1.1 \times 6.0 \times 10^3/154 = 42.86$$

For $L/r = 42.86$ and $f_y = 250$ MPa, using Table 9c of the code,

$$f_{cd} = 193.71\ \text{MPa}$$

Capacity of the built-up column = $193.71 \times 2 \times 6380/1000 = 2471\ \text{kN} > 2400\ \text{kN}$

Hence the column is safe.

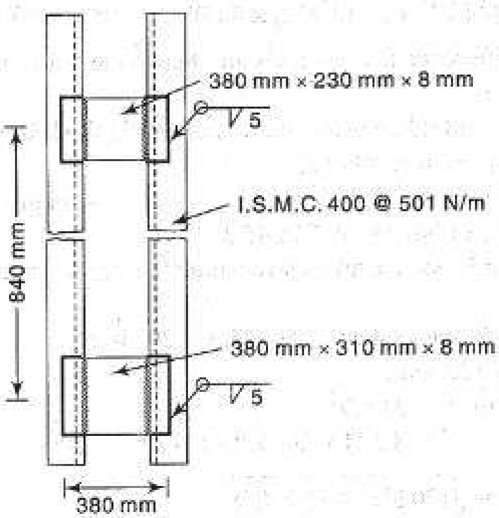
Spacing of channels

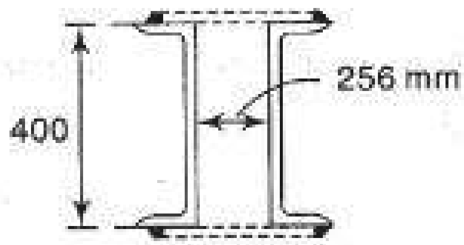
$$2I_{zz} = 2[I_{yy} + A(S/2 + c_{yy})^2]$$

$$2 \times 15200 \times 10^4 = 2[508 \times 10^4 + 6380\,(S/2 + 24.2)^2]$$

$$(S/2 + 24.2)^2 = 23028$$

Hence $S = 255.1$ mm. Therefore, provide two ISMC 400 at a spacing of 256 mm back-to-back as shown in Fig. 9.65.





Spacing of battens

L_o/r_{yy} should be less than 0.7 × slenderness ratio of the built-up column. Hence,

$$L_o/r_{yy} < 0.7(L/r)$$
$$L_o < 0.7(L/r)r_{yy}$$
$$< 0.7 \times 42.86 \times 28.2$$
$$< 846 \text{ mm}$$

Also, L_o/r_{yy} should be less than 50. Therefore,

$$L_o/r_{yy} < 50$$

i.e., $L_o < 50 \times 28.2 = 1210$ mm

Provide the batten at a spacing of 840 mm.

Size of end battens

Overall depth of batten $= 256 + 2 \times c_{yy}$

$$= 256 + 2 \times 24.2 = 304.4 \text{ mm}$$

Provide a 62 mm overlap of batten on channel flange for welding.

Length of batten $= 256 + 2 \times 62 = 380$ mm

Thickness of batten $= 1/50 \times 380 = 7.60$ mm

Provide 380 × 310 × 8 mm end batten plate.

Size of intermediate battens

Overall depth $= 3/4 \times 304.4 = 228.3$ mm

Provide 380 × 230 × 8 mm intermediate battens.

Design forces

Transverse shear = 2.5% of axial load

$$= 2.5 \times 2400 \times 10^3/100 = 60{,}000 \text{ N}$$

Longitudinal shear $V_b = V_t L_o/ns$

$$= 60{,}000 \times 840/[2(256 + 2 \times 62/2)]$$
$$= 79{,}245 \text{ N}$$

Moment $M = V_t L_o/2n$

$$= 60{,}000 \times 840/(2 \times 2) = 12.6 \times 10^6 \text{ N mm}$$

Check

(a) For end battens,

Shear stress $= 79{,}245/(310 \times 8)$

$$= 31.9 \text{ MPa} < 250/(\sqrt{3} \times 1.1) = 131.2 \text{ MPa}$$

Bending stress $= 12.6 \times 10^6 \times 6/(8 \times 310^2)$

$$= 98.3 \text{ MPa} < 250/1.1 = 227 \text{ MPa}$$

(b) For intermediate battens,

Shear stress $= 79{,}245/(230 \times 8)$

$$= 43.1 \text{ MPa} < 131.2 \text{ MPa}$$

Bending stress $= 12.6 \times 10^6 \times 6/(8 \times 230^2)$

$$= 178.6 \text{ MPa} < 227 \text{ MPa}$$

Hence the battens are safe.

Design of weld

Let t be the throat thickness of weld.

$I_{zz} = 2[(62 \times t^3/12) + (62 \times t)(230/2)^2] + 2 \times t \times 230^3/12$

Neglecting $62 \times t^3/12$, which will be insignificant, we get

$$I_{zz} = 366.77 \times 10^4 t \text{ mm}^4$$

$$I_{yy} = 2[t \times 62^3/12] + 2 \times 230 \times t^3/12 + 2 \times 230 \times t \times 31^2$$

Again neglecting $2 \times 230 \times t^3/12$, which will be very small, we get

$$I_{yy} = 48.18 \times 10^4\ t \text{ mm}^4$$

$$I_p = I_{zz} + I_{yy} = t(366.77 \times 10^4 + 48.18 \times 10^4) = 414.95 \times 10^4 t \text{ mm}^4$$

$$r = \sqrt{[(230/2)^2 + (62/2)^2]} = 119.1 \text{ mm}$$

$$\cos\theta = 31/119.1 = 0.260$$

Direct shear stress $= 79245/(2 \times 62 + 2 \times 230)t$

$$= 135.7/t \text{ N/mm}^2$$

Shear stress due to bending moment

$$= 12.6 \times 10^6 \times 119.1/(414.95 \times 10^4 \times t)$$

$$= 361.65/t \text{ N/mm}^2$$

Combined stress

$[(135.7/t)^2 + (361.65/t)^2 + 2 \times (135.7/t)(361.65/t) \times 0.260]^{0.5}$

$$= 418/t < 410/(\sqrt{3} \times 1.25) = 189.4$$

or $t = 418/189.4 = 2.2$ mm

Size of weld = 2.2/0.7 = 3.14 mm

The size of weld should not be less than 5 mm for 15.3 mm flange. Hence, provide a 5 mm weld to make the connections.

Students must prepare answers for the following questions at the end of the lecture.

Qn No	Question	Marks	CO	Bloom's Knowledge Level
1	Design a built-up column 9m long to carry a factored axial compressive load of 1100kN. the column is restrained in position but not in direction in both the ends. Design the column with connecting system as battens with bolted connections. Use two channel sections back-to-back. Use steel of grade Fe410 (Nov Dec 2021)	13	2	K3

Reference Book:

- N Subramaniam , Design of Steel Structures, Oxford University press, New Delhi, 2011

Unit	2. DESIGN OF TENSION AND COMPRESSION MEMBERS	Lecture No	18
Topic	Design of Column Support		
Learning Outcome (LO) At the end of this lecture, students will be able to		Bloom's Knowledge Level	
LO1	Design a support for Compression Members	K3	

Example 9.21 *Design the base plate for an ISHB 350 column to carry a factored load of 1200 kN. Assume Fe 410 grade steel and M25 concrete.*

Solution

For ISHB 350 section,

h = 350 mm; b = 250 mm; t_f = 11.6 mm; t_w = 8.3 mm

Bearing strength of concrete = $0.45 f_{ck} = 0.45 \times 25 = 11.25$ N/mm^2

Required area of base plate = $1200 \times 10^3/11.25$ = 1,06,667 mm^2

Use a base plate of size 450 × 350 mm with area = 1,57,000 mm^2. If the ISHB 350 is kept at the centre, the projection will be 50 mm on each side as shown in Fig. 9.67.

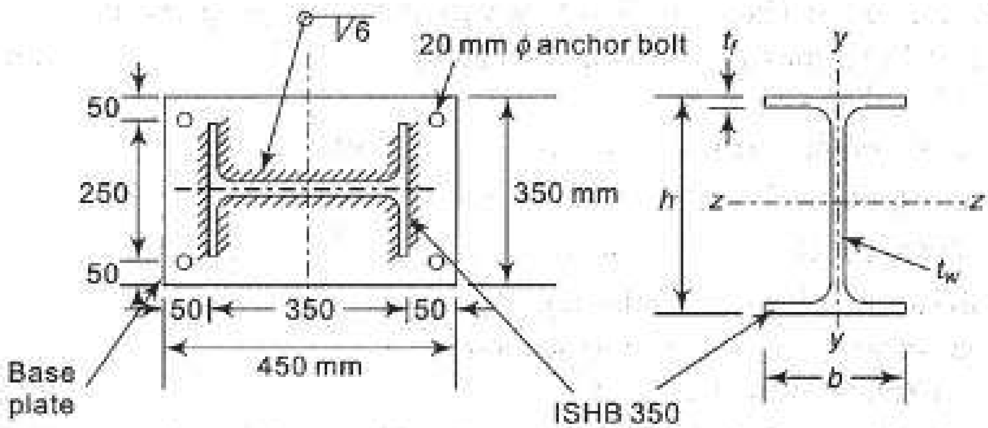


Fig. 9.67

$$w = (1200 \times 10^3)/(450 \times 350) = 7.62 \text{ MPa}$$

$$t_s = \sqrt{[2.5w(a^2 - 0.3b^2)\gamma_{m0}/f_y]}$$

$$= \sqrt{[2.5 \times 7.62 \times (50^2 - 0.3 \times 50^2) \times 1.10/250]}$$

= 12.11 mm > 11.6 mm (flange thickness)

Hence provide 450 × 350 × 14 mm plate. Also provide four 20-mm diameter and 300-mm long anchor bolts to connect the base plate to the foundation concrete.

Weld connecting base plate to column

Use a 6-mm fillet weld all round the column section to hold the base plate in place. Note that the surfaces are to be machined for direct bearing.

Total length available for welding along the periphery of ISHB 350

$$= 2(250 + 250 - 8.3 + 350 - 11.6) = 1660.2 \text{ mm}$$

After deducting end returns of the weld, at the rate of two times the size of the weld at each end, we get,

$$L_{eff} = 1660.2 - 2(4 + 2)2a = 16660.2 - 24 \times 6 = 1516.2 \text{ mm}$$

Capacity of the weld = $0.7 \times 6 \times 189/1000 = 0.7938$ kN/mm

Required length of the weld = 1200/0.7938

= 1511 mm < 1516.2 mm

Hence a 6 mm weld is adequate.

Note This is a fillet weld on the edge and not on the rounded ends of the member. Hence the limitation 0.75 times thickness (clause 10.5.8.2 of the code) will not apply here.

Students must prepare answers for the following questions at the end of the lecture.

Qn No	Question	Marks	CO	Bloom's Knowledge Level
1	Design a gusseted base to carry a factored axial load of 2800kN. The column consists of ISHB 450 @ 0.855kN/m with two cover plates 250mm x 20mm on either side. Take the effective height of column as 4m. (Nov Dec 2021)	13	2	K3

Reference Book:

- N Subramaniam , Design of Steel Structures, Oxford University press, New Delhi, 2011

Unit	3. DESIGN OF BEAMS	Lecture No	19
Topic	Design of Laterally Supported beams		
Learning Outcome (LO) At the end of this lecture, students will be able to		**Bloom's Knowledge Level**	
LO1	Design a beam with lateral supports	K3	

SECTION 8
DESIGN OF MEMBERS SUBJECTED TO BENDING

8.1 General

Members subjected to predominant bending shall have adequate design strength to resist bending moment, shear force, and concentrated forces imposed upon and their combinations. Further, the members shall satisfy the deflection limitation presented in Section 5, as serviceability criteria. Member subjected to other forces in addition to bending or biaxial bending shall be designed in accordance with Section 9.

8.1.1 *Effective Span of Beams*

The effective span of a beam shall be taken as the distance between the centre of the supports, except where the point of application of the reaction is taken as eccentric at the support, when it shall be permissible to take the effective span as the length between the assumed lines of the reactions.

8.2 Design Strength in Bending (Flexure)

The design bending strength of beam, adequately supported against lateral torsional buckling (laterally supported beam) is governed by the yield stress (*see* **8.2.1**). When a beam is not adequately supported against lateral buckling (laterally un-supported beams) the design bending strength may be governed by lateral torsional buckling strength (*see* **8.2.2**).

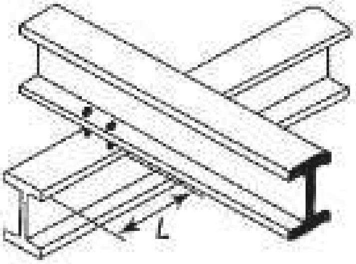
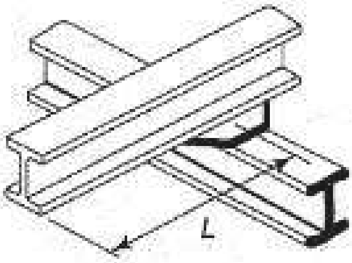
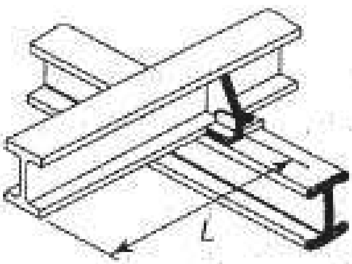
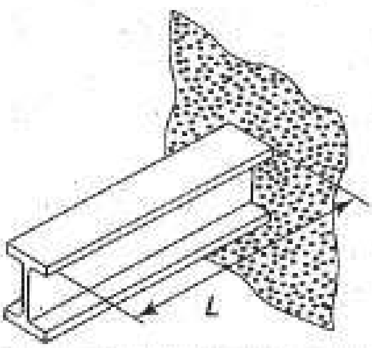
The factored design moment, M at any section, in a beam due to external actions, shall satisfy

$$M \leq M_d$$

where

M_d = design bending strength of the section, calculated as given in **8.2.1.2**.

Table 10.5 Effective length L_E for cantilevers without intermediate restraint (adapted from BS 5950-1:2000)

Restraint conditions		Loading conditions	
At support	At tip	Normal	Destabilizing (top flange loading)
(a) Continuous, with lateral restraint to top flange 	1. Free	3.0L	7.5L
	2. Lateral restraint to top flange	2.7L	7.5L
	3. Torsional restraint	2.4L	4.5L
	4. Lateral and torsional restraint	2.1L	3.6L
(b) Continuous, with partial torsional restraint 	1. Free	2.0L	5.0L
	2. Lateral restraint to top flange	1.8L	5.0L
	3. Torsional restraint	1.6L	3.0L
	4. Lateral and torsional restraint	1.4L	2.4L
(c) Continuous, with lateral and torsional restraint 	1. Free	1.0L	2.5L
	2. Lateral restraint to top flange	0.9L	2.5L
	3. Torsional restraint	0.8L	1.5L
	4. Lateral and torsional restraint	0.7L	1.2L
(d) Restrained laterally, torsionally and against rotation on plane 	1. Free	0.85L	1.4L
	2. Lateral restraint to top flange	0.7L	1.4L
	3. Torsional restraint	0.75L	0.6L
	4. Lateral and torsional restraint	0.5L	0.5L

Example 10.1 *Design a simply supported beam of span 5 m carrying a reinforced concrete floor capable of providing lateral restraint to the top compression flange. The uniformly distributed load is made up of 20 kN/m imposed load and 20 kN/m dead load (section is stiff against bearing). Assume Fe 410 grade steel.*

Solution

Step 1: Calculation of factored loads

Dead load = 1.5 × 20 = 30 kN/m

Live load = 1.5 × 20 = 30 kN/m

Total factored load on the beam = 60 kN/m (see Fig. 10.47)

Step 2: Calculation of maximum bending moment and shear force

Maximum bending moment = $60 \times 5^2/8 = 187.5$ kN m

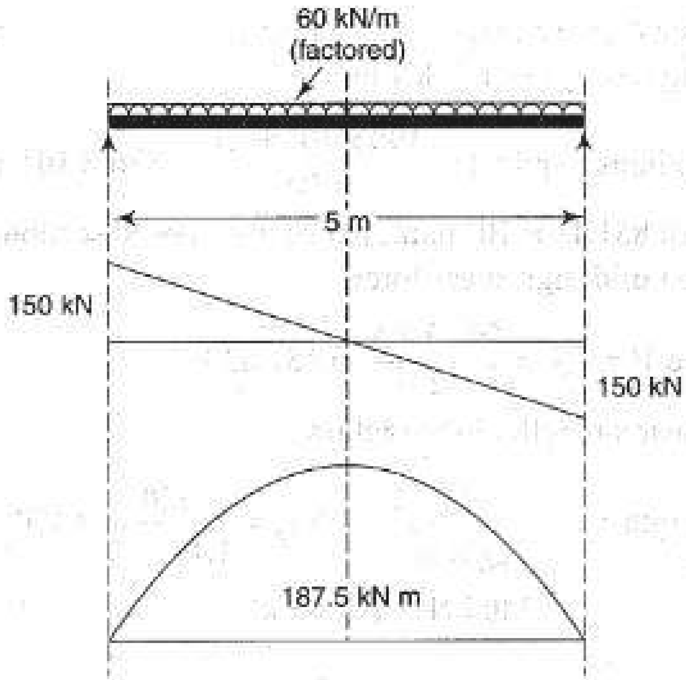


Fig. 10.47

Step 3: Section modulus required (Z_p)

$$Z_p \text{ (required)} = \frac{M \times \gamma_{m0}}{f_y} = \frac{187.5 \times 10^6 \times 1.1}{250} = 825 \times 10^3 \text{ mm}^3$$

Step 4: Selection of suitable section

Choose a trial section of ISLB 350 @ 0.486 kN/m

The properties of the section are as follows:

Depth of section (h) = 350 mm

Width of flange (b) = 165 mm

Thickness of flange (t_f) = 11.4 mm

Depth of web (d) = $h - 2(t_f + R) = 350 - 2(11.4 + 16) = 295.2$ mm

Thickness of web (t_w) = 7.4 mm

Moment of inertia about major axis $I_{zz} = 13200 \times 10^4$ mm^4

Elastic section modulus (Z_e) = 751.9×10^3 mm^3

Plastic section modulus (Z_p) = 851.11×10^3 mm^3

Section classification

$$\varepsilon = \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{250}} = 1$$

$$\frac{b}{t_f} = \frac{82.5}{11.4} = 7.23 < 9.4$$

$$\frac{d}{t_w} = \frac{295.2}{7.4} = 39.9 < 84$$

Hence the section is classified as a plastic section.

Step 5: Adequacy of the section including self weight of the beam

Total load acting on the beam = 60.73 kN/m

Maximum bending moment = 190 kN m

$$\text{Plastic section modulus required} = \frac{190\times10^6\times1.1}{250} = 836\times10^3\ \text{mm}^3$$

Since it is less than 851.11×10^3 mm^3, hence, the chosen section is adequate.

Step 6: Calculation of design shear force

$$\text{Design shear force } V = \frac{wl}{2} = \frac{60.73\times5}{2} = 151.83\ \text{kN}$$

Step 7: Design shear strength of the section

$$\text{Design shear strength } V_d = \frac{f_y}{\gamma_{m0}\times\sqrt{3}}\times h\times t_w = \frac{250}{1.1\times\sqrt{3}}\times350\times7.4$$

$$= 340\ \text{kN} > 151.83\ \text{kN}$$

Also,

$$0.6V_d = 204$$

Therefore, the design shear force $V < 0.6V_d$.

Step 8: Check for design capacity of the section

$$\frac{d}{t_w} = 39.9\ (\text{which is less than } 67\varepsilon)$$

Hence,

$$M_d = \beta_b Z_p\times\frac{f_y}{\gamma_{m0}}$$

$\beta_b = 1.0$, since the section is plastic section.

Therefore,

$$M_d = \frac{1.0\times851.11\times10^3\times250}{1.1} = 193.43\ \text{kN m}$$

$$193.43\ \text{kN m} \le \frac{1.2\times Z_e\times f_y}{\gamma_{m0}} = \frac{1.2\times751.9\times10^3\times250}{1.1} = 205\ \text{kN m}$$

Hence the design capacity of the member is more than maximum bending moment M_d (193.43 kN m > 190 kN m).

Step 9: Check for deflection

Deflection (which is a serviceability limit state) must be calculated on the basis of the unfactored imposed loads.

$$\delta = \frac{5wl^4}{384EI} = \frac{5\times20\times5000^4}{384\times2\times10^5\times13200\times10^4} = 6.165\ \text{mm}$$

$$\text{Allowable maximum deflection max} = \frac{L}{300} = \frac{5000}{300} = 16.67\ \text{mm}$$

Example 10.7 *A proposed cantilever beam is built into a concrete wall and free at its end. It supports dead load of 20 kN/m and a live load of 10 kN/m. The length of the beam is 5 m. Select an available section with necessary checks. Assume bearing length of 100 mm.*

Solution

Step 1: Calculation of load

Dead load = 1.5 × 20 = 30 kN/m

Live load = 1.5 × 10 = 15 kN/m

Total load = 45 kN/m

Step 2: Calculation of bending moment and shear force

$$BM = \frac{wl^2}{2} = \frac{45 \times 5^2}{2} = 562.5 \text{ kN m}$$

$$SF = wl = 45 \times 5 = 225 \text{ kN}$$

Step 3: Selection of initial section

Assume $\lambda = 80$; $h/t_f = 20$

$$f_{cr,b} = \left(\frac{1473.5}{\lambda}\right)^2 \left[1 + \frac{1}{20}\left(\frac{\lambda}{h/t_f}\right)^2\right]^{0.5} = \left(\frac{1473.5}{80}\right)^2 \left[1 + \frac{1}{20}\left(\frac{80}{20}\right)^2\right]^{0.5}$$

$$= 455.1 \text{ N/mm}^2$$

$$\lambda_{LT} = \sqrt{\frac{f_y}{f_{crb}}} = \sqrt{\frac{250}{455.1}} = 0.741$$

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

$$= 0.5[1 + 0.21(0.741 - 0.2) + 0.741^2]$$

$$= 0.831$$

$$\chi_{LT} = \frac{1}{\phi_{LT} + [\phi_{LT}^2 - \lambda_{LT}^2]^{0.5}} \le 1.0$$

$$= \frac{1}{0.831 + [0.831^2 - 0.741^2]^{0.5}} \le 1.0$$

$$= 0.828$$

$$f_{bd} = \frac{\chi_{LT} f_y}{\gamma_{m0}} = \frac{0.828 \times 250}{1.10} = 188.27 \text{ N/mm}^2$$

Therefore required section modulus $= \dfrac{562.5 \times 10^6}{188.27} = 2987.6 \times 10^3 \text{ mm}^3$

Choose a section of ISWB 600 @ 1.337 kN/m

Overall depth (h) = 600 mm

Width of flange (b) = 250 mm

Thickness of flange (t_f) = 21.3 mm

Thickness of web (t_w) = 11.2 mm

Depth of web (d) = $h - 2(t_f + R) = 600 - 2(21.3 + 17) = 523.4$ mm

Moment of inertia about major axis $I_z = 106199 \times 10^4$ mm^4

Moment of inertia about minor axis $I_y = 4702.5 \times 10^4$ mm^4

Elastic section modulus (Z_e) = 3540×10^3 mm^3

Plastic section modulus (Z_p) = 3986.6×10^3 mm^3

Section classification

Outstand of compression flange = $125/21.3 = 5.86 < 9.4\varepsilon$

Web with N.A at mid depth = $523.4/11.2 = 46.74 < 83.9\varepsilon$

Therefore, the section is plastic.

Since flange thickness is > 20 mm, we should use f_y = 240 MPa, as per Table 1 of IS 800.

Students must prepare answers for the following questions at the end of the lecture.

Qn No	Question	Marks	CO	Bloom's Knowledge Level
1	A beam is simply supported over a span of 6m. It supports one iron beam at 10 midspan exerting 90kN. Design a laterally unsupported beam with ISWB section with flange plates (Nov Dec 2021)	13	3	K3

Reference Book:

- N Subramaniam , Design of Steel Structures, Oxford University press, New Delhi, 2011

Unit	3. DESIGN OF BEAMS	Lecture No	20
Topic	Design of Continuous beams		
Learning Outcome (LO) At the end of this lecture, students will be able to		Bloom's Knowledge Level	
LO1	Design a beams on continuous supports	K3	

Example 10.8 *Design a continuous beam of span 4.9 m, 6 m and 4.9 m carrying a total uniformly distributed load of 32.5 kN/m and laterally unrestrained with a bearing length of 100 mm.*

Solution

Step 1: Load calculation

$$\text{Factored load} = 1.5 \times 32.5 = 48.75 \text{ kN/m}$$

For the bending moment and shear force diagram see Fig. 10.50.

Maximum bending moment = 146.25 kN m

Maximum shear force = 146.25 + 146.25 = 292.5 kN m

Step 2: Selection of initial section

Assume $\lambda = 100$ and $h/t_f = 25$. Therefore $f_{crb} = 291.4$ N/mm² (from Table 14 of the code).

$$\lambda_{LT} = \sqrt{\frac{f_y}{f_{crb}}} = \sqrt{\frac{250}{291.4}} = 0.926$$

$$\phi_{LT} = 0.5[1 + \alpha_{LT}(\lambda_{LT} - 0.2) + \lambda_{LT}^2]$$
$$= 0.5[1 + 0.21(0.926 - 0.2) + 0.926^2]$$
$$= 1.005$$

$$\chi_{LT} = \frac{1}{\phi_{LT} + [\phi_{LT}^2 - \lambda_{LT}^2]^{0.5}} \leq 1.0$$

$$= \frac{1}{1.005 + [1.005^2 - 0.926^2]^{0.5}} \leq 1.0$$

$$= 0.716 \leq 1$$

$$f_{bd} = \frac{\chi_{LT} f_y}{\gamma_{m0}} = \frac{0.716 \times 250}{1.10} = 162.7 \text{ N/mm}^2$$

$$\text{Therefore required section} = \frac{146.25 \times 10^6}{162.7} = 898.7 \times 10^3 \text{ mm}^3$$

Choose a section of ISLB 400 @ 0.57 kN/m

Overall depth (D) = 400 mm

Width of flange (B) = 165 mm

Thickness of flange (t_f) = 12.5 mm

Thickness of web (t_w) = 8 mm

Depth of web (d) = $h - 2(t_f + R) = 400 - 2(12.5 + 16) = 343$ mm

Moment of inertia about major axis $I_{zz} = 19281.5 \times 10^4$ mm^4

Moment of inertia about minor axis $I_{yy} = 716 \times 10^4$ mm^4

Elastic section modulus (Z_{ez}) = 964.1×10^3 mm^3

Plastic section modulus (Z_{py}) = 1098.2×10^3 mm^3

Section classification

Outstand of compression flange = $b/t_f = 82.5/12.5 = 6.6 < 9.4$

Web with N.A at mid depth = $d/t_w = 343/8 = 42.8 < 84$

Therefore the section is plastic.

Step 3: Calculation of lateral torsional buckling moment

$$M_{cr} = \sqrt{\frac{\pi^2 EI_y}{(KL)^2}\left(GI_t + \frac{\pi^2 EI_w}{(KL)^2}\right)}$$

$$G = \frac{E}{2(1+\mu)} = \frac{2\times10^5}{2\times(1+0.3)} = 76.923\times10^3 \text{ N/mm}^2$$

$$I_t = \sum\frac{b_i t_i^3}{3} = \left[\frac{2\times165\times12.5^3}{3} + \frac{(400-12.5)\times8^3}{3}\right] = 2.81\times10^5 \text{ mm}^4$$

$$I_w = (1-\beta_f)\beta_f I_y h_f^2$$

$$h_f = D - t_f = 400 - 12.5 = 387.5 \text{ mm}$$

$$\beta_f = \frac{I_{fc}}{I_{fc}+I_{ft}} = 0.5$$

$$I_w = (1-0.5)\times0.5\times716\times10^4\times387.5^2 = 2.69\times10^{11} \text{ mm}^6$$

$$M_{cr} = \sqrt{\frac{\pi^2\times2\times10^5\times716\times10^4}{6000^2}\left(76.923\times10^3\times2.81\times10^5 + \frac{\pi^2\times2\times10^5\times2.69\times10^{11}}{6000^2}\right)}$$

$$= 119.48 \text{ kN m}$$

$$\lambda_{LT} = \sqrt{\frac{\beta_b Z_p f_y}{M_{cr}}} = \sqrt{\frac{1.0\times1098.2\times10^3\times250}{119.48\times10^6}} = 1.516$$

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

$$= 0.5[1 + 0.21(1.516 - 0.2) + 1.516^2] = 1.79$$

$$\chi_{LT} = \frac{1}{\phi_{LT} + [\phi_{LT}^2 - \lambda_{LT}^2]^{0.5}} \le 1.0$$

$$= \frac{1}{1.79 + [1.79^2 - 1.518^2]^{0.5}} = 0.365 \le 1$$

$$f_{bd} = \frac{\chi_{LT} f_y}{\gamma_{m0}} = \frac{0.365 \times 250}{1.10} = 83 \text{ N/mm}^2$$

Therefore required section $= \dfrac{146.25 \times 10^6}{83} = 1762 \times 10^3 \text{ mm}^3$

Choose a section of ISLB 500 @ 0.75 kN/m.

Overall depth (D) = 500 mm

Width of flange (B) = 180 mm

Thickness of flange (t_f) = 14.1 mm

Depth of the web (d) = $D - 2(t_f + R) = 500 - 2(14.1 + 17) = 437.8$ mm

Thickness of web (t_w) = 9.2 mm

Moment of inertia about major axis $I_z = 38579 \times 10^4 \text{ mm}^4$

Moment of inertia about minor axis $I_y = 1060 \times 10^4 \text{ mm}^4$

Elastic section modulus (Z_{ez}) = $1543 \times 10^3 \text{ mm}^3$

Plastic section modulus (Z_{pz}) = $1770.8 \times 10^3 \text{ mm}^3$

Radius of gyration $r_y = 33.4$ mm

Section classification

Outstand of compression flange = $b/t_f = 90/14.1 = 6.38 < 9.4$

Web with N.A at mid depth = $d/t_w = 437.8/9.2 = 47.58 < 83.9$

Therefore the section is plastic.

Step 4: Calculation of moment carrying capacity of the section

$$M_{cr} = \sqrt{\frac{\pi^2 EI_y}{(KL)^2}\left(GI_t + \frac{\pi^2 EI_w}{(KL)^2}\right)}$$

$$G = \frac{E}{2(1+\mu)} = \frac{2 \times 10^5}{2 \times (1 + 0.3)} = 76.923 \times 10^3 \text{ N/mm}^2$$

$$I_t = \sum \frac{b_i t_i^3}{3} = \left[\frac{2 \times 180 \times 14.1^3}{3} + \frac{485.8 \times 9.2^3}{3}\right] = 4.63 \times 10^5 \text{ mm}^4$$

$$I_w = (1 - \beta_f)\beta_f I_y h_f^2$$

$$h_f = D - t_f = 500 - 14.1 = 485.9$$

$$\beta_f = \frac{I_{fc}}{I_{fc} + I_{ft}} = 0.5$$

$$I_w = (1 - 0.5) \times 0.5 \times 1060 \times 10^4 \times 485.9^2$$
$$= 6.256 \times 10^{11} \text{ mm}^6$$

Students must prepare answers for the following questions at the end of the lecture.

Qn No	Question	Marks	CO	Bloom's Knowledge Level
1	Design an I section purlin of the span of 4 m subjected to an LTDL of 1.5 kN/m in the plane of the minor axis and O, 5 kym in the plane of the major axis under service condition. Assume that the purlin is continuous over the supports and no lateral buckling occurs. grade of Steel is Fe 250 (Apr May 2022)	13	3	K3

Reference Book:

- N Subramaniam , Design of Steel Structures, Oxford University press, New Delhi, 2011

Unit	3. DESIGN OF BEAMS	Lecture No	21
Topic	Design of Continuous beams – Continuation of Lecture 20		
Learning Outcome (LO) At the end of this lecture, students will be able to		**Bloom's Knowledge Level**	
LO1	Design a beams on continuous supports	K3	

Example 10.8 *Design a continuous beam of span 4.9 m, 6 m and 4.9 m carrying a total uniformly distributed load of 32.5 kN/m and laterally unrestrained with a bearing length of 100 mm.*

$$I_w = (1 - 0.5) \times 0.5 \times 1060 \times 10^4 \times 485.9^2$$
$$= 6.256 \times 10^{11}\ \text{mm}^6$$

$$M_{cr} = \sqrt{\frac{\pi^2 \times 2 \times 10^5 \times 1060 \times 10^4}{6000^2}\left(76.923 \times 10^3 \times 4.63 \times 10^5 + \frac{\pi^2 \times 2 \times 10^5 \times 6.256 \times 10^{11}}{6000^2}\right)}$$

$$= 201.6\ \text{kN m}$$

Conservatively,

$$M_{cr} = \frac{\pi^2 E I_y h}{2(KL)^2}\left[1 + \frac{1}{20}\left(\frac{KL/r_y}{h/t_f}\right)^2\right]^{0.5}$$

$$\frac{KL}{r_y} = \frac{6000}{33.4} = 179.64$$

$$\frac{h}{t_f} = \frac{500}{14.1} = 35.46$$

$$M_{cr} = \frac{\pi^2 \times 2\times10^5 \times 1060\times10^4 \times 500}{2\times6000^2}\left[1+\frac{1}{20}\left(\frac{179.64}{35.46}\right)^2\right]^{0.5}$$

$$= 219.55 \text{ kN m}$$

From Table 10.9, for $KL/r_y = 179.64$ and $h/t_f = 35.46$, $f_{cr,b} = 101.66$ N/mm^2
Therefore $M_{cr} = 101.66 \times 1770.8 \times 10^3 = 180.02$ kN m.
Note The variation in the values obtained by the approximate and accurate method is about 8.9 to 10.7%.

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

$$= \sqrt{\frac{1.0\times1770.8\times10^3\times250}{201.6\times10^6}} = 1.48$$

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

$$\alpha_{LT} = 0.21$$

$$\phi_{LT} = 0.5[1 + 0.21(1.48 - 0.2) + 1.48^2] = 1.7296$$

$$\chi_{LT} = \frac{1}{\phi_{LT} + [\phi_{LT}^2 - \lambda_{LT}^2]^{0.5}} \le 1.0$$

$$= \frac{1}{1.7296 + [1.7296^2 - 1.48^2]^{0.5}} = 0.381 \le 1.0$$

$$f_{bd} = \frac{\chi_{LT} f_y}{\gamma_{m0}} = \frac{0.381\times250}{1.10} = 86.6 \text{ N/mm}^2$$

$$M_d = Z_p f_{bd}$$

$$= 1770.8\times10^3\times86.6 = 153.34 \text{ kN m} > 146.25 \text{ kN m}$$

Step 5: Calculation of shear capacity of section

$$V_d = \frac{f_y}{\gamma_{m0}\sqrt{3}} \times D \times t_w = \frac{250}{1.1\times\sqrt{3}} \times 500 \times 9.2 = 603.59 \text{ kN}$$

$$0.6\,V_d = 362.15 \text{ kN} > 292.5 \text{ kN}$$

Step 6: Calculation of deflection

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

$$w = 32.5 \text{ kN/m}$$

$$\delta_b = \frac{5\times32.5\times6000^4}{384\times2\times10^5\times38579\times10^4} = 7.10 \text{ mm}$$

$$\text{Allowable deflection} = \frac{l}{300} = \frac{6000}{300} = 20 \text{ mm}$$

Step 7: Check for web buckling

$A_b = (b_1 + n_1)t_w$, $b_1 = 100$ mm, $n_1 = D/2 = 500/2 = 250$ mm

$\therefore A_b = (100 + 250) \times 9.2 = 3220$ mm^2

Effective length $= 0.7d = 0.7 \times 437.8 = 306.46$ mm

$$I = \frac{bt^3}{12} = \frac{100 \times 9.2^3}{12} = 6489 \text{ mm}^4$$

$$A = 100 \times 9.2 = 920 \text{ mm}^2$$

$$r_{min} = \sqrt{\frac{I}{A}} = \sqrt{(6489/920)} = 2.66 \text{ mm}$$

$$\lambda = \frac{L_{eff}}{r_{min}} = \frac{306.46}{2.66} = 115.21$$

$\therefore$ From Table 9c of the code, $f_{cd} = 88.9$ N/mm^2

Strength of the section against web buckling $= 88.9 \times 3220 = 286.26$ kN $= 292.5$ kN

Hence the section is safe against web buckling.

Step 8: Check for web bearing

$F_w = (b_1 + n_2)t_w f_y/\gamma_{m0}$

$b_1 = 100$ mm

$n_2 = 2.5(t_f + R) = 2.5 \times (14.1 + 17) = 77.75$

$F_w = (100 + 77.75) \times 9.2 \times 250/1.10 = 371.65$ kN > 292.5 kN

Hence the section is safe against web bearing.

Students must prepare answers for the following questions at the end of the lecture.

Qn No	Question	Marks	CO	Bloom's Knowledge Level
1	Design an I section purlin of the span of 4 m subjected to an LTDL of 1.5 kN/m in the plane of the minor axis and O, 5 kym in the plane of the major axis under service condition. Assume that the purlin is continuous over the supports and no lateral buckling occurs. grade of Steel is Fe 250 (Apr May 2022)	13	3	K3

Reference Book:

- N Subramaniam , Design of Steel Structures, Oxford University press, New Delhi, 2011

Unit	3. DESIGN OF BEAMS	Lecture No	22
Topic	Design of Laterally Unsupported Beams		
Learning Outcome (LO) At the end of this lecture, students will be able to		**Bloom's Knowledge Level**	
LO1	Design a beams without lateral supports	K3	

Example 10.9 *Design a laterally unrestrained beam to carry a uniformly distributed load of 50 kN/m. The beam is unsupported for a length of 1.5 m and is simply placed on longitudinal beams at its ends.*

Solution

Step 1: Calculation of load

Factored load = 1.5 × 50 = 75 kN/m

Step 2: Calculation of bending moment and shear force

$$BM = \frac{wl^2}{8} = \frac{75 \times 1.5^2}{8} = 21.09 \text{ kN m}$$

$$SF = \frac{wl}{2} = \frac{75 \times 1.5}{2} = 56.25 \text{ kN}$$

Step 3: Choosing an initial section

Assume $\lambda = 100$; $h/t_f = 25$ and hence from Table 10.9, $f_{cr,b} = 291.31 \text{ N/mm}^2$

$$\lambda_{LT} = \sqrt{\frac{f_y}{f_{cr,b}}} = \sqrt{\frac{250}{291.31}} = 0.926$$

$$\phi_{LT} = 0.5[1 + \alpha_{LT}(\lambda_{LT} - 0.2) + \lambda_{LT}^2] = 0.5[1 + 0.21(0.926 - 0.2) + 0.926^2]$$
$$= 1.005$$

$$\chi_{LT} = \frac{1}{\phi_{LT} + [\phi_{LT}^2 - \lambda_{LT}^2]^{0.5}} \leq 1.0$$

$$= \frac{1}{1.005 + [1.005^2 - 0.926^2]^{0.5}} = 0.716 \leq 1.0$$

$$f_{bd} = \frac{\chi_{LT} f_y}{\gamma_{m0}} = \frac{0.716 \times 250}{1.10} = 162.7 \text{ N/mm}^2$$

$$\text{Therefore required section} = \frac{21.09 \times 10^6}{162.7} = 129.6 \times 10^3 \text{ mm}^3$$

Choose a section of ISMB 175 @ 0.19 kN/m.

Overall depth (D) = 175 mm
Width of flange (B) = 90 mm
Thickness of flange (t_f) = 8.6 mm
Thickness of web (t_w) = 5.5 mm
Depth of web (d) = $D - 2(t_f + R) = 175 - 2(8.6 + 10) = 137.8$ mm
Moment of inertia about major axis $I_z = 1270.6 \times 10^4$ mm^4
Moment of inertia about minor axis $I_y = 85.1 \times 10^4$ mm^4
Elastic section modulus (Z_{ez}) = 145.2×10^3 mm^3
Plastic section modulus (Z_{pz}) = 161.65×10^3 mm^3
Minimum radius of gyration (r_y) = 18.6 mm

Section classification

Outstand of compression flange = 45/8.6 = 5.23 < 9.4
Web with N.A at mid depth = 137.8/5.5 = 25.05 < 83.9
Therefore the section is plastic.

Step 4: Calculation of lateral-torsional buckling moment

$$M_{cr} = \sqrt{\frac{\pi^2 EI_y}{(KL)^2}\left(GI_t + \frac{\pi^2 EI_w}{(KL)^2}\right)}$$

$$G = \frac{E}{2(1+\mu)} = \frac{2 \times 10^5}{2 \times (1 + 0.3)} = 76.923 \times 10^3 \text{ N/mm}^2$$

$$I_t = \sum \frac{b_i t_i^3}{3} = \left[\frac{2 \times 90 \times 8.6^3}{3} + \frac{(175 - 8.6) \times 5.5^3}{3}\right] = 47.39 \times 10^3 \text{ mm}^3$$

$$I_w = (1 - \beta_f)\beta_f I_y h_f^2$$

$$\beta_f = \frac{I_{fc}}{I_{fc} + I_{ft}} = 0.5$$

$$I_w = (1 - 0.5) \times 0.5 \times 85.1 \times 10^4 \times 166.4^2 = 5.89 \times 10^9 \text{ mm}^6$$

$$M_{cr} = \sqrt{\frac{\pi^2 \times 2 \times 10^5 \times 85.1 \times 10^4}{1500^2}\left(76.923 \times 10^3 \times 47.39 \times 10^3 + \frac{\pi^2 \times 2 \times 10^5 \times 5.89 \times 10^9}{1500^2}\right)}$$

$= 81.11$ kNm

Check As per the approximate equation given in the code

$$M_{cr} = \frac{\pi^2 E I_y h}{2(KL)^2}\left[1 + \frac{1}{20}\left(\frac{KL/r_y}{h/t_f}\right)^2\right]^{0.5}$$

$$\frac{KL}{r_y} = \frac{1500}{18.6} = 80.65; \frac{h}{t_f} = \frac{175}{8.6} = 20.34$$

$$M_{cr} = \frac{\pi^2 \times 2 \times 10^5 \times 85.1 \times 10^4 \times 175}{2 \times 1500^2}\left[1 + \frac{1}{20}\left(\frac{80.65}{20.34}\right)^2\right]^{0.5} = 87.30 \text{ kN m}$$

This value is comparable to 81.11 kN m obtained by the exact method (7.6% difference)

$$\lambda_{LT} = \sqrt{\frac{Z_p f_y}{M_{cr}}} = \sqrt{\frac{161.65 \times 10^3 \times 250}{81.11 \times 10^6}} = 0.7$$

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

$$\alpha_{LT} = 0.21$$

$$\phi_{LT} = 0.5[1 + 0.21(0.7 - 0.2) + 0.7^2] = 0.7975$$

$$\chi_{LT} = \frac{1}{\phi_{LT} + [\phi_{LT}^2 - \lambda_{LT}^2]^{0.5}} = \frac{1}{0.7975 + [0.7975^2 - 0.7^2]^{0.5}} = 0.848 \le 1.0$$

$$f_{bd} = \frac{\chi_{LT} f_y}{\gamma_{m0}} = \frac{0.848 \times 250}{1.10} = 192.67 \text{ N/mm}^2$$

$$M_d = Z_p f_{bd} = 161.65 \times 10^3 \times 192.67 = 21.09 \text{ kN m}$$

Step 5: Calculation of shear capacity of section

$$V_d = \frac{f_y}{\gamma_{m0}\sqrt{3}} \times D \times t_w = \frac{250}{1.1 \times \sqrt{3}} \times 175 \times 5.5 = 126.3 \text{ kN}$$

$0.6\ V_d = 75.78$ kN > 56.25 kN

Step 6: Calculation of deflection

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

$w = 50$ kN/m

$$\delta_b = \frac{5 \times 50 \times 1500^4}{384 \times 2 \times 10^5 \times 1270.6 \times 10^4} = 1.3 \text{ mm}$$

Allowable deflection $= \frac{l}{300} = \frac{1500}{300} = 5$ mm

Hence the section is safe against deflection.

Step 7: Check for web buckling

Assuming that longitudinal beams are of the same size,

$$A_b = (b_1 + n_1)t_w$$

$$b_1 = (b_f - t_w)/2 = (90 - 5.5)/2 = 42.25 \text{ mm}$$

$$n_1 = D/2 = 175/2 = 87.5 \text{ mm}$$

$$\therefore A_b = (42.25 + 87.5) \times 5.5 = 713.6 \text{ mm}^2$$

$$I = \frac{bt_w^3}{12} = \frac{42.25 \times 5.5^3}{12} = 585.8 \text{ mm}^4$$

$$A = 42.25 \times 5.5 = 232.38 \text{ mm}^2$$

$$r_{min} = \sqrt{\frac{I}{A}} = \sqrt{\frac{585.8}{232.38}} = 1.59 \text{ mm}$$

$$\lambda = \frac{l_{eff}}{r_{min}} = \frac{0.7 \times 137.8}{1.59} = 60.67$$

$\therefore f_{cd} = 167$ N/mm^2 (From Table 9c of the code)

Strength of the section against web buckling $= 167 \times 713.6 = 119.17$ kN > 56.25 kN

Hence the section is safe against web buckling.

Step 8: Check for web bearing

$$F_w = (b_1 + n_2)t_w f_y / \gamma_{m0}$$

$$b_1 = 42.25 \text{ mm};$$

$$n_2 = 2.5(t_f + R) = 2.5(8.6 + 10) = 46.5 \text{ mm}$$

$$F_w = (42.25 + 46.5) \times 5.5 \times 250/1.10 = 110.9 \text{ kN} > 56.25 \text{ kN}$$

Hence the section is safe against web bearing.

Students must prepare answers for the following questions at the end of the lecture.

Qn No	Question	Marks	CO	Bloom's Knowledge Level
1	A beam is simply supported over a span of 6m. It supports one iron beam at 10 midspan exerting 90kN. Design a laterally unsupported beam with ISWB section with flange plates (Nov Dec 2021)	13	3	K3
2	Calculate the moment carrying capacity of a laterally unrestrained ismB400 member of 3 m length.(apr may 2021)	13	3	K3

Reference Book:

- N Subramaniam , Design of Steel Structures, Oxford University press, New Delhi, 2011

Unit	3. DESIGN OF BEAMS	Lecture No	23
Topic	Design of Laterally Unsupported Beams		
Learning Outcome (LO) At the end of this lecture, students will be able to		Bloom's Knowledge Level	
LO1	Design a beams without lateral supports	K3	

Example 10.6 *Calculate the moment carrying capacity of a laterally unrestrained ISMB 400 member of length 3m.*

Solution

Section properties:

Overall depth (h) = 400 mm
Width of flange (b) = 140 mm
Thickness of flange (t_f) = 16 mm
Height of web (d) = $h - 2(t_f + R) = 400 - 2(16 + 14) = 340$ mm
Thickness of web (t_w) = 8.9 mm
Moment of inertia about major axis $I_z = 20458.4 \times 10^4$ mm^4
Moment of inertia about minor axis $I_y = 622.1 \times 10^4$ mm^4
Elastic section modulus (Z_e) = 1020×10^3 mm^3
Plastic section modulus (Z_p) = 1175.2×10^3 mm^3

As the beam is laterally unsupported, the design bending strength is found by calculating the lateral torsional buckling moment:

$$M_{cr} = \sqrt{\frac{\pi^2 EI_y}{(KL)^2}\left(GI_t + \frac{\pi^2 EI_w}{(KL)^2}\right)}$$

$$G = \frac{E}{2(1+\mu)} = \frac{2\times 10^5}{2\times(1+0.3)} = 76.923 \times 10^3 \text{ N/mm}^2$$

$$I_t = \sum \frac{b_i t_i^3}{3} = \left[\frac{2\times 140\times 16^3}{3} + \frac{(400-16)\times 8.9^3}{3}\right] = 4.725 \times 10^5 \text{ mm}^4$$

$KL = 3000$ mm

$h_f = 400 - 16 = 384$ mm

$I_w = (1 - \beta_f)\beta_f I_y h_f^2$

$$\beta_f = \frac{I_{fc}}{I_{fc} + I_{ft}} = 0.5$$

$$I_w = (1 - 0.5)\times 0.5 \times 622.1 \times 10^4 \times 384^2$$
$$= 2.29 \times 10^{11} \text{ mm}^6$$

$$M_{cr} = \sqrt{\frac{\pi^2 \times 2 \times 10^5 \times 622.1 \times 10^4}{3000^2}\left(76.923 \times 10^3 \times 4.725 \times 10^5 + \frac{\pi^2 \times 2 \times 10^5 \times 2.29 \times 10^{11}}{3000^2}\right)}$$

$= 343.68$ kN m

Section classification

Outstanding element of compression flange $= b/t_f = 70/16 = 4.38 < 9.4$

Web with neutral axis at mid section $= d/t_w = 340.4/8.9 = 38.2 < 84\varepsilon$

Hence the section is plastic.

Calculation of moment carrying capacity

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

$$= \sqrt{\frac{1.0 \times 1175.2 \times 10^3 \times 250}{343.68 \times 10^6}} = 0.925$$

$\phi_{LT} = 0.5\,[1 + \alpha_{LT}(\lambda_{LT} - 0.2) + \lambda_{LT}^2]$

$\alpha_{LT} = 0.21$

$\phi_{LT} = 0.5[1 + 0.21\,(0.925 - 0.2) + 0.925^2] = 1.004$

$$\chi_{LT} = \frac{1}{\phi_{LT} + [\phi_{LT}^2 - \lambda_{LT}^2]^{0.5}} \le 1.0$$

$$= \frac{1}{1.004 + [1.004^2 - 0.925^2]^{0.5}} = 0.717 \le 1.0$$

$$f_{bd} = \frac{\chi_{LT} f_y}{\gamma_{m0}}$$

$$= \frac{0.717 \times 250}{1.10} = 163 \text{ N/mm}^2$$

$M_d = \beta_b Z_p f_{bd}$

$= 1.0 \times 1175.2 \times 10^3 \times 163$

$= 191.56$ kN m

The moment carrying capacity of the section = 191.56 kN m.

<u>Students must prepare answers for the following questions at the end of the lecture.</u>

Qn No	Question	Marks	CO	Bloom's Knowledge Level
1	Check whether an ISMB200 section can be used a laterally unrestrained beam of length 1.5 m (simply supported) to carry a factored UDL of 50 kN/m (Apr May 2021)	13	3	K3

Reference Book:

- N Subramaniam , Design of Steel Structures, Oxford University press, New Delhi, 2011

Unit	3. DESIGN OF BEAMS	Lecture No	24
Topic	Design of Built-up Beams		
Learning Outcome (LO) At the end of this lecture, students will be able to		**Bloom's Knowledge Level**	
LO1	Design a beam with built-up sections	K3	

Example 10.10 *Steel beams having a clear span of 9 m are resting on 150-mm wide end bearings. The beams spacing is 3 m and the beams carry a dead load of 5 kN/m², including the weight of the section. The imposed load on the beam is 15 kN/m². The beam depth is restricted to 575 mm and the yield strength of the steel is 250 N/mm².*

Solution

Factored loads

Dead load = $1.5 \times 5 = 7.5$ kN/m²

Imposed load = $1.5 \times 15 = 22.5$ kN/m²

The beams are spaced at 3 m intervals, therefore the load per meter

Dead load = $7.5 \times 3 = 22.5$ kN/m

Imposed load = $22.5 \times 3 = 67.5$ kN/m

Total load = 90 kN/m

Effective span = clear span + 2 × half the width of the end bearing

$= 9 + 2 \times 0.075 = 9.15$ m

$$\text{Reactions at support} = \frac{90 \times 9.15}{2} = 411.75 \text{ kN}$$

$$\text{Mid span moment} = \frac{wl^2}{8} = \frac{90 \times 9.15^2}{8} = 941.878 \text{ kN m}$$

Selection of the section

Plastic section modulus required

$$Z_p = \frac{M \times \gamma_{m0}}{f_y} = \frac{941.878 \times 10^6 \times 1.10}{250} = 414.43 \times 10^4 \text{ mm}^3$$

The section with the largest plastic modulus under 575 mm depth restriction is ISWB 550 @ 1.125 kN/m. The plastic section modulus of this section is 306.6×10^4 mm^3 which is less than the required value. The section must be strengthened with additional plates to provide the required plastic section modulus. The stiffness required to be provided to restrict the deflection problem can be calculated as follows:

Maximum deflection = Effective span/360 = 9150/360 = 25.42 mm.

The moment of inertia of the required beam section that can satisfy the deflection requirement mentioned earlier due to unfactored imposed load

$$I_z = \frac{5}{384} \times \frac{45 \times 9150^4}{2 \times 10^5 \times 25.42} = 80784 \times 10^4 \text{ mm}^4$$

Hence provide ISWB 550 with 10 mm cover plates which gives $I_z = 106{,}266.1 \times 10^4$ mm^4.

Check for shear

Shear capacity of the section

$$V_d = \frac{f_y}{\gamma_{m0} \times \sqrt{3}} \times h \times t_w = \frac{250}{1.1 \times \sqrt{3}} \times 550 \times 10.5 = 757.7 \text{ kN}$$

$0.6V_d = 0.6 \times 757.7 = 454.663$

The maximum shear force is 411.75 kN and it is less than $0.6V_d$.

Check for plastic modulus

The plastic modulus required for the section is 414.43×10^4 mm^3. ISWB 550 provides 306.6×10^4 mm^3. Assumed thickness of the plate is 10 mm and the total depth of the beam is 570 mm. Distance between the c/c of the plates is 560 mm.

Additional plastic section modulus to be provided by the plate = 107.83×10^4 mm^3

$$\text{Required area of the plate} = \frac{107.83 \times 10^4}{560} = 1925.535 \text{ mm}^2$$

The provided area of the plate = 2000 mm^2. The plastic modulus Z_p of the compound section = $306.6 \times 10^4 + 2000 \times (550 + 10) = 418.6 \times 10^4$ mm^3, which is

Check for deflection

Minimum I_z required is 80784×10^4 mm^4

I_z provided by ISWB 550 = 74906.1×10^4 mm^4

I_z to be provided by the plates = 5878×10^4 mm^4

I_z provided by the plates = $2 \times 200 \times 10 \times 280^2 = 31360 \times 10^4$ mm^4.

Total I_z provided = 106266.1×10^4 mm^4 greater than the I_z required.

Curtailment of flange plates

In the case of plated beams, the flange plates may be curtailed near the supports of the beam to save some steel. The theoretical cut off points may be calculated using the algebraic method or by a geometrical construction. For the above problem the theoretical cut off points are computed using the algebraic method. The bending moment capacity for the beam ISWB 550

$$M = 306.6 \times 10^4 \times 250 = 766.5 \text{ kN m}$$

At any point, distance x from the support

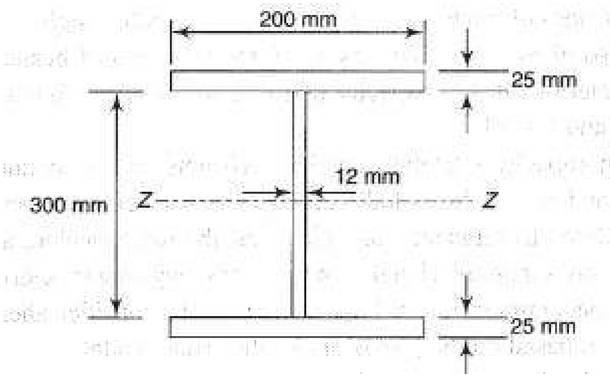
$$M_z = Rx - \frac{wx^2}{2}, \text{ where } M_z = 766.5 \text{ kN m}, R = 411.75 \text{ kN, and } w = 90 \text{ kN/m}$$

Substituting the values

$$766.5 \times 10^6 = 411.75 \times 10^3 \times x - \frac{90 \times x^2}{2}$$

Solving this equation x = 6500 mm or 2600 mm. Hence the theoretical cut-off point is 2600 from either side of the support.

<u>Students must prepare answers for the following questions at the end of the lecture.</u>

Qn No	Question	Marks	CO	Bloom's Knowledge Level
1	Determine the following for this built-up beam:  a. Elastic Section Modulus Z_e and yield Moment b. Plastic Section Modulus Z_p and Plastic Moment c. Bending moment about Z-Z axis considering f_y=250N/mm^2.	13	3	K3

Reference Book:

- N Subramaniam , Design of Steel Structures, Oxford University press, New Delhi, 2011

Unit	3. DESIGN OF BEAMS	Lecture No	25
Topic	Design of Plate Girders		
Learning Outcome (LO) At the end of this lecture, students will be able to		**Bloom's Knowledge Level**	
LO1	Design a Plate Girders - Normal	K3	

Example 11.4 *Design a welded plate girder for a simply supported bridge deck beam with clear span of 20 m, subjected to the following:*

Dead load including self weight = 20 kN/m

Imposed load = 10 kN/m

Two moving loads = 150 kN each spaced 2 m apart

Assume that the top compression flange of the plate girder is restrained laterally and prevented from rotating. Use mild steel with f_y = 250 MPa. Design as an unstiffened plate girder with thick webs.

Solution

(1) Loading

Total factored udl on girder = 1.5 × (20 + 10) = 45 kN/m

Factored moving loads = 1.5 × 150 = 225 kN

(2) Maximum Bending Moment

$$R_A \text{ and } R_B \text{ due to UDL} = \left(\frac{45 \times 20}{2}\right) = 450 \text{ kN}$$

The maximum bending moment due to UDL occurs at mid span; whereas the maximum bending moment due to the two moving loads will occur under one of the loads, when one of the loads is placed such that the middle of the beam divides the distance between this load (under which maximum bending moment occurs) and CG of the moving loads equally {see Fig. 11.41(b)}.

CG of the loads = $a/2$(from the loads under consideration)

where a is the distance between the moving loads. The distance of the loads from middle of the beam = $a/4$. Hence,

$$\text{Distance of the CG of loads from right support} = L/2 - a/4 = 20/2 - 2/4 = 9.5 \text{ m}$$

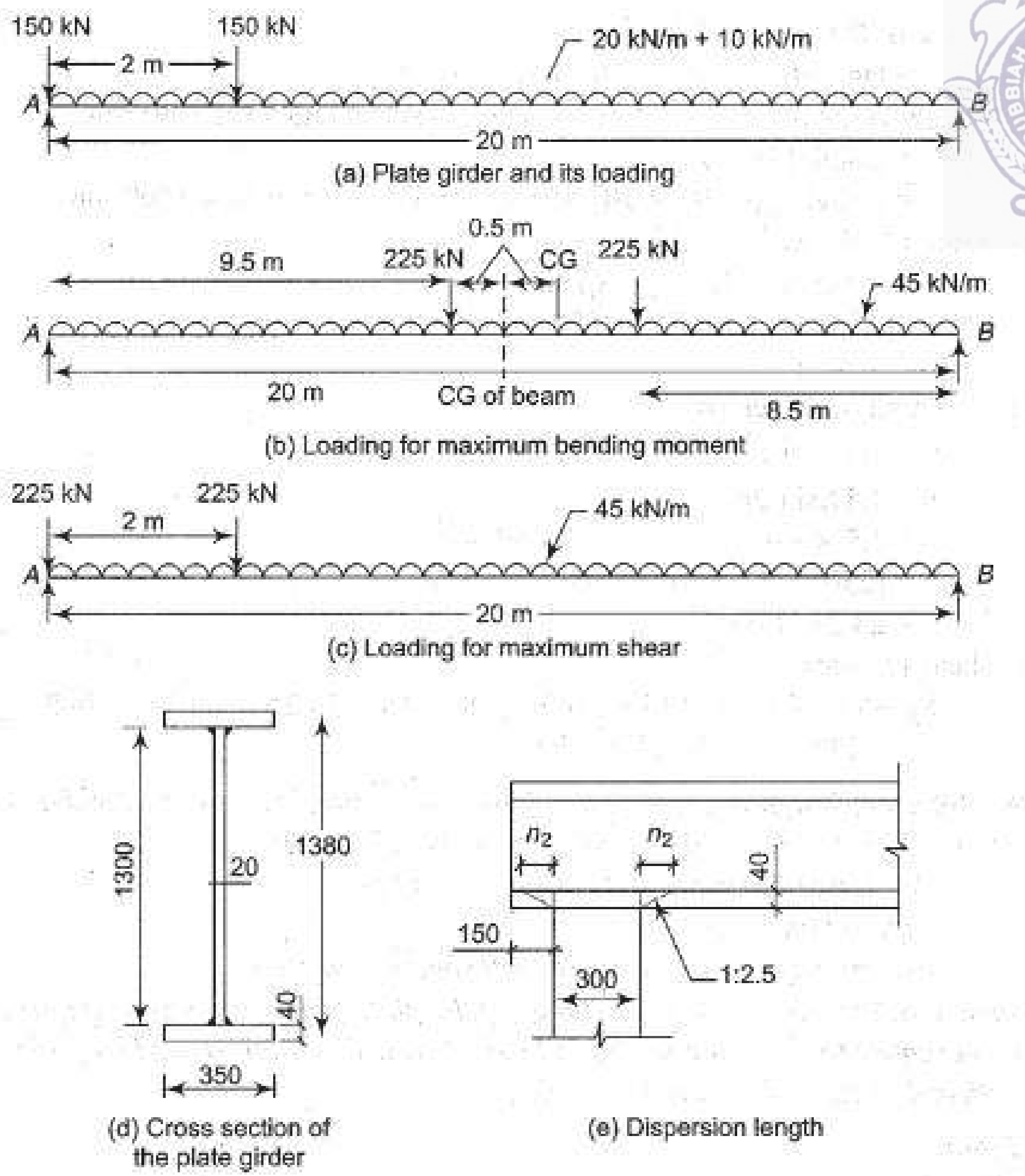


Fig. 11.41

Since the maximum BM under UDL is at a different location from that of maximum BM under moving loads, we have to compute the bending moment in both cases and choose the higher of the two. However, in our case the bending moment occurred under one of the moving loads.

$$R_A \text{ due to moving load} = \frac{1}{20}[(225 \times 10.5) + (225 \times 8.5)] = 213.75 \text{ kN}$$

$$\therefore \quad R_B \text{ due to moving load} = (2 \times 225) - 213.75 = 236.25 \text{ kN}$$

$$\therefore \quad \text{Total } R_B = 450 + 236.25 = 686.25 \text{ kN}$$

$$\text{Total } R_A = 450 + 213.75 = 663.75 \text{ kN}$$

$$\text{Maximum BM} = (663.75 \times 9.5) - \left(\frac{45 \times 9.5^2}{2}\right) = 4275 \text{ kN m}$$

(3) Maximum Shear Force

For maximum shear force, the wheel loads should be placed such that one of the wheel loads should be very close to the supports.

$$R_A \text{ due to moving load} = \frac{1}{20}[(225 \times 20) + (225 \times 18)] = 427.5 \text{ kN}$$

Total $R_A = 427.5 + 450 = 877.5$ kN

$\therefore$ Maximum shear force, $V_z = 877.5$ kN

Optimum depth of plate girder

$$d = (Mk/f_{yf})^{0.33} = (4275 \times 10^6 \times 180/250)^{0.33} = 1352 \text{ mm}$$

(Assuming $k = 180$)

Assume $d = 1300$ mm.

Optimum value of thickness of web

$$t_w = [M/(f_y k^2)]^{0.33} = [4275 \times 10^6/(250 \times 180^2)]^{0.33} = 22.97 \text{ mm}$$

There are two design requirements regarding the minimum web thickness for the condition of no intermediate stiffeners (clause 8.6.1.1 and 8.6.1.2 of code)

$d/t_w < 200\varepsilon$ (for serviceability)

$d/t_w \le 345\varepsilon_f^2$ (to avoid flange buckling)

If the web is deliberately made thick, i.e., $d/t < 67\varepsilon$ (clause 8.4.2.1), these requirements are automatically met. Thus, the minimum web thickness should be as follows:

$$t_w > \frac{1300}{67 \times 1} \text{ or } t_w > 19.403 \text{ mm}$$

Hence, provide $t_w = 20$ mm

In order to maximize the moment capacity, the cross section of the plate girder should be so proportioned that it satisfies the requirements of plastic/compact section. Thus b_f/t_f should be less than 8.4ε or 9.4ε for plastic and compact sections respectively (see Table 2 of the code). Assuming b_f as 0.3 times the depth of web $b_f = 0.3 \times 1300 = 390$ mm. Provide $b_f = 350$ mm; $t_f > b_f/(2 \times 8.4) = 20.8$ mm. Provide

$t_f = 40$ mm.

Use 1300 mm $\times$ 20 mm web plates with flange plates of 350 mm $\times$ 40 mm.

As discussed earlier, it is a *plastic section.*

(4) Shear capacity

As per clause 8.4 of the code

$$V \le V_d$$

$$V_d = \frac{V_n}{\gamma_{m0}}$$

Nominal plastic shear resistance, $V_n = V_p = \dfrac{A_v f_{yw}}{\sqrt{3}}$

As per clause 8.4.1.1 of the code, for welded section, $A_v = dt_w$

$$\therefore \quad V_n = \frac{dt_w f_{yw}}{\sqrt{3}} = \frac{1300 \times 20 \times 250}{\sqrt{3} \times 1.1 \times 10^3} = 3411.6 \text{ kN} > V_z = 877.5 \text{ kN}$$

(5) Moment Capacity

According to clause 8.2.1.2 of the code, design bending strength,

$$M_d = \beta_b Z_p \frac{f_y}{\gamma_{m0}} \leq 1.2 Z_e \frac{f_y}{\gamma_{m0}}$$

Plastic modulus, $Z_p = 2b_f t_f (D - t_f)/2 + t_w d^2/4$

$= 2 \times 350 \times 40 \times 1340/2 + 20 \times 1300^2/4$

$= 27.21 \times 10^6$ mm^3

$$M_d = 1.0 \times 27.21 \times 10^6 \times \frac{250}{1.10 \times 10^6} = 6184.09 \text{ kN m} > M_z = 4275 \text{ kN m}$$

Hence safe to carry the applied moment.

As the compression flange is restrained, there is no need to check for lateral-torsional buckling.

(6) Check for bearing stiffeners

(a) At the supports [see Fig. 11.41(e)]

Assume that the width of support is 300 mm and that the minimum stiff bearing provided by the support $b_1 = 300/2 = 150$ mm,

Dispersion length (1:2.5), $n_2 = 2.5 \times 40 = 100$ mm

Loc: ' capacity of the web, $F_w = (b_1 + n_2) t_w \frac{f_{yw}}{\gamma_{m0}}$

$$F_w = (100 + 150) \times 20 \times \frac{250}{1.10 \times 10^3} = 1136.35 \text{ kN} > \text{support reaction}$$

$= 877.5$ kN

(b) At position of moving wheel loads

$$F_w = (b_1 + n_2) t_w \frac{f_{yw}}{\gamma_{m0}}$$

Assuming, $b_1 = 0$

$$F_w = (0 + 2.5 \times 2 \times 40) \times 20 \times \frac{250}{1.10 \times 10^3} = 909.09 \text{ kN} > 225 \text{ kN}$$

The associated buckling resistance F_{qd} is dependent on the slenderness of the unstiffened web (clause 8.7.1.5).

Slenderness ratio of the web $= L_e/r_y = 0.7L/r_y$ [with $r_y = t/2\sqrt{3}$, see Eqn (10.52)]

$= 2.5d/t = 2.5 \times 1300/20 = 162.5$

From Table 9c of the code $f_{cd} = 52$ N/mm^2 and as per clause 8.7.1.3 of the code,

Stiff bearing length = 0 (assumed)

45° dispersion length (to the level of half the depth of beam) = 1380/2 = 690 mm

$$F_{qd} = (0 + 690) \times 20 \times 52/10^3 = 717.6 \text{ kN} > 225 \text{ kN}$$

The web is adequate at both supports and positions of concentrated loads. Hence, there is no need to provide bearing stiffeners.

Students must prepare answers for the following questions at the end of the lecture.

Qn No	Question	Marks	CO	Bloom's Knowledge Level
1	Design an 18-m long simply supported welded plate girder carrying a uniformly distributed load of 50 kN/m excluding self weight, and two concentrated loads of 350 kN each at quarter points of the span. Assume that the girder is laterally supported throughout.	13	3	K3

Reference Book:

- N Subramaniam , Design of Steel Structures, Oxford University press, New Delhi, 2011

Unit	3. DESIGN OF BEAMS	Lecture No	26
Topic	Design of Plate Girders – Thin Web		
Learning Outcome (LO) At the end of this lecture, students will be able to		**Bloom's Knowledge Level**	
LO1	Design a Plate Girders with Thin web members	K3	

Example 11.5 *Redesign the plate girder given in Example 11.4 using thin web plates.*

Solution

From Example 11.4,

$$M_z = 4275 \text{ kNm}$$

$$F_z = 877.5 \text{ kNm}$$

As shown in Example 11.4,

optimum depth = 1352 mm and $t_w = 22.97$ mm

Let us assume web plate of 1200 mm × 12 mm size. Assuming that the bending moment is carried by the flanges,

$$A_f = M_z \times \gamma_{mo}/(f_y \times D) = 4275 \times 10^6 \times 1.1/(250 \times 1200) = 15675 \text{ mm}^2$$

Limiting 8.4ε for the plastic section, the approximate flange thickness is

$$t_f = \sqrt{15675/(2 \times 8.4)} = 30.5 \text{ mm.}$$

Provide $t_f = 35$ mm.

$$b_f = 15675/35 = 447.86 \text{ mm}$$

Provide $b_f = 450$ mm.

The cross section of the plate girder is shown in Fig. 11.42.

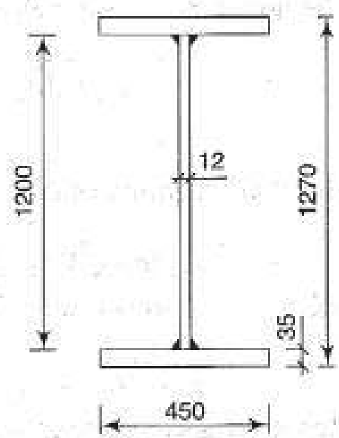


Fig. 11.42 Cross section of plate girder

According to clause 8.2.1 of the code,

$$\frac{d}{t_w} = \frac{1200}{12} = 100 > 67\varepsilon$$

(1) Moment capacity check

As discussed earlier, the flanges are *plastic*

$$\text{Moment of resistance (clause 8.2.1.2)} = \frac{\beta_b Z_p f_y}{\gamma_{m0}}$$

Considering that the flanges only resist the bending moment,

$$Z_p = 2b_f t_f (D - t_f)/2 = 2 \times 450 \times 35(1270 - 35)/2 = 19.45 \times 10^6 \text{ mm}^3$$

$$M_d = 1 \times 19.45 \times 10^6 \times \frac{250}{1.10 \times 10^6} = 4420 \text{ kN m} > 4275 \text{ kNm}$$

(2) Shear resistance of the web

As per clause 8.6.1.1a of the code, for serviceability

$$\frac{d}{t_w} = \frac{1200}{12} = 100 < 200\varepsilon$$

As per clause 8.6.1.2a of the code to avoid flange buckling

$$\frac{d}{t_w} \le 345\varepsilon_f^2$$

$$\frac{1200}{12} = 100 < 345\varepsilon_f^2$$

Hence the minimum web thickness requirements are met.
Let us consider the simple post-critical method. According to clause 8.4.2.2(a) of the code,

Nominal shear strength $V_n = V_{cr}$

$k_v = 5.35$, when transverse stiffeners are provided only at supports.

The elastic critical shear stress of the web

$$\tau_{cr} = \frac{k_v \pi^2 E}{12(1-\mu^2)\left(\frac{d}{t_w}\right)^2}$$

With $\mu = 0.3$,

$$\tau_{cr} = \frac{5.35 \times \pi^2 \times 2 \times 10^5}{12(1-0.3^2) \times 100^2} = 96.7 \text{ N/mm}^2$$

The non-dimensional web slenderness ratio for shear buckling stress,

$$\lambda_w = \sqrt{[f_{yw}/(\sqrt{3}\tau_{cr})]} = \sqrt{\frac{250}{(\sqrt{3} \times 96.7)}} = 1.22 > 1.20$$

Shear stress corresponding to buckling $\tau_b = f_{yw}/(\sqrt{3}\lambda_w^2)$

$$\tau_b = 250/(\sqrt{3} \times 1.22^2) = 96.97 \text{ N/mm}^2$$

Shear force corresponding to web buckling,

$$V_{cr} = d t_w \tau_b$$
$$= 1200 \times 12 \times 96.97/10^3 = 1396.3 \text{ kN} > 877.5 \text{ kN}$$

Hence the shear strength is greater than the applied shear.

(3) Lateral torsional buckling

As the compression flange is restrained, there is no need to check for lateral torsional buckling.

(4) Design of load carrying stiffeners

(i) At the position of applied wheel load:

As per clause 8.7.4 of the code, the local capacity of web

$$F_w = (b_1 + n_2) t_w \frac{f_{yw}}{\gamma_{m0}}$$

$$n_2 = 2 \times 35 \times 2.5 = 175 \text{ mm}$$
$$b_1 = 0$$

$$F_w = [0 + 175] \times 12 \times \frac{250}{1.10 \times 10^3} = 477.2 \text{ kN} > 225 \text{ kN}$$

The buckling resistance depends on the slenderness ratio of the web.
According to clause 8.7.1.5 of the code,

$$\lambda = 2.5\frac{d}{t} = 2.5 \times \frac{1200}{12} = 250$$

$n_1 = D/2 = 1270/2 = 635$

From Table 9c of the code, $f_{cd} = 24.3\text{N/mm}^2$

$\therefore$ Buckling resistance $= (0 + 2 \times 635) \times 12 \times 24.3/10^3 = 370.3$ kN > 225 kN

No stiffeners are necessary to prevent local buckling failure of the web at the point load position.

(ii) At the supports:

The reaction is 877.5 kN, which is much greater than the point load of 225 kN. Hence stiffeners are necessary for both the supports.

Try a pair of 220 × 12 mm flats.

As per clause 8.7.1.2 of the code,

Maximum outstand of stiffener $= 20t_q\varepsilon = 20 \times 12 \times 1 = 240$ mm > 220 mm

$= 14 \times 12 \times 1 = 168$ mm < 220 mm

This means that the core area of the stiffener should be reduced to 2 × 168 mm × 12 mm.

(a) Check stiffener for buckling

As per clause 8.7.1.5, the effective section is the core area of the stiffener together with an effective length of web, on either side of the centre line of the stiffener of $20 \times t_w$. At the support, assuming that the web is available only on one side of the edge stiffener,

Local buckling resistance,

Area $= 2 \times 168 \times 12 + 20 \times 12 \times 12 = 6912$ mm^2

$$I_x = \frac{12 \times 336^3}{12} + \frac{(20 \times 12)12^3}{12} = 37.967 \times 10^6 \text{ mm}^4$$

$$r = \sqrt{I_x/A} = \sqrt{\frac{37.967 \times 10^6}{6912}} = 74.11 \text{ mm}$$

$$\lambda = 0.7 \times \frac{1200}{74.11} = 11.33$$

From Table 9c of the code, $f_{cd} = 226$ N/mm^2

Buckling resistance $= 226 \times 6912/10^3 = 1562.1$ kN > 877.5 kN

Hence the stiffener is safe.

Provide a pair of 220 × 12 mm stiffener at both the ends of the girder. Now calculate the bearing capacity of the end stiffener.

$$F_{psd} = A_q f_{yq}/(0.8 \times \gamma_{m0}) = 2(220 - 15) \times 12 \times 250/(0.8 \times 1.1) \times 1000$$
$$= 1397.7 > 877.5 \text{ kN}$$

Hence the stiffener is safe.

In the preceding equation, A_q is the area of the stiffener in contact with the flanges. Normally, the flanges and web will be welded together, before the stiffeners are fitted. This means that the inside corners of the stiffener need to be coped/chamfered at the junction of the web and flanges, so that they will not foul with the web/flange weld. The chamfered width is taken as 15 mm.

(b) Torsional restraint provided by end stiffener

However, it might be deemed necessary that the ends of the plate girder are by checking the second moment of area of the end bearing stiffeners at the supports as per clause 8.7.9 of the code.

$$I_s \geq 0.34\alpha_s D^3 t_{cf}$$

Assuming that there is no restraint to the compression flange for the above situation,

$$r_y \text{ of the girder} = \sqrt{\frac{I_y}{A}}$$

$$I_y = \frac{2t_f b_f^3}{12} + \frac{dt_w^3}{12} = 2 \times 35 \times 450^3/12 + 1200 \times 12^3/12$$

$$= 531.73 \times 10^6 \text{ mm}^4$$

$$A = 2 \times 450 \times 35 + 1200 \times 12 = 45900 \text{ mm}^2$$

$$r_y = \sqrt{(531.73 \times 10^6/45900)} = 107.6 \text{ mm}$$

$$\lambda = 20 \times 1000/107.6 = 185.8 > 100$$

Hence,

$$\alpha_s = 30/\lambda^2 = 30/185.8^2 = 8.69 \times 10^{-4}$$

$$I_s \geq 0.34 \times 8.69 \times 10^{-4} \times 1270^3 \times 35 = 21.183 \times 10^6 \text{ mm}^4$$

Now $I_s = 12 \times (2 \times 220)^3/12 = 85.184 \times 10^6 \text{ mm}^4 > 21.183 \times 10^6 \text{ mm}^4$

Hence the provided stiffener has the necessary torsional restraint.

(c) Weld at web flange junction

Assuming fillet weld on each side of the web,

$$q_w = VA_f\bar{y}/2I_z$$

$$I_z = b_f D^3/12 - (b_f - t_w)d^3/12$$

$$= 450 \times 1270^3/12 - (450 - 12)1200^3/12$$

$$= 13742.3 \times 10^6 \text{ mm}^4$$

$$q_w = 877.5(450 \times 35) \times 635/(2 \times 13742.3 \times 10^6)$$

$$= 0.319 \text{ kN/mm}$$

From Table 6.6, provide 4 mm fillet weld (0.442 kN/mm)

(d) Weld for end stiffener

The minimum weld size required for connecting the stiffeners to the web, assuming a weld on each side of the stiffener is (clause 8.7.2.6 of the code)

$$q_1 = t_w^2/5b_s = 12^2/(5 \times 220) = 0.13 \text{ kN/mm}$$

when the stiffener also is subjected to external loading, then shear due to such loading must be added to the above value. The stiffeners have to resist the difference between the applied load and the minimum load that can be carried safely by the unstiffened web (370.3 kN in this case).

Length of weld = 1200 – 2 × 15 = 1170 mm

$$q_2 = (877.5 - 370.3)/1170 = 0.43 \text{ kN/mm}$$

$$q_w = q_1 + q_2 = 0.13 + 0.43 = 0.56 \text{ kN/mm}$$

Force on each weld = 0.56/2 = 0.28 kN/mm

From Table 6.6, provide a pair of 4 mm fillet welds (0.442 kN/mm).

<u>Students must prepare answers for the following questions at the end of the lecture.</u>

Qn No	Question	Marks	CO	Bloom's Knowledge Level
1	Determine the buckling resistance moment for a welded plate girder consisting of 500 x 30 mm flange plates and a 1250 x 12 mm web plate in grade 4 IO steel. Assume a laterally braced span of5.5 m.	13	3	K3

Reference Book:

- N Subramaniam , Design of Steel Structures, Oxford University press, New Delhi, 2011

Unit	3. DESIGN OF BEAMS	Lecture No	27
Topic	Design of Plate Girders – with Intermediate Stiffeneres		
Learning Outcome (LO) At the end of this lecture, students will be able to		**Bloom's Knowledge Level**	
LO1	Design a Plate Girders with Intermediate Stiffeners	K3	

Example 11.6 *Redesign the plate girder in Example 11.5 with intermediate stiffeners and not using tension field action.*

Solution

From Example 11.5,

$M_z = 4275$ kN m

$F_z = 877.5$ kN m

Assume a web plate of size 1400 × 8 mm and a flange plate of size 450 × 35 mm for each of the flanges. Since the flanges are of the same size as in Example 11.5, the calculations for moment capacity check are not repeated here.

(1) Check for shear

According to clause 8.4.2.2 (a) of the code,

Nominal shear strength $V_n = V_{cr}$

Assume $c/d = 1.4$; Hence stiffener spacing = 1.4 × 1400 = 1960 mm

Adopt a stiffener spacing of 2000 mm. Provide 10 panels @ 2000 mm c/c (see Fig. 11.43).

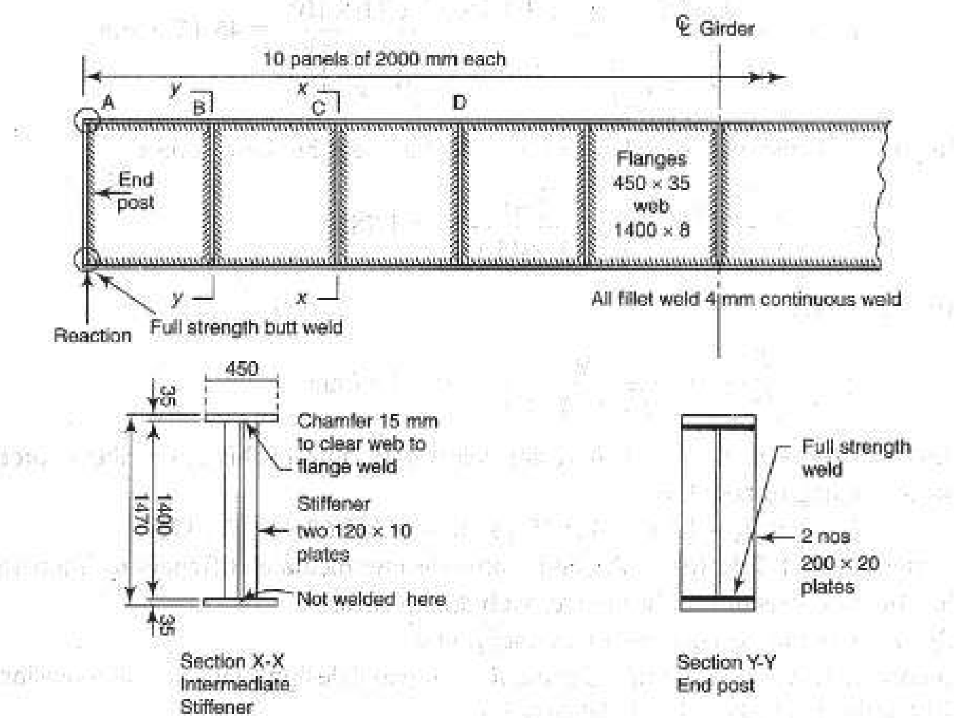


Fig. 11.43

Panel AB is the most critical panel for maximum shear. So design checks for the web are made for panel AB only. For the panel AB,

$$\frac{d}{t} = \frac{1400}{8} = 175$$

Since c (2000 mm) > d (1400 mm) and $d/t_w < 200$, the girder satisfies serviceability requirements.

$$\frac{c}{d} = \frac{2000}{1400} = 1.4286$$

$$d/t_w = 175 < 345\varepsilon_f$$

Hence as per clause 8.6.1.2 buckling of compression flange into the web will be avoided.

$$\text{For } \frac{c}{d} > 1.0;\ k_v = 5.35 + \frac{4.0}{\left(\frac{c}{d}\right)^2} = 5.35 + \frac{4.0}{(1.4286)^2} = 7.3099$$

with $\mu = 0.3$

Average shear stress

$$q_v = \frac{877.5 \times 10^3}{1400 \times 8} = 78.35 \text{ N/mm}^2$$

The elastic critical shear stress of the web

$$\tau_{cr} = \frac{k_v \pi^2 E}{12(1-\mu^2)\left(\frac{d}{t_w}\right)^2} = \frac{7.3099 \times \pi^2 \times 2.0 \times 10^5}{12(1-0.3^2)\left(\frac{1400}{8}\right)^2} = 45.15 \text{ N/mm}^2$$

The non-dimensional web slenderness ratio for shear buckling stress,

$$\lambda_w = \sqrt{\frac{250}{(\sqrt{3}\tau_{cr,e})}} = \sqrt{\frac{250}{(\sqrt{3} \times 45.15)}} = 1.788$$

when $\lambda_w > 1.2$

$$\tau_b = \frac{f_{yw}}{(\sqrt{3}\lambda_w^2)} = \frac{250}{(\sqrt{3} \times 1.788^2)} = 45.15 \text{ N/mm}^2$$

Note that the values of τ_{cr} and τ_b are nearly the same in this case. Shear force corresponding to buckling

$$V_{cr} = dt_w\tau_b = 1400 \times 8 \times 45.15/1000 = 505.68 < 877.5 \text{ kN}$$

As per clause 8.7.1.1(a) of the code, provide intermediate stiffeners to improve the buckling strength of the slender web due to shear.

(2) Check for shear capacity of the end panel

According to clause 8.5.1 of the code, for end panel design (without using tension field action), we should use clause 8.5.3,

$$H_q = 1.25V_{dp}\left(1 - \frac{V_{cr}}{V_{dp}}\right)^{1/2}$$

where

$$V_{dp} = dtf_{yw}/\sqrt{3} = \frac{(1400 \times 8)250}{\sqrt{3}} = 1616.58 \text{ kN}$$

$V_{cr} = 505.68$ kN

$\therefore$ Longitudinal shear

$$H_q = 1.25 \times 1616.58\left(1 - \frac{505.68}{1616.58}\right)^{1/2} = 1675.12 \text{ kN}$$

$$R_{tf} = \frac{H_q}{2} = \frac{1675.12}{2} = 837.56 \text{ kN}$$

$$A_v = t_w d = 8 \times 1400 = 11200 \text{ mm}^2$$

$$V_n = \frac{A_v f_{yw}}{\sqrt{3}\gamma_{m0}} = \frac{250}{\sqrt{3} \times 1.10} \times \frac{11200}{1000} = 1469.6 \text{ kN} > 837.56 \text{ kN}$$

The end panel is safe to carry the shear due to anchoring forces. Check for the moment capacity of end panel:

$$M_{tf} = \frac{H_q d}{10} = \frac{1675.12 \times 1400 \times 10^3}{10 \times 10^6} = 234.5 \text{ kN m}$$

$$y = \frac{c}{2} = \frac{2000}{2} = 1000 \text{ mm}$$

$$I = \frac{1}{12} t_w \times c^3 = \frac{1}{12} \times 8 \times 2000^3 = 533.33 \times 10^7 \text{ mm}^4$$

$$M_q = \frac{I}{y}\frac{f_y}{\gamma_{m0}} = \frac{533.33 \times 10^7}{1000 \times 10^6} \times \frac{250}{1.10} = 1212.12 \text{ kN m} > M_{tf} = 234.5 \text{ kN m}$$

$$M_{tf} < M_q$$

Hence, the end panel can carry the bending moment due to anchor forces.

(3) Design of stiffeners

Load bearing stiffener at point A (see Fig.11.43),

Reaction at A = 877.5 kN

$$\text{Force } F_m \text{ due to } M_{tf} = \frac{M_{tf}}{c} = \frac{234.5 \times 10^6}{2000 \times 10^3} = 117.25 \text{ kN}$$

Total compression $F_c = 877.5 + 117.25 = 994.75$ kN

According to clause 8.7.5.2 of the code,

$$\text{Area of stiffener } A_q > \left(\frac{0.8 \times F_c \times \gamma_{m0}}{f_{yq}}\right)$$

$$A_q > \left(\frac{0.8 \times 994.75 \times 1.1 \times 10^3}{250}\right) \text{ or } A_q > 3501 \text{ mm}^2$$

Provide stiffener of two flats of size 200 × 20 mm.

Area = 8000 mm^2 > 3501 mm^2

(a) Check for outstand: (clause 8.7.1.2 of the code)

Outstand $b_s = 200 \text{ m} < 20 t_q \varepsilon = 20 \times 20 \times 1 = 400$ mm

$14 t_q \varepsilon = 14 \times 20 \times 1 = 280$ mm

$b_s = 200 \text{ mm} < 14 t_q \varepsilon$

(b) Buckling check: (clause 8.7.1.5 of the code)
Neglecting the buckling resistance of the web for simplicity,

$$I_x = \left(\frac{20 \times 408^3}{12}\right) - \left(\frac{1}{12} \times 20 \times 8^3\right)$$

$$= 113.19 \times 10^6 \text{ mm}^4$$

Effective area $= 200 \times 20 \times 2 = 8000 \text{ mm}^2$

$$\text{Radius of gyration} = \sqrt{\frac{I_x}{A}} = \sqrt{\frac{113.19 \times 10^6}{8000}} = 118.951$$

Flange is restrained against rotation and lateral deflection.

$$L_e = 0.7L = 0.7 \times 1400 = 980 \text{ mm}$$

$$\lambda = \frac{L_e}{r_x} = \frac{980}{118.951} = 8.238 < 10$$

From Table 9c, for $f_y = 250 \text{ N/mm}^2$

$$f_{cd} = 227 \text{ N/mm}^2$$

Buckling resistance of the stiffener

$$P_d = f_{cd} A_e = 227 \times 2 \times 200 \times 20 = 1816 \text{ kN} > 994.75 \text{ kN}$$

Hence, the stiffener is safe against buckling.

(c) Check stiffener as load bearing stiffener

Assume stiff bearing length,

$$b_1 = 0$$

$$n_2 = 2.5 \times 35 = 87.5 \text{ mm}$$

According to clause 8.7.4 of the code,

$$\text{Local capacity of web } F_w = (b_1 + n_2) t_w \frac{f_{yw}}{\gamma_{m0}}$$

$$F_w = (87.5) \times 8 \times \frac{250}{1.10 \times 10^3} = 159.09 \text{ kN}$$

Bearing stiffener is designed for $(F_c - F_w) = (994.75 - 159.09) = 835.66$ kN

Bearing capacity of stiffener alone

$$= \frac{250}{1.10} \times \frac{200 \times 20 \times 2}{1000} = 1818.18 \text{ kN} > 835.66 \text{ kN}$$

Hence the stiffener is safe as a load bearing stiffener.

At the location of concentrated loads local capacity of the web

$$= (87.5 \times 2) \times 8 \times 250/(1.10 \times 10^3) = 318.16 \text{ kN} > 225 \text{ kN}$$

Hence, the stiffener is safe.

(4) Design of intermediate stiffener at 'B' (clause 8.7.2.4 of the code)

Stiffener B is the most critical intermediate stiffener.

(a) Minimum Stiffeners

$$\frac{c}{d} = \frac{2000}{1400} = 1.429 > \sqrt{2}$$

$$I_s \geq 0.75 \times 1400 \times 8^3 = 0.5376 \times 10^6 \text{ mm}^4$$

Try intermediate stiffener of two flats of 120 × 10 mm.

$$\text{Provided } I_s = \left(\frac{10 \times 248^3}{12}\right) - \left(\frac{10 \times 8^3}{12}\right) = 12.71 \times 10^6 \text{ mm}^4$$

Hence, the stiffeners have more than the minimum required stiffeness.

(b) Check for outstand (clause 8.7.1.2):

Outstand of the stiffener = b_s = 120 mm

$14 t_q \varepsilon = 14 \times 10 = 140$ mm

$b_s < 14 t_q \varepsilon$

Hence the outstand provisions of the code are satisfied.

(c) Buckling check (clause 8.7.2.5):

$$F_q = \left[\frac{V - V_{cr}}{\gamma_{m0}}\right]$$

where V is the factored shear force and V_{cr} is the shear buckling resistance of the web panel designed without using tension field action = 505.68 kN

Shear force @ B,

$$V_B = 877.5 - 45 \times 2 = 787.5 \text{ kN}$$

$$F_q = \frac{(787.5 - 505.68)}{1.10} = 256.2 \text{ kN}$$

Buckling resistance of intermediate stiffener at B (clause 8.7.1.5):

As per clause 8.7.1.5, an effective length of web equal to 20 × t_w on each side of the centre line of stiffener can be considered along with the stiffener.

$$20 t_w = 20 \times 8 = 160 \text{ mm}$$

$$I_x = \left(\frac{1}{12} \times 10 \times 248^3\right) + \left(\frac{320 \times 8^3}{12}\right) - \left(\frac{10 \times 8^3}{12}\right) = 12.724 \times 10^6 \text{ mm}^4$$

$$\text{Area} = (240 \times 10) + (320 \times 8) = 4960 \text{ mm}^2$$

$$r_x = \sqrt{\frac{12.724 \times 10^6}{4960}} = 50.649 \text{ mm}$$

$$L_e = 0.7L = 0.7 \times 1400 = 980 \text{ mm}$$

$$\lambda = \frac{L_e}{r_x} = \frac{980}{50.649} = 19.349$$

From Table 9c of the code, for f_y = 250 MPa (buckling curve c) and λ = 19.349,

$$f_{cd} = 224.195 \text{ N/mm}^2$$

Buckling resistance = 224.195 × 4960 = 1112.01 kN > 256.2 kN

Hence, the stiffener is safe against buckling.

According to clause 8.7.2.5 of the code:

Intermediate stiffener @ D subjected to external load should satisfy the following interaction equation.

$$\left(\frac{F_q - F_x}{\ldots}\right) + \left(\frac{F_x}{\ldots}\right) + \left(\frac{M_q}{\ldots}\right) \leq 1.0$$

where F_q is the stiffener force = 256.2 kN, F_{qd} is the design resistance of an intermediate web stiffener corresponding to buckling about an axis parallel to web = 1112.01 kN, F_x is the external load or reaction at the stiffener = 225 kN, F_{xd} is the design resistance of a load carrying stiffener corresponding to buckling about axis parallel to web at D = 1112.01 kN, and M_q is the moment on the stiffener due to eccentrically applied load = 0.

$$(F_q - F_x) = (256.2 - 225) = 31.2 \text{ kN}$$

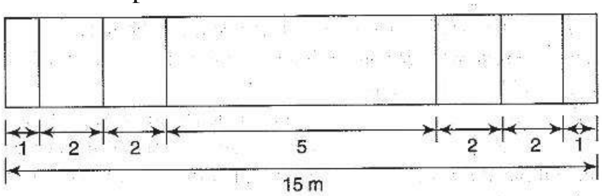
Thus, the above check reduces to

$$\frac{31.2}{1112.01} + \left(\frac{225}{1112.01}\right) = 0.23 < 1.0$$

Hence, the stiffener at D is safe.

Note Intermediate stiffener should extend to the compression flange, but not necessarily be connected to it. Stiffeners not subjected to external load or moment can be terminated at a distance of about $4t$ ($4 \times 10 = 40$ mm) from the tension flange. However in our case, since we have a moving load, we may connect the intermediate stiffener to the top (compression) flange and just extend it up to the bottom flange. The calculation for weld at web-flange junction and weld for end and intermediate stiffener are similar to those given in Example 11.5.

<u>Students must prepare answers for the following questions at the end of the lecture.</u>

Qn No	Question	Marks	CO	Bloom's Knowledge Level
1	A plate girderofgrade410 steel is composed of 10 x 2000mm web and 30 x 500mm flanges. The girder span is 15 m (see Fig. 11.45). Stiffeners are placed at 1 m, 3 m, and 5 m from both ends. Determine the shear strength of each of the panels.  Fig. 11.45	13	3	K3

Reference Book:

- N Subramaniam , Design of Steel Structures, Oxford University press, New Delhi, 2011

Unit	4. Industrial Structures	Lecture No	28
Topic	Introduction to Roof Trusses		
Learning Outcome (LO) At the end of this lecture, students will be able to		**Bloom's Knowledge Level**	
LO 1	Identify the basic concepts and applications of roof trusses in industrial buildings.	K1	
LO 2	Describe different types of roof trusses and their structural components.	K2	

Introduction to Roof Trusses

Roof trusses are critical components in many industrial buildings. They provide a framework that supports the roof, ensuring the structural integrity of the building while allowing for large, unobstructed interior spaces.

Basic Concepts of Roof Trusses

A roof truss is a structural framework composed of triangular units connected at joints called nodes. These trusses are designed to support the weight of the roof and to distribute this load to the building's walls or columns. The triangular arrangement is crucial because it provides stability and distributes forces evenly, preventing deformation under load.

Applications of Roof Trusses in Industrial Buildings

Roof trusses are widely used in various types of buildings, from residential houses to large industrial warehouses. In industrial buildings, they are particularly advantageous due to their ability to span large distances without intermediate supports, creating open spaces that can be used for manufacturing, storage, or other industrial activities.

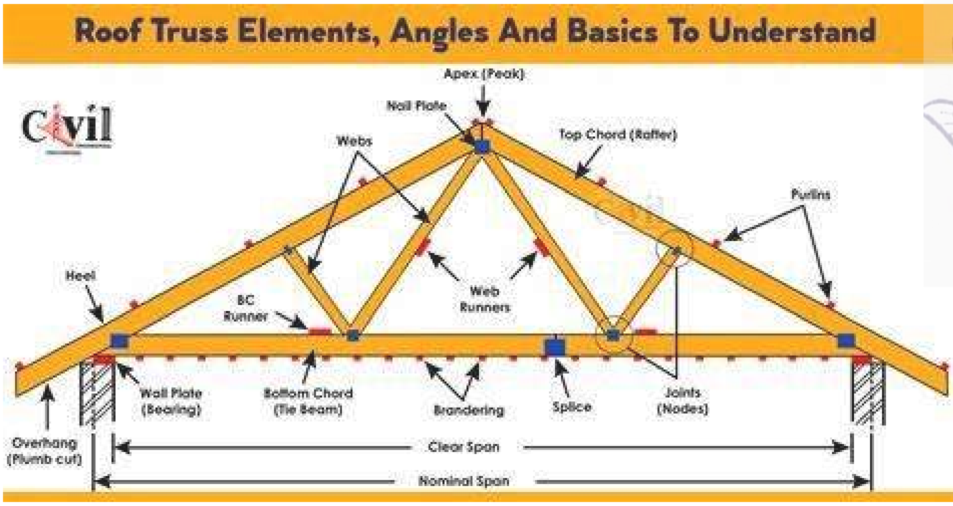
Advantages of Using Roof Trusses

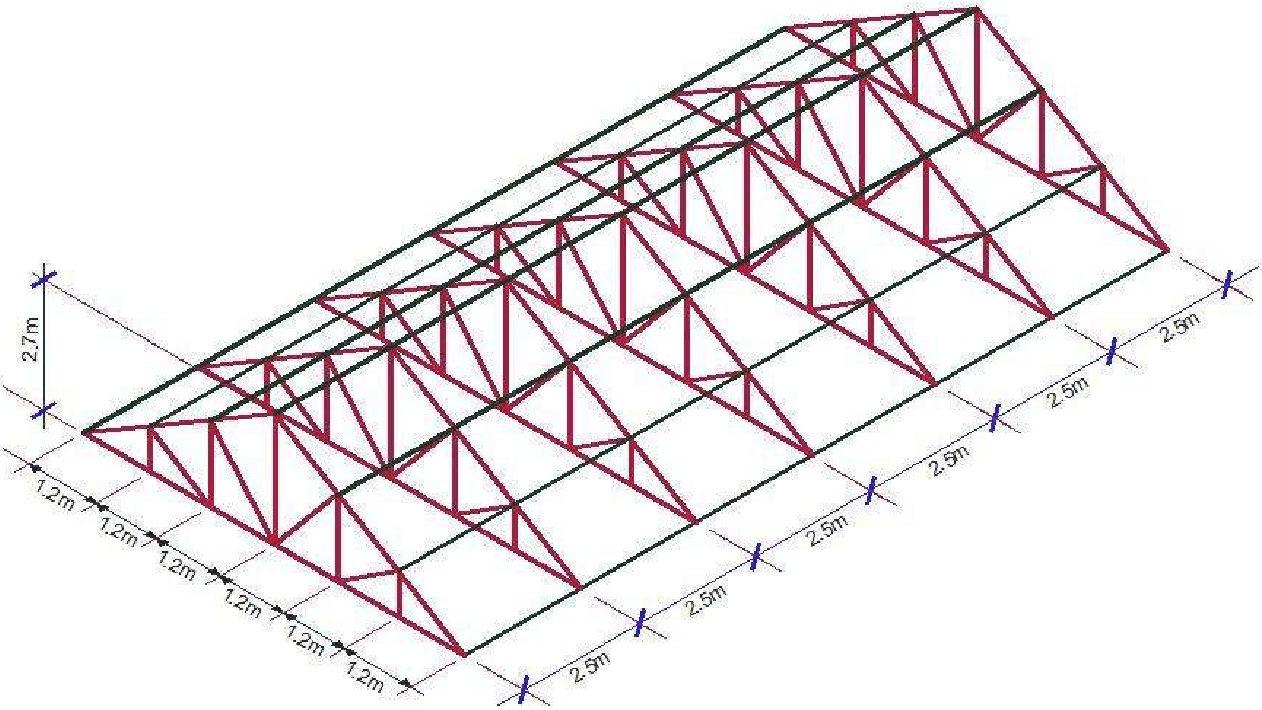
1. **Strength and Stability**: The triangular units in a truss distribute loads efficiently, providing high strength and stability.
2. **Material Efficiency**: Trusses require less material than traditional framing methods, making them cost-effective.
3. **Flexibility in Design**: Trusses can be designed in various shapes and sizes to fit different architectural requirements.
4. **Ease of Installation**: Prefabricated trusses can be manufactured off-site and quickly assembled on-site, reducing construction time.

Components of Roof Trusses

Understanding the components of a roof truss is essential for designing and analyzing its behavior under load. The main components include:

1. **Top Chord**: The uppermost member of the truss, which experiences compression forces. It forms the sloping or horizontal top edge of the truss.
2. **Bottom Chord**: The lower member, typically under tension, forming the base of the truss.





3. **Web Members**: The intermediate members that connect the top and bottom chords. These can be under either compression or tension, depending on their orientation and the applied loads.

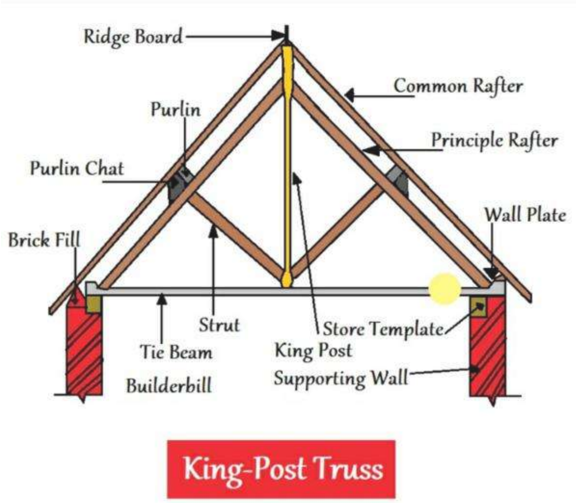
4. **Connections (Joints or Nodes)**: Points where the truss members meet. Connections can be bolted, welded, or nailed, depending on the materials and the design requirements.

Types of Roof Trusses

Different types of trusses are used in construction, each with unique characteristics and suitable applications:

1. **King Post Truss**

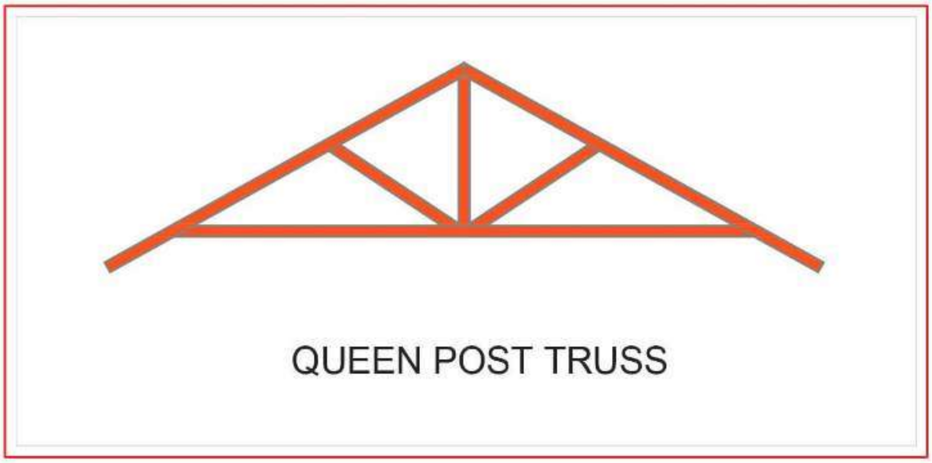
   - **Description**: The simplest type of truss, consisting of a central vertical post (king post), two top chords, and a bottom chord.



- **Application**: Ideal for short spans, typically used in residential buildings and small structures.

2. **Queen Post Truss**

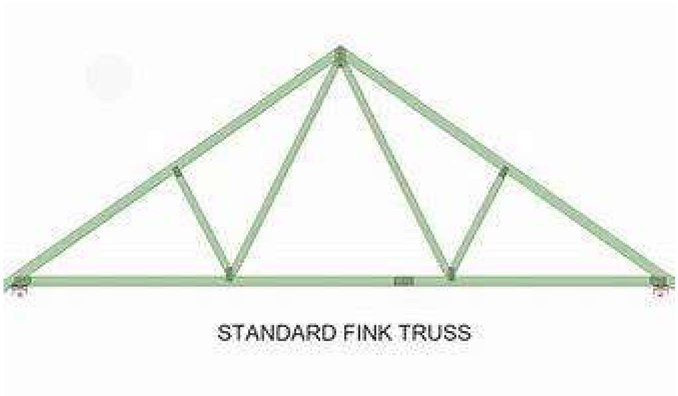
- **Description**: Similar to the King Post Truss but with two vertical posts (queen posts), providing additional support.



- **Application**: Suitable for medium spans, offering more stability and strength than the King Post Truss.

3. **Fink Truss**

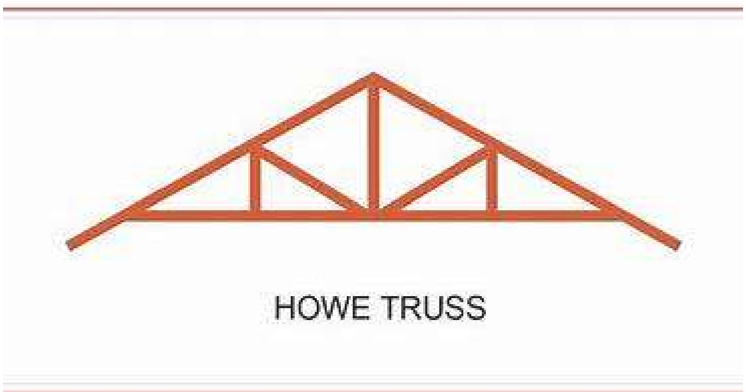
- **Description**: Characterized by a W-shaped web configuration, providing high strength and rigidity.



- **Application**: Commonly used in residential construction for roof framing.

4. **Howe Truss**

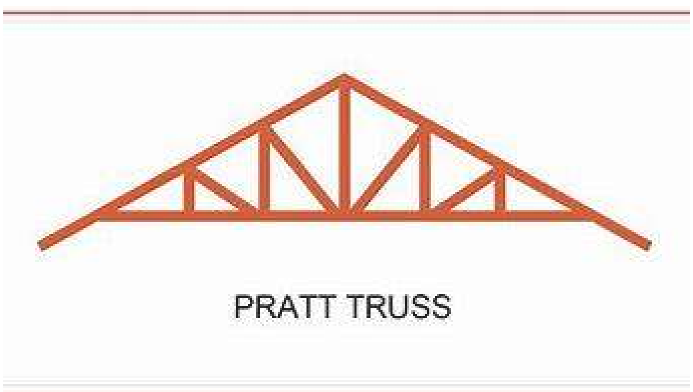
- **Description**: Features a combination of diagonal and vertical web members, with the diagonals typically under tension and the verticals under compression.



- **Application**: Suitable for large spans, often used in bridges and industrial buildings.

5. **Pratt Truss**

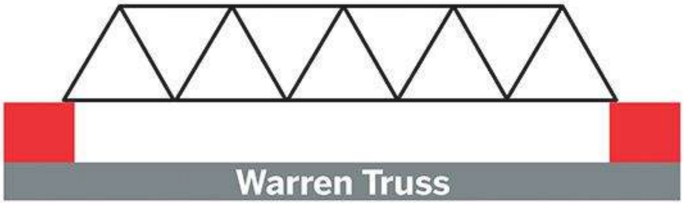
- **Description**: Has diagonal members sloping towards the center, with the vertical members under tension and the diagonals under compression.



- **Application**: Used for both short and long spans, common in bridge construction.

6. **Warren Truss**

- **Description**: Consists of equilateral triangles, with no vertical members, providing uniform load distribution.



- **Application**: Versatile and used in various applications, including bridges and buildings.

Design Considerations for Roof Trusses

When designing roof trusses, several factors need to be considered to ensure safety, functionality, and cost-effectiveness:

1. **Load Considerations**: Trusses must be designed to support various loads, including dead loads (weight of the truss and roofing materials), live loads (snow, wind, and maintenance workers), and environmental loads (wind, seismic activity).
2. **Material Selection**: The choice of materials (typically wood or steel) affects the strength, weight, and cost of the truss. Steel trusses are preferred for industrial buildings due to their higher strength and durability.
3. **Span and Spacing**: The span (distance between supports) and spacing (distance between adjacent trusses) must be optimized to balance structural integrity and material efficiency.
4. **Deflection Limits**: Trusses should be designed to limit deflection under load to prevent damage to roofing materials and ensure structural stability.
5. **Connections**: The design of connections is critical as they transfer loads between members. Proper detailing and selection of connection methods (bolting, welding) are essential for the truss's performance.

Conclusion

Roof trusses are indispensable components in modern construction, particularly for industrial buildings. Their ability to span large distances without intermediate supports, coupled with their strength and material efficiency, makes them ideal for various applications. Understanding the basic concepts, types, and components of roof trusses is crucial for civil engineering students to design and analyze these structures effectively.

Students must prepare answers for the following questions at the end of the lecture.

Qn No	Question	Marks	CO	Bloom's Knowledge Level
1	Explain the different types of roof trusses and their applications in detail. (April/May 2019)	13	4	K2

2	Discuss the components of a roof truss and their roles in providing structural stability. (Nov/Dec 2020)	13	4	K2

Reference Book:

- N Subramaniam , Design of Steel Structures, Oxford University press, New Delhi, 2011
- IS 875 (Part 1 to 5): Code of Practice for Design Loads (Other than Earthquake) for Buildings and Structures. Bureau of Indian Standards.
- IS 800:2007: General Construction in Steel - Code of Practice. Bureau of Indian Standards.

Unit	4. Industrial Structures	Lecture No	29
Topic	Loads on Trusses		
Learning Outcome (LO) At the end of this lecture, students will be able to		**Bloom's Knowledge Level**	
LO 1	List different types of loads acting on roof trusses, such as dead, live, wind, and snow loads.	K1	
LO 2	Explain how load distribution on roof trusses is calculated using standard codes.	K2	

Introduction to Loads on Trusses

Understanding the different types of loads that act on trusses is fundamental in structural engineering. These loads must be accurately determined and properly distributed to ensure the safety and stability of the structure. In this session, we will discuss the various loads that affect roof trusses and the methods used to calculate and distribute these loads according to standard codes.

Types of Loads on Roof Trusses

1. **Dead Loads**

   - **Definition**: Dead loads are permanent static loads that are relatively constant over time. These include the weight of the structural elements themselves, as well as any permanent fixtures attached to the structure.

   - **Components**: Dead loads typically consist of the weight of the roofing materials, the trusses, purlins, insulation, and other permanent attachments.

2. **Live Loads**

   - **Definition**: Live loads are temporary or movable loads that can vary in magnitude and location. These loads are not constant and can change over time.

   - **Components**: Live loads include the weight of maintenance workers, movable equipment, and temporary storage. In some regions, live loads also account for people, furniture, and other movable objects in the structure.

3. **Wind Loads**

   - **Definition**: Wind loads are lateral forces exerted by wind pressure on the structure. These forces can cause significant stress on the trusses and must be carefully considered in the design.

   - **Components**: Wind loads depend on the building's height, geographical location, and exposure to wind. The load varies with the shape and orientation of the building.

4. **Snow Loads**

   - **Definition**: Snow loads are vertical loads due to the accumulation of snow on the roof. The amount of snow load depends on the geographic location and the design of the roof.

- **Components**: Factors affecting snow loads include the roof slope, thermal properties of the building, and the local climate.

5. **Seismic Loads**

   - **Definition**: Seismic loads are horizontal and vertical forces exerted on the structure due to ground motion during an earthquake. These loads are particularly important in earthquake-prone regions.
   - **Components**: The magnitude of seismic loads depends on the building's mass, stiffness, and the characteristics of the seismic activity in the region.

Calculation of Load Distribution

Accurate calculation and distribution of loads are critical to ensuring the structural integrity of trusses. The Indian Standards (IS) codes provide guidelines for determining these loads.

1. Dead Load Calculation

- **IS Code Reference**: IS 875 Part 1
- **Steps**:
  1. Identify all permanent elements of the roof structure.
  2. Determine the unit weight of each element.
  3. Calculate the total dead load by summing the weights of all elements.

Example Calculation:

- **Given**: Roofing material = 0.5 kN/m², Trusses = 0.2 kN/m², Insulation = 0.1 kN/m²
- **Total Dead Load** = 0.5 + 0.2 + 0.1 = 0.8 kN/m²

2. Live Load Calculation

- **IS Code Reference**: IS 875 Part 2
- **Steps**:
  1. Determine the intended use of the roof and the corresponding live load values from the code.
  2. Apply the live load uniformly over the roof area or as specified.

Example Calculation:

- **Given**: Live load = 1.5 kN/m²
- **Total Live Load** = 1.5 kN/m²

3. Wind Load Calculation

- **IS Code Reference**: IS 875 Part 3

- **Steps**:
    1. Determine the basic wind speed for the location.
    2. Calculate the design wind pressure using factors such as terrain, height, and structure shape.
    3. Apply the wind load to the structure.

Example Calculation:

- **Given**: Basic wind speed = 50 m/s, Height factor = 1.0, Terrain factor = 1.0, Shape factor = 0.8
- **Design Wind Pressure** = 0.6×Basic wind speed2×Factors0.6×Basic wind speed2×Factors = $0.6 \times 50^2 \times 1.0 \times 1.0 \times 0.8 = 1200$ N/m²

4. Snow Load Calculation

- **IS Code Reference**: IS 875 Part 4
- **Steps**:
    1. Determine the ground snow load for the location.
    2. Adjust the load based on the roof's slope and other factors.
    3. Apply the snow load to the roof.

Example Calculation:

- **Given**: Ground snow load = 2 kN/m², Roof slope factor = 0.9
- **Design Snow Load** = Ground snow load × Roof slope factor = $2 \times 0.9 = 1.8$ kN/m²

5. Seismic Load Calculation

- **IS Code Reference**: IS 1893
- **Steps**:
    1. Determine the seismic zone factor for the location.
    2. Calculate the building's natural period and damping.
    3. Apply the seismic load to the structure using the response spectrum method.

Example Calculation:

- **Given**: Seismic zone factor = 0.16, Building weight = 500 kN
- **Base Shear (V)** = Seismic zone factor × Building weight = $0.16 \times 500 = 80$ kN

Illustrations and Diagrams

To enhance understanding, include diagrams showing how different loads act on a truss. These should illustrate:

- The distribution of dead loads from roofing materials and structural components.
- Live loads applied uniformly or at specific points on the truss.
- Wind loads acting laterally and how they vary with height.
- Snow loads distributed based on roof slope and climate conditions.
- Seismic loads represented as lateral and vertical forces due to ground motion.

Problem Solving Example

Refer to Subramaniam's "Design of Steel Structures"

Problem: Calculate the load distribution on a truss with a span of 8m, including dead and live loads.

1. **Given**: Span = 8m, Dead Load = 3 kN/m, Live Load = 2 kN/m
2. **Solution**:
    - **Step 1**: Calculate the total load per meter length.
    - **Step 2**: Distribute the load to each truss member.
    - **Step 3**: Verify the load distribution using IS codes.

Detailed Step-by-Step Solution:

1. **Draw the Truss**:
    - Create a simple sketch of the truss indicating the span and the load distribution.
2. **Calculate the Total Load**:
    - **Total Load** = Dead Load + Live Load = 3 kN/m + 2 kN/m = 5 kN/m
3. **Distribute the Load to Members**:
    - If the truss has 4 equally spaced panels, each panel carries an equal portion of the total load.
    - **Load per Panel** = Total Load × Span / Number of Panels = 5 kN/m × 8m / 4 = 10 kN per panel.
4. **Verify Load Distribution**:
    - Use standard methods like influence lines or load tables for accurate distribution.
    - Check compliance with IS 875 and IS 800 codes.

Conclusion

Accurately calculating and distributing loads on trusses is crucial for ensuring the structural integrity of industrial buildings. Understanding the types of loads, such as dead, live, wind, snow, and seismic loads, and applying the appropriate IS code guidelines are essential skills for civil engineering students.

Students must prepare answers for the following questions at the end of the lecture.

Qn No	Question	Marks	CO	Bloom's Knowledge Level
1	Discuss the types of loads acting on roof trusses and the methods to calculate them. (April/May 2018)	13	4	K2
2	Elaborate the procedure of the load distribution on a given truss with specified dead and live loads.	13	4	K2

Reference Book:

- N Subramaniam , Design of Steel Structures, Oxford University press, New Delhi, 2011
- IS 875 (Part 1 to 5): Code of Practice for Design Loads (Other than Earthquake) for Buildings and Structures. Bureau of Indian Standards.
- IS 800:2007: General Construction in Steel - Code of Practice. Bureau of Indian Standards.

Unit	4. Industrial Structures	Lecture No	30
Topic	**Purlin Design Using Angle Sections**		
Learning Outcome (LO) At the end of this lecture, students will be able to		**Bloom's Knowledge Level**	
LO 1	Understand the design principles for purlins using angle sections.	K2	
LO 2	Apply design calculations for purlins using angle sections under various loads.	K3	

Introduction to Purlins

Purlins are horizontal structural members in a roof that support the roof deck or sheeting. They transfer loads from the roof to the primary structural elements such as beams and columns. Purlins are typically used in industrial buildings with steel frameworks due to their efficiency in spanning large distances and supporting significant loads.

Design Principles of Purlins

The design of purlins involves determining the size and type of purlin that can safely support the loads imposed on the roof structure. The main loads considered in purlin design include dead loads, live loads, wind loads, and snow loads.

Types of Purlins

1. **Z-Section Purlins**: Commonly used due to their ability to interlock and create continuous spans.
2. **C-Section Purlins**: Typically used in simple span conditions without continuous spans.
3. **Angle Section Purlins**: Used less frequently but suitable for certain applications.

Design Codes and References

The design of purlins should comply with the following Indian Standards:

- **IS 800:2007**: General Construction in Steel – Code of Practice
- **IS 875 (Part 1 to 5)**: Code of Practice for Design Loads (Other than Earthquake) for Buildings and Structures

Step-by-Step Design Process

1. **Determine Loads on Purlins**
2. **Select Purlin Type and Size**
3. **Calculate Bending Moment, Shear Force, and Deflection**
4. **Check Section Capacity and Serviceability Limits**
5. **Detail Connections and Supports**

Step-by-Step Problem Solving

Example Problem: Design of a Z-Section Purlin

Given Data:

- Span of Purlin: 6 meters
- Roof slope: 10 degrees
- Dead Load (including self-weight): 0.6 kN/m²
- Live Load: 0.75 kN/m²
- Wind Load: 0.9 kN/m²
- Purlin Spacing: 1.5 meters
- Material: Steel Grade Fe 410 (fy = 250 MPa)

Step 1: Determine Loads on Purlins

1. **Dead Load (DL)**

$$DL = 0.6\,\text{kN/m}^2 \times 1.5\,\text{m} = 0.9\,\text{kN/m}$$

2. **Live Load (LL)**

$$LL = 0.75\,\text{kN/m}^2 \times 1.5\,\text{m} = 1.125\,\text{kN/m}$$

3. **Wind Load (WL)**

$$WL = 0.9\,\text{kN/m}^2 \times 1.5\,\text{m} = 1.35\,\text{kN/m}$$

4. **Total Load (TL)**

$$TL = DL + LL + WL$$

$$TL = 0.9 + 1.125 + 1.35 = 3.375\,\text{kN/m}$$

Step 2: Select Purlin Type and Size

From IS 800:2007, we select a Z-section purlin. Assume initial dimensions and check against bending and deflection requirements.

Step 3: Calculate Bending Moment, Shear Force, and Deflection

1. **Bending Moment (M)**

$$M_{\max} = \frac{wL^2}{8}$$

$$M_{\max} = \frac{3.375 \times 6^2}{8} = 15.1875\,\text{kN.m}$$

2. **Shear Force (V)**

$$V_{\max} = \frac{wL}{2}$$

$$V_{\max} = \frac{3.375 \times 6}{2} = 10.125\,\text{kN}$$

3. **Deflection (δ)**

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

$$\delta_{\max} = \frac{5 \times 3.375 \times 6000^4}{384 \times 2.1 \times 10^5 \times 800 \times 10^4}$$

$$\delta_{\max} = 5.07\,\text{mm}$$

$$\delta_{\text{allowable}} = \frac{6000}{300} = 20\,\text{mm}$$

$$\delta_{\max} \leq \delta_{\text{allowable}}$$

Calculate II for selected section and use the above formula to ensure it is within allowable limits.

Step 4: Check Section Capacity and Serviceability Limits

1. **Bending Stress ($fbfb$)**

$$f_b = \frac{M}{Z}$$

Select appropriate section modulus ZZ from steel tables and verify $f_b \leq f_y$

2. **Shear Stress (fv)**

$$f_v = \frac{V}{A_w}$$

$$f_v \leq \frac{f_y}{\sqrt{3}}$$

3. **Deflection Check**

$$\delta_{\max} = \frac{5 \times 3.375 \times 6000^4}{384 \times 2.1 \times 10^5 \times 800 \times 10^4}$$

$$\delta_{\max} = 5.07\,\text{mm}$$

$$\delta_{\text{allowable}} = \frac{6000}{300} = 20\,\text{mm}$$

$\delta_{\max} \leq \delta_{\text{allowable}}$, section is adequate.

Step 5: Detail Connections and Supports

Detail the connections at the supports and joints. Ensure bolts or welds are designed to handle calculated shear forces.

Clause References from IS 800:2007

- **Bending Stress**: Clause 8.2.1
- **Shear Stress**: Clause 8.4.1
- **Deflection Limits**: Clause 8.5.1

Conclusion

The design of purlins involves understanding the loads they will carry and ensuring the selected section can safely handle these loads. Following the steps outlined and using standard codes ensures that the purlins will be both safe and efficient.

Students must prepare answers for the following questions at the end of the lecture.

Qn No	Question	Marks	CO	Bloom's Knowledge Level
1	Design a Z-section purlin for an industrial roof with a span of 8 meters, roof slope of 15 degrees, dead load of 0.7 kN/m², live load of 0.9 kN/m², wind load of 1.1 kN/m², and purlin spacing of 2 meters." (April/May 2019)	13	4	K3
2	Explain the procedure for designing purlins in steel structures and solve an example with the following data: span of 5 meters, roof slope of 12 degrees, dead load of 0.5 kN/m², live load of 0.8 kN/m², wind load of 1.0 kN/m², and purlin spacing of 1.2 meters." (Nov/Dec 2018)	13	4	K3

Reference Book:

- N Subramaniam , Design of Steel Structures, Oxford University press, New Delhi, 2011
- IS 875 (Part 1 to 5): Code of Practice for Design Loads (Other than Earthquake) for Buildings and Structures. Bureau of Indian Standards.
- IS 800:2007: General Construction in Steel - Code of Practice. Bureau of Indian Standards.

Unit	4. Industrial Structures	Lecture No	31
Topic	**Design of Purlins using Channel Sections**		
Learning Outcome (LO) At the end of this lecture, students will be able to		**Bloom's Knowledge Level**	
LO 1	Understand the design principles for purlins using angle sections.	K2	
LO 2	Apply design calculations for purlins using angle sections under various loads.	K3	

Design Principles of Purlins Using Channel Sections

The design of purlins involves ensuring that the selected section can safely support the imposed loads without excessive deflection or failure. The main loads considered in purlin design include dead loads, live loads, wind loads, and snow loads.

Step-by-Step Problem Solving

Example Problem: Design of a C-Section Purlin

Given Data:

- Span of Purlin: 5 meters
- Dead Load (including self-weight): 0.5 kN/m²
- Live Load: 0.6 kN/m²
- Wind Load: 0.8 kN/m²
- Purlin Spacing: 1.2 meters
- Material: Steel Grade Fe 410 (fy = 250 MPa)

Step 1: Determine Loads on Purlins

1. **Dead Load (DL)**

$$DL = 0.5\,\mathrm{kN/m^2} \times 1.2\,\mathrm{m} = 0.6\,\mathrm{kN/m}$$

2. **Live Load (LL)**

$$LL = 0.6\,\mathrm{kN/m^2} \times 1.2\,\mathrm{m} = 0.72\,\mathrm{kN/m}$$

3. **Wind Load (WL)**

$$WL = 0.8\,\mathrm{kN/m^2} \times 1.2\,\mathrm{m} = 0.96\,\mathrm{kN/m}$$

4. **Total Load (TL)**

$$TL = DL + LL + WL$$

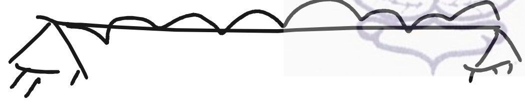
$$TL = 0.6 + 0.72 + 0.96 = 2.28\,\mathrm{kN/m}$$

Step 2: Select Purlin Type and Size

From IS 800:2007, we select a C-section purlin. Assume initial dimensions and check against bending and deflection requirements.

Step 3: Calculate Bending Moment, Shear Force, and Deflection

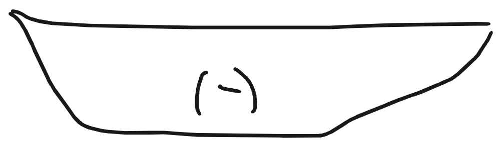
1. **Bending Moment (M)**



$$M_{\text{max}} = \frac{wL^2}{8}$$

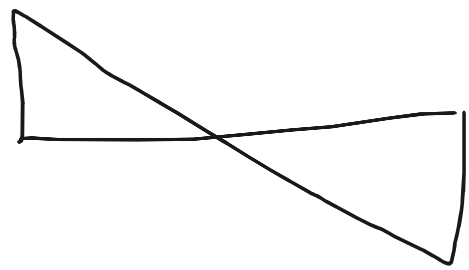
$$M_{\text{max}} = \frac{2.28 \times 5^2}{8} = 7.125 \text{ kN.m}$$

2. **Shear Force (V)**



$$V_{\text{max}} = \frac{wL}{2}$$

$$V_{\text{max}} = \frac{2.28 \times 5}{2} = 5.7 \text{ kN}$$



3. **Deflection (δ)**

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

Calculate I for selected section and use the above formula to ensure it is within allowable limits.

Step 4: Check Section Capacity and Serviceability Limits

1. **Bending Stress (fb)**

$$f_b = \frac{M}{Z}$$

Select appropriate section modulus Z from steel tables and verify $fb \leq fy$

2. **Shear Stress (fv)**

$$f_v = \frac{V}{A_v}$$

Ensure $fv \leq fy/3$

3. **Deflection Check**

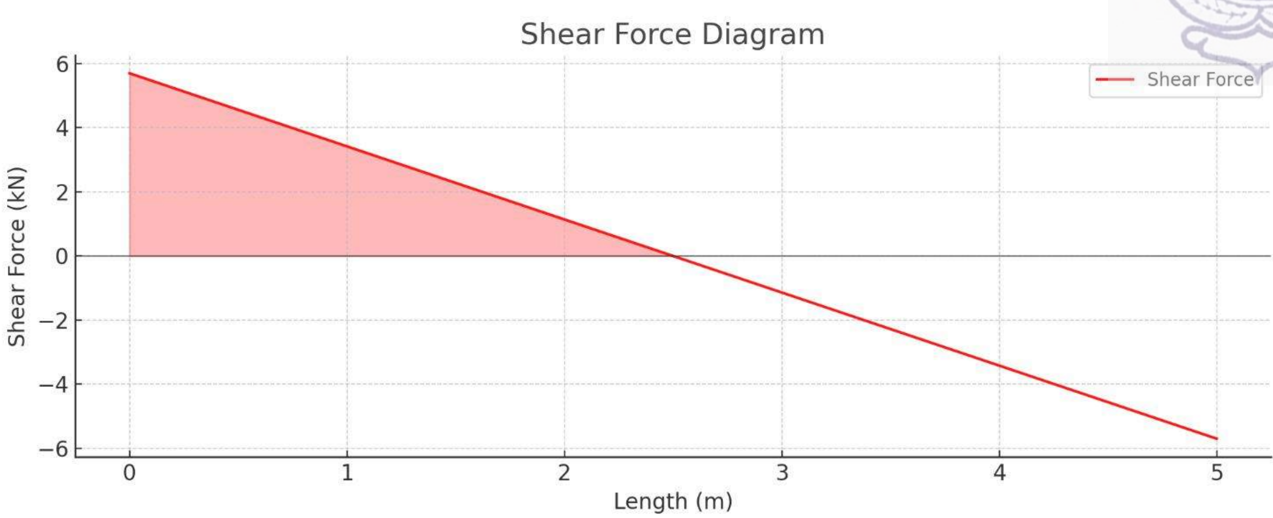
$$\delta_{\text{max}} = \frac{5 \times 2.28 \times 5000^4}{384 \times 2.1 \times 10^5 \times I}$$

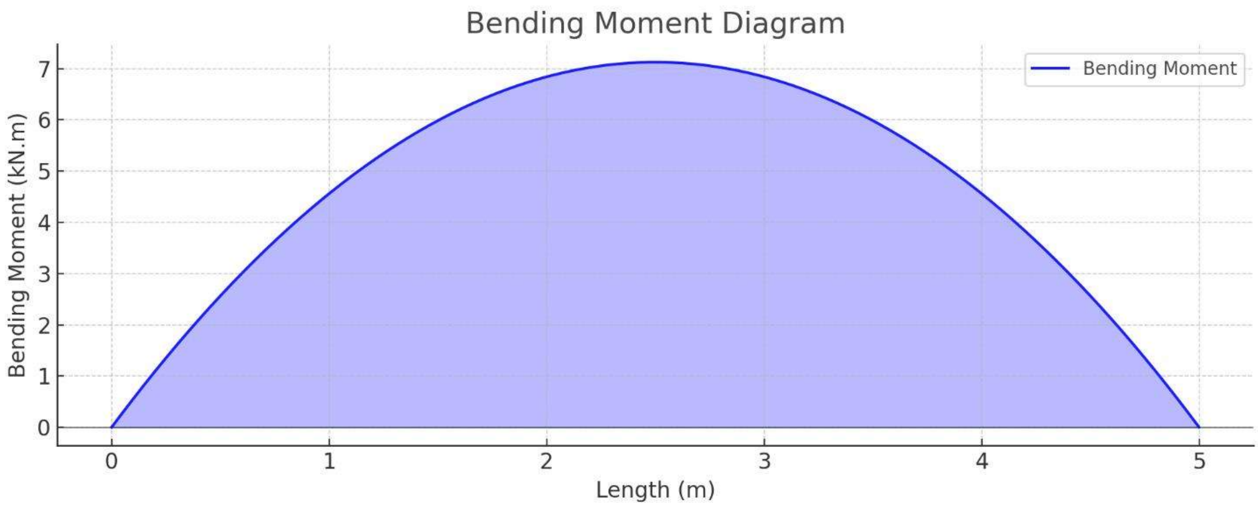
Step 5: Detail Connections and Supports

Detail the connections at the supports and joints. Ensure bolts or welds are designed to handle calculated shear forces.

Clause References from IS 800:2007

- **Bending Stress**: Clause 8.2.1
- **Shear Stress**: Clause 8.4.1
- **Deflection Limits**: Clause 8.5.1





Conclusion

The design of purlins involves understanding the loads they will carry and ensuring the selected section can safely handle these loads. Following the steps outlined and using standard codes ensures that the purlins will be both safe and efficient.

<u>Students must prepare answers for the following questions at the end of the lecture.</u>

Qn No	Question	Marks	CO	Bloom's Knowledge Level
1	Design a C-section purlin for an industrial roof with a span of 6 meters, roof slope of 10 degrees, dead load of 0.7 kN/m², live load of 0.8 kN/m², wind load of 1.0 kN/m², and purlin spacing of 1.5 meters.	13	4	K3
2	Explain the procedure for designing purlins using channel sections and solve an example with the following data:	13	4	K3

	span of 4 meters, roof slope of 12 degrees, dead load of 0.5 kN/m², live load of 0.6 kN/m², wind load of 0.9 kN/m², and purlin spacing of 1.2 meters.			

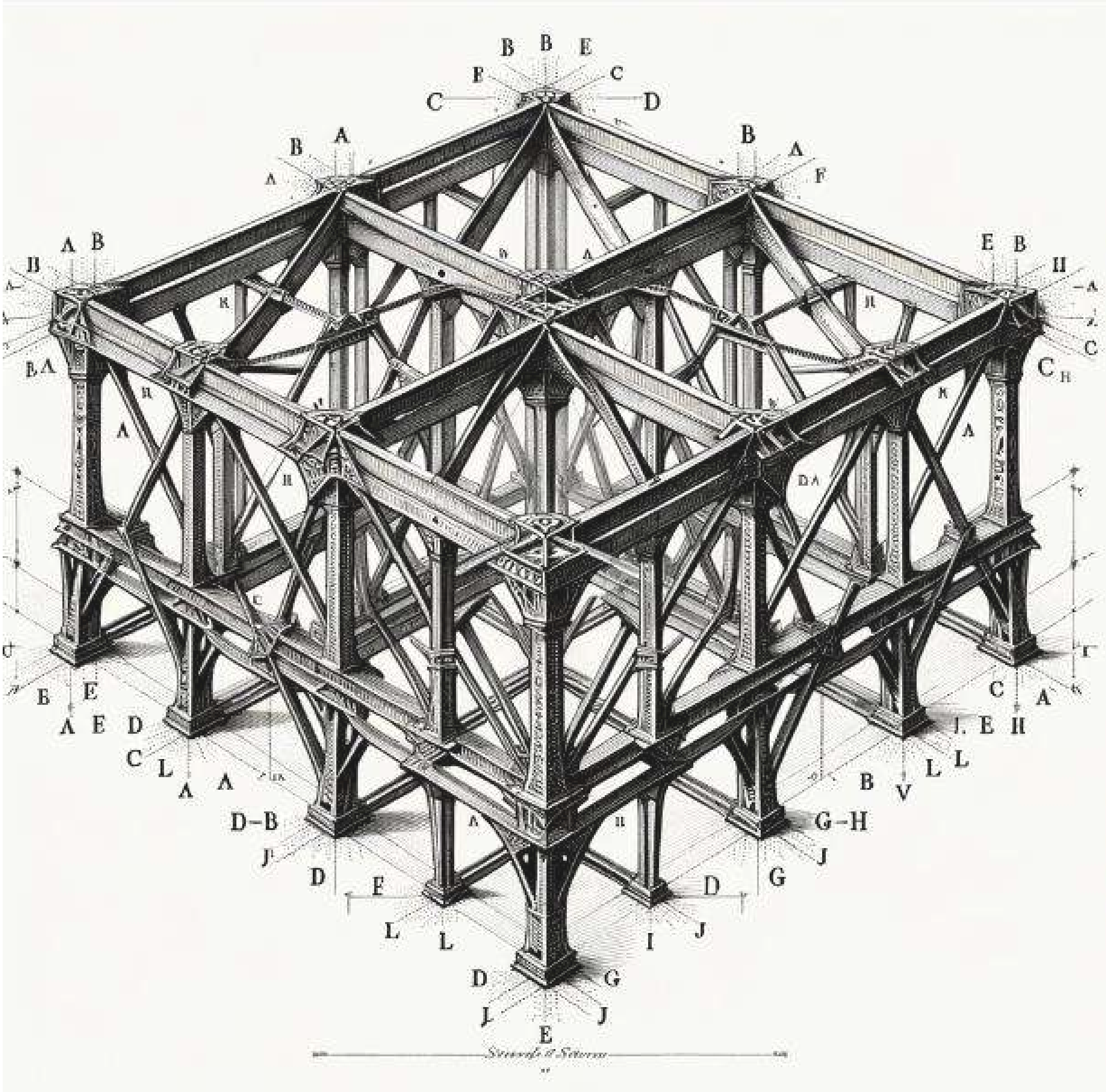
Reference Book:

- N Subramaniam , Design of Steel Structures, Oxford University press, New Delhi, 2011
- IS 875 (Part 1 to 5): Code of Practice for Design Loads (Other than Earthquake) for Buildings and Structures. Bureau of Indian Standards.
- IS 800:2007: General Construction in Steel - Code of Practice. Bureau of Indian Standards.

Unit	4. Industrial Structures	Lecture No	32
Topic	**Truss Design Principles Truss Design Principles**		
Learning Outcome (LO) At the end of this lecture, students will be able to		**Bloom's Knowledge Level**	
LO 1	Explain the design process for industrial roof trusses considering various loading scenarios.	K2	
LO 2	Apply the strength and deflection criteria in the design of trusses.	K3	

Introduction to Truss Design

Trusses are structural frameworks composed of triangular units. They are widely used in industrial buildings due to their ability to span large distances and efficiently distribute loads. Trusses are made of individual members connected at joints or nodes, forming a rigid structure capable of carrying heavy loads over large spans without excessive deflection.



Design Process for Industrial Roof Trusses

The design of trusses involves several key steps:

1. **Load Assessment**: Determining the various loads that the truss will be subjected to, including dead loads, live loads, wind loads, and snow loads.
2. **Selection of Truss Type**: Based on span, load, and architectural requirements, choosing the appropriate type of truss.
3. **Member Design**: Calculating the forces in each truss member and selecting appropriate sections to carry these forces.
4. **Connection Design**: Ensuring that connections between members can transfer the required forces.

Types of Trusses

1. **Pratt Truss**
2. **Warren Truss**
3. **Howe Truss**
4. **Fink Truss**

Load Assessment

1. Dead Load (DL)

- Permanent loads including the weight of the truss itself, roofing material, and any permanent fixtures.
- Dead Load Calculation (IS 875 Part 1): $DL=\text{Unit weight of material}\times\text{Area of application}$$DL=\text{Unit weight of material}\times\text{Area of application}$

2. Live Load (LL)

- Temporary or movable loads including occupancy loads, maintenance loads, and equipment loads.
- Live Load Calculation (IS 875 Part 2): $LL=\text{Load per unit area}\times\text{Area of application}$$LL=\text{Load per unit area}\times\text{Area of application}$

3. Wind Load (WL)

- Lateral loads due to wind pressure.
- Wind Load Calculation (IS 875 Part 3): $WL=Cp\cdot Cf\cdot A\cdot P$$WL=Cp\cdot Cf\cdot A\cdot P$ Where $Cp$$Cp$ is the pressure coefficient, $Cf$$Cf$ is the force coefficient, $A$$A$ is the area, and $P$$P$ is the wind pressure.

4. Snow Load (SL)

- Loads due to accumulated snow on the roof.
- Snow Load Calculation (IS 875 Part 4):
  SL=Load per unit area×Area of applicationSL=Load per unit area×Area of application

Selection of Truss Type

1. Pratt Truss

- Diagonals slope towards the center, ideal for longer spans.

2. Warren Truss

- Equilateral triangles, providing uniform load distribution.

3. Howe Truss

- Diagonals slope away from the center, suitable for heavy loads.

4. Fink Truss

- Complex triangular shapes, used for roof structures.

Member Design

1. Calculate Axial Forces in Truss Members

Using the method of joints, calculate the forces in each member of the truss. Assume the truss is simply supported and symmetrical.

Example Problem: Design of a Pratt Truss

Given Data:

- Span of Truss: 15 meters
- Height of Truss: 3 meters
- Dead Load: 1.2 kN/m²
- Live Load: 1.5 kN/m²
- Wind Load: 0.8 kN/m²
- Material: Steel Grade Fe 410 (fy = 250 MPa)

Step 1: Load Assessment

1. **Dead Load (DL)**

$$DL = \text{Unit weight of material} \times \text{Area of application}$$

2. **Live Load (LL)**

$$LL = \text{Load per unit area} \times \text{Area of application}$$

3. **Wind Load (WL)**

$$WL = C_p \cdot C_f \cdot A \cdot P$$

4. **Total Load (TL)**

$$TL = DL + LL + \text{WL}$$

$$TL = 18 + 22.5 + 12 = 52.5 \text{ kN/m}$$

Step 2: Selection of Truss Type

For this example, we will design a Pratt truss.

Step 3: Member Design

1. Calculate Axial Forces in Truss Members

For simplicity, let's assume the truss has 5 panels. Using the method of joints or method of sections, the axial forces in the members can be calculated. Here's a simplified approach using the method of joints:

- Calculate reactions at supports:

$$R_A = R_B = \frac{TL \times \text{Span}}{2}$$

$$R_A = R_B = \frac{52.5 \times 15}{2} = 393.75 \text{ kN}$$

- At joint A:

$$\sum F_y = 0 \backslash implies V_{A1} = R_A = 393.75 \text{ kN}$$

$$\sum F_x = 0 \backslash implies H_{A1} = 0 \text{ kN}$$

- At joint B (considering symmetry):

$$V_{B1} = 393.75 \text{ kN}$$

$$H_{B1} = 0 \text{ kN}$$

Assume suitable members from SP6

For the given forces, select appropriate sections for the truss members from SP6.

Step 4: Connection Design

1. Bolted Connections

Design the bolted connections to ensure they can transfer the calculated forces. Refer to IS 800:2007 for bolt strength and spacing requirements.

$$F_{\text{bolt}} = \frac{F_{\text{member}}}{n_{\text{bolts}}}$$

where n_{bolts} is the number of bolts in the connection.

2. Welded Connections

If using welded connections, design the welds to safely transfer the forces. Refer to IS 800:2007 for weld strength and size requirements.

$$F_{\text{weld}} = \frac{F_{\text{member}}}{L_{\text{weld}}}$$

where L_{weld} is the length of the weld.

Step 5: Check Deflection

Ensure that the deflection of the truss is within permissible limits.

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

where:

- δ_{max} = maximum deflection
- w = uniformly distributed load
- L = span of the truss
- E = modulus of elasticity
- I = moment of inertia

Clause References from IS 800:2007

- **Bending Stress**: Clause 8.2.1
- **Shear Stress**: Clause 8.4.1
- **Deflection Limits**: Clause 8.5.1

Conclusion

The design of trusses involves understanding the loads they will carry and ensuring the selected sections can safely handle these loads. Following the steps outlined and using standard codes ensures that the trusses will be both safe and efficient.

Students must prepare answers for the following questions at the end of the lecture.

Qn No	Question	Marks	CO	Bloom's Knowledge Level
1	Design a Pratt truss for an industrial building with a span of 18 meters, roof slope of 10 degrees, dead load of 1.0 kN/m^2, live load of 1.2 kN/m^2, wind load of 0.7 kN/m^2, and truss height of 4 meters.	13	4	K3
2	Explain the procedure for designing a Warren truss and solve an example with the following data: span of 20 meters, dead load of 1.3 kN/m^2, live load of 1.5 kN/m^2, wind load of 0.9 kN/m^2, and truss height of 3.5 meters	13	4	K3

Reference Book:

- N Subramaniam , Design of Steel Structures, Oxford University press, New Delhi, 2011
- IS 875 (Part 1 to 5): Code of Practice for Design Loads (Other than Earthquake) for Buildings and Structures. Bureau of Indian Standards.
- IS 800:2007: General Construction in Steel - Code of Practice. Bureau of Indian Standards.

X``Unit	4. Industrial Structures	Lecture No	33
Topic	Design of Joints in Trusses		
Learning Outcome (LO) At the end of this lecture, students will be able to		Bloom's Knowledge Level	
LO 1	Identify the types of joints used in truss structures, including bolted and welded joints	K1	
LO 2	Design efficient and safe connections for truss members	K3	

Introduction to Truss Joints

Joints in trusses are critical as they transfer loads between members and ensure the stability of the structure. Joints can be either bolted or welded, depending on the requirements and the materials used.

Types of Joints

1. **Bolted Joints**: Commonly used in steel structures for ease of assembly and disassembly. Bolts must be designed to withstand shear and tensile forces.
2. **Welded Joints**: Provide a rigid connection between members. Welds must be designed to handle the required loads and prevent failure.

Design Principles for Truss Joints

Bolted Joints

Bolted joints must be designed to handle shear, tension, and combined stresses. The strength of a bolted joint is determined by the bolt material, diameter, and the number of bolts.

Design Steps for Bolted Joints

1. **Determine the type and grade of bolt** (e.g., Grade 4.6, Grade 8.8).
2. **Calculate the number of bolts required** using the design forces.
3. **Check the bolt shear capacity** using the formula:

$$V_b = \frac{\pi d^2}{4} \cdot \tau_{\text{allow}}$$

Where τ_{allow} is the allowable shear stress.

4. **Check the bearing capacity of the bolt** on the connected plates using:

$$P_b = d \times t \times f_u$$

Where tt is the thickness of the plate and $fufu$ is the ultimate tensile strength.

5. **Check for bolt tension capacity** if the joint is subjected to tensile forces.

Example Problem: Design of a Bolted Joint in a Truss

Given Data:

- Axial force in member: 150 kN
- Bolt diameter: 20 mm

- Bolt material: Grade 4.6 (fy = 240 MPa)
- Number of bolts: 4

Step-by-Step Solution:

1. **Shear Strength of Bolt**

$$V_b = \frac{\pi d^2}{4} \cdot \tau_{\text{allow}}$$

$$Where\ d = 20\,mm\ and\ \tau_{\text{allow}} = \frac{240}{\sqrt{3}} = 138.56\,\text{MPa}$$

$V_b = \frac{\pi \times (20)^2}{4} \times 138.56 = 43.63\,\text{kN}$

2. **Bearing Strength of Bolt**

$$P_b = d \times t \times f_u$$

Assuming plate thickness t=10 mm and fu=410 MPa

$$P_b = 20 \times 10 \times 410 = 82\,\text{kN}$$

3. **Total Number of Bolts Required**

$$n_{\text{bolts}} = \frac{F_{\text{member}}}{\min(V_b, P_b)}$$

$$n_{\text{bolts}} = \frac{150}{\min(43.63{,}82)} = \frac{150}{43.63} \approx 3.44$$

Hence, use 4 bolts.

Welded Joints

Welded joints must be designed to handle shear and tensile forces. The strength of a welded joint is determined by the weld material, size, and length.

Design Steps for Welded Joints

1. **Determine the type and size of weld** (e.g., fillet weld, butt weld).
2. **Calculate the weld length required** using the design forces.
3. **Check the weld shear capacity** using the formula:

$$V_w = l \times \frac{t_w}{\sqrt{3}} \times \tau_{\text{allow}}$$

Where l is the length of the weld and tw is the throat thickness.

4. **Check the tensile capacity of the weld** if the joint is subjected to tensile forces.

Example Problem: Design of a Welded Joint in a Truss

Given Data:

- Axial force in member: 200 kN
- Weld type: Fillet weld
- Weld size: 6 mm
- Weld material: E60 (fu = 410 MPa)

Step-by-Step Solution:

1. **Shear Strength of Weld**

$$V_w = l \times \frac{t_w}{\sqrt{3}} \times \tau_{\text{allow}}$$

Where $t_w = 0.7 \times \text{weld size}$

$$t_w = 0.7 \times 6 = 4.2\ \text{mm}$$

$$\tau_{\text{allow}} = \frac{410}{\sqrt{3}} = 236.6\ \text{MPa}$$

Assuming the weld length l

$$V_w = l \times \frac{4.2}{\sqrt{3}} \times 236.6$$

2. **Calculate the Required Weld Length**

$$l = \frac{F_{\text{member}}}{V_w}$$

$$l = \frac{200 \times 10^3}{4.2 \times 236.6} \approx 201.8\ \text{mm}$$

Hence, use a weld length of approximately 202 mm.

Clause References from IS 800:2007

- **Bolted Joints**: Clause 10.3
- **Welded Joints**: Clause 10.5

Conclusion

Designing efficient and safe joints in trusses involves understanding the loads they will carry and ensuring the selected joints can safely handle these loads. Following the steps outlined and using standard codes ensures that the joints will be both safe and efficient.

<u>Students must prepare answers for the following questions at the end of the lecture.</u>

Qn No	Question	Marks	CO	Bloom's Knowledge Level
1	Design a bolted joint for a truss member with an axial force of 180 kN. Assume Grade 8.8 bolts with a diameter of 22 mm and a plate thickness of 12 mm.	13	4	K3

2	Design a welded joint for a truss member with an axial force of 250 kN. Assume a fillet weld with a size of 8 mm and weld material E70	13	4	K3

Reference Book:

- N Subramaniam , Design of Steel Structures, Oxford University press, New Delhi, 2011
- IS 875 (Part 1 to 5): Code of Practice for Design Loads (Other than Earthquake) for Buildings and Structures. Bureau of Indian Standards.
- IS 800:2007: General Construction in Steel - Code of Practice. Bureau of Indian Standards.

Unit	4. Industrial Structures	Lecture No	34
Topic	**Design of End Bearings**		
Learning Outcome (LO) At the end of this lecture, students will be able to		**Bloom's Knowledge Level**	
LO 1	Describe the purpose and types of end bearings in truss structures.	K2	
LO 2	Apply design principles to end bearing components to support trusses.	K3	

Introduction

End bearings are critical components in truss structures as they facilitate the transfer of loads from the truss to the support structures. Proper design and installation of end bearings ensure the stability and longevity of trusses, preventing structural failures.

Purpose of End Bearings

The primary purpose of end bearings is to:

- Support the truss and distribute loads to the foundation or supporting structure.
- Allow for slight movements due to thermal expansion, contraction, or other factors.
- Reduce stress concentrations at the support points, minimizing potential damage.

Types of End Bearings

End bearings can be classified into several types based on their design and functionality:

1. **Fixed Bearings:** These bearings restrict all translational and rotational movements. They are used where no relative movement between the truss and support is allowed.
2. **Roller Bearings:** These bearings allow translational movement in one direction, accommodating thermal expansion and contraction.
3. **Pinned Bearings:** These bearings allow rotational movement but restrict translational movement, providing a degree of flexibility while maintaining structural integrity.

Design Principles for End Bearings

The design of end bearings involves several key steps and considerations to ensure they can adequately support the truss.

Step-by-Step Design Process

1. **Determine Load Requirements:**
    - Identify the total load on the truss, including dead loads, live loads, and any additional forces (wind, seismic, etc.).
    - Calculate the reactions at the supports using static equilibrium equations.

Example: For a simple span truss with a uniform load w (kN/m) over a span LL (m):

$$R_A = R_B = \frac{wL}{2}$$

Where R_A and R_B are the reactions at supports A and B respectively.

2. **Select Bearing Type:**
   - Choose the appropriate type of bearing based on the truss configuration and expected movements.
3. **Design Bearing Components:**
   - Calculate the bearing area required to support the load without exceeding the allowable stress of the bearing material

$$A_{bearing} = \frac{P}{f_{allow}}$$

Where P is the reaction force and f_{allow} is the allowable bearing stress as per IS 800:2007, Clause 10.3.3.

   - Check for bearing pressure:

$$p_{bearing} = \frac{P}{A_{bearing}}$$

Ensure $p_{bearing} \leq f_{allow}$.

4. **Detailing and Connections:**
   - Design the connection between the truss and the bearing, considering bolts, welds, or other connection methods as per IS 800:2007, Clause 10.3.4.
   - Ensure that the connections can transfer the forces without slipping or failing.

Example Problem 1:

Problem: Design a bearing plate for a truss with a fixed end bearing on a concrete support. The vertical load on the bearing is 150 kN, and the horizontal load is 50 kN. The concrete support has a bearing strength of 10 MPa.

Solution:

1. **Determine the bearing area:**

$$\text{Bearing Area} = \frac{\text{Vertical Load}}{\text{Bearing Strength of Concrete}}$$

$$\text{Bearing Area} = \frac{150 \text{ kN}}{10 \text{ MPa}} = \frac{150{,}000 \text{ N}}{10 \times 10^6 \text{ N/m}^2} = 0.015 \text{ m}^2 = 150 \text{ cm}^2$$

2. **Assume a square bearing plate:**

$$\text{Side length} = \sqrt{\text{Bearing Area}} = \sqrt{150 \text{ cm}^2} = 12.25 \text{ cm}$$

Choose a plate size of 150 mm x 150 mm to simplify the design.

3. **Calculate the thickness of the plate:**

- Using the maximum bending stress for steel (σ_b) of 250 MPa.
- Use the formula for the thickness of the plate under uniformly distributed load:

$$t = \sqrt{\frac{6M}{b \times \sigma_b}}$$

where M is the moment, b is the width of the plate, and σ_b is the permissible bending stress.

The moment M for a simply supported plate with uniformly distributed load can be approximated as:

$$M = \frac{w \times L^2}{8}$$

where w is the load per unit area and L is the side length of the plate.

Load per unit area, w:

$$w = \frac{150 \text{ kN}}{150 \text{ cm}^2} = \frac{150{,}000 \text{ N}}{0.015 \text{ m}^2} = 10{,}000 \text{ N/m}^2$$

Moment MM:

$$M = \frac{10{,}000 \text{ N/m}^2 \times (0.15 \text{ m})^2}{8} = \frac{10{,}000 \times 0.0225}{8} = 28.125 \text{ Nm}$$

Now, calculate the thickness tt:

$$t = \sqrt{\frac{6 \times 28.125 \text{ Nm}}{0.15 \text{ m} \times 250 \times 10^6 \text{ N/m}^2}}$$

$$t = \sqrt{\frac{168.75}{37.5 \times 10^6}} = \sqrt{4.5 \times 10^{-6}} \approx 2.12 \text{ mm}$$

Considering practical limitations, choose a plate thickness of 10 mm for adequate safety and structural integrity.

4. **Check for bearing capacity of the concrete support:**
   - Ensure the concrete support can withstand the load using the IS 800:2007 code for reference.
   - Verify that the stress on the concrete support does not exceed its permissible bearing stress.

Example Problem 2:

Problem: Design a roller bearing for a truss that carries a vertical load of 100 kN. The bearing is supported on a steel column. The steel column has a permissible bearing stress of 150 MPa.

Solution:

1. **Determine the bearing area:**

$$\text{Bearing Area} = \frac{100\ \text{kN}}{150\ \text{MPa}} = \frac{100{,}000\ \text{N}}{150 \times 10^6\ \text{N/m}^2} = 0.00067\ \text{m}^2 = 6.67\ \text{cm}^2$$

2. **Assume a rectangular bearing plate:** Choose a practical size for the plate. Let's assume dimensions of 100 mm x 70 mm:

$$\text{Bearing Area} = 100\ \text{mm} \times 70\ \text{mm} = 7000\ \text{mm}^2 = 70\ \text{cm}^2$$

3. **Calculate the thickness of the plate:**
   - Using the maximum bending stress for steel (σ_b) of 250 MPa.
   - Use the formula for the thickness of the plate under uniformly distributed load:

$$t = \sqrt{\frac{6M}{b \times \sigma_b}}$$

The moment MM for a rectangular plate under uniform load is:

$$M = \frac{w \times L^2}{8}$$

where w is the load per unit area and L is the shorter side length of the plate.

Load per unit area w:

$$w = \frac{100\ \text{kN}}{70\ \text{cm}^2} = \frac{100{,}000\ \text{N}}{0.007\ \text{m}^2} = 14{,}285.71\ \text{N/m}^2$$

Moment M:

$$M = \frac{14{,}285.71\ \text{N/m}^2 \times (0.1\ \text{m})^2}{8} = \frac{14{,}285.71 \times 0.01}{8} = 17.86\ \text{Nm}$$

Now, calculate the thickness tt:

$$t = \sqrt{\frac{6 \times 17.86\ \text{Nm}}{0.07\ \text{m} \times 250 \times 10^6\ \text{N/m}^2}}$$

$$t = \sqrt{\frac{107.14}{17.5 \times 10^6}} = \sqrt{6.12 \times 10^{-6}} \approx 2.47\ \text{mm}$$

Considering practical limitations, choose a plate thickness of 10 mm for adequate safety and structural integrity.

4. **Verify the design:**
   - Ensure the steel column can support the load without local failure.
   - Check the stress on the steel column using IS 800:2007 code to ensure it is within permissible limits.

Students must prepare answers for the following questions at the end of the lecture.

Qn No	Question	Marks	CO	Bloom's Knowledge Level
1	Describe the different types of end bearings used in truss structures and explain their specific applications. Provide a detailed design example for a fixed bearing on a concrete support.	13	4	K2
2	Explain the design principles for end bearings in truss structures. Write the Design procedure of a roller bearing for a truss with a given load and support characteristics, and verify the bearing capacity according to IS 800:2007.	13	4	K2

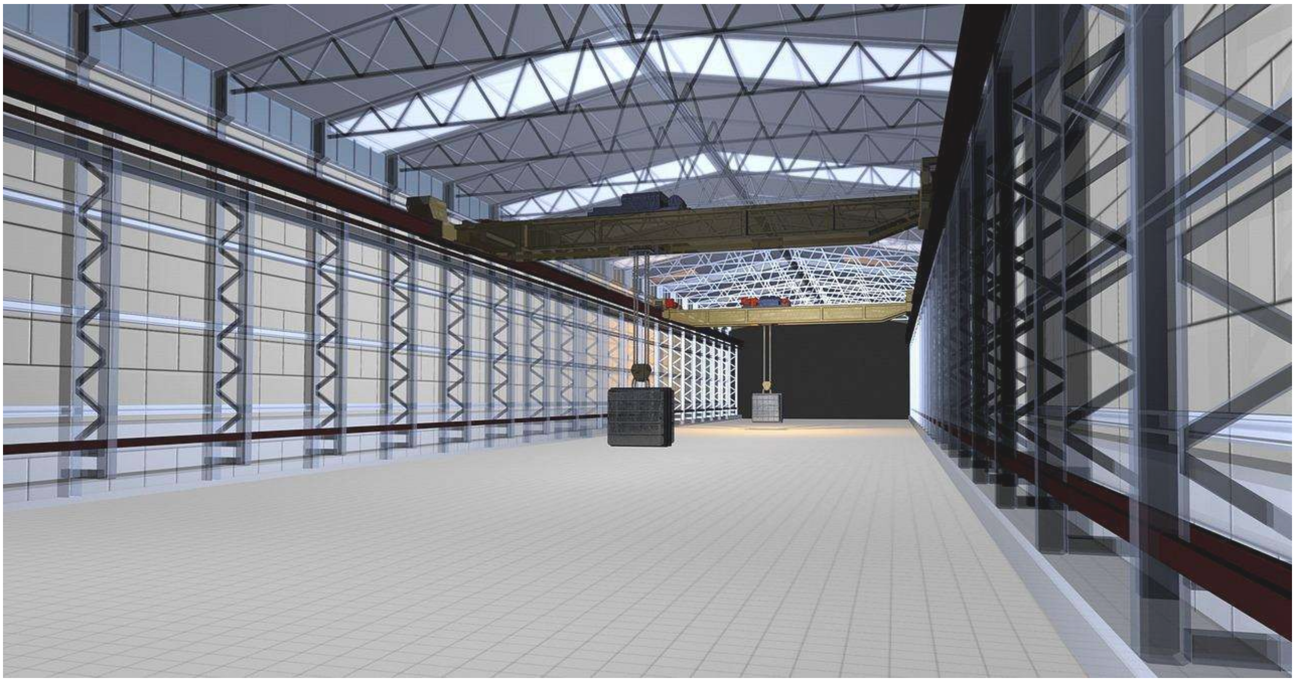
Reference Book:

- N Subramaniam , Design of Steel Structures, Oxford University press, New Delhi, 2011
- IS 875 (Part 1 to 5): Code of Practice for Design Loads (Other than Earthquake) for Buildings and Structures. Bureau of Indian Standards.
- IS 800:2007: General Construction in Steel - Code of Practice. Bureau of Indian Standards.

Unit	4. Industrial Structures	Lecture No	35
Topic	**Design of Gantry Girders**		
Learning Outcome (LO) At the end of this lecture, students will be able to		**Bloom's Knowledge Level**	
LO 1	Understand the load conditions and design requirements for gantry girders in industrial buildings..	K2	
LO 2	Apply design calculations for gantry girders to support overhead crane loads..	K3	

1. Introduction to Gantry Girders

Gantry girders are horizontal structural members used in industrial buildings to support overhead crane loads. They transfer loads from the crane to the building columns and ultimately to the foundation. Understanding the load conditions and design requirements is crucial for ensuring the safety and functionality of the structure.



2. Load Conditions and Design Requirements

2.1 Load Conditions

The primary loads acting on gantry girders include:

- **Dead Load (DL):** The self-weight of the gantry girder.
- **Live Load (LL):** The weight of the crane and the load it carries.
- **Impact Load:** Dynamic effects due to the moving crane.
- **Longitudinal Load:** Horizontal forces due to crane acceleration or braking.
- **Lateral Load:** Sideways forces due to crane movement or wind.

2.2 Design Requirements

The design of gantry girders involves:

- **Selecting a suitable section:** I-section, box-section, or built-up section.
- **Checking for bending stresses, shear stresses, and deflections.**
- **Ensuring stability against lateral-torsional buckling.**
- **Adhering to the guidelines of IS 800:2007 code.**

3. Design Calculations for Gantry Girders

3.1 Example Problem 1: Design of a Simply Supported Gantry Girder

Given:

- Span of gantry girder: 10 m
- Crane capacity: 100 kN
- Self-weight of crane: 20 kN
- Self-weight of trolley: 10 kN
- Wheelbase of crane: 3 m
- Impact factor: 25%

Step-by-Step Solution:

Step 1: Calculation of Loads

1. **Crane Load (P):**

$$P = 100\,\text{kN}$$

2. **Impact Load (I):**

$$I = 0.25 \times P = 0.25 \times 100 = 25\,\text{kN}$$

3. **Total Vertical Load (V):**

$$V = P + I + \text{Self-weight of crane} + \text{Self-weight of trolley}$$

$$V = 100 + 25 + 20 + 10 = 155\,\text{kN}$$

4. **Longitudinal Load (L):**

$$L = 0.1 \times P = 0.1 \times 100 = 10\,\text{kN}$$

Step 2: Maximum Bending Moment

$$M_{\text{max}} = \frac{V \times \text{Span}}{4}$$

$$M_{\text{max}} = \frac{155 \times 10}{4} = 387.5\,\text{kNm}$$

Step 3: Selection of Section (Refer IS 800:2007, Clause 9.1)

Assume an I-section with:

- Depth = 600 mm
- Flange width = 250 mm
- Flange thickness = 20 mm

- Web thickness = 10 mm

Step 4: Check for Bending Stress (Refer IS 800:2007, Clause 8.2)

$$f_b = \frac{M_{\max}}{Z}$$

Where Z is the section modulus.

Assume section modulus Z=5000 cm^3.

$$f_b = \frac{387.5 \times 10^6}{5000 \times 10^3} = 77.5\,\text{MPa}$$

Check against permissible stress

$$f_y/\gamma_m = 250/1.15 = 217.4\,\text{MPa}$$

Since 77.5 MPa<217.4 MPa, the section is safe in bending.

Step 5: Check for Shear Stress (Refer IS 800:2007, Clause 8.4)

$$f_v = \frac{V_{\max}}{A_w}$$

Where Aw is the web area.

Assume Aw=600×10=6000 mm2.

$$f_v = \frac{155 \times 10^3}{6000} = 25.83\,\text{MPa}$$

Check against permissible shear stress

$$f_{vy} = 250/\sqrt{3}/1.15 = 125.6\,\text{MPa}$$

Since 25.83 MPa<125.6 MPa, the section is safe in shear.

Step 6: Deflection Check (Refer IS 800:2007, Clause 9.1.2)

$$\delta = \frac{5WL^4}{384EI}$$

Where W is the total load, E is the modulus of elasticity, and I is the moment of inertia.

Assume I=3000×10^6 mm^4 and E=2.1×10^5 MPa.

$$\delta = \frac{5 \times 155 \times 10^3 \times 10000^4}{384 \times 2.1 \times 10^5 \times 3000 \times 10^6} = 9.28\,\text{mm}$$

Check against permissible deflection

$$L/750 = 10000/750 = 13.33\,\text{mm}.$$

Since 9.28 mm<13.33mm, the section is safe in deflection.

3.2 Example Problem 2: Design of a Continuous Gantry Girder

Given:

- Span of each segment: 6 m
- Crane capacity: 50 kN

- Self-weight of crane: 15 kN
- Self-weight of trolley: 5 kN
- Wheelbase of crane: 2 m
- Impact factor: 20%

Step-by-Step Solution:

Step 1: Calculation of Loads

1. **Crane Load (P):**

$$P = 50\,\text{kN}$$

2. **Impact Load (I):**

$$I = 0.2 \times P = 0.2 \times 50 = 10\,\text{kN}$$

3. **Total Vertical Load (V):**

$$V = P + I + \text{Self-weight of crane} + \text{Self-weight of trolley}$$

$$V = 50 + 10 + 15 + 5 = 80\,\text{kN}$$

4. **Longitudinal Load (L):**

$$L = 0.1 \times P = 0.1 \times 50 = 5\,\text{kN}$$

Step 2: Maximum Bending Moment (Using continuous beam analysis)

For a continuous beam with equal spans:

$$M_{\text{max}} = \frac{V \times L}{8}$$

$$M_{\text{max}} = \frac{80 \times 6}{8} = 60\,\text{kNm}$$

Step 3: Selection of Section (Refer IS 800:2007, Clause 9.1)

Assume an I-section with:

- Depth = 450 mm
- Flange width = 200 mm
- Flange thickness = 15 mm
- Web thickness = 8 mm

Step 4: Check for Bending Stress (Refer IS 800:2007, Clause 8.2)

$$f_b = \frac{M_{\text{max}}}{Z}$$

Where Z is the section modulus.

Assume section modulus Z=3000cm3.

$$f_b = \frac{60 \times 10^6}{3000 \times 10^3} = 20\,\text{MPa}$$

Check against permissible stress

$$f_y/\gamma_m = 250/1.15 = 217.4\,\text{MPa}$$

Since 20 MPa<217.4 MPa, the section is safe in bending.

Step 5: Check for Shear Stress (Refer IS 800:2007, Clause 8.4)

$$f_v = \frac{V_{\max}}{A_w}$$

Where Aw is the web area.

Assume Aw=450×8=3600 mm2.

$$f_v = \frac{80 \times 10^3}{3600} = 22.22\,\text{MPa}$$

Check against permissible shear stress

$$f_{vy} = 250/\sqrt{3}/1.15 = 125.6\,\text{MPa}$$

Since 22.22 MPa<125.6 MPa, the section is safe in shear.

Step 6: Deflection Check (Refer IS 800:2007, Clause 9.1.2)

$$\delta = \frac{5WL^4}{384EI}$$

Where W is the total load, E is the modulus of elasticity, and I is the moment of inertia.

Assume I=2000×10^6 mm^4 and E=2.1×10^5 MPa.

$$\delta = \frac{5 \times 80 \times 10^3 \times 6000^4}{384 \times 2.1 \times 10^5 \times 2000 \times 10^6} = 7.89\,\text{mm}$$

Check against permissible deflection

$$L/750 = 6000/750 = 8\,\text{mm}$$

Since 7.89 mm<8 mm, the section is safe in deflection.

Students must prepare answers for the following questions at the end of the lecture.

Qn No	Question	Marks	CO	Bloom's Knowledge Level
1	Design a gantry girder for an industrial building with a span of 8 m, crane capacity of 80 kN, self-weight of crane 15 kN, self-weight of trolley 5 kN, and a wheelbase of 2.5 m. Consider an impact factor of 25%. Perform a detailed step-by-step design and check for bending stress, shear stress, and deflection as per IS 800:2007.	13	4	K3
2	A continuous gantry girder spans 5 meters between supports and carries a crane with a capacity of 60 kN. The self-weight of the crane is 10 kN, and the self-weight of the trolley is 5 kN. The wheelbase of the crane is 2 m, and the impact factor is 20%. Design the gantry girder, ensuring to include checks for bending stress, shear stress, and deflection according to IS 800:2007.	13	4	K3

Reference Book:

- N Subramaniam , Design of Steel Structures, Oxford University press, New Delhi, 2011

- IS 875 (Part 1 to 5): Code of Practice for Design Loads (Other than Earthquake) for Buildings and Structures. Bureau of Indian Standards.
- IS 800:2007: General Construction in Steel - Code of Practice. Bureau of Indian Standards.

Unit	4. Industrial Structures	Lecture No	34
Topic	**Introduction to Pre-engineered Buildings (PEBs)**		
Learning Outcome (LO) At the end of this lecture, students will be able to		**Bloom's Knowledge Level**	
LO 1	Describe the concept and advantages of pre-engineered buildings in industrial construction.	K2	
LO 2	Identify the components and materials used in pre-engineered buildings.	K1	

1. Introduction to Pre-engineered Buildings (PEBs)

Definition

Pre-engineered buildings (PEBs) are structures designed and fabricated in factories and then assembled at the construction site. These buildings are designed based on a predetermined inventory of raw materials and manufacturing methods that can efficiently satisfy a wide range of structural and aesthetic design requirements.

Concept

The concept of PEB involves the design and manufacture of steel buildings in the factory and assembling them at the site. This approach leverages the efficiencies of mass production and modular construction, resulting in faster, cost-effective, and high-quality construction.

Advantages of PEBs in Industrial Construction

1. **Cost-Effective**: Reduces construction cost due to the efficient use of materials and labor.
2. **Speed of Construction**: Quick assembly and erection as components are pre-fabricated.
3. **Flexibility**: Easily expandable and modifiable for future needs.
4. **Quality Control**: High quality is maintained through factory production.
5. **Durability**: Steel structures are robust and can withstand adverse conditions.
6. **Reduced Maintenance**: Minimal maintenance required compared to conventional buildings.
7. **Energy Efficiency**: Can be designed for improved thermal efficiency.

IS Code

Refer to **IS 800: 2007** for the general construction in steel – Code of Practice.

2. Components and Materials Used in PEBs

Components

1. **Primary Frames**: Main load-bearing frames, including columns and rafters.
2. **Secondary Members**: Purlins, girts, and eave struts that support roof and wall sheeting.
3. **Roof and Wall Panels**: Sheets made from steel or other materials to cover the structure.
4. **Bracing Systems**: Provide stability against lateral loads.

5. **Accessories**: Include doors, windows, ventilators, skylights, etc.

Materials

1. **Steel**: The primary material used for frames and panels.
2. **Insulation**: Used for thermal efficiency, typically fiberglass or foam panels.
3. **Paints and Coatings**: For corrosion protection and aesthetic purposes.

Types/Classifications of PEBs

1. **Clear Span Buildings**: No internal columns, ideal for large open spaces.
2. **Multi-span Buildings**: Internal columns provide additional support, allowing for larger buildings.
3. **Low-rise Buildings**: Single or two-story buildings used for factories, warehouses, etc.

Principles of PEB Design

1. **Modular Design**: Utilizing standard modules to simplify design and construction.
2. **Load Analysis**: Calculating loads (dead, live, wind, seismic) as per IS 875.
3. **Efficient Use of Materials**: Minimizing waste through precise fabrication.

Example Problem

Design of a Simple PEB Structure

Problem Statement: Design a PEB structure with a span of 20 meters, a length of 50 meters, and a height of 6 meters. The building is to be used as a warehouse with minimal internal columns.

Steps:

1. **Load Calculation**: Determine the loads based on usage, location, and building dimensions.
   - **Dead Load**: Weight of structural components.
   - **Live Load**: Usage load as per IS 875 Part 2.
   - **Wind Load**: As per IS 875 Part 3.
2. **Material Selection**: Choose steel grade and section sizes.
3. **Design of Primary Frame**: Calculate moments, shear forces, and axial forces.
   - Use **IS 800: 2007** for designing steel structures.
4. **Design of Secondary Members**: Design purlins, girts, and bracing systems.
5. **Connection Design**: Design bolt and weld connections as per IS codes.
6. **Detailing**: Prepare detailed drawings for fabrication and assembly.

Calculation Example

Assuming the building to be located in a moderate wind zone:

1. **Dead Load (DL)**: Weight of steel frames and roof sheeting.

$$DL = 0.15\ \mathrm{kN/m^2}$$

2. **Live Load (LL)**: Load due to warehouse usage.

$$LL = 0.5\ \mathrm{kN/m^2}$$

3. **Wind Load (WL)**: Based on wind speed and building dimensions.

$$WL = 0.3\ \text{kN/m}^2$$

Design of a Rafter (Example Calculation):

- **Span**: 20 meters.
- **Rafter Section**: Assume ISMB 400.

Moment Calculation:

$$M = \frac{wL^2}{8}$$

$$w = DL + LL = 0.65\ \text{kN/m}^2$$

$$M = \frac{0.65 \times 20^2}{8}$$

$$M = 32.5\ \text{kNm}$$

Check against permissible stress:

$$f_b = \frac{M}{Z}$$

Where Z is the section modulus of ISMB 400.

$$Z = 1096\ \text{cm}^3 = 1096 \times 10^{-6}\ \text{m}^3$$

$$f_b = \frac{32.5}{1096 \times 10^{-6}}$$

$$f_b = 29.64\ N/mm^2$$

Since f_b is less than the permissible stress for the steel grade used, the section is adequate.

Students must prepare answers for the following questions at the end of the lecture.

Qn No	Question	Marks	CO	Bloom's Knowledge Level
1	Describe the concept of pre-engineered buildings (PEBs) and discuss the advantages of using PEBs in industrial construction. Include references to IS codes where applicable.	13	4	K3
2	Identify and explain the main components and materials used in pre-engineered buildings. Provide examples of how these components are designed, including any relevant calculations.	13	4	K3

Reference Book:

- N Subramaniam , Design of Steel Structures, Oxford University press, New Delhi, 2011
- IS 875 (Part 1 to 5): Code of Practice for Design Loads (Other than Earthquake) for Buildings and Structures. Bureau of Indian Standards.
- IS 800:2007: General Construction in Steel - Code of Practice. Bureau of Indian Standards.