LECTURE NOTES ON

STRUCTURAL DESIGN-II

(STEEL STRUCTURES)

DEPARTMENT OF CIVIL ENGG.

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Chapters

CHAPTER 1

INTRODUCTION

In this course, we focus on one of the most important engineering material – **STRUCTURAL STEEL**.

Low Carbon Steel (carbon content: 0.1 – 0.25%) is useful as a structural material (as load bearing frames & trusses), and hence known as ***Structural Steel***.

- Besides carbon (C), it also contains Manganese (Mn), Sulphur (S) & Phosphorus (P).
- Molten metal is rendered free of impurities by oxidation and then casted into semi-finished products (like slabs, blooms and billets). These semi-finished products are heated at 1200°C to make it malleable and then rolled into structural steel sections (I, C, T, L, tubes, bars, flats, plates etc).
- Types of Structural Steel:
    - **Standard Structural Steel** (IS 2062)
        - C & Mn are the main strengthening elements
        - Max C – 0.23%
        - Max Mn – 1.5%
    - **Medium/High Strength Micro-alloyed Steel** (IS 8500)
        - Micro-alloying elements such as Niobium, Vanadium, Titanium etc are added to achieve high strength
        - Total micro-alloying elements restricted to 0.25%

PROPERTIES OF STRUCTURAL STEEL

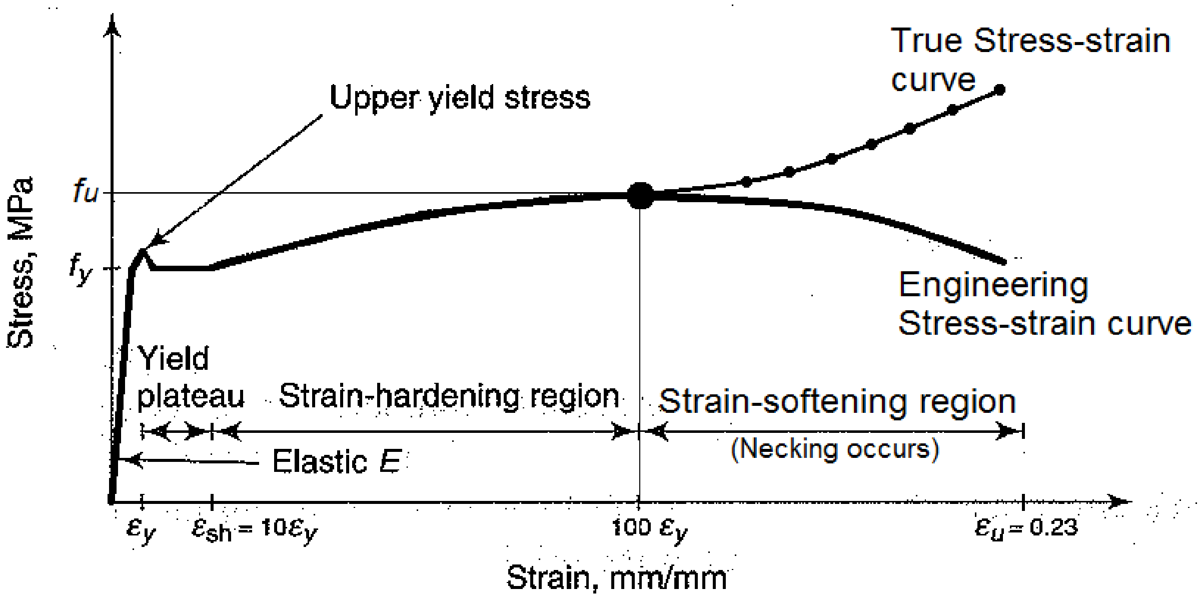
1) PHYSICAL PROPERTIES (Cl. 2.2.4.1 of IS 800-2007, Page 12)

- Modulus of Elasticity E = 2×10^5 N/mm^2
- Modulus of Rigidity G = 0.77×10^5 N/mm^2
- Density ρ = 7850 kg/m^3
- Poisson's ratio ϑ = 0.30 (elastic range)
  = 0.50 (plastic range)
- Coefficient of thermal expansion α = 12×10^{-6} /°C
- Brinell Hardness Number = 150 – 190
- Vickers Hardness Number = 150 – 190
- Approximate Melting Point = 1530 °C

2) Mechanical properties

a) Ultimate Strength (or Tensile Strength) - f_u

- It is the minimum guaranteed ultimate tensile strength at which the steel fails.
- It is obtained from Tension test on a standard specimen.
- In this test, Gauge length ($L_g = 5.65\sqrt{A}$) & Initial crossectional area (A_o) are the important parameters.
- Stress-strain diagram for a mild steel specimen subjected to gradually increasing tensile load is obtained as shown:

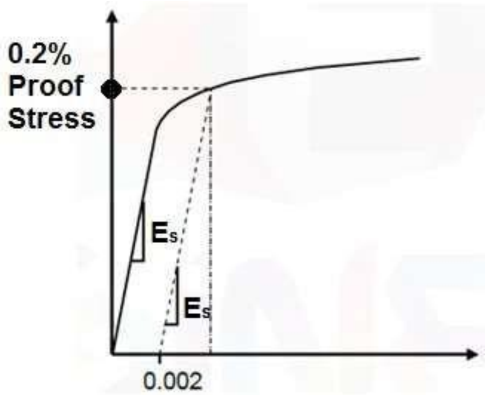


- Engineering Stress-strain curve uses Initial cross-sectional area of specimen; while True stress-strain curve uses actual cross-sectional area.
- Steel is designated in terms of ***Characteristic Ultimate Tensile Strength*** **in MPa** (It is the stress below which not more than 5% of test results are expected to fall), ***Grade of steel*** (Grade A – used in structures subjected to normal load conditions; Grade B – used for structures subjected to brittle fracture, cyclic loading etc.; Grade C – for those subjected to low temperatures, severe impacts, brittle fracture etc.) and Weldability (denoted by W if steel is weldable).

  Eg: Fe 410 WA
  This is a structural steel section of characteristic ultimate tensile strength = 410MPa, of Grade A and is Weldable.

b) Yield Stress (or Proof Stress) - f_y

- It is the stress level at which the material undergoes large deformations.
- In mild steel, there is a well-defined yield point, as shown in above figure.
- In case of high strength steel, there may be no well-defined yield points. In such case, stress corresponding to 0.2% proof strain is adopted as Yield stress, hence known as Proof stress. (see figure shown)



c) Ductility

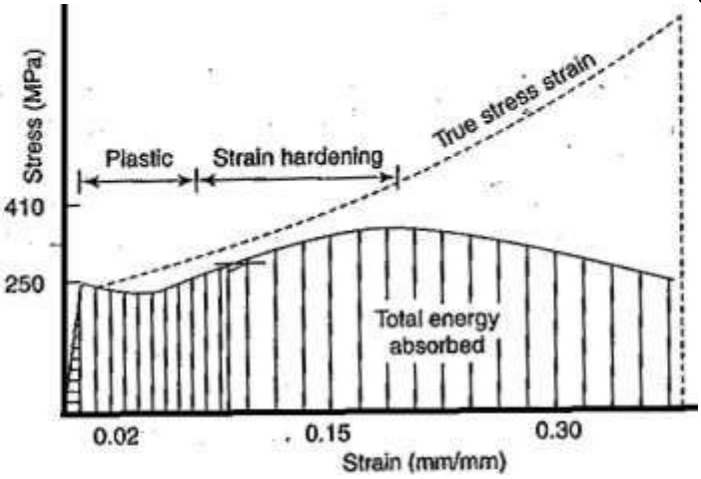
- Ductility is the capacity to undergo large inelastic deformations without significant loss of stiffness.
- Ductility is measured by measuring the % elongation of the tension test specimen.

d) Hardness

- Hardness is the resistance to indentation & scratching.
- Tested by 3 methods
  - Brinell Hardness - Steel ball indentor is used
  - Vicker Hardness - Diamond square pyramid indentor of included angle 135^o
  - Rockwell Hardness- Diamond indentor with included angle 120^o

e) Toughness

- Toughness is the ability to resist fracture under impact loading.
- Area under stress - strain curve is a measure of toughness.



- Important design parameter for structures subjected to impact loads (Eg: Bridges) and those subjected to seismic loads.

f) Weldability

Steel structural elements may be connected by weld. But the steel used must be weldable. Steel is weldable, if :

- Hardness is low
- There is adequate elongation & notch toughness

This is taken care of, if *Carbon Equivalency* of the steel used is under control.

Carbon equivalency is the equivalent carbon that produce the same effect as that of chemical components present in steel used. This is given by:

$$C_{eq} = \frac{C + Mn}{6} + \frac{(Cr + Mo + V)}{5} + \frac{(Ni + Cu)}{15}$$

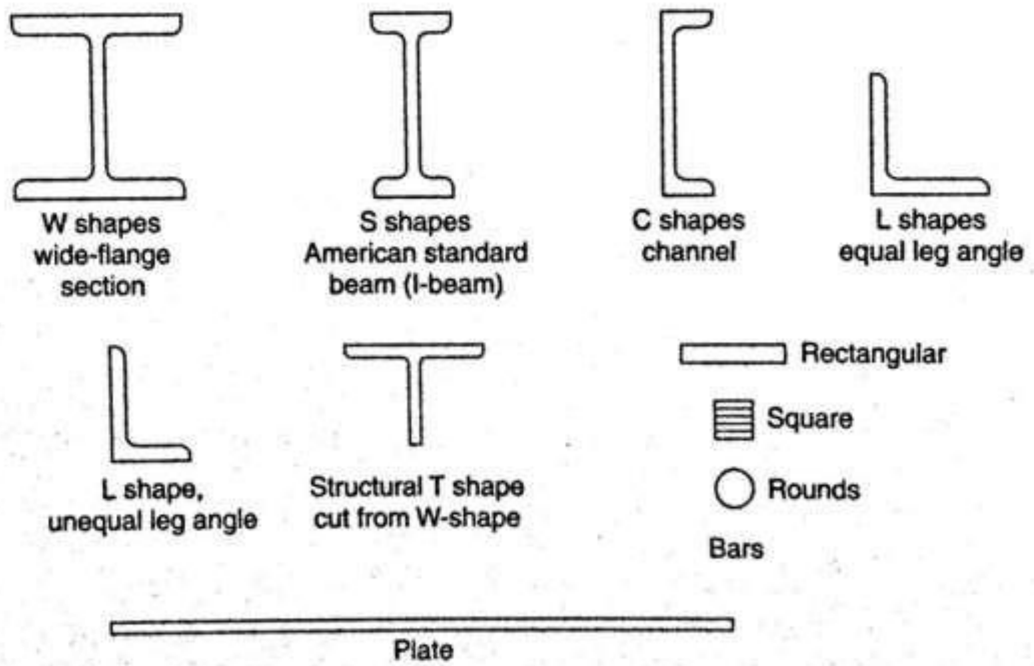
where C is carbon, Mn is manganese, Cr is chromium, Mo is molybdenum, V is vanadium, Ni is nickel, and Cu is copper. Each symbol refers to % weight of each element.

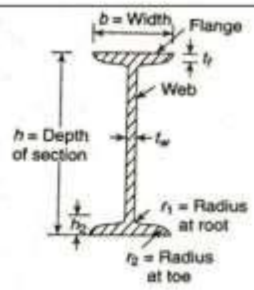
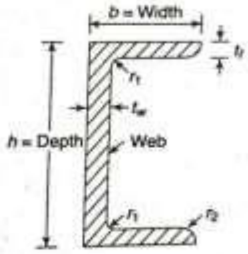
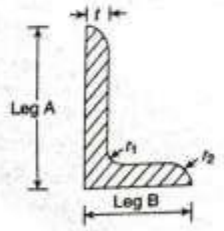
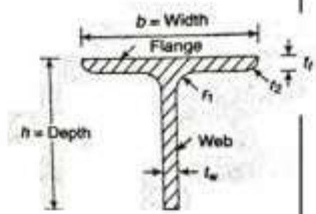
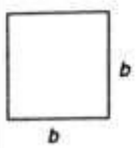
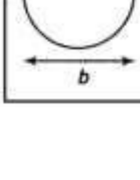
Smaller the Carbon Equivalency, better is the Weldability of steel.

STRUCTURAL STEEL SECTIONS

Various hot-rolled structural steel sections are as follows:

- Rolled beams
    - Junior beams (ISJB, meaning Indian Standard Junior Beams)
    - Lightweight beams (ISLB)
    - Medium-weight beams (ISMB)
    - Wide-flange beams (ISWB)
    - Heavyweight beams/columns (ISHB)
    - Column sections (ISSC)
- Channels: Junior, light, and medium and parallel flange (ISJC, ISLC, ISMC, ISMCP)
- Equal angles (ISEA or ISA)
- Unequal angles (ISA)
- T sections (ISJT, ISLT, ISST, ISNT and ISHT)
- Rolled bars
    - Round (ISRO)
    - Square (ISSQ)
- Tubular sections (ISLT, ISMT, ISHT)
- Plates (ISPL)
- Strips (ISST)
- Flats (ISFl)



ROLLED SECTION	INDIAN STANDARD DESIGNATION	REMARKS
	ISJB ISLB ISMB ISWB ISHB	A beam section referred to as ISMB 400 at 0.616 kN/m is an ISMB with a depth of 400 mm and a weight of 0.616 kN per metre length.
	ISJC ISLC ISMC	Channel sections are referred to, for example, as ISMC 200 at 0.221 kN/m.
	ISA	Angles are equal or unequal. For equal angles, $A = B$. For unequal angles $A > B$. Angles are referred to, for example, as ISA 60 × 60 × 6, indicating equal angles with legs 60 mm each and thickness 6 mm. An example of an unequal angle is ISA 100×75×6.
	ISNT ISHT ISST ISLT ISJT	Tee sections are referred to, for example, as ISNT 100 at 0.147 kN/m, indicating that the depth of the section is 100 mm.
Plates	$t \geq 5$ mm	Plates are referred to in terms of width × thickness, e.g., 900 × 10 indicates a plate 900 mm wide and 10 mm thick.
Strips	$t < 5$ mm	Strips are referred to in terms of width × thickness.
Flats	$t \geq 5$ mm $b \leq 250$ mm	Flats are referred to in terms of $b \times t$.
		A square bar is referred to in terms of its sides, e.g., a 20-mm square bar.
		A round bar is referred to in terms of its diameter, e.g., a 20-mm diameter bar.

Advantages of Steel structures over RCC structures

1. Strength – weight ratio of structural steel is very high compared to RCC. Hence structural steel requires smaller cross sections to resist external loads.
2. Precast structural steel sections are easily available and erection becomes faster.
3. Since steel is a ductile material, failure of structures is neither abrupt nor catastrophic.
4. It has 100% scrap value. It is recyclable; can be reused even after dismantling.
5. It has longer life, if maintained properly.
6. Since sections are all factory made, quality control is ensured.
7. Strengthening of structures is relatively simpler. This can be performed by connecting additional sections to the existing sections.

Disadvantages of Steel structures

1. Less fire resistance
2. more susceptible to corrosion.
3. High maintenance cost
4. High initial cost of investment/installation
5. Strength of steel sections reduce if subjected to large number of stress reversals (fatigue)

Types of Load

- Dead Loads (permanent; including self-weight, floor covering, suspended ceiling, partitions,)
- Live Loads (not permanent; the location is not fixed; including furniture, equipment, and occupants of buildings)
- Wind Load (exerts a pressure or suction on the exterior of a building)

Types of Load Continued

- Earthquake Loads (the effects of ground motion are simulated by a system of horizontal forces)
- Snow Load (varies with geographical location and drift)
- Other Loads (hydrostatic pressure, soil pressure)
- If the load is applied suddenly, the effects of IMPACT must be accounted for.
- If the load is applied and removed many times over the life of the structure, FATIGUE stress must be accounted for

Load Factors

- The values are based on extensive statistical studies
  - DL only 1.4D
  - DL+LL+SL (LL domin.) 1.2D+1.6L+0.5S
  - DL+LL+SL (SL domin.) 1.2D+0.5L+1.6S

Design Philosophies

- Allowable Stress **Design** Method (ASD)
- Load and Resistance Factor **Design** (LRFD)

- A member is selected such that the max stress due to working loads does not exceed an allowable stress.

- It is also called elastic **design** or working stress **design**.
- allowable stress=yield stress/factor of safety
- actual stress $\subseteq$ allowable stress

CHAPTER 2

STRUCTURAL STEEL FASTENERS AND CONNECTION

DESIGN OF BOLTED CONNECTIONS

INTRODUCTION

Any steel structure is an assemblage of different members or sections, which are connected to one another at its ends, using connections – BOLTS, WELDS and/or RIVETS.

Connections have the following characteristics:

- Facilitate the flow of forces and/or moments from one member to another
- Also used to extend lengths of different members
  - It may be considered as a weak point of a structure, because connection failure is undesirable. This is due to the fact that the connection failure is not ductile, unlike member failure. It is a catastrophic failure. Connection failure is avoided by adopting a high factor of safety for joints than members.

TYPES OF BOLTS

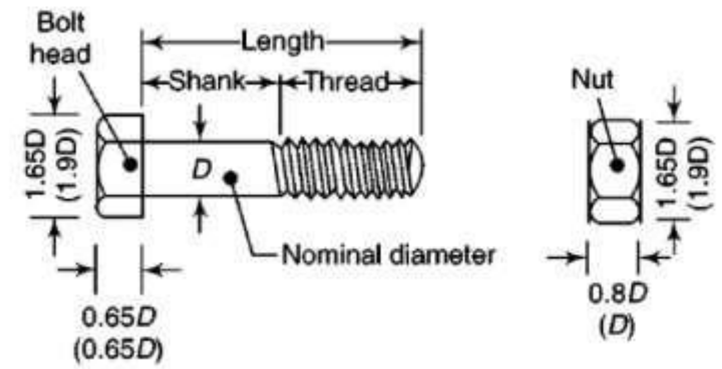
Bolts are available of the following types:

a) Black bolts b) Turned bolts c) Ribbed bolts d) High strength bolts

a) Black bolts/ Unfinished Bolts/ C-grade bolts

- Ordinary, unfinished, rough or common bolts
- Least expensive bolts
- Generally used in structures subjected to static loads; not recommended if subjected to impact, fatigue or dynamic loads.
- Made of mild steel rods with square or hexagonal heads & nuts
- Designated by its Diameter (i.e., Shank Diameter in mm) as M5 to M36.
- Cross-sectional area at threads = 0.78 x Cross-sectional area at shank
- Generally used grade(or property class) = 4.6 grade bolts

  where f_{yb} represents yield strength of bolt & f_{ub} represents ultimate strength of bolt



Hexagonal head black bolt and nut. Figures in brackets are for high-strength bolts and nuts.

- Here, slip occurs between connected plates, when force is applied. As a result, the plates bear against the bolts. Such type of connections are known as Bearing-type connections.

b) Turned Bolts (Close Tolerance Bolts)

- These are similar to unfinished bolts, with the difference that the shanks of these bolts are formed from a hexagonal rod.
- The surface of these bolts are prepared and machined carefully to fit in the hole. Tolerances allowed are about 0.15 mm to 0.5 mm. Since the tolerance available is small, these bolts are expensive. The small tolerance necessitates the use of special methods to ensure that all the holes align correctly.
- These bolts (precision and semi-precision) are used when no slippage is permitted between connected parts and where accurate alignment of components is required.
- They are mainly used in special jobs (in some machines and where there are dynamic loads).

c) Ribbed Bolts

- Have rounded head and raised ribs parallel to shank.
- Actual dia of ribbed bolts is slightly larger than the hole dia. Hence while driving into the hole, the ribs cit the edges around the hole, thereby producing a relatively tight fit.

d) High strength bolts

- High-strength bolts are made from bars of medium carbon steel.
- Their high strength is achieved through quenching and tempering process or by alloying steel. Hence, they are less ductile than black bolts. The material of the bolts do not have a well-defined yield point. Instead of using yield stress, a so-called *proof stress* is used. In IS **800,** the proof stress is taken **as** 0.7 times the ultimate tensile stress of the bolt.
- Bolts of sizes M16 – M36 are available.
- Grades (or property class) available are 8.8S, 10.9 S etc.
    - Number defined similar to black bolts
    - Letter 'S' denotes high-strength structural bolts
- % elongation of High strength bolts < % elongation of black bolts
- Here, slip between connected plates is prevented by applying initial pretension (to bolts) using torque wrenches, which induces friction. Such high-strength bolts are called High Strength Friction Grip (HSFG) bolts. Such connections are known as Non-slip connections or Friction-type connections.

ADVANTAGES OF BOLTED CONNECTIONS

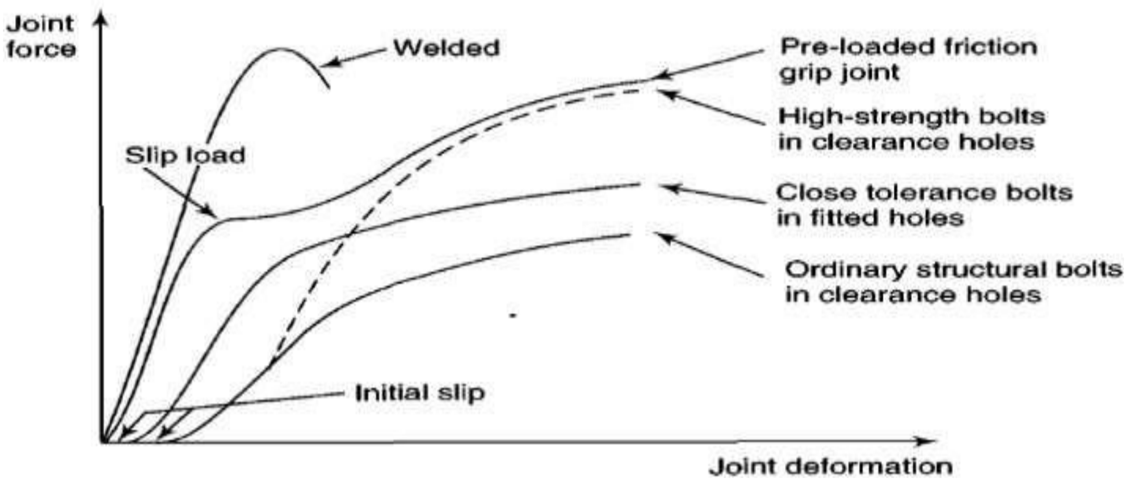
Advantages of Black bolts over riveted or welded connections:

- Use of unskilled labour and simple tools
- Noiseless and quick fabrication
- No special equipment/process needed for installation

- Fast progress of work
- Accommodates minor discrepancies in dimensions
- The connection supports loads as soon as the bolts are tightened
- Unlike riveted joints, few persons are required for making the connections.
- No heating is required and no danger of tossing of bolt. Thus, the safety of the workers is enhanced.
- Alterations, if any (e.g. replacement of the defective bolt) are done easily than in welded or riveted connections.
- Dismantling of structures is easy

Advantages of HSFG bolt over black bolts:

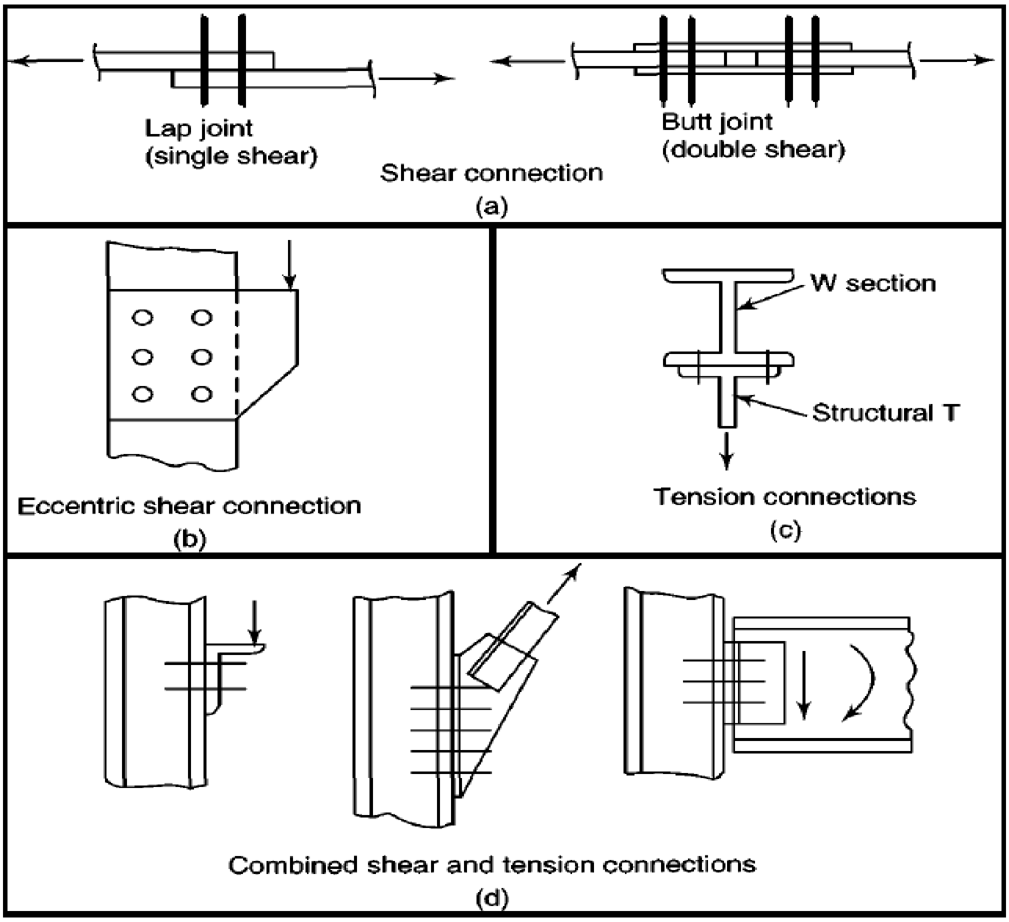
- HSFG bolts do not allow any slip between the elements connected, thus providing rigid connections.
- Due to the clamping action, load is transmitted by friction only and the bolts are not subjected to shear and bearing.
- Due to the smaller number of bolts, the gusset plate sizes are reduced.
- Deformation is minimized.
- Noiseless fabrication, since the bolts are tightened with wrenches.
- The possibility of failure at the net section under the working loads is eliminated.
- Since the loads causing fatigue will be within proof load, the nuts are prevented from loosening and the fatigue strength of the joint will be greater and better than welded and riveted joints.
- Since the load is transferred by friction, there is no stress concentration in the holes.



TYPES OF BOLTED CONNECTIONS

Bolted connections are of the following types, based on the mode of load transfer:

(a) Shear connections – Lap joint & Butt joint
(b) Eccentric shear connections
(c) Tension connections
(d) Combined Shear & Tension connections

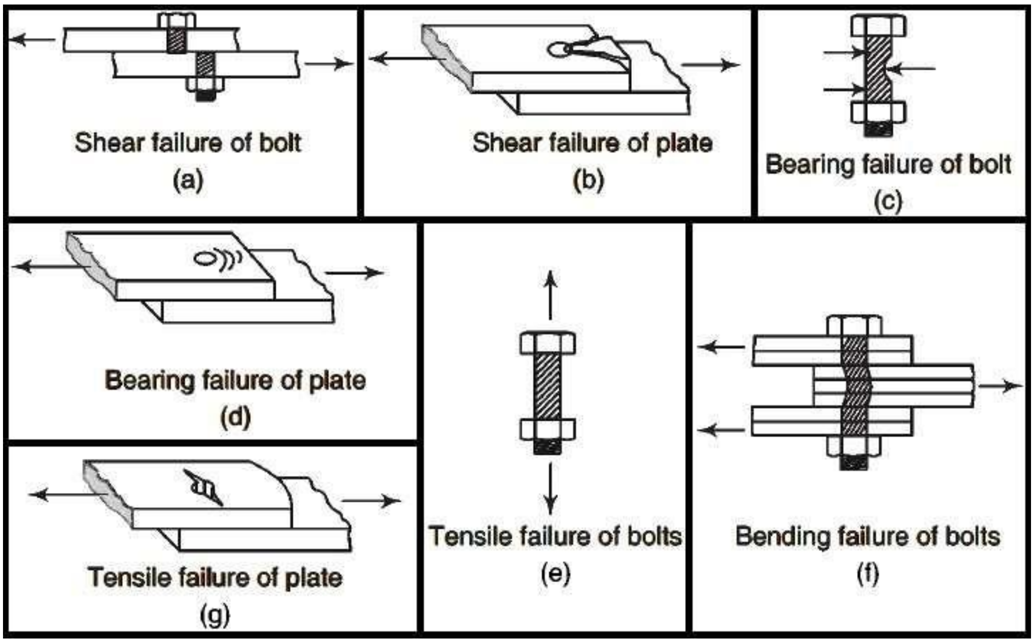


PROBABLE MODES OF FAILURE OF BOLTED CONNECTIONS

The possible limit states or failure modes of bolted connections are:

a) Shear failure of bolt
b) Shear failure of plate
c) Bearing failure of bolt
d) Bearing failure of plate
e) Tensile failure of bolts

f) Bending of bolts
g) Tensile failure of plate



IMPORTANT TERMINOLOGIES & RELEVANT CODAL PROVISIONS OF IS800

i. **Nominal diameter of the Fastener** (d) = Diameter of Bolt at the shank region

ii. **Bolt hole Diameter** (d_o) = d + clearance (Cl. 10.2.1 & Table 19)

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
12 – 14	1.0	3.0	4.0	2.5 *d*
16 – 22	2.0	4.0	6.0	2.5 *d*
24	2.0	6.0	8.0	2.5 *d*
Larger than 24	3.0	8.0	10.0	2.5 *d*

iii. **Pitch** (p): centre-to-centre distance between two adjacent rows of bolts, measured in the direction of application of force (Cl. 10.2.2 & 10.2.3)

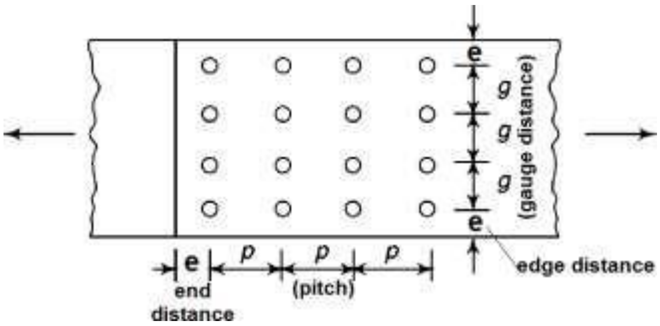
- Min. p = 2.5 d
- Max. p = 32t or 300mm, whichever is less (generally)
  = 16t or 200mm, whichever is less (tension members)
  = 12t or 200mm, whichever is less (compression members)
  Where t is the thickness of the thinner plate in mm

iv. **Gauge distance** (g): centre-to-center distance between bolts measured perpendicular to the direction of application of force.

- Max. g = (4t + 100)mm or 200mm, whichever is less (generally)

v. **End distance** (e): distance measured in the direction of stress from the centre of a hole to the end of the element. (Cl. 10.2.4) *vi.* **Edge distance** (e): distance measured at right angles to the direction of stress from the centre of a hole to the adjacent edge.

- Min. e = $1.7d_o$ (for hand-flame cuts)
  = $1.5d_o$ (for machine-flame cuts)
- Max. e = $12\varepsilon t$, where ε = ——



DESIGN STRENGTH OF ORDINARY BLACK BOLTS:

We have already discussed that, bolted connections fail by 3 modes – shear, bearing & tension. Hence, the design strength of bolts depend on these modes of failure.

❖ **DESIGN SHEAR CAPACITY OF BOLTS** (Cl. 10.3.3)

10.3.3 *Shear Capacity of Bolt*

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

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

where

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

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

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.

The Design shear capacity of Bolt Vdsb shall be reduced in case of Long joints, larger Grip lengths and usage of packing plates. Their reducing factors are given below:

10.3.3.1 *Long joints*

When the length of the joint, l_j (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: $\boxed{\beta_{lj} = 1.075 - l_j / (200\,d)}$ but $0.75 \le \beta_{lj} \le 1.0$

where d = Nominal diameter of the fastener.

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

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

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

❖ DESIGN BEARING CAPACITY OF BOLTS (Cl. 10.3.4)

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:

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

where

$$\boxed{\begin{aligned} V_{npb} &= \text{nominal bearing strength of a bolt} \\ &= 2.5\, k_b\, d\, t\, f_u \end{aligned}}$$

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.

❖ **DESIGN TENSION CAPACITY OF BOLTS** (Cl. 10.3.5)

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.

NOTE: Design capacity of bolts is calculated from the above three criteria, depending upon the kind of forces the bolt is subjected to. And finally, the BOLT VALUE is reported as the least of these design capacities.

Strength of bolt hole in bearing $= dt\ \sigma_{pf}$

$$= 20 \times 14 \times 300 \times 10^{-3} = 84 \text{ kN}$$

Hence, strength of the bolt = 62.830 kN

Example 2.12 Design a butt joint to connect two plates 240 × 12 mm (f_y = 250 N/mm²) using ordinary M20 bolts. Arrange the bolts to give maximum efficiency.

Solution Let us provide a double cover butt joint.

$$\text{Thickness of cover plate } = \frac{5}{8} \times 12 = 7.5 \text{ mm} = 8 \text{ mm}$$

The tensile force (T) the main plate can carry

$$= bt\ \sigma_{at} = 240 \times 12 \times 0.6 \times 250/10^3 = 432 \text{ kN}$$

$$\text{Shear strength of bolt} = 2\frac{\pi}{4}\, d^2\, \tau_{vf}$$

$$= 2 \times \frac{\pi}{4} \times 20^2 \times 80/10^{-3} = 50.265 \text{ kN}$$

Bearing strength of bolt $= dt\ \sigma_{pf}$

$$= 20 \times 12 \times 250 \times 10^{-3} = 60 \text{ kN}$$

Hence, strength of the bolt = 50.265 kN

$$\text{Number of bolts required} = \frac{432}{50.265} = 8.594 \approx 9$$

Provide a diamond joint as shown in Fig. Ex. 2.12.

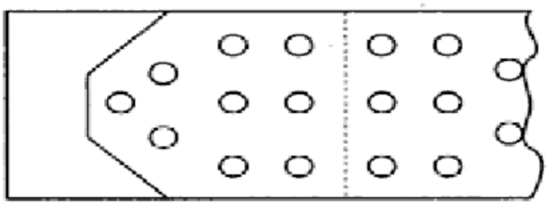


Fig. Ex. 2.12

Example 2.13 An ISA 100 × 100 × 10 mm is subjected to a tension of 66 kN. It is to be jointed to a 12 mm thick gusset plate. Design a high strength bolted joint.

Solution Design load = 66 kN

$$F = \frac{n\,\mu\, t}{F_s}$$

$$66 = \frac{1 \times 0.5 \times T}{1.4}$$

Proof load, T = 184.8 kN

The bolts are under single shear. Let us try High Strength M 14–8.8 G bolts. From Table 2.5, the proof load for the bolt is 68.8 kN.

$$\text{Number of bolts required} = \frac{184.8}{68.8} = 2.68 \approx 3$$

Use 3, High Strength M 14–8.8 G bolts.

Example 2.14 Design a bolted connection using HSFG bolts to connect flange of the bracket-Tee with the column flange as shown in Fig. Ex. 2.14. The double angle tie member is connected to the web of Tee bracket as shown in the Fig. Ex. 2.14 Assume a slip factor of 0.5.

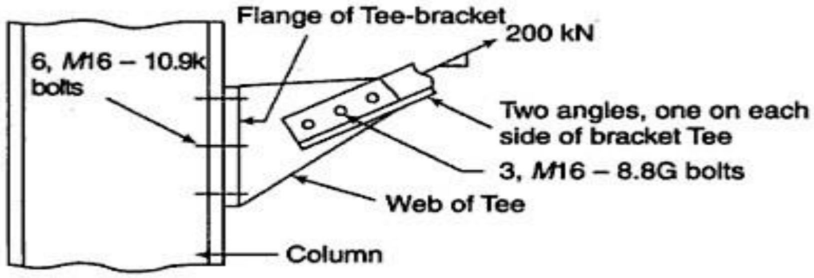


Fig. Ex. 2.14

Solution

Connection of tie member with Tee-bracket

There are two interfaces.

Member force = 200 kN

$$F = \frac{n\mu T}{F_s}$$

$$200 = \frac{2 \times 0.50 \times T}{1.4}$$

Proof load, $T = 280.0$ kN

Let us try M 16–8.8 G bolts

The proof load for the above bolt is 94.5 kN.

$$\text{Number of bolts required} = \frac{280.0}{94.5} = 2.96 \approx 3$$

Provide 3 M 16–8.8 G HSFG bolts for making the connection.

Connection of Tee-bracket with column flange

$$\text{Tension on the connection} = 200 \times \frac{4}{5} = 160 \text{ kN}$$

$$\text{Shear on the connection} = 200 \times \frac{3}{5} = 120 \text{ kN}$$

Let us try total 6 HSFG M 16–10.9 K bolts, three on each part of the Tee-flange on the two sides of the bracket web.

$$\text{Tension per bolt} = \frac{160}{6} = 26.66 \text{ kN}$$

$$\text{Shear per bolt} = \frac{120}{6} = 20 \text{ kN}$$

Example 2.15 Design a lap joint to connect two plates each of width 100 mm, if the thickness of one plate is 12 mm and the other is 10 mm. The joint has to transfer a working load of 100 kN. The plates are of fe 410 grade. Use bearing type of bolts and draw connection details.

Solution:

Using M16 bolts of grade 4.6, d = 16 mm do = 18 mm fub = 400 N/mm2
Since it is a lap joint, the bolt is in single shear, the critical section being at the roots of the thread of the bolts.

Nominal strength of a bolt in shear

$$V_{nsb} = \frac{fub}{\sqrt{3}} 1 \text{ x0+ } 0.78 \quad \frac{\pi}{4} d^2$$

$$- \frac{400}{\sqrt{3}} \times 0.78 \times \frac{\pi}{4} \times 16^2$$

= 36218 N

Design strength of a bolt in shear

Minimum pitch to be provided = 2.5 *d* -- 2.5 x 16 = 40 mm

Minimum edge distance = 1.5 *d* = 1.5 x 18 = 27 mm

Provide *p* ---- 40 mm and *e* ---30 mm.

Strength in bearing:

k_p is least of $\frac{30 \quad 40}{3 \times 18' \; 3 \times 18}$ -- 0.25, $\frac{\mathbf{400}}{\mathbf{410}}$ and 1.0

i.e., k_b ----0.4907

Now, thickness of thinner plate = 10 mm, f_u = 400 N/mm^2

. Normal bearing strength of a bolt

V_{erb} ---- 2.5 $k_b\, dt\, f$

= 2.5 x 0.4907 x 16 x 10 x 400

= 78512 N

. Design strength of M16 bolts= 28974 N

Working (nominal load) = 100 kN.

Design load ioo i.s = iso i«.

Hence, no. of bolts required= $\frac{150 \times 1000}{28974}$ = 5.18

Check for the Strength of Plate

$$T_{dn} = \frac{0.9\, A_n f_u}{\gamma_{ml}}$$

There are two holes along the critical section,

$$\therefore \quad T_d \text{ ——} \frac{0.9 \times (100 — 2 \times 18) \times 10 \times 410}{1.25}$$

= 188928 N = 188.928 kN > 150 kN.

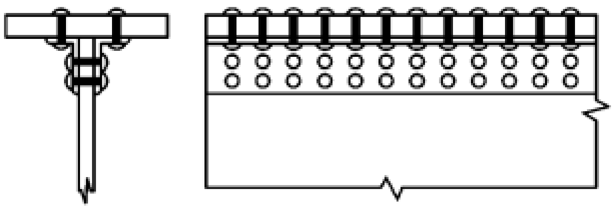
DESIGN OF WELDED CONNECTIONS

Welding is the method of connecting two pieces of metal by heating to a plastic or fluid state (with or without pressure), so that fusion occurs.

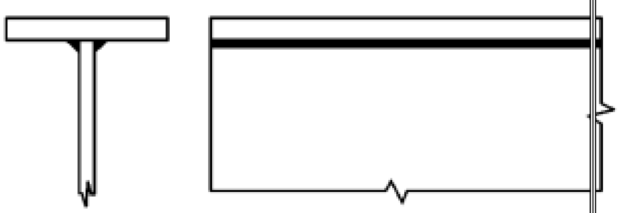
ADVANTAGES & DISADVANTAGES OF WELDING

Advantages of Welding over bolting or riveting

- Welded connections eliminate the need for making holes in the members. Hence the calculation of net section is eliminated.
- Welding offers airtight and watertight jointing of plates and hence is employed in the construction of water/oil storage tanks, ships, etc.
- Welded joints are economical, since they enable direct transfer of stresses between the members. Moreover, the splice plates and bolts are eliminated, which otherwise would make the connection expensive. The required size of gusset plates is also smaller, because of reduced connection length. Due to the elimination of operations such as drilling and punching, welding results in less fabrication costs.
- Time is also saved in fabrication, and field erection. Welding also requires considerably less labour for executing the work. It is estimated that the total overall savings by employing welding over bolting may be up to **15%.**
- Welded structures are more rigid (due to the direct connection of members by welding) as compared to bolted joints. In bolted joints, the cover plates, connecting angle, etc. may deflect with the member during load transfer thus making a structure flexible. Rigid structures are always more economical than flexible structures, due to the transfer of moments from one member to another.
- Welded connections are usually aesthetic in appearance and appear less cluttered in contrast to bolted connections.



Bolted girder section



Welded girder section

- Welding is practicable even for complicated shapes of joints. Eg: connections with tubular sections can be made easily by welding, whereas it is difficult to make them using bolting.
- Alterations can be made with less expense in case of welding as compared to bolting. It is also easy to correct mistakes in fabrication during erection, whereas a mismatch of holes in a bolted connection is very difficult to correct.
- The efficiency of a welded joint is more than a bolted joint. In fact 100% efficiency can be obtained using welding. Due to the elimination of holes, stress concentration effect is considerably less in welded connections.
- The process of welding is relatively silent compared to riveting and bolting (drilling holes) and requires less safety precautions.

Disadvantages of Welding over bolting or riveting

- Welding requires highly skilled human resources.
- The inspection of welded joints is difficult and expensive, whereas inspection of bolted joints is simple. Moreover, non-destructive testing is required in important structures.
- Costly equipment is necessary to make welded connections.
- Welded connections are prone to cracking under fatigue loading.
- Proper welding may not be done in field conditions, especially in vertical and overhead positions.
- The possibility of brittle fracture is more in the case of welded joints than in bolted connections.
- The welding performed in the field is expensive than performed in the shop.
- Welding at the site may not be feasible due to lack of power supply.

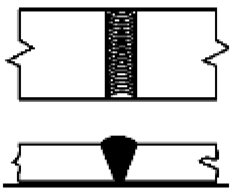
TYPES OF WELD

The welds may be grouped into four types as follows:

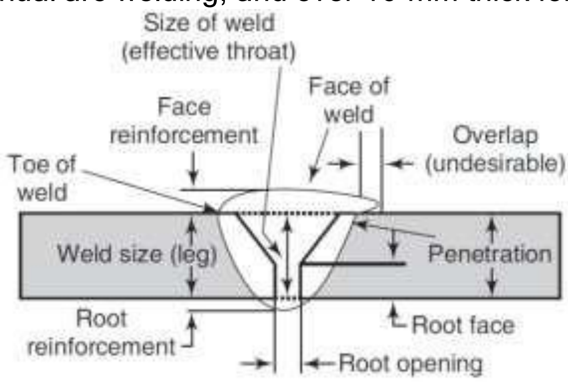
(a) Groove welds (b) Fillet welds (c) Slot welds (d) Plug welds

(A) Groove Welds

- Groove welds are used to connect structural members that are aligned in the same plane and often used in butt joints.



- The grooves have a slope of ***30°-60°.*** Edge preparation becomes necessary for plates over 10-mm thick for manual arc welding, and over 16-mm thick for automatic welding.

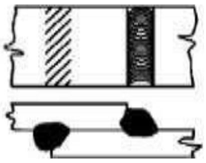


- Since groove welds will transmit the full load of the members they join, they should have the same strength **as** the members they join. Hence, only full penetration groove welds are often used. Partial penetration groove welds should not be used especially in fatigue situations.
- The choice between single or double penetration depends on access on both sides, the thickness of the plate, the type of welding equipment, the position of the weld, and the means by which the distortion is controlled.

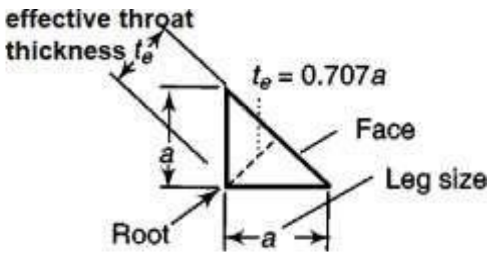
- For a groove weld, the root opening or gap**,** is provided for the electrode to access the base of the joint. The smaller the root opening, the greater will be the angle of the bevel.

(B) Fillet Welds

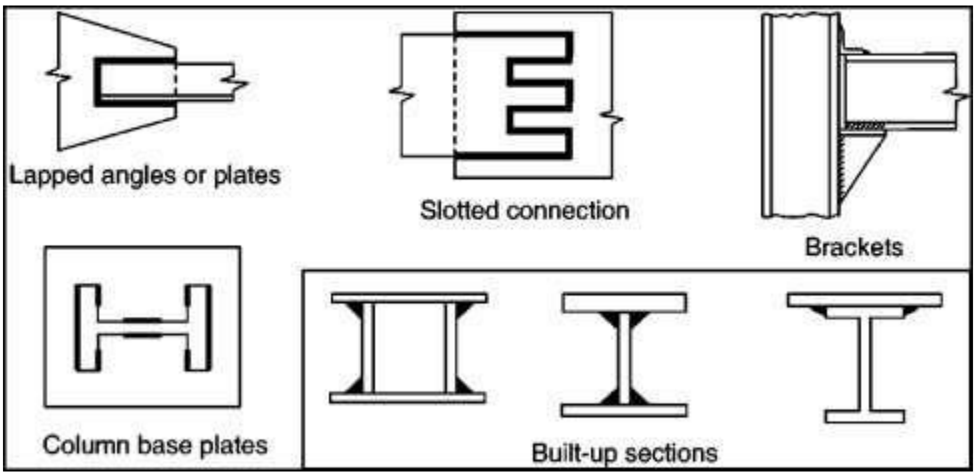
- They are approximately triangular in cross section.



- Unlike groove welds, they require less precision in 'fitting up' two sections, due to the overlapping of pieces. Hence, they are adopted in field as well as shop welding.
- Since they do not require any edge preparation (edge conditions resulting from flame cutting or shear cutting procedures are generally adequate), they are cheaper than groove welds.
- In connections, members generally intersect at right angles, but intersection angles between 60° and 120° can be used.

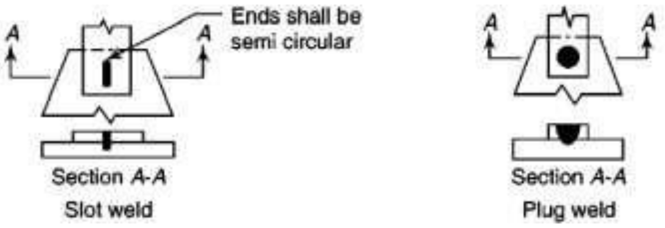


- They fail in shear.
- Fillet welds are most widely used due to their economy, ease of fabrication, and adoptability at site. Hence, fillet welds are used extensively (about 80%) followed by groove welds (15%).
- They have the following application.



(C) Slot And Plug Welds

- Slot and plug welds are not used exclusively in steel construction.
- When it becomes impossible to use fillet welds or when the length of the fillet weld is limited, slot and plug welds are used to supplement the fillet welds.
- They are also assumed to fail in shear. Thus, their design strength is similar to that of fillet welds.



DESIGN OF WELDS (Based on provisions of IS 800-2007 - Pg78-80)

The following assumptions are usually made in the design of welded joints.

- The welds connecting the various parts are homogenous, isotropic, and elastic.
- The parts connected by the welds are rigid and their deformation is, therefore, neglected.
- Only stresses due to external forces are considered. The effects of residual stresses, stress concentrations, and the shape of the weld are neglected.

(a) Groove welds

- The groove welds in butt joints will be **treated as parent metal with a thickness equal to the throat thickness** and the stresses shall not exceed those permitted in the parent metal. (Cl. 10.5.7.1.2)
- Here, failure occurs by **yielding** of weld material. Hence, **yield strength (f_y)** is considered.
- Design strength of weld subjected to tension or compression

$$T_{dw} = f_y L_w t_e / \gamma_{mw}$$

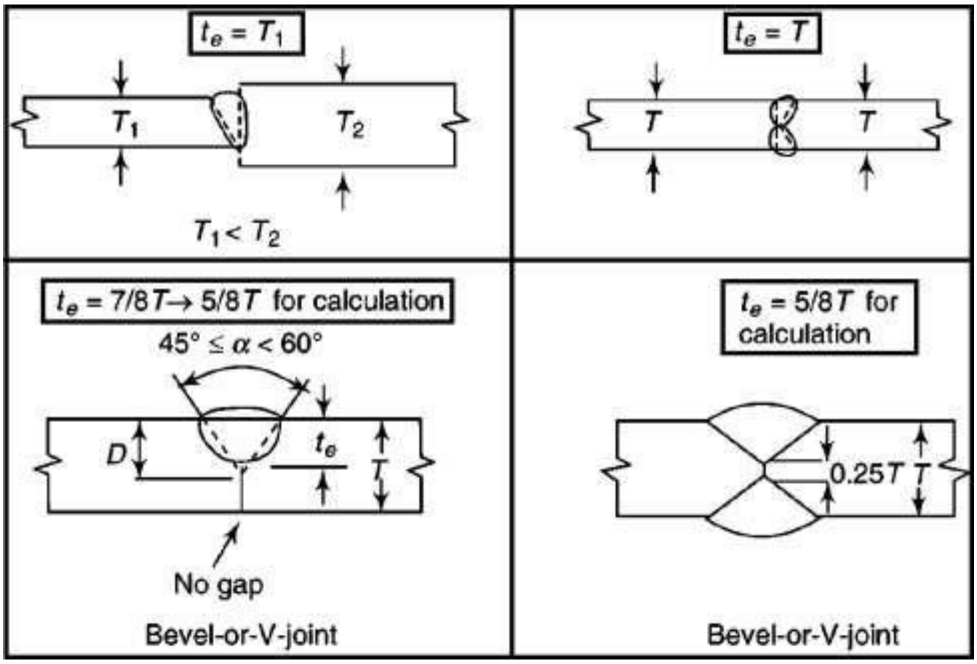
where T_{dw} is the design strength of the weld in tension, f_y is the smaller of yield stress of the weld and the parent metal in MPa, t_e is the effective throat thickness of the weld in mm, L_w is the effective length of the weld in mm, and γ_{mw} is the partial safety factor taken as 1.25 for shop welding and as 1.5 for site welding.

- Design strength of weld subjected to shear

$$V_{dw} = L_w t_e f_y / (\sqrt{3} \times \gamma_{mw})$$

where V_{dw} is the design strength of the weld in shear. Other quantities have been defined already.

- The effective throat thickness is computed from



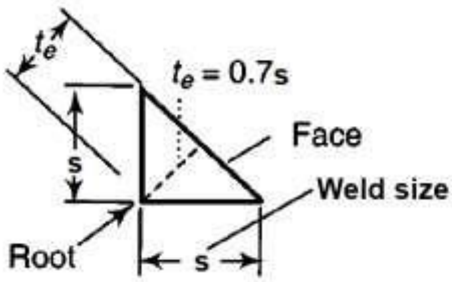
- The effective length of butt weld shall be taken as the length of the continuous full size weld, but **not less than four times the size of the weld.** (Cl. 10.5.4.2)

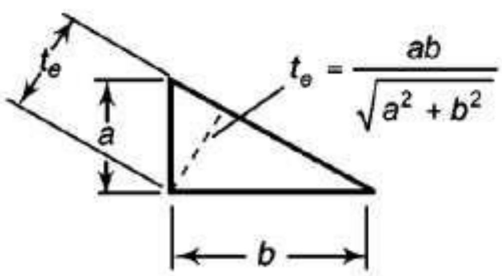
(b) Fillet welds

- Here, failure occurs by **rupture** of weld material. Hence, **ultimate strength (f_u)** is considered.
- Since fillet weld fails only in shear, the design strength is given by: (Cl. 10.5.7.1.1)

$$P_{dw} = L_w t_e f_u / (\sqrt{3}\gamma_{mw})$$

- The effective throat dimension of a fillet weld is the shortest distance from the root to the face of the weld. (shown below welds have fusion faces perpendicular to each other)





(Cl. 10.5.3.1)

- If the fusion faces are inclined to each other at some angle, then the effective throat thickness shall be taken as *K* times the fillet size (s), where *K is* a constant, depending upon the angle between fusion faces, as given in Table below (Cl. 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

- 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 below (Cl. 10.5.2.3)

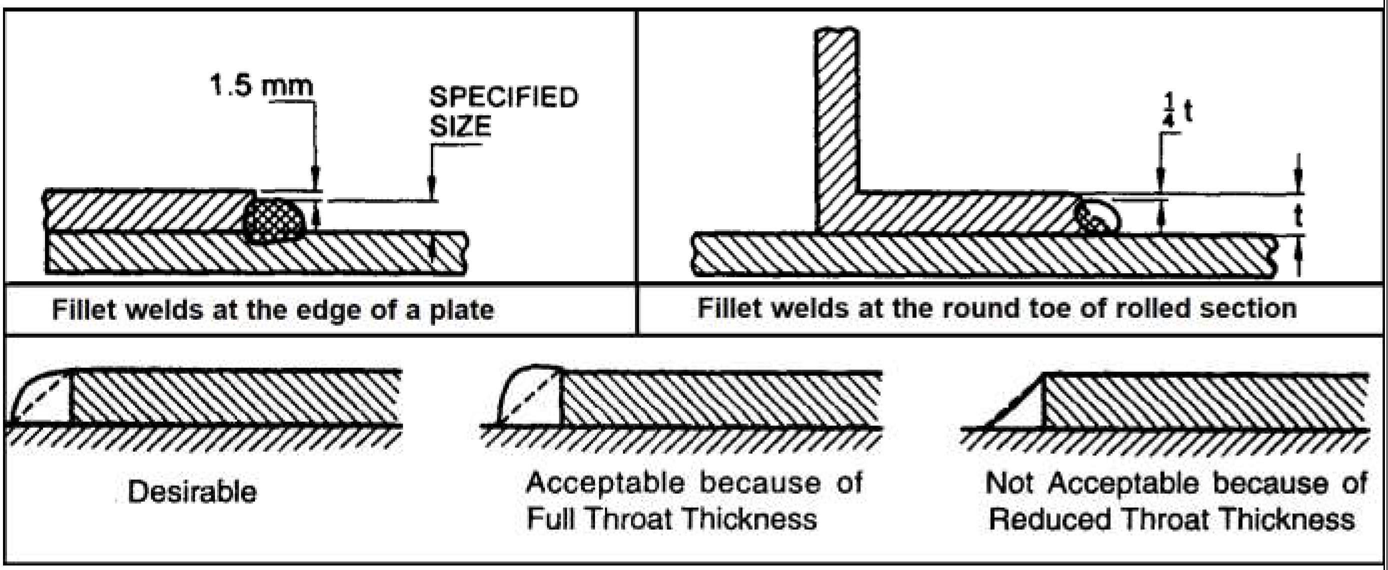
Sl No.	Thickness of Thicker Part mm		Minimum Size mm
	Over	Up to and Including	
(1)	(2)	(3)	(4)
i)	–	10	3
ii)	10	20	5
iii)	20	32	6
iv)	32	50	8 of first run 10 for minimum size of weld

NOTES

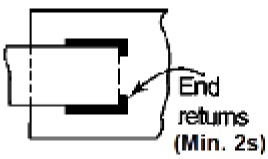
1 When the minimum size of the fillet weld given in the table is greater than the thickness of the thinner part, the minimum size of the weld should be equal to the thickness of the thinner part. The thicker part shall be adequately preheated to prevent cracking of the weld.

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

- In practice the actual length of weld is made of the effective length plus two times the weld size, but not less than four times the size of the weld. (Cl. 10.5.4.1)
- Fillet Weld Applied to the Edge of a Plate or Section should satisfy the following (Cl. 10.5.8)



- ***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. (Cl. 10.5.1.1)



1. Two plates of size 200 x 10 mm and 200 x 8 mm are connected by a weld groove having (i) Single – V groove weld joint, and (ii) Double – V groove weld joint. Determine the Maximum tension which the joints can resist. The steel plates are of grade Fe 410 grade with yield strength of 250 MPa. Assume shop welding.

SOLUTION:

Case (i) : Single – V groove weld
In this case, incomplete penetration results due to single – V groove
Single V is an incomplete penetration welding. Hence the throat thickness is $5/8^{th}$ of the thickness of thinner plate
$t_e = 5/8\ t = (5/8) \times 8 = 5$ mm
Effective length of weld L_W = width of plate = 200 mm
Strength of weld, $P = L \times t_t \times f_y / \Upsilon_{mw}$
= 200 x 5 x 200/1.25 = 200,000 N = 200 kN
Case (ii) : Double – V groove weld (Fig. 40)
In this case, complete penetration results due to Double – V groove
Effective throat thickness is 8mm which is the thickness of the thinner plate. t_e = 8 mm
Strength of weld, $P = L \times t_t \times f_y / \gamma_{mw}$
= 200 x 8 x 200/1.25 = 320,000 N = 320 kN

2. Find the size and length of the fillet weld for the lap joint to transmit a factored load of 120 kN as shown in Fig. 41. Assume site welds, Fe 410 grade steel and E41 electrode. Assume width of plate as 75 mm and thickness as 8 mm

Solution

Minimum size of weld for 8 mm thick section = 3 mm (Table 5, Cl. 10.5.2.3)
Maximum size of weld = 8 – 1.5 = 6.5 mm (Cl. 10.5.8.1)
Choose the size of weld, a = 6 mm
Effective throat thickness = t_e = 0.70 a = 4.2 mm
Strength of 6 mm weld / mm length = 4.2 x 410 / ($\sqrt{3}$ x 1.5)
Cl. 10.5.7.1.1
= 662.7 N/mm
assuming only two longitudinal welds along the sides
Required length of weld = 120 x 10^3/662.7 = 181 mm
Length to be provided on each side = 181/2 = 90.5 mm

> 75 mm (width of plate)

Hence, provide 90.5 mm weld on each side with an end return of 2x 6 = 12 mm Overall length of the weld provided = 2 x (90.5 + 2 x 6) = 205 mm

3. Two plates are connected to form a fillet joint using 6mm weld. Welding is provided on three sides with a lap of 300mm as shown in Fig.42. Find the strength of the joint. If welding is provided on all four sides (Fig. 44), determine the strength of the joint. Also find the percentage increase in the strength. Use Fe 410 steel with yield stress 250 MPa. Assume shop welding

Solution

Case (i) : Welding on three sides (Fig. 42)

Lw = 300 + 200 + 300 = 800 mm

Design strength of fillet weld joint, $P_1 = 0.7a \times f_{wd} \times L_w / \Upsilon_{mw}$

$= 0.7 \times 6 \times (410 / \sqrt{3}) \times$
$= 800 / 1.25 = 6, 36,286$ N

Hence, allowable load = 6, 36,286 / 1.5 = 4, 24,191 N

Case (ii): Welding on four sides

Lw = 300 + 200 + 300 + 200 = 1000 mm

Design strength of fillet weld joint, $\mathbf{P_2 = 0.7a\ f_{wd} \times L_W / \Upsilon_{MW}}$

$= 0.7 \times 6 \times (410 / \sqrt{3}) \times$
$1000 / 1.25 = 7, 95,358$ N

Hence, allowable load = 7,95,358 / 1.5 = 5,30,239 N

Percentage increase in strength = $(P_2 - P_1) / P_1 \times 100 = 25$ %

CHAPTER 3

DESIGN OF TENSION MEMBERS

INTRODUCTION

Tension members are **structural elements that are subjected to axial tensile forces**. Examples of tension members are bracing for buildings and bridges, truss members, and cables in suspended roof systems. Tension members are those subjected to direct axial tensile loads.

SLENDERNESS RATIO

Although, tension members donot buckle locally or overall, IS800 stipulates maximum slenderness ratio for tension members, inorder to ensure a minimum stiffness to prevent undesirable lateral movement & excessive vibrations. The ***slenderness ratio*** of a tension member is defined **as** the ratio of its unsupported length *(L)* to its least radius of gyration.

Member	Maximum effective slenderness ratio (L/r)
A tension member in which a reversal of direct stress occurs due to loads other than wind or seismic forces	180
A member subjected to compressive forces resulting only from a combination of wind/earthquake actions, provided the deformation of such a member does not adversely affect the stresses in any part of the structure	250
A member normally acting as a tie in a roof truss or a bracing member, which is not considered effective when subject to reversal of stress resulting from the action of wind or earthquake forces	350
Members always in tension (other than pre-tensioned members)	400

MODES OF FAILURE OF TENSION MEMBERS

(a) Gross section Yielding

- Athough a tension member without bolt holes can resist loads up to the ultimate load, it becomes unserviceable by undergoing large deformation (yielding).

- Hence, **yield strength of material f_y** is the deciding parameter here. Design strength is

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

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

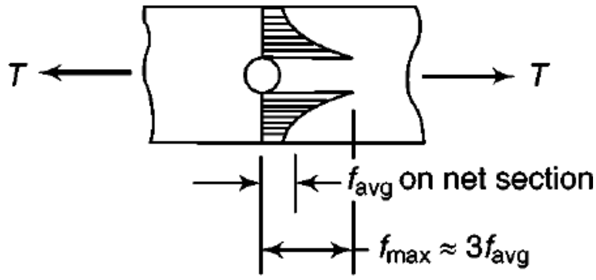
f_y = yield stress of the material,

A_g = gross area of cross-section, and

γ_{m0} = partial safety factor for failure in tension by yielding **= 1.10**

(b) Net section Rupture

- Holes in the members cause stress concentration at service loads, as shown.



- Since stress adjacent to the hole is much higher than that at the periphery, the material adjacent to the hole would have ruptured as the stress at the peripheral fibre reach yield stress. Hence, **ultimate strength of the material f_u** is the deciding parameter here.
- Design strength of **plates** due to net section rupture is given by: (Cl. 6.3 of IS800)

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

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

where

γ_{m1} = partial safety factor for failure at ultimate stress = 1.25

f_u = ultimate stress of the material, and

A_n = net effective area of the member given by,

$$A_n = (b - nd_h)t \quad \text{(for non-staggered holes)}$$

$$A_n = \left[b - nd_h + \sum_{i=1}^{m} p_{si}^2/(4g_i) \right] t \quad \text{(for staggered holes)}$$

b, t = width and thickness of the plate

d_h = diameter of the bolt hole

g = gauge length between the bolt holes

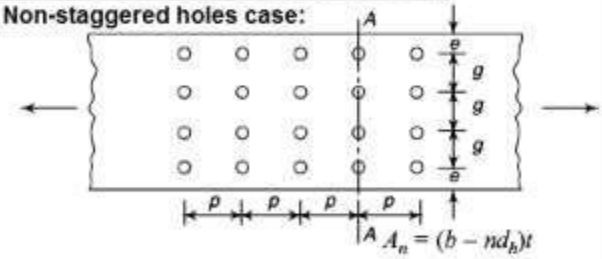
p_s = staggered pitch length

n = number of bolt holes in the critical section

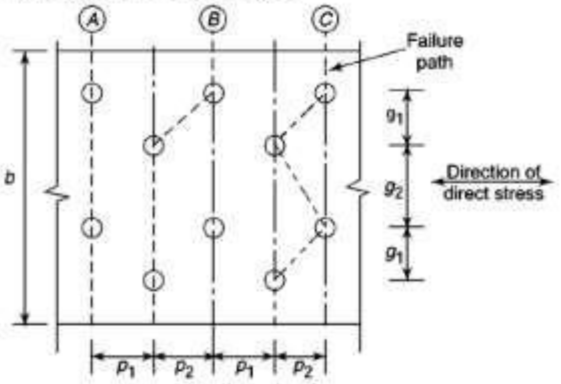
i = subscript for summation of all the inclined legs.

EXAMPLES OF COMPUTING NET AREA

(a) Non-staggered holes case:



(b) Staggered holes case



On line A:

$A_n = t[b - 2d_h]$

On line B:

$A_n = t[b - 3d_h + 0.25p_2^2/g_1]$

On line C:

$A_n = t[b - 4d_h + 0.5p_2^2/g_1 + 0.25p_2^2/g_2]$

- For **single angle sections** connected through one leg
    - Design strength due to net section rupture is given by:

6.3.3 ***Single Angles***

The rupture strength of an angle connected through one leg is affected by shear lag. The design strength, $T_{dn,}$ as governed by rupture at net section is given by:

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

(for connected leg) **(for outstanding leg)**

where

β = $1.4 - 0.076\,(w/t)\,(f_y/f_u)\,(b_s/L_c) \leq (f_u \gamma_{m0} / f_y \gamma_{m1})$ ≥ 0.7

w = outstand leg width,

L_c = length of the end connection, that is the distance between the outermost bolts in the end joint measured along the load direction or length of the weld along the load direction.

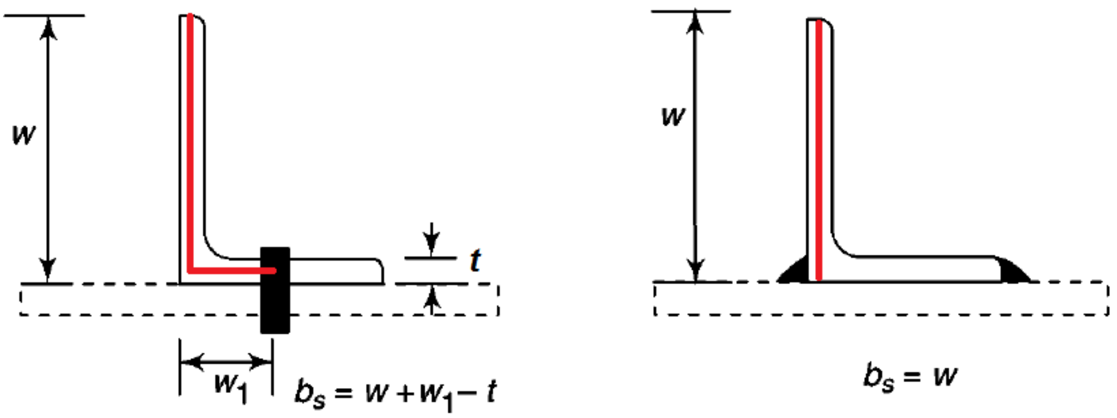
A_n = net area of the total cross-section

A_{nc} = net area of the connected leg

A_{go} = gross area of the outstanding leg

t = thickness of the leg

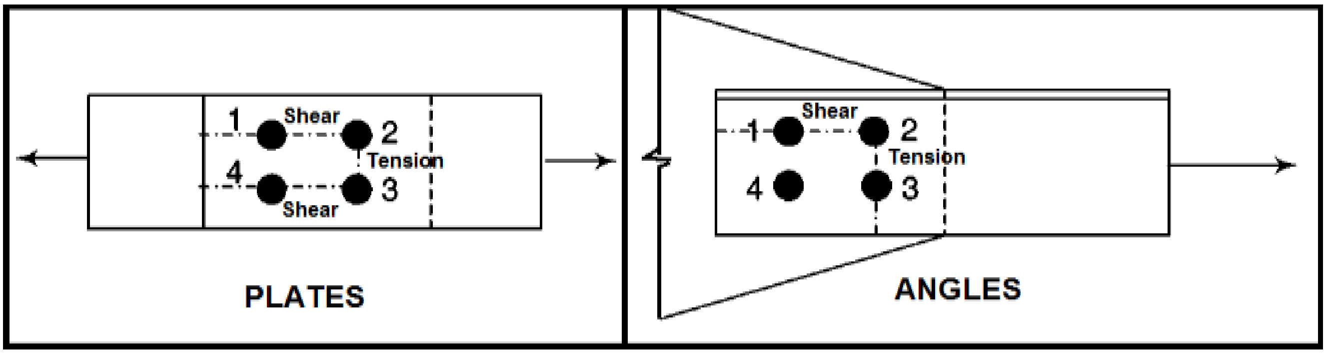
    - Here, b_s is the shear lag width, i.e., the shear distance from the edge of the outstanding leg to the nearest line of fasteners, measured along the centre line of the legs in the cross section.



(a) Bolted connection (b) Welded connection

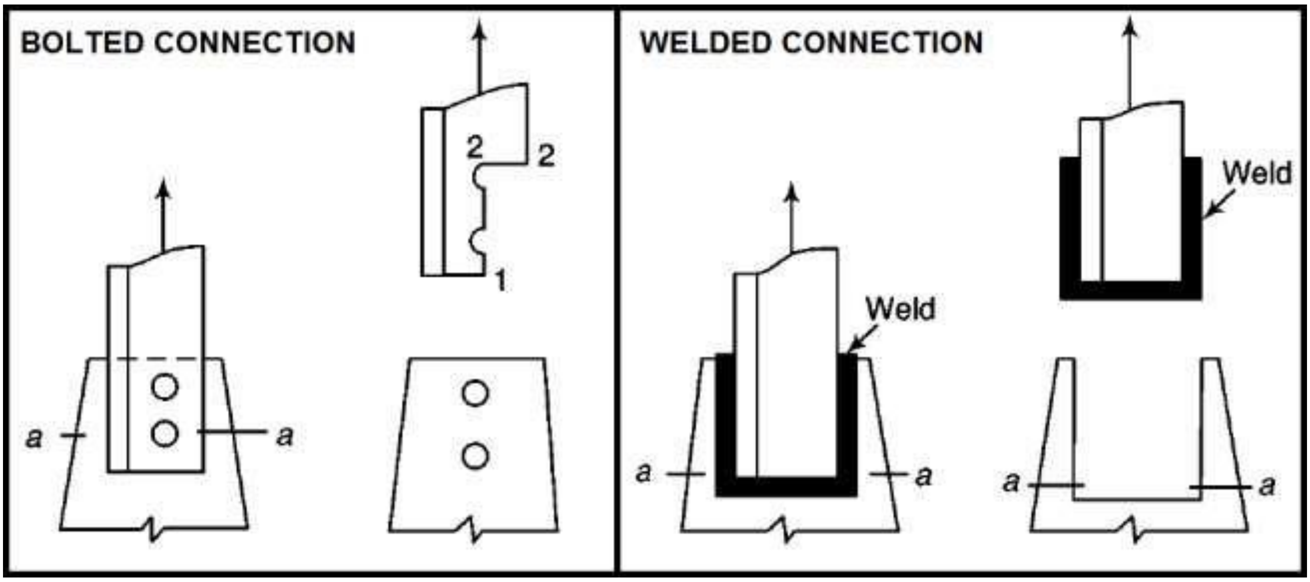
(c) Block Shear Failure

- In this failure mode, the failure of the member occurs along a path involving tension on one plane and shear on a perpendicular plane along the fasteners as shown.

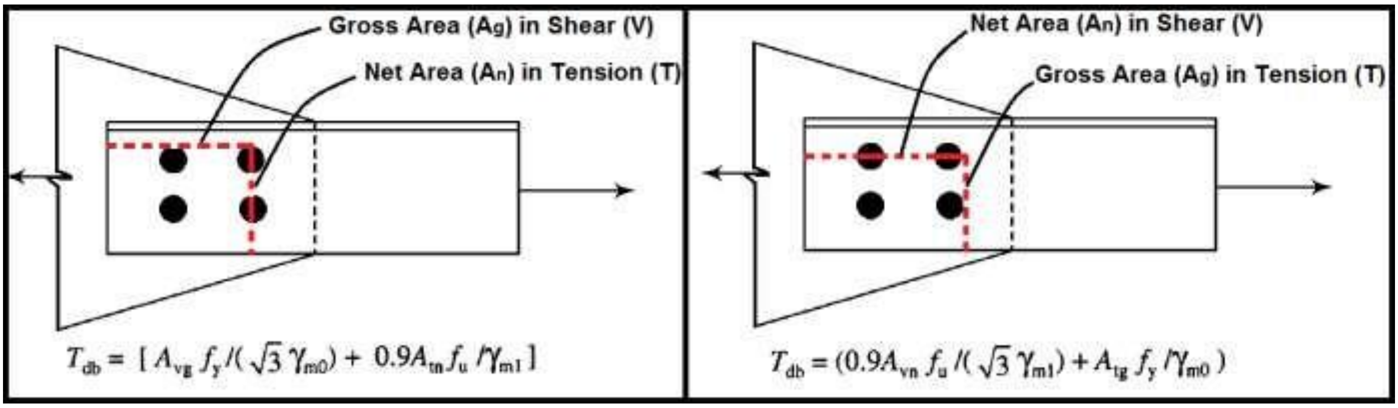


- Block shear Failure occurs under the following circumstances:
    - When the bearing strength of plate as well as the shear strength of bolt are higher, only fewer bolts are required in connection. Thus, smaller connection length may result in Block shear failure.
    - It may also occur when bolts re closely spaced.

- Failure possibilities for different connections is shown below



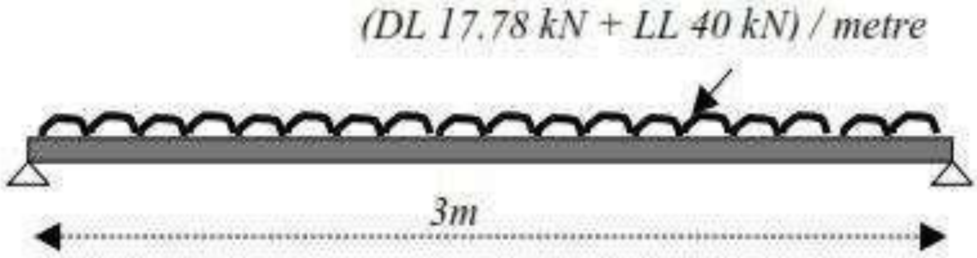
- The **design block shear strength (T_{db})** depends on the following probable failure mechanisms in block shear failure in bolted connections. (Cl. 6.4 of IS800)



where $\boldsymbol{A_{vg}}$ and $\boldsymbol{A_{vn}}$ are the minimum gross and net area in shear along a line of transmitted force, respectively; $\boldsymbol{A_{tg}}$ and $\boldsymbol{A_{tn}}$ are the minimum gross and net area in tension from the hole to the toe of the angle or next last row of bolt in plates, perpendicular to the line of force respectively.

- In welded connections, since no net-areas are involved in block shear failures, the above formula for T_{db} is modified by replacing A_{vn} by A_{vg} and A_{tn} by A_{tg}.

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



Design load calculation:

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

in this example the following load factors are chosen.

γ_{LD} *and* γ_{LL} *are taken as 1.50 and 1.50 respectively.*

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

Total factored load $= 1.50 \times 17.78 + 1.50 \times 40.0 = 86.67$ kN / m

Factored bending moment $= 86.67 \times 3^2 / 8 = 97.504$ kN – m

Z—value required for f_y=250 MPa ; γ_m =1.10

$$Z_{reqd} = \frac{97.5 \times 1000 \times 1000 \times \gamma_m}{250}$$

$Z_{reqd} = 429.02 \text{ cm}^3$

Try ISMB 250

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

$B = 125\ mm$

$t = 6.9\ mm$

$T = 12.5\ mm$

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

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

Section classification:

Flange criterion $= B/2T \qquad = 5.0$

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

Since $B/2T < 9.4\ \varepsilon$ & $(D\text{-}2T)/t < 83.9\ \varepsilon$

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

Moment of resistance of the cross section:

Since the section considered is 'PLASTIC'

$$M_d = \frac{Z_p \times f_y}{\gamma_m}$$

Where Z_p *is the plastic modulus*

'Z_p' *for ISMB 250* $= 459.76\ cm^3$

$M_d = 459.76 \times 1000 \times 250 / 1.10$

$= 104.49\ kN\text{-}m > 97.504\ kN\text{-}m$

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

Shear resistance of the cross section:

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

Shear capacity $V_{c} = \dfrac{0.6\, f_y A_v}{\gamma_m}$

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

$V_c = 0.6 \times 250 \times 1725 / 1.10 = 235.3 kN$

V = factored max shear $= 86.67 \times 3 / 2 = 130.0\ kN$

$V / V_c = 130/235.3 = 0.55 < 0.6$

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

Check for Web Buckling:

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

$= 70.33$

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

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

Stiff bearing length $= 100\ mm$

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

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

$= 315.16\ kN$

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

Check for web crippling at support

Root radius of ISMB 250 = 13 mm

Thickness of flange + root radius = 25.5 mm

Dispersion length (1:2.5) n_2 = 2.5 x 25.5 = 63.75 mm

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

$= 245.63\ kN > 126 kN$

Hence ISMB 250 has adequate web crippling resistance

Check for serviceability – Deflection:

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

design load = 57.78 kN/m.

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

$$= 5.65\ mm$$

Max deflection $= \dfrac{L}{531}$

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

Hence serviceability is satisfied

Result: -- Use ISMB – 250.

CHAPTER 4

DESIGN OF COMPRESSION MEMBERS

INTRODUCTION

Compression members are commonly used as **columns in building structures**, chords or webs in trusses, bridge piers or braces in framed structures. The maximum strength of a steel compression member depends, to a large extent, on the member length and end support conditions. Compression members are those subjected to compressive forces along its axis.

TYPES

- **Axially Loaded Columns:** subjected to true axial loads and no bending moments along its length.
- **Beam-columns:** subjected to both axial loads and bending moments along its length.

TERMINOLOGIES

- **Columns / Stanchions:** They support floors in the building and carry very heavy loads.
- **Struts:** Short compression members, used in roof trusses & bracings in buildings. Top chord member of roof truss is also a compression member known as Principal Rafter.
- **Boom:** Principal compression member in a crane

NOTE: Generally, columns are connected to adjacent members by bolts. It is assumed that the bolts replace the material removed for bolt holes. Hence, bolt holes are generally ignored in the design of compression members.

POSSIBLE FAILURE MODES

The possible failure modes of an axially loaded column are discussed as follows:

1) ***Local Buckling:*** Failure occurs by buckling of one or more individual plate elements. This failure mode may be prevented by selecting suitable width-to-thickness ratios of component plates.

2) ***Squashing:*** When the length is relatively small (stocky column) and its component plate elements are prevented from local buckling, then the column will be able to attain its full strength or 'squash load' (yield stress x area of cross section).

3) *Overall Flexural Buckling:* This mode of failure normally controls the design of most compression members. In this mode, failure of the member occurs by excessive deflection

CLASSIFICATION OF COMPRESSION MEMBERS

- ***Short compression members*** – No buckling occurs. This member fails by squashing (yielding). Failure stress will be equal to yield stress.
- ***Long compression members*** (or Slender Columns) – Fails by elastic buckling. Its strength is predicted with the help of Euler's Formula.
- ***Intermediate length compression members*** - Fails by both yielding and buckling.

BUCKLING CURVES & DESIGN STRENGTH

- **Euler's Buckling load** or crippling load P_{cr} is given by:

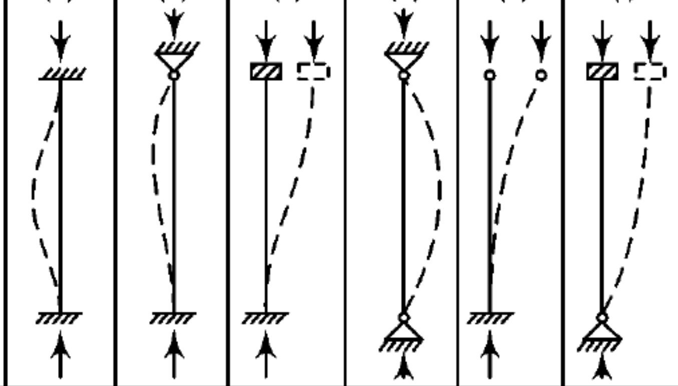
$$P_{cr} = \pi^2 EI/L_e^2$$

Where I = Least Moment of Inertia for the given column section

L_e = Effective length of column = K L

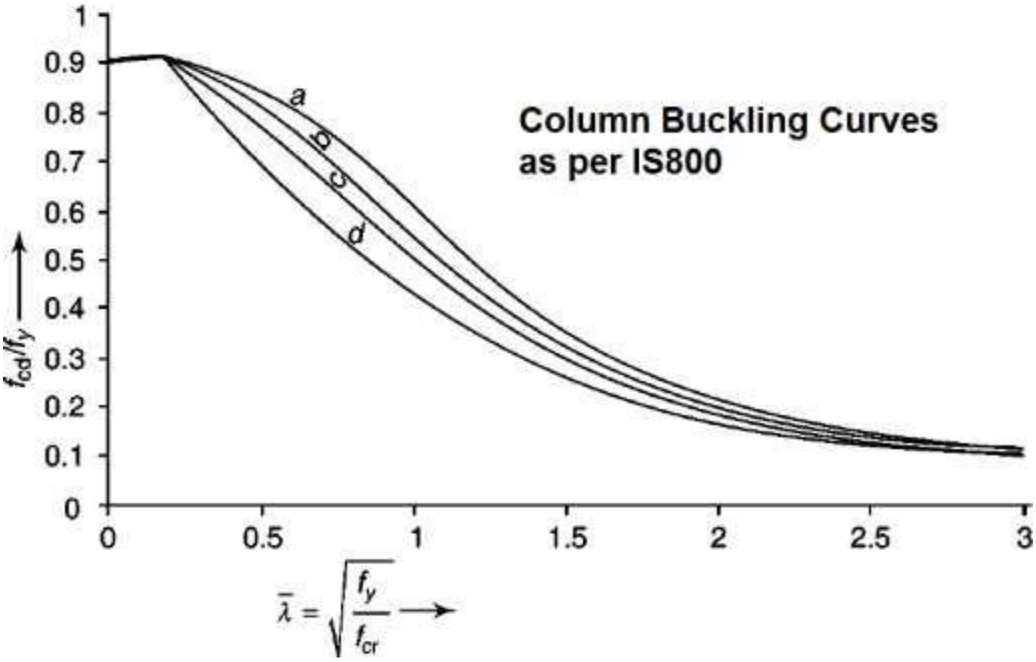
L = Actual length of column

Value of K for different support conditions is given below.

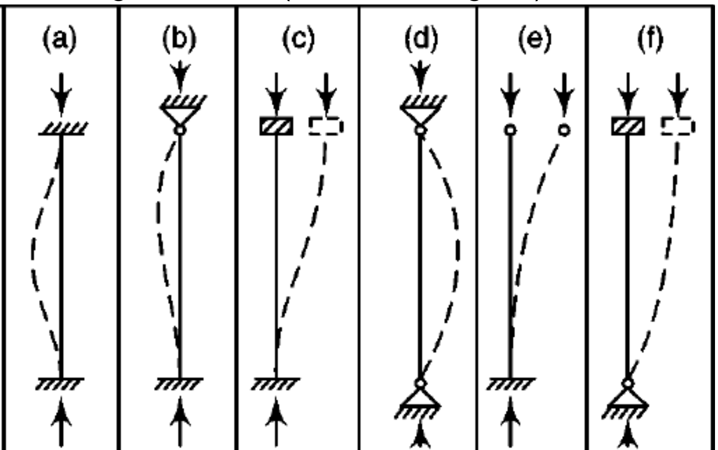


	(a)	(b)	(c)	(d)	(e)	(f)
Buckled shape of column is shown by dashed line						
Theoretical *K* value	0.5	0.7	1.0	1.0	2.0	2.0

- **Behaviour of Real Compression Members:** The experimental results diverged substantially from the solutions of Euler's approach. This is because, the effects of initial crookedness of the member, accidental eccentricity of load, end restraint, local or lateral buckling, residual stress, etc were not considered for Euler's approach and is hence ignored for design.
- In order to represent the real strength of columns, Indian Standard Code (IS 800) has adopted the **multiple column curves** (shown below), depending on different cross sections of columns. This is based on the **Perry-Robertson approach**. Based on this, the columns are categorized as **a, b, c** and **d**, depending on their buckling behaviour.

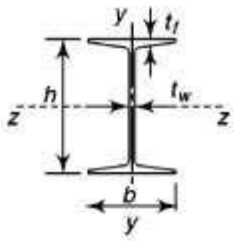
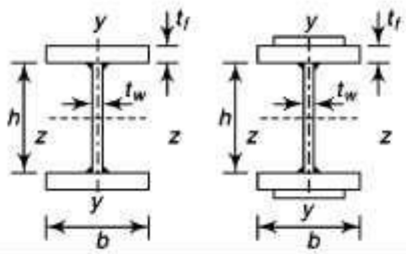
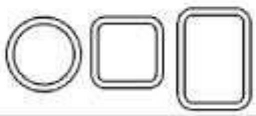
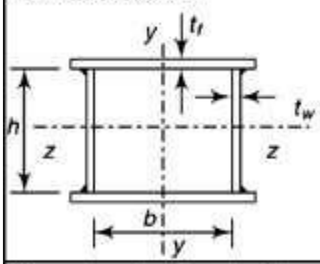
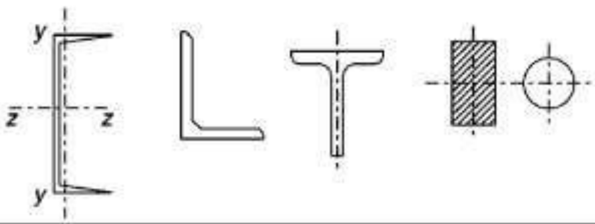
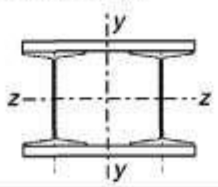


- The effective length of column *KL, is* calculated from the actual length *L,* of the member, considering the rotational and relative translational boundary conditions at the ends. The value of K, as recommended by IS 800 is given below (Table 11- Page45).

	(a)	(b)	(c)	(d)	(e)	(f)
Buckled shape of column is shown by dashed line						
Value of K recommended by IS 800-2007	0.65	0.80	1.2	1.0	2.0	2.0
End condition code		Rotation fixed, translation fixed Rotation free, translation fixed Rotation fixed, translation free Rotation free, translation free				

- The actual length shall be taken as the length from centre-to-centre of its intersections with the supporting members in the plane of the buckling deformation. In the case of a member with a free end, the free standing length from the center of the intersecting member at the supported end, shall be taken as the actual length.

- **Classification of steel sections based on buckling curves** (Table 10 - Pg 44)

Cross section	Limits		Buckling about axis	Buckling curve
Rolled I-sections 	$h/b > 1.2$:	$t_f \le 40$ mm	z-z	*a*
			y-y	*b*
	40 mm $< t_f \le$ 100 mm		z-z	*b*
			y-y	*c*
	$h/b \le 1.2$:	$t_f \le 100$ mm	z-z	*b*
			y-y	*c*
		$t_f > 100$ mm	z-z	*d*
			y-y	*d*
Welded I-section 		$t_f \le 40$ mm	z-z	*b*
			y-y	*c*
		$t_f > 40$ mm	z-z	*c*
			y-y	*d*
Hollow section 	Hot rolled	Any		*a*
	Cold formed	Any		*b*
Welded box section 	Generally (except as below)		Any	*b*
	Thick welds and	$b/t_f < 30$	z-z	*c*
		$h/t_w < 30$	y-y	*c*
Channel, angle, T- and solid sections 			Any	*c*
Built-up member 			Any	*c*

- **Design compressive strength of column** is computed from Section 7 (Pg 34 of IS 800)

7.1.2 The design compressive strength P_d, of a member is given by: $\boxed{P_d = A_e \times f_{cd}}$

where A_e = effective sectional area =Gross-sectional area of the compression member

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

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

where

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

α = imperfection factor **(Table 7 - Pg35)**

Buckling Class	a	b	c	d
α	0.21	0.34	0.49	0.76

λ = non-dimensional effective slenderness ratio

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

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

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

χ = stress reduction factor

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

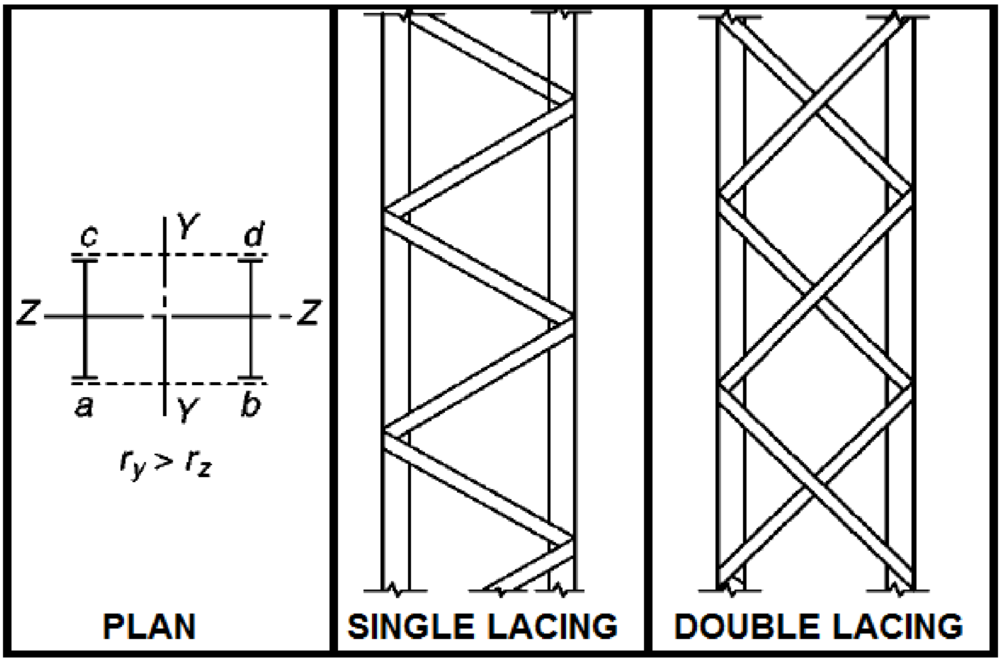
γ_{m0} = partial safety factor for material strength = **1.10**

BUILT-UP COMPRESSION MEMBERS

They are generally made up of two or more individual sections such as angles, channels, or I-sections and properly connected along their length by lacing or battening so that they act together as a single unit. They are used generally for large loads and for efficient use of material. They are also known as *combined columns* or *open-web columns.* Design for shear is the most critical for them. *Eg:* Laced columns, Battened columns, etc.

(1) **Laced Columns** (Cl. 7.6 – Page 48)

The inclined members used to brace the built-up compression members are called lacings.



(a) The radius of gyration of the combined column about the axis perpendicular to the plane of lacing should be greater than the radius of gyration about the axis parallel to the plane of lacing.

(b) Lacing system should be uniform throughout the length of the column.

(c) Single and double laced systems should not be provided on the opposite sides of the same member. Similarly lacings and battens should not be provided on opposite sides of the same member.

(d) Single laced system on opposite sides of the main component shall be in the same direction viewed from either side so that one is the shadow of the other.

(e) The lacing shall be designed to resist a **total transverse shear V**, at any point in the member, equal to 2.5% of the axial force in the member; and this shear shall be divided among the lacing systems in parallel planes.

(f) The lacings in addition should be designed to resist any shear due to bending moment or lateral load on the member.

(g) The **slenderness ratio** of lacing shall not exceed 145.

(h) The **effective length** shall be taken as the length between inner end bolts/rivets of the bar for single lacings and 0.7 times the length for double lacings effectively connected at intersections. For welded bars, the effective length is taken as 0.7 times the distance between the inner ends of the welds connecting the single bars to the members.

(i) The **minimum width** of the lacing bar shall not be less than approximately three times the diameter of the connecting bolt/rivet; the thickness shall not be less than 1/40th of the effective length for single lacing and 1/60th for double lacing.

(j) The **spacing** of lacing bars shall be such that the maximum slenderness ratio of the components of the main member between two consecutive lacing connections is not greater than 50 or 0.7 times the most unfavourable slenderness ratio of the combined column.

(k) When welded lacing bars overlap the main members, the amount of lap should be not less than four times the thickness of the bar and the welding is to be provided along each side of the bar for the full length of lap.

(l) Plates shall be provided at the ends of laced compression members and shall be designed as battens.

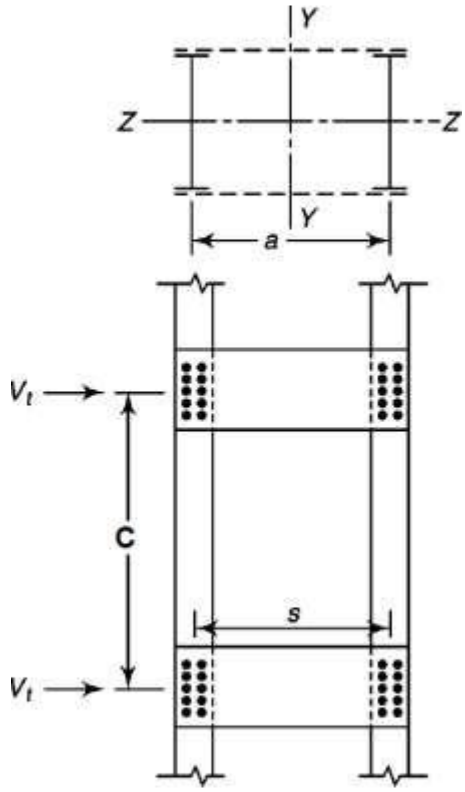
(m) Flats, angles (normally adopted in practice), channels, or tubes may be used as lacings.

(n) Lacing bars, whether in double or single shear shall be inclined at an angle of 40^o to 70^o to the axis of the built-up member.

(o) The **effective slenderness ratio** $(KL/r)_e$, of the laced column shall be taken as 1.05 times $(KL/r)_o$,where $(KL/r)_o$ is the maximum actual slenderness ratio of the column, to account for shear deformation effects.

2) **<u>Battened Columns</u>** (Cl. 7.7 – Page 50)

Horizontal members used to brace the built-up compression members are called battens.



a) It should to be noted that the battened columns have the less resistance to shear compared to laced columns, and may experience an appreciable reduction in strength. Hence, laced columns are preferred over battened columns.
b) The **number of battens** shall be such that the member is divided into not less than three bays.
c) Battens shall be **designed** to resist simultaneously the following:

$$\text{Longitudinal shear } V, = \boldsymbol{V_t\, C/NS}$$

and

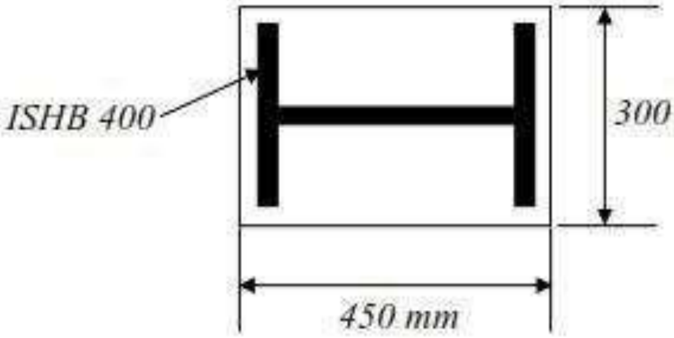
$$\text{Moment } M = \boldsymbol{V_t\, C/2N}$$

where V_t is the transverse shear force, C is the distance between centre-to-centre of battens, longitudinally, N is the number of parallel planes of battens, and S is the minimum transverse distance between the centroids of the bolt/rivet group/welding connecting the batten to the main member.

d) When plates are used for battens, the **effective depth** between the end bolts/ rivets or welds shall not be less than twice the width of one member in the plane of battens; nor less than three quarters of the perpendicular distance between centroids of the main members for intermediate battens; and not less than the perpendicular distance between the centroids of main members for end battens.
e) The **thickness** of batten plates shall not be less than 1/50th of the distance between the innermost connecting transverse bolts, rivets or welds.
f) When connected to main members by welds, the length of the weld connecting each end of the batten shall not be less than half the depth of the plate; at least one third of its length should be placed at each end of the edge; in addition the weld shall be returned along the other two edges for a length not less than the minimum lap (i.e., not less than four times the thickness of the plate). The length of the weld and depth of batten should be measured along the longitudinal axis of the main member.
g) The **effective slenderness ratio** of battened column shall be taken as 1.10 times $(KL/r)_o$, here $(KL/r)_o$ is the maximum actual slenderness ratio of the column, to account for shear deformation effects.

Design a simple base plate for a ISHB400 @ 0.822 kN/m column to carry a factored load of 1800 kN.

[$f_{cu} = 40\ N/mm^2$; $f_y = 250\ N/mm^2$; $\gamma_m = 1.10$]



Thickness of Flange for ISHB400 $= T = 12.7\ mm$

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

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

Use plate of 450 X 300 mm (135000 mm^2)

Assuming projection of 25 mm on each side, $a = b = 25\ mm$

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

Now thickness of Slab Base, t_s

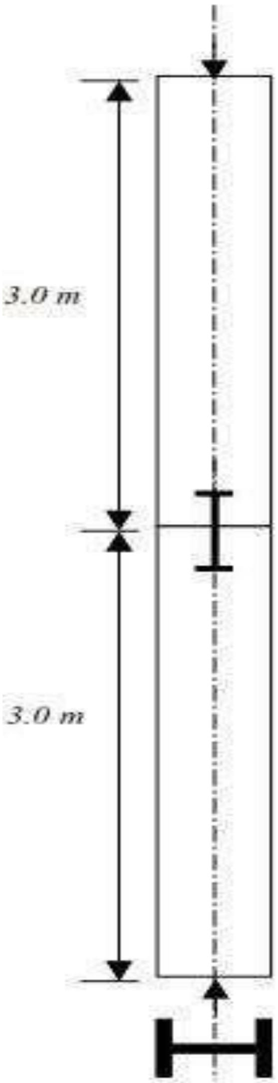
$$\boldsymbol{t_s = \sqrt{2.5\,w\,(a^2 - 0.3b^2)\gamma_{m0} / f_y} \quad > \; T}$$

$$= \sqrt{\frac{2.5w\left(a^2 - 0.3b^2\right) \times 1.10}{f_y}} = \sqrt{\frac{2.5 \times 13.33 \times \left(25^2 - 0.3 \times 25^2\right) \times 1.10}{250}} = 8.01\ mm$$

$< T = 12.7$ mm, Hence provide a base plate of thickness not less than 12.7 mm and since the available next higher thickness of plate is 16 mm

Use 450 X 300 X 16 mm plate.

Obtain maximum axial load carried by the column shown when ISHB 400 is employed. The column is effectively restrained at mid-height in the y-direction, but is free in x-axis. The data is the same as in problem1.
[$f_y = 250$ N/mm^2 ; $E = 2.0 \times 10^5$ N/mm^2 ; $\gamma_m = 1.10$]



(i) Type of section:

Section is "COMPACT" from previous example.

(ii) Effective lengths:

As, both ends are pin-jointed effective length $KL_x = 1.0 \times 6000 = 6000$ mm
$KL_y = 1.0 \times 3000 = 3000$ mm

(iii) Slenderness ratios:

$$KL_x / r_x = \frac{6000}{166.1} = 36.12$$

$$KL_y / r_y = \frac{3000}{51.6} = 58.1$$

(iv) Non-dimensional Effective Slenderness ratio, λ :

As calculated in previous example,
$\lambda = 0.654$ *(for worst slenderness ratio)*

(v) Value of ϕ:

As calculated in previous example,
$\phi = 0.791$ *(for $\lambda = 0.654$)*

(vi) Value of χ:
As calculated in previous example,
$\chi = 0.809$ *(for $\phi = 0.791$)*

(vii) Calculation of f_{cd}:

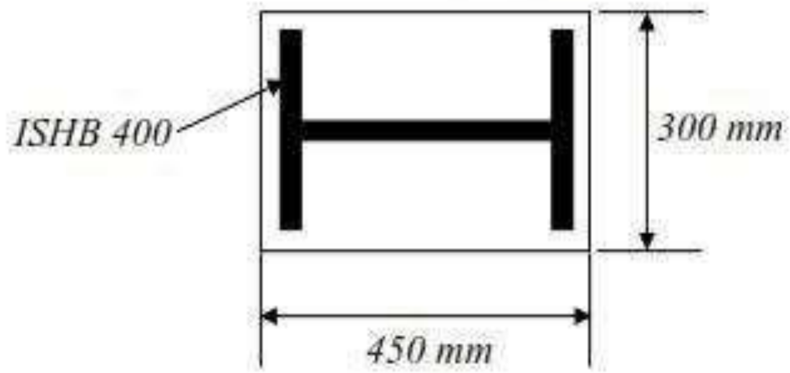
As calculated in previous example,
$f_{cd} = 183.86$ N/mm^2

(viii) Calculation of Factored Load:

Factored Load = $f_{cd} \times A_e/\gamma_m$ = 183.86/1.10 x10466/1000= 1924.28 kN

Design a simple base plate for a ISHB400 @ 0.822 kN/m column to carry a factored load of 1800 kN.

[f_{cu} = 40 N/mm^2 ; f_y = 250 N/mm^2 ; γ_m = 1.10]



Thickness of Flange for ISHB400 = T = 12.7 mm

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

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

Use plate of 450 X 300 mm (135000 mm^2)

Assuming projection of 25 mm on each side, a = b = 25 mm

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

Now thickness of Slab Base, t_s

$$\mathbf{t_s = \sqrt{2.5\,w\,(a^2 - 0.3b^2)\gamma_{m0}\,/\,f_y} \quad > \ T}$$

$$= \sqrt{\frac{2.5w\left(a^2 - 0.3b^2\right)\times 1.10}{f_y}} = \sqrt{\frac{2.5\times 13.33\times\left(25^2 - 0.3\times 25^2\right)\times 1.10}{250}} = 8.01\ mm$$

< T = 12.7 mm, Hence provide a base plate of thickness not less than 12.7 mm and since the available next higher thickness of plate is 16 mm

Use 450 X 300 X 16 mm plate.

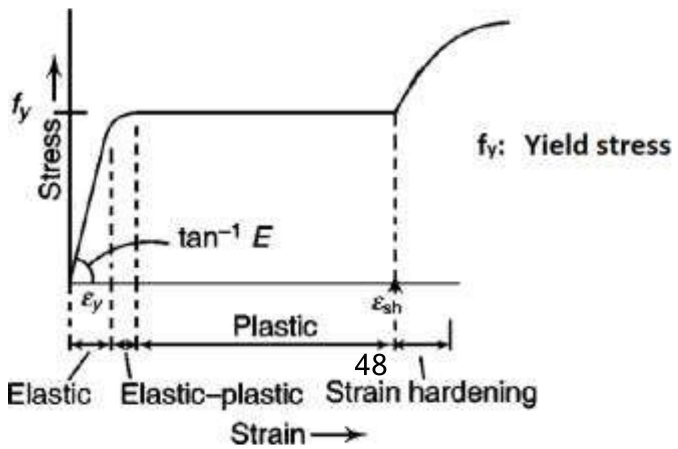
CHAPTER 6

DESIGN OF STEEL BEAMS

INTODUCTION

The following points should be considered in the design of a beam.

- Bending moment consideration: The section of the beam must be able to resist the maximum bending moment to which it is subjected.
- Shear force consideration: The section of the beam must be able to resist the maximum shear force to which it is subjected.
- Deflection consideration: The maximum deflection of a loaded beam should be within a certain limit so that the strength and efficiency of the beam should not be affected. Limiting the deflection within a safe limit will also prevent any possible damage to finishing. As per the I.S. code, generally the maximum deflection should not exceed 1/325 of the span.
- Bearing stress consideration: The beam should have enough bearing area at the supports to avoid excessive bearing stress which may lead to crushing of the beam or the support itself.
- Buckling consideration: The compression flange should be prevented from buckling. Similarly the web, the beam should also be prevented from crippling. Usually these failures do not take place under normal loading due to proportioning of thickness of flange and web. But under considerably heavy loads, such failures are possible and hence in such cases the member must be designed to remain safe against such failures
- The idealized stress - strain behavior of steel is given by:



- In working stress method, the stress in the material is restricted well within the Yield stress (f_y), such that the stress-strain relationship is linear and hence simpler for analysis. This region of graph is ideally termed as Elastic Region and hence the analysis is called **Elastic Analysis**.
    - Here, since the strength of the material beyond the yield point is not utilized, the structures designed using this method is generally heavier, and hence less economical.
- Hence, the method of **Plastic Analysis** is introduced here.
    - This utilizes the strength of the material beyond yield point. The material behavior upto strain-hardening strain (ε_{sh}) is considered here (see above figure).
    - Instead of yield stress, this method uses ultimate load as the design criterion. Hence this method is also known as *Ultimate Load method* or *Load Factor Design method*.
    - This is relatively a simpler method for the analysis of Statically Indeterminate Method.

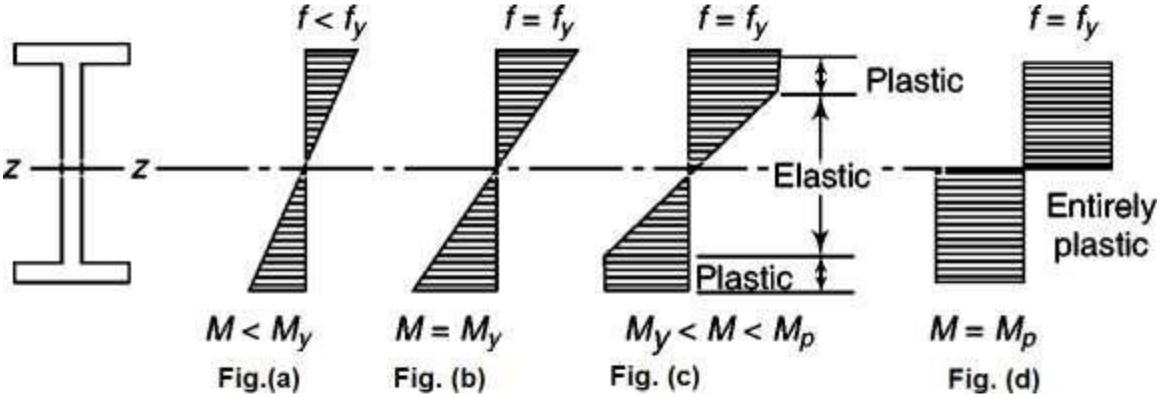
PLASTIC THEORY

Consider an I-beam subjected to a steadily increasing bending moment.

- In the service load range, the section is elastic as shown in Fig(a).
- The elastic condition exists until the stress at the extreme fibre reaches the yield stress [Fig(b)]. When the yield stress reaches the extreme fibre, the nominal moment strength of the beam is referred to as the ***yield moment M_y*** and is given by

$$\boxed{M_y = Z_e \times f_y}$$

where, Z_e is the elastic section modulus = $\frac{I}{y_{max}}$, I is the moment of inertia of the section and y_{max} is the maximum distance between extreme fibre and the neutral axis.



- A further increase in the bending moment causes the yield to spread inwards from the lower and upper surfaces of the beam as shown in Fig(c). This is because of the yielding of the outer fibres without increase in stresses, as shown by the horizontal line of the idealized stress-strain diagram (see above fig).
- Upon increasing the bending moment further, the whole section yields as shown in Fig.(d). When this condition is reached, every fibre has a strain equal to or greater than $\varepsilon_y = f_y / E_s$. The nominal moment strength of the beam at this stage is referred to as the ***plastic moment*** $\mathbf{M_p}$ and is given by

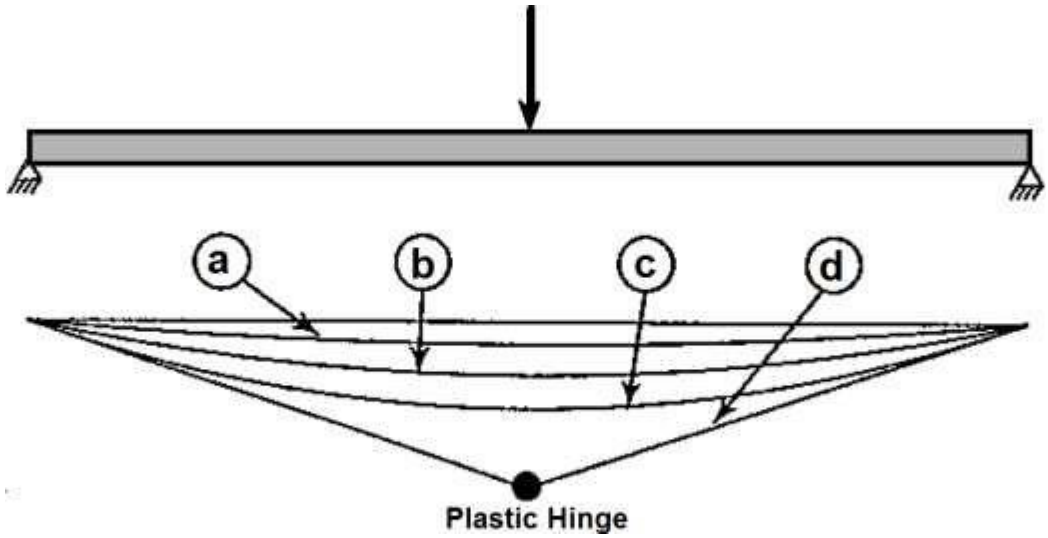
$$\boxed{M_p \quad = \quad Z_p \times f_y}$$

where, Z_p is the plastic section modulus = % & '

- Any further increase in the bending moment results only in rotation, since no greater resisting moment than the fully plastic moment can be developed until strain hardening occurs.
- In the simple plastic theory, it is conservatively assumed that **when the maximum moment in the beam reaches Plastic Moment *M_p*, the curvature increases indefinitely without increase in moment, i.e., neglecting the effects of strain hardening**.

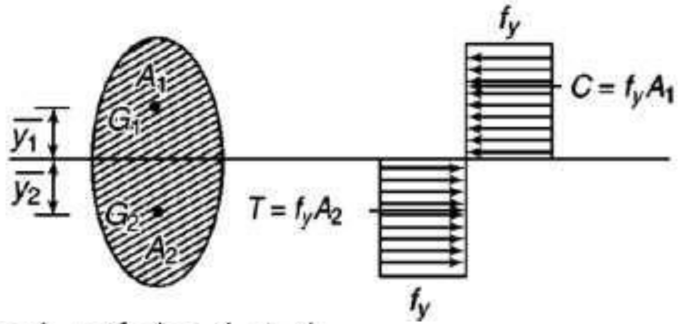
- The portion of the member where ***M_p*** occurs is termed **"plastic hinge"**
  - Plastic hinge is defined as a yielded zone, which can cause an infinite rotation to take place at a constant plastic moment ***M_p*** of the section.
  - Plastic hinges form in a member at the maximum bending moment locations.
  - However, at the intersections of two members, where the bending moment is the same, a hinge forms in the weaker member.
  - Generally, hinges are located at restrained ends, intersections of members, and point loads.
  - Generally the number of plastic hinges are ***n* = *r* + 1,** where ***r*** is the degree of static indeterminacy.

- The deflection curve of the beam at different stages of loading and the consequent formation of plastic hinge is shown below.



- **To determine the Plastic section modulus Z_p**, consider the following:
  - Consider a beam cross-section of area A, as shown in figure given below. At equilibrium, compressive force C = Tensile force T

    => $f_y A_1 = f_y A_2$ => $A_1 = A_2$



Since $A_1 = A_2$ and $A = A_1 + A_2$,

$$A_1 = A_2 = A/2$$

Plastic moment of resistance

$$M_p = f_y A_1 \, \overline{y}_1 + f_y A_2 \, \overline{y}_2$$

$$= f_y A/2 \, (\overline{y}_1 + \overline{y}_2)$$

Since $M_p = f_y Z_p$

=> $\boxed{Z_p = (A/2)\ (\overline{y}_1 + \overline{y}_2)}$ is the plastic modulus of the section.

  - Thus, the *plastic modulus* of the section may be defined as the combined static moment of the cross-sectional area above and below the equal-area axis. It is also referred to as the ***resisting modulus of the completely plasticized section***.

SHAPE FACTOR

Shape Factor (υ) is the ratio of plastic moment to yield moment.

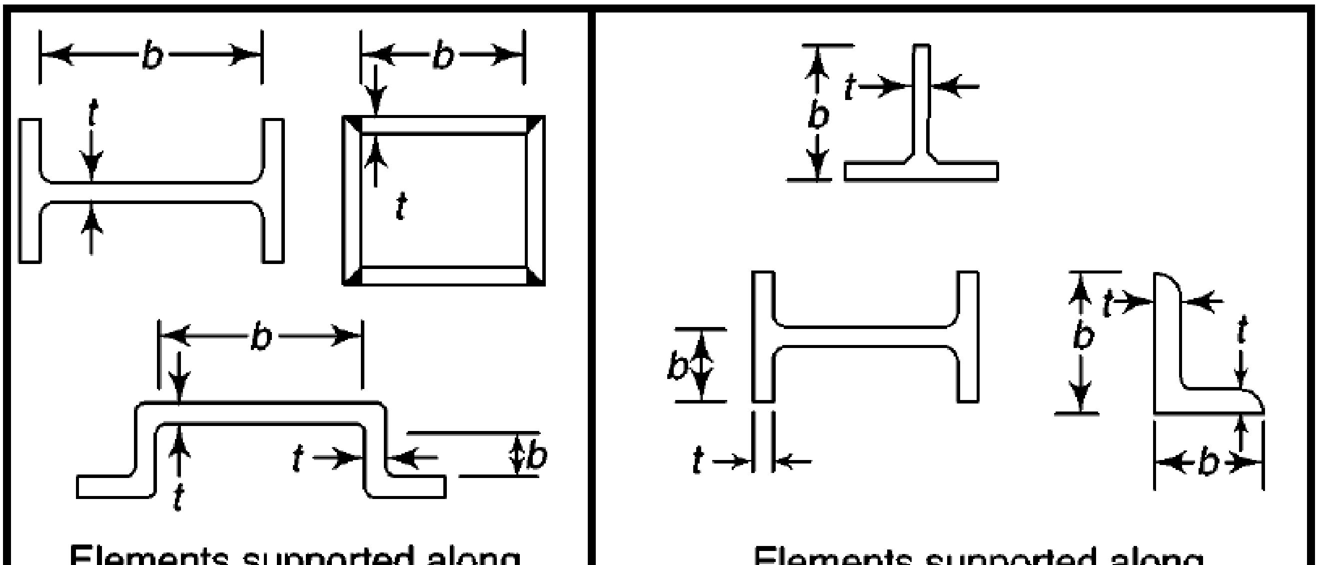
$$\upsilon = \frac{M_p}{M_y} = \frac{Z_p}{Z_e}$$

- It is a property of the cross-sectional shape and is independent of the material properties.

LOCAL BUCKLING OF PLATES

- *Buckling* may be defined as a structural behaviour in which a mode of deformation develops in a direction or plane perpendicular to that of the loading which produces it; such a deformation changes rapidly with increase in the magnitude of the applied loading. It occurs mainly in members or elements that are subjected to compressive forces. As a result, changes in geometry can arise at three levels**,**
  1) Deformation within the cross section of a member (known as ***Local Buckling*** or ***Local Instability***) <= WE RESTRICT OUR CURRENT DISCUSSION TO THIS CATEGORY ONLY
  2) Deflections/displacements within the length of the member relative to straight lines drawn between the corresponding points of the end supports due to the bending (known as ***Buckling of Column*** or ***Member Instability***) <= ALREADY DISCUSSED IN CHAPTER ON COMPRESSION MEMBERS
  3) Overall change of geometry of the structure, causing the joints to displace relative to each other, such as the sway deformation in multi-storey frames (known as **Frame Instability**)

- In the study on Local Buckling, any structural steel section may be considered to be composed of plate elements. These plate elements can be divided into two categories – **Unstiffened elements** & **Stiffened elements**.



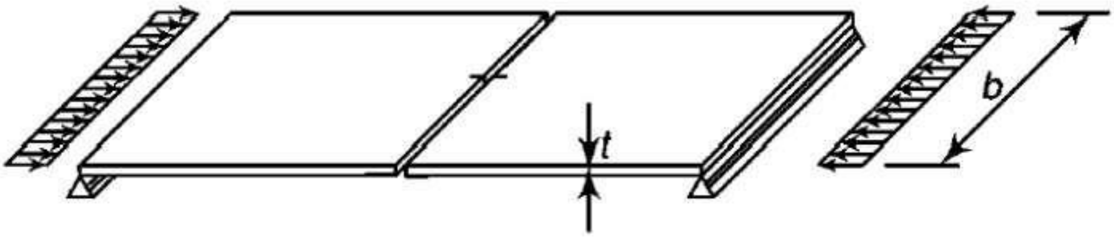
Elements supported along two edges (stiffened elements)

Elements supported along one edge only (unstiffened elements)

- As far as the local buckling of plates is concerned, the critical buckling stress in a plate element is given by:

$$f_{cr} = (k\,\pi^2\,E)/[12\,(1-\mu^2)(b/t)^2]$$

where μ is Poisson's ratio of the material, b/t is the width-to-thickness ratio of the plate, *k* is the buckling coefficient, and *E* is Young's modulus of rigidity of the material.



Since f_{cr} is inversely proportional to (b/t) ratio, **Local buckling of plate elements can be prevented by choosing those having smaller value of width-to-thickness ratio**.

- One of the major assumptions in Plastic Theory is that the beam is supported continuously laterally to prevent the failure of compression flange by local buckling. This is because, local buckling of compression flange (an unstiffened plate element) of the I-section reduces the rotation capacity of the beam and prevents the formation of plastic hinge, instead causes early failure. Hence, **Local Buckling causes premature failure of beam and hence should be avoided, otherwise the beam section shall not be utilized upto its plastic moment capacity**.

- **Classification of Cross-section:**
In IS 800-2007***,*** cross sections are placed into four behavioural classes depending upon the material yield strength, the width-to-thickness ratios of the individual components (e.g., webs and flanges), loading arrangement, and the capacity to form plastic hinges. The four classes of sections are defined as follows:

1) ***Plastic*** *or* ***Class 1*** **Cross sections**
   - They can develop Plastic Moment (i.e., Maximum Moment = M_p) as well as sufficient rotation capacity (i.e., $\Theta_2 > 6\Theta_1$; where Θ_1 is the rotation at the onset of plasticity and Θ_2 is the minimum rotation required to qualify as a plastic section) [See the moment-curvature graph below]
   - Thus, they can develop Plastic hinges
   - Hence, fully effective under pure compression
   - only these sections are used in plastic analysis and design
2) ***Compact*** *or* ***Class 2*** **Cross sections**
   - They can develop Plastic Moment (i.e., Maximum Moment = M_p) but cannot attain sufficient rotation capacity (i.e., $\Theta_2 > \Theta_1$ but $\Theta_2 < 6\Theta_1$)
   - Thus, they cannot develop Plastic Hinges, because of local buckling.

3) ***Semi-compact*** *or* ***Class 3*** **Cross sections**

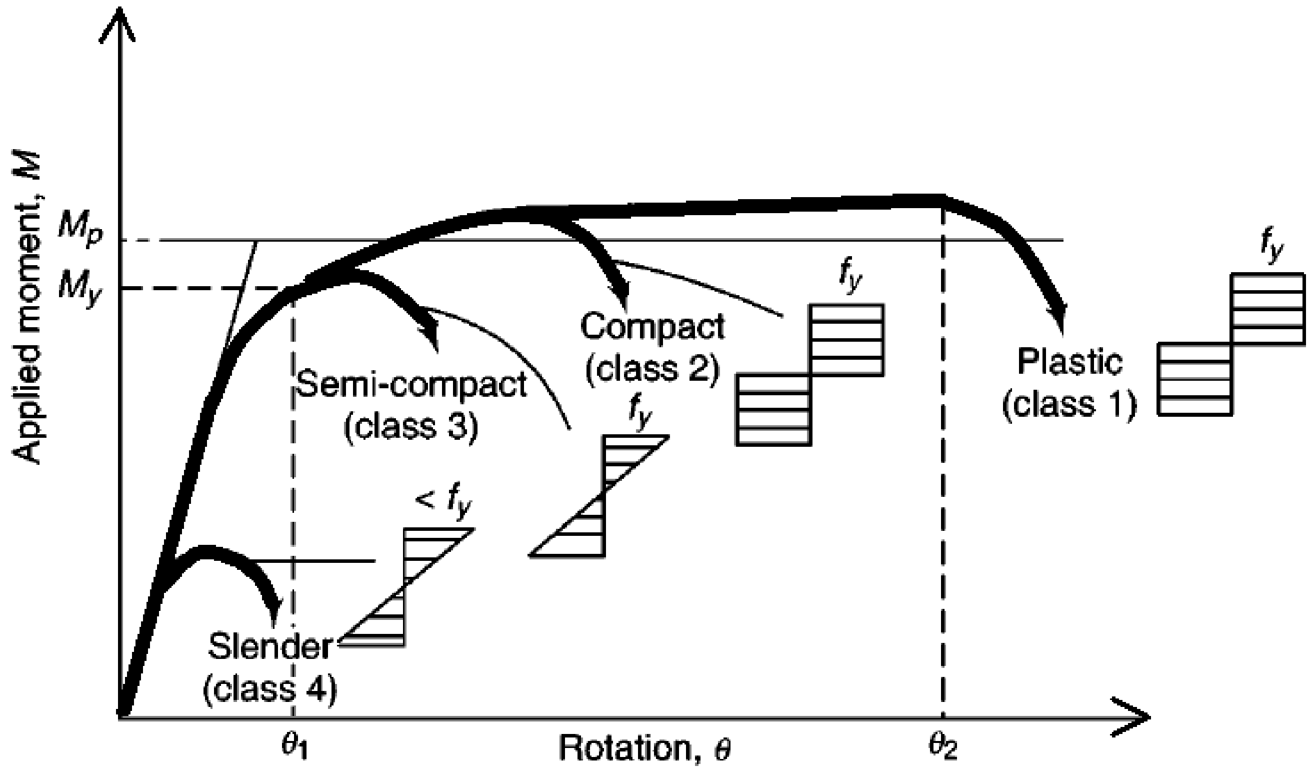
- They can develop Elastic Moment (i.e., Maximum Moment = M_y) only.
- Thus, at that section, only the extreme fibres reach yield stress. This is because local buckling prevents the development of the plastic moment resistance.

4) ***Slender*** *or* ***Class 4*** **Cross sections**

- Local buckling will occur even before the attainment of yield stress in one or more parts of the cross section.
- They cannot even develop Elastic Moment (i.e., Maximum Moment < M_y).

- The design moment capacity $\boldsymbol{M_d}$ of each of the four classes of sections is:

Plastic:	$M_d = Z_p f_y$
Compact:	$M_d = Z_p f_y$
Semi-compact:	$M_d = Z_e f_y$
Slender:	$M_d < Z_e f_y$



DESIGN OF STEEL BEAMS

- The design bending strength of beam, which is adequately supported against *lateral-torsional buckling* (local buckling of the compression flange) is known as laterally supported beam. It is governed by the yield stress .
- Design strength of beam is given under Cl. 8.2 of IS 800
  - A beam may be assumed to be adequately supported at the supports, provided the compression flange has full lateral restraint and nominal torsional restraint at supports supplied by web cleats, partial depth end plates, fin plates or continuity with the adjacent span. Full lateral restraint to compression flange may be assumed to exist if the frictional or other positive restraint of a floor connection to the compression flange of the member is capable of resisting a lateral force not less than 2.5 percent of the maximum force in the compression flange of the member. This may be considered to be uniformly distributed along the flange, provided gravity loads constitute the dominant loading on the member and the floor construction is capable of resisting this lateral force.
  - The **design bending strength** of a section which is not susceptible to web buckling under shear before yielding (where $d/t_w < 67\varepsilon$, where $\varepsilon =$ ——) shall be determined according to Cl. 8.2.1.2.

8.2.1.2 When the factored design shear force does not exceed 0.6 V_d, where V_d is the design shear strength of the cross-section (*see* **8.4**), the design bending strength, M_d shall be taken as:

$$\boxed{M_d = \beta_b Z_p f_y / \gamma_{m0}}$$

To avoid irreversible deformation under serviceability loads, M_d shall be less than 1.2 $Z_e f_y / \gamma_{m0}$ incase of simply supported and 1.5 $Z_e f_y / \gamma_{m0}$ in cantilever beams;

where

β_b = 1.0 for plastic and compact sections;

β_b = Z_e / Z_p for semi-compact sections;

Z_p, Z_e = plastic and elastic section modulii of the cross-section, respectively;

f_y = yield stress of the material; and

γ_{m0} = partial safety factor (*see* **5.4.1**).

- **Design for shear strength** of beam is calculated based on Cl.8.4 in IS 800, as:

8.4 Shear

The factored design shear force, V, in a beam due to external actions shall satisfy

$$V \leq V_d$$

where

V_d = design strength

= V_n / γ_{m0}

γ_{m0} = partial safety factor against shear failure **=1.10**

8.4.1 The nominal plastic shear resistance under pure shear is given by:

$$V_n = V_p = \frac{A_v f_{yw}}{\sqrt{3}}$$

A_v = shear area, and

f_{yw} = yield strength of the web.

8.4.1.1 The shear area may be calculated as given below:

I and channel sections:

Major Axis Bending:

Hot-Rolled — $h\, t_w$

Welded — $d\, t_w$

Minor Axis Bending:

Hot-Rolled or Welded — $2b\, t_f$

Rectangular hollow sections of uniform thickness:

Loaded parallel to depth (h) — $A\, h / (b + h)$

Loaded parallel to width (b) — $A\, b / (b + h)$

Circular hollow tubes of uniform thickness — $2\, A / \pi$

Plates and solid bars — A

where

A = cross-section area,

b = overall breadth of tubular section, breadth of I-section flanges,

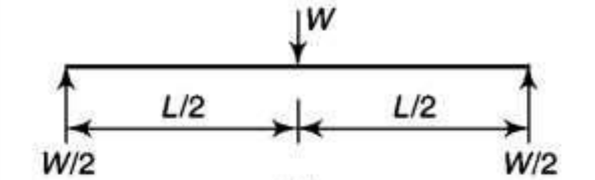
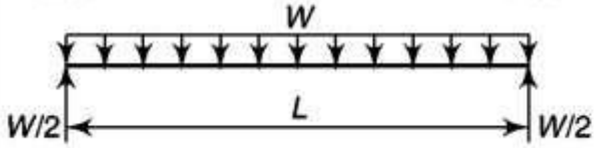
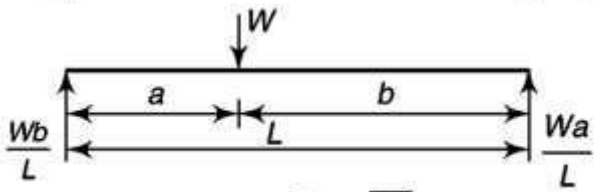
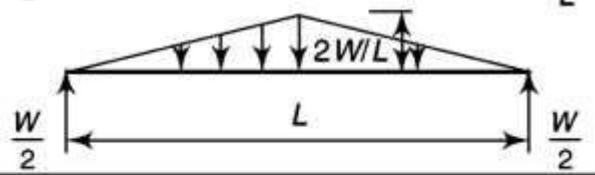
d = clear depth of the web between flanges,

h = overall depth of the section,

t_f = thickness of the flange, and

t_w = thickness of the web.

- Deflections for some common load cases for simply supported beams together with the maximum moments, can be determined using the following formulae.

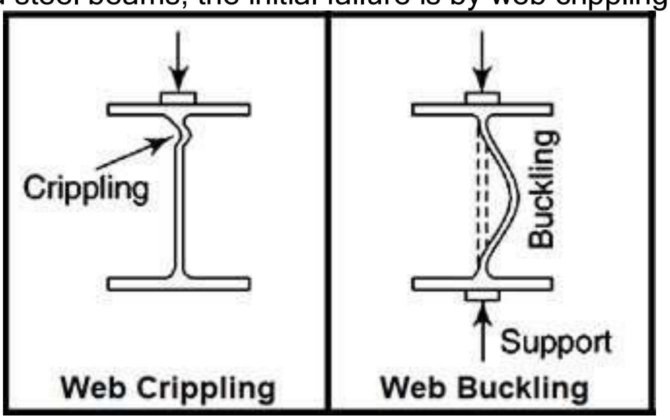
Beam and Load	Maximum Moment	Deflection at Centre
	$WL/4$	$\frac{WL^3}{48EI}$
	$WL/8$	$\frac{5WL^3}{384EI}$
	Wab/L	$\frac{WL^3}{48EI}\left[\frac{3a}{L}-4\left(\frac{a}{L}\right)^3\right]$
	$WL/6$	$\frac{WL^3}{60EI}$

The deflection thus found out should not exceed the deflection limits, as restricted by the code (Table 5 – Page 31).

MODES OF FAILURES OF WEB OF BEAM (*Additional Topic only for reference)*

A heavy load or reaction concentrated on a short length produces a region of high compressive stresses in the web either under the load or at the support. This shall failure of web in any of the following modes:

- **Web Buckling / Vertical Buckling**: occurs when the intensity of vertical compressive stress near the centre of the section becomes greater than the critical buckling stress of the web. It is very common in built-up beams having greater depth-to-thickness ratios.
- **Web Crippling / Web Crushing** : involves yielding of the web material directly adjacent to the flange. In rolled steel beams, the initial failure is by web crippling.



QUESTION: *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

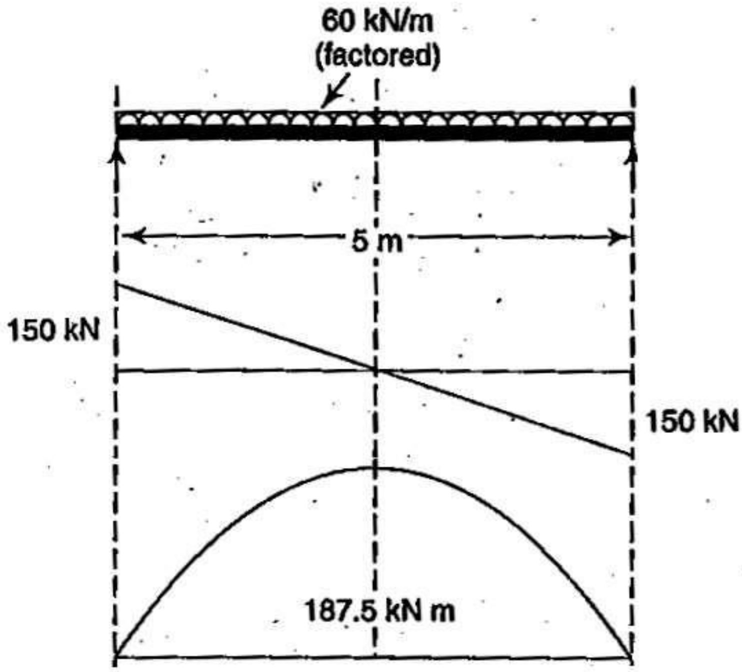
Dead load = $1.5 \times 20 = 30$ kN/m

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

Total factored load on the beam = 60 kN/m (see Fig)

Step 2: Calculation of maximum bending moment and shear force

Maximum bending moment = $60 \times 5^2/8 = 187.5$ kN m



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 **(From Steel Tables)**

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

Factored self weight of the beam = $1.5 \times 0.486 = 0.73$ kN/m

Total load acting on the beam = 60.73 kN/m

Maximum bending moment = 190 kN m

Plastic section modulus required = $\dfrac{190 \times 10^6 \times 1.1}{250} = 836 \times 10^3$ 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

Design shear force $V = \frac{wl}{2} = \frac{60.73 \times 5}{2} = 151.83$ kN

Step 7: Design shear strength of the section

Design shear strength $V_d = \frac{f_y}{\gamma_{m0} \times \sqrt{3}} \times h \times t_w = \frac{250}{1.1 \times \sqrt{3}} \times 350 \times 7.4$

$= 340 \text{ kN} > 151.83 \text{ kN}$

Also,

$0.6V_d = 204$

Therefore, the design shear force $V < 0.6V_d$.

Step 8: Check for design capacity of the section

$\frac{d}{t_w} = 39.9$ (which is less than 67ε)

Hence,

$$M_d = \beta_b Z_p \times \frac{f_y}{\gamma_{m0}}$$

$\beta_b = 1.0$, since the section is plastic section.

Therefore,

$$M_d = \frac{1.0 \times 851.11 \times 10^3 \times 250}{1.1} = 193.43 \text{ kN m}$$

$$193.43 \text{ kN m} \le \frac{1.2 \times Z_e \times f_y}{\gamma_{m0}} = \frac{1.2 \times 751.9 \times 10^3 \times 250}{1.1} = 205 \text{ kN m}$$

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.

$$\delta = \frac{5wl^4}{384EI} = \frac{5 \times 20 \times 5000^4}{384 \times 2 \times 10^5 \times 13200 \times 10^4} = 6.165 \text{ mm}$$

Allowable maximum deflection max $= \frac{L}{300} = \frac{5000}{300} = 16.67$ mm

The deflection is less than the allowable maximum deflection.

Example 14.1 The effective length of compression flange of simply supported beam MB 500,@0.869 kN/m is 8 m. Determine the safe uniformly distributed load per meter length which can be placed over the beam having an effective span of 8 meters. Adopt maximum permissible stresses as per IS 800-1984. The ends of beam are restrained against rotation at the bearings.

Solution:

Step 1: Permissible bending stress

MB 500,@0.869 kN/m has been used as simply supported beam. The effective span of beam is 8 m. The effective length of compression flange is also 8 m.

From the steel section table, the section modulus of beam $Z = 1808.7 \times 10^3\ mm^3$

Mean thickness of compression flange $t_f = T = 17.2$ mm

Thickness of web $t_w = 10.2$ mm

It is assumed that the value of yield stress, f_y for the structural steel of MB 500,@0.869 kN/m is 250 N/mm^2(MPa).

Ratio $\left(\frac{T}{t_w} = \frac{17.2}{10.2}\right) = 1.686 < 2.00$

Ratio $\left(\frac{d_t}{t_w}\right) = \left(\frac{h-2h_2}{t_w}\right) = \left(\frac{500-2\times37.95}{10.2}\right) = \left(\frac{h_1}{t_w}\right) = \left(\frac{424.1}{10.2}\right) = 41.578 < 85$

Ratio $\left(\frac{D}{T}\right) = \left(\frac{500}{17.0}\right) = 29.07$

The effective length of compression flange is 8 m.

Ratio $\frac{l}{r_y} = \left(\frac{0.7\times8\times1000}{35.2}\right) = 159.1$

From IS: 800-1984, the maximum permissible bending stress, for above ratios (by linear interpolation) $\sigma_{bc} = 65.121$ N/mm^2(MPa)

Step 2: Load supported over beam

$$M_r = (\sigma_{bc} \times Z) = \left(\frac{88.566 \times 1808.7 \times 10^3}{1000 \times 1000}\right) = 160.189\ m - kN$$

MB 500,@0.869 kN/m can resist maximum bending moment equal to moment of resistance. Therefore the maximum bending moment M=160.189 m-kN

Step 3: Load supported over beam

The effective span of the beam is 8 meters. Let w be the uniformly distributed load per meter length. The maximum bending moment, M for the beam occurs at the centre..

$$M = \left(\frac{w.l^2}{8}\right) = 160.189 = \left(\frac{w \times 8 \times 1000}{8}\right)$$

$$\text{Therefore } w = \left(\frac{160.189 \times 8}{8 \times 8 \times 1000} \times 1000\right) = 20.02\ kN/m$$

The self-weight of the beam is 0.869 kN/m. Therefore, the safe uniformly distributed load which can be placed over the beam (20.02-0.869)=19.15 kN.

Example 14.2 Design a simply supported beam to carry a uniformly distributed load of 44 kN/m. The effective span of beam is 8 meters. The effective length of compression flange of the beam is also 8 m. The ends of beam are not free to rotate at the bearings.

Design:

Step 1: Load supported, bending moment and shear force

Uniformly distributed load = 44 kN/m

Assume self weight of beam = 1.0 kN/m

Total uniformly distributed load w= 45 kN/m

The maximum bending moment, M occurs at the centre

$$M = \left(\frac{w.l^2}{8}\right) = \left(\frac{45 \times 8 \times 8 \times 1000}{8 \times 1000}\right) = 360\ m - kN$$

The maximum shear force, F occurs at the support $F = \left(\frac{w.l}{2}\right) = \left(\frac{45 \times 8}{2}\right) = 180\ kN$

Step 2: Permissible bending stress

It is assumed that the value of yield stress, f_y for the structural steel is 250 N/mm^2 (MPa). The ratios (T/t_w) and (d_1/t_w) are less than 2.0 and 85 respectively. The maximum permissible stress in compression or tension may be assumed as $\sigma_{bc} = \sigma_{bt}$ = (0.66 x 250) = 165 N/mm^2

Section modulus required, $Z = \frac{M}{\sigma_{bc}} = \left(\frac{360\times1000\times1000}{165}\right) = 2181.82 \times 10^3\ mm^3$

The steel beam section shall have (D/T) and (l/r_y) ratios more than 8 and 40 respectively. The trial section of beam selected may have modulus of section, Z more than that needed (about 25 to 50 per cent more).

Step 3: Trial section modulus

1.50 x 2181.82 x 10^3 mm^3=3272.73 x 10^3 mm^3

From steel section tables, try WB 600,@1.337 kN/m

Section modulus, Z_{xx}=3540.0 x 10^3 mm^3

Moment of inertia, I_{xx}=106198.5 x 10^4 mm^4

Thickness of web, t_w=11.2 mm

Thickness of flange, $T=t_f$=21.3 mm

Depth of section, h=600 mm

Step 4: Check for section modulus

$$\frac{D}{T} = \frac{600}{21.3} = 28.169$$

$$\frac{T}{t_w} = \frac{21.3}{11.2} = 1.901 < 2.0, also \left(\frac{d_1}{t_w} < 85\right)$$

The effective length of compression flange of beam is 8 m.

$$\frac{l}{r_y} = \left(\frac{0.7 \times 8 \times 10.0}{52.5}\right) = 106.66$$

From IS: 800-1984, the maximum permissible bending stress σ_{bc}=118.68 N/mm^2(MPa)

Section modulus required

$$\left(\frac{360 \times 1000 \times 1000}{118.68}\right) = 3033.34 \times 10^3 mm^3 < 3540 \times 10^3 mm^3 provided$$

Further trial may give more economical section.

Step 5: Check for shear force

Average shear stress, $\tau_{v.cal} = \left(\frac{F}{h \times t_w}\right) = \left(\frac{180 \times 1000}{600 \times 11.2}\right) = 26.78\ N/mm^2$

Permissible average shear stress

$0.4 \text{ x } f_y = (0.4 \text{ x } 250) = 100$ N/mm^2 > Actual average shear stress

Step 6: Check for deflection

Maximum deflection of the beam

$$y_{max} = \left(\frac{5}{384} \times \frac{w.l^4}{EI}\right) = \frac{5}{384} \times \left(\frac{45 \times 8 \times 8^3 \times 1000^3 \times 1000}{2.047 \times 10^5 \times 106198.5 \times 10^4}\right) = 24.53\ mm$$

Allowable deflection $\frac{1}{325} \times span = \left(\frac{1}{325} \times 8000\right) = 24.60\ mm$

The maximum deflection is less than allowable deflection, hence the beam is safe. Provide WB 600,@1.337 kN/m

CHAPTER 6

DESIGN OF TUBULAR STEEL STRUCTURE

INTRODUCTION

In structural engineering, the **tube** is a system where, to resist lateral loads (wind, seismic, impact), a building is designed to act like a hollow cylinder, cantilevered perpendicular to the ground. This system was introduced by Fazlur Rahman Khan while at the architectural firm Skidmore, Owings & Merrill (SOM), in their Chicago office.

Construction of high-rise buildings used to be driven by the demand for space in densely populated land areas. Advancements in structural engineering and technology have greatly pushed the height limit. Combined with the improvement in fabrication and construction methods, the construction of skyscrapers not only has become more relevant and feasible; it has pushed the height limit even further. Over the years, Nations and major companies have been constantly pursuing the title of the tallest building in the world.

Major advancements in structural engineering have been the development of different structural systems that allow for higher buildings. As the height of the building increase, the lateral resisting system becomes more important than the structural system that resists the gravitational loads.

A. Framed Tube System

Sufficient design of exterior frame for the purpose of resisting lateral loads allows the interior part of the building to only resist the gravity loads. Not only this concept is a clever solution for resisting lateral loads but also leaves the architect with the wonderful choices for the interior space of the buildings. Framed-tube and braced-tube structural systems are popular structural systems among the exterior lateral load resisting systems. Beedle et al. defines this type of structural system as following: “In structural engineering, the tube is the system where in order to resist lateral loads (wind, seismic, etc.) a building is designed to act like a hollow cylinder, cantilevered perpendicular to the ground” This system was introduced by Fazlur Rahman Khan while at Skidmore, Owings and Merrill's (SOM) Chicago office. The first example of the tube‟s use is the 43-story Khan-designed DeWitt-Chestnut Apartment Building in Chicago, Illinois, completed in 1963

- **Bundled Tube System**

Instead of one tube, a building consists of several tubes tied together to resist the lateral forces. The system allows for the greatest height and the most floor area. In this system, introduction of the internal webs greatly reduces the shear lag in the flanges

Hence, their columns are more evenly stressed than in the single tube structure and their contribution to the lateral stiffness is greater. This allows columns of the frames to be spaced further apart and to be less obstructive. This structural system was used in Sears Tower in Chicago introduced by Fazlur Rahman Khan. In the Sears Tower,
advantage was taken of the bundled form to discontinue some of the tubes, and so reduce the plan of the building at
stages up the height. Types of bundle tube system are mentioned below

1 Framed Tube 2 Bundled Tube

Single framed tube is connected with more than one tube which makes framed bundled tube. Example of Framed Bundle Tube is sears tower shown in Same as Framed Bundle Tube, in this system single braced tube connect with more than one tube and make bundle 0f Braced tube. Example of Braced Bundle Tube shown in Combination of Braced Tube and Framed Tube connect with each other and make Bundle Tube. Like two framed tube and two braced tube connect and make bundle tube having four tubes

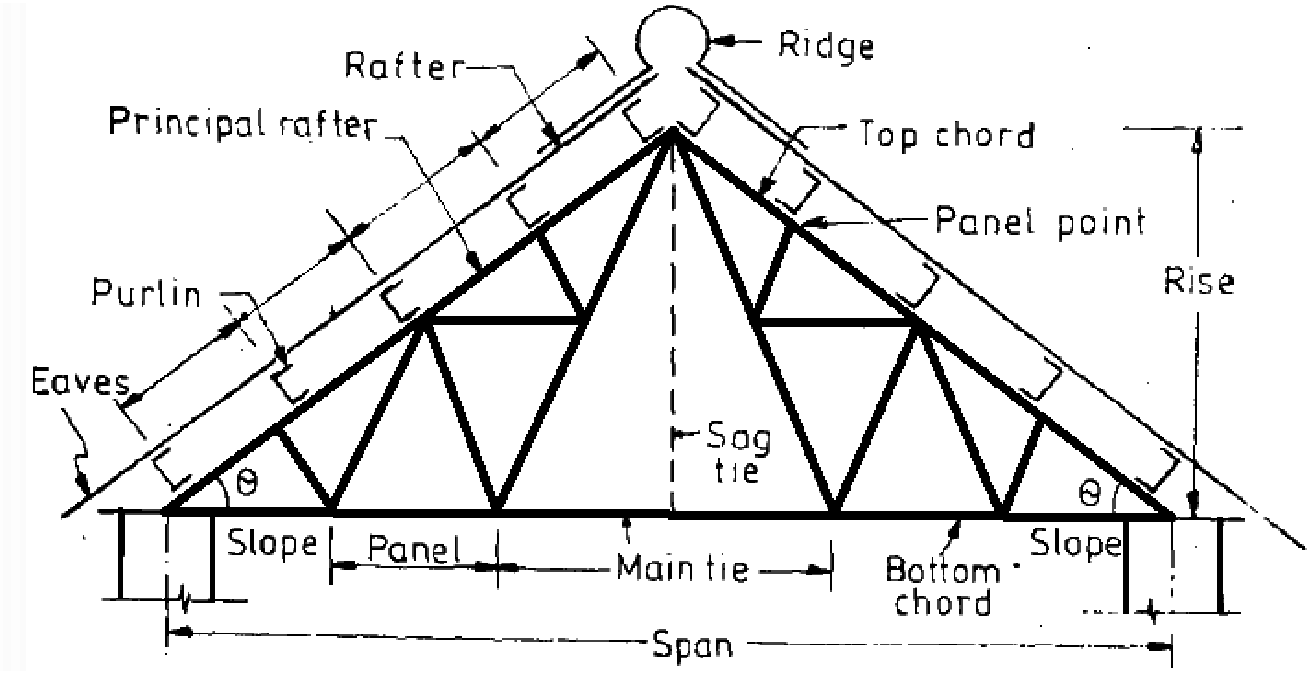
TUBE AND FRAMED TUBE

Framed Bundled Tube System

Braced Bundled Tube System

For the parametric comparison, a symmetrical building is selected. One steel building for 64 story are modelled, analyzed and designed in ETABS for two structural systems; framed bundled tube and framed tube. Analysis and design are carried out for dead load, live load, lateral earthquake load and lateral wind load. For earthquake loads, both static and response spectrum analysis are done. To consider extreme conditions of lateral loads, the buildings are considered to be located in Zone V. The parameters selected for the comparison are fundamental time period, maximum top story lateral displacement, maximum base shear, maximum story displacement and maximum story drift. Further, governing lateral force is also determined.

PARTS OF ROOF TRUSS



1) Top Chord

The uppermost line of members that extend from one support to the other through the apex is called top chord. The top chord is also known as the upper chord of the roof truss. Such members are called Principal Rafters.

2) Bottom Chord

The lowermost line of members of truss extending from one support to the other is called bottom chord. The bottom chord is also known as lower chord of the root truss.

3) Rise

The rise of a roof truss is the vertical distance measured from the apex to the line joining supports.

4) Pitch

The ratio of the rise to the span is called the pitch of a roof truss. It is also expressed sometimes as the angle between the lower and the upper chords. Roofs are pitched to facilitate drainage of water. The pitches 1/3 and 1/5 and corresponding to an angle of inclination of 30^{O} is commonly employed.

5) Slope

The slope of a roof is defined as the tangent of the angle that the plane of the roof makes with horizontal. The slope of the roof therefore is not equal to the pitch and greater care should be taken to see that the two terms are not used synonymously. The slope of the roof is equal to twice the numerical value of pitch in all the cases whether truss is symmetrical or unsymmetrical.

6) Panel

The portion of the truss lying between two consecutive joints in a principal rafter of a roof truss is called a panel. It is also defined as the distance between the two adjacent purlins.

7) Bay

The portion of a roof truss contained between any two consecutive trusses is called as Bay.

8) Purlins

The purlins are horizontal beams spanning between the two adjacent trusses. These are the structural members subjected to transverse loads and rest on the top chords of root trusses. The purlins aremeant to carry the loads of the roofing material and to transfer it on the panel points.

9) Sub-purlins

The sub-purlins are the secondary system of purlins resting on the rafter. These are spaced to support the tiles or slate coverings.

10) Rafters

The rafters are beams and rest on the purlins. The rafters support the sheathing. They may support sub-purlins directly. These are called common rafters to distinguish from principal rafter.

11) Ridge Line

The ridge line is a line joining the vertices of the trusses.

12) Eaves

The bottom edges of an inclined roof surface or a pitched roof is termed as eaves.

LOADS ON ROOF TRUSS

1. Dead Load: Based on IS 875 (Part 1) : 1987
2. Live Load: Based on IS 875 (Part 2) : 1987
3. Wind Load: Based on IS 875 (Part 3) : 1987
    - Wind pressure (in N/m2) at height z is given by: $p_z = 0.6 (V_z)^2$
    where V_z is the design wind speed (in m/s) at height z, calculated as:
    $$V_z = V_b \times k_1 \times k_2 \times k_3 \times k_4$$
    V_b is the basic wind speed (in m/s)
    k_1 is the probability factor or risk factor
    k_2 is the terrain, height and structure size factor
    k_3 is the topography factor
    k_4 is the importance factor for cyclonic regions
4. Snow Load: Based on IS 875 (Part 4) : 1987
5. Special loads and load combinations: Based in IS 875 (Part 5) : 1987
6. Earthquake Loads: Based on IS 1893 : 2002

TYPES OF TRUSSES AND TRUSS CONFIGURATIONS

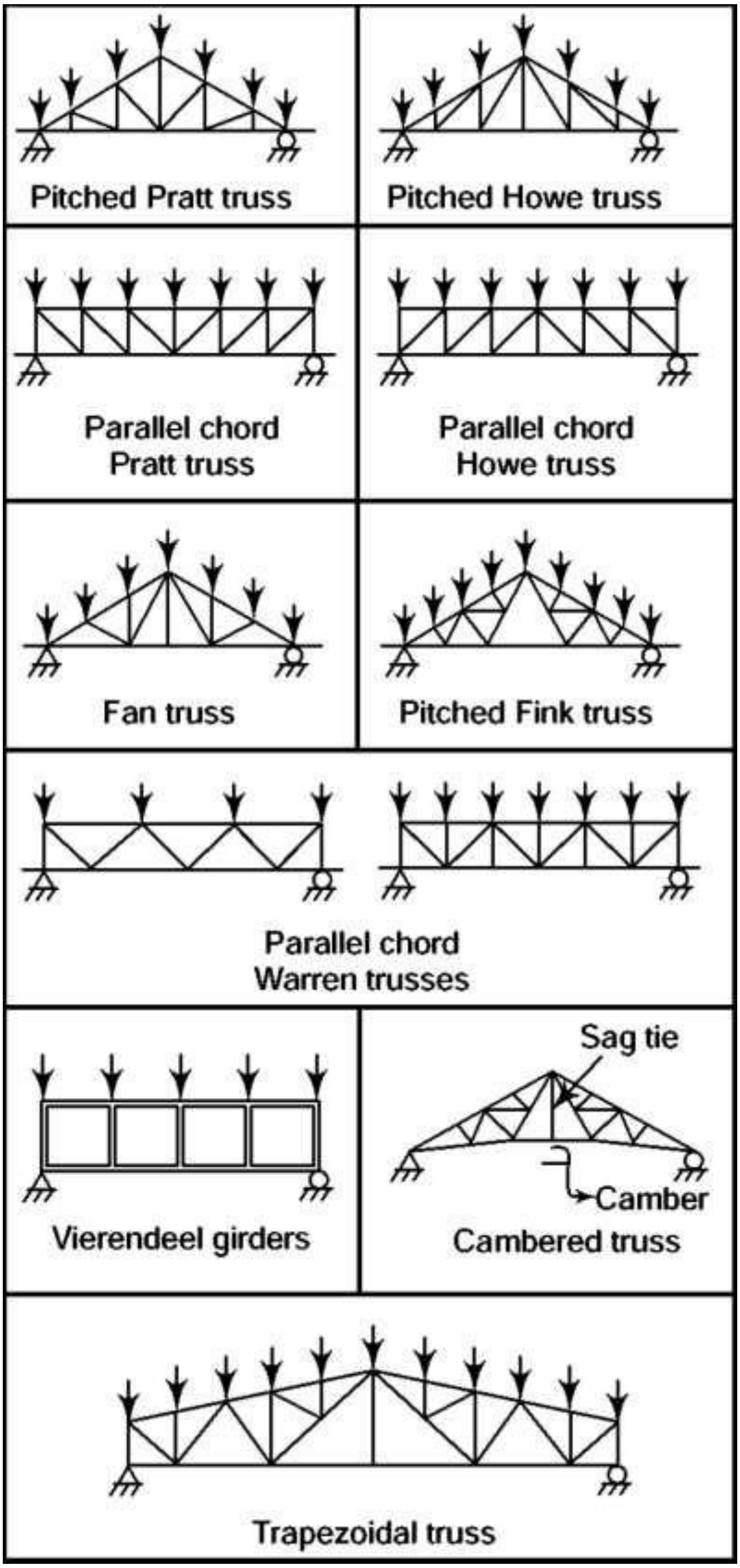
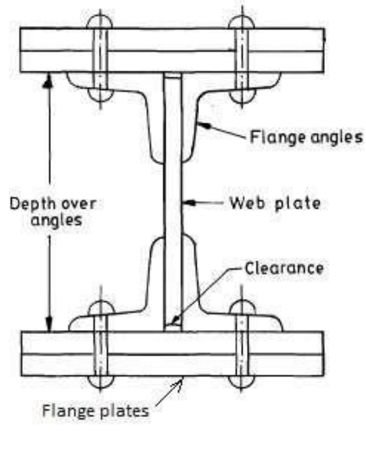


PLATE GIRDERS

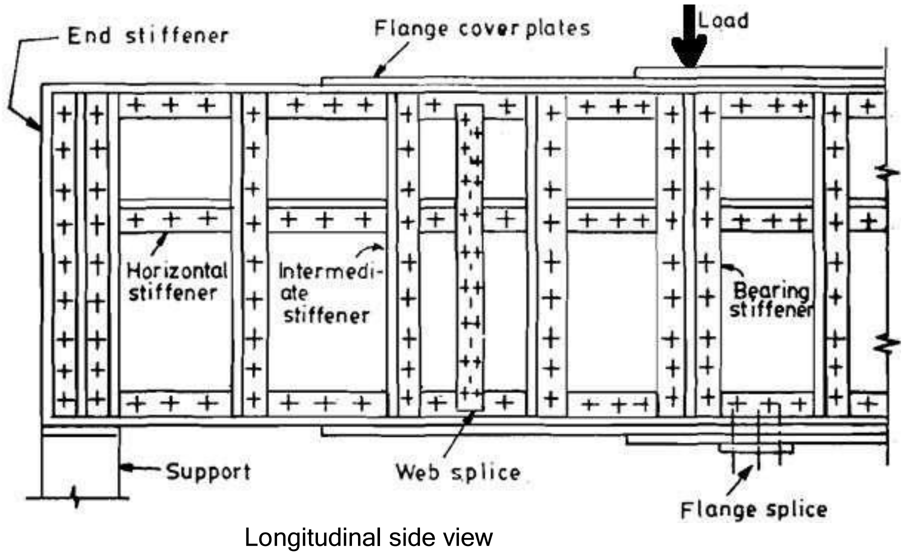
The plate girders are essentially built-up beams to carry heavier loads over large spans. They are deep structural members subjected to transverse loads. The plate girders consist of plates and angles riveted together. Plates and angles form an I-Section. They are used in building constructions and also in bridges. When the span and load combination is such that the rolled steel beams become insufficient to furnish the requirement and built-up beam becomes uneconomical, then plate-girders are used.

A plate girder essentially consists of

- Web plate - a vertical plate which connects flange plates by means of angles top and bottom flange angles. It keeps flange plates in position and is capable to resist shear force in beams.
- Flange plate - two horizontal plates in the built-up beam, used to resist bending moment acting on the beam. One flange develops tension, while the other develops compression. The web and flange plates are thin, and hence likely to buckle under compression.
- Stiffeners - In order to avoid buckling of web due to shear and bending, and buckling of web at points of concentrated loads, the web has to be stiffened by means of stiffeners. Stiffeners are classified into four:
    - Bearing stiffeners – provided at points of concentrated loads, to protect the web from direct compressive stress.
    - End posts or End bearing stiffeners – since at the end of beams, certain portion of web acts as compression member, these are provided to prevent crushing of web.
    - Intermediate transverse stiffeners – provided to prevent buckling of web and enhance its stiffness.
    - Longitudinal stiffeners or Horizontal stiffeners – provided to prevent buckling of web. If a single stiffener is required, it is provided at the level of 0.2d from the compression flange, where d is the depth of web plate.



Cross-sectional view



Longitudinal side view

DESIGN OF MASONARY STRUCTURES

INTRODUCTION

The strength of a masonry wall depends primarily upon the strength of the masonry units and the strength of the mortar. In addition, the **quality of workmanship** and the method of bonding is also important. thickness.

- Height of the wall.
- Length of the wall.
- Thickness of wall and.
- Support conditions.

Load bearing masonry buildings

ADVANTAGES AND DEVELOPMENT OF LOADBEARING MASONRY

The basic advantage of masonry construction is that it is possible to use the same element to perform a variety of functions, which in a steelframed building, for example, have to be provided for separately, with consequent complication in detailed construction. Thus masonry may, simultaneously, provide structure, subdivision of space, thermal and acoustic insulation as well as fire and weather protection. As a material, it is relatively cheap but durable and produces external wall finishes of very acceptable appearance. Masonry construction is flexible in terms of building layout and can be constructed without very large capital expenditure on the part of the builder. In the first half of the present century brick construction for multistory buildings was very largely displaced by steel- and reinforced concrete-framed structures, although these were very often clad in brick. One of the main reasons for this was that until around 1950 load bearing walls were proportioned by purely empirical rules, which led to excessively thick walls that were wasteful of space and material and took a great deal of time to build. The situation changed in a number of countries after 1950 with the introduction of structural codes of practice which made it possible to calculate the necessary wall thickness and masonry strengths on a more rational basis. These codes of practice were based on research programs and building experience, and, although initially limited in scope, provided a sufficient basis for the design of buildings of up to thirty storey. A considerable amount of research and practical experience over the past 20 years has led to the improvement and refinement of the various structural codes. As a result, the structural design of masonry buildings is approaching a level similar to that applying to steel and concrete.

BASIC DESIGN CONSIDERATIONS

Load bearing construction is most appropriately used for buildings in which the floor area is subdivided into a relatively large number of rooms of small to medium size and in which the floor plan is repeated on each storey throughout the height of the building. These considerations give ample opportunity for disposing loadbearing walls, which are continuous from foundation to roof level and, because of the moderate floor spans, are not called upon to carry unduly heavy concentrations of vertical load. The types of buildings which are compatible with these requirements include flats, hostels, hotels and other residential buildings. The form and wall layout for a particular building will evolve from functional requirements and site conditions and will call for collaboration between engineer and architect. The arrangement chosen will not usually be critical from the structural point of view provided that a reasonable balance is allowed between walls oriented in the principal directions of the building so as to permit the development of adequate resistance to lateral forces in both of these directions. Very unsymmetrical arrangements should be avoided as these will give rise to torsional effects under lateral loading which will be difficult to calculate and which may produce undesirable stress distributions. Stair wells, lift shafts and service ducts play an important part in deciding layout and are often of primary importance in providing lateral rigidity. The great variety of possible wall arrangements in a masonry building makes it rather difficult to define distinct types of structure, but a rough classification might be made as follows: • Cellular wall systems • Simple or double cross-wall systems • Complex arrangements. A cellular arrangement is one in which both internal and external walls are loadbearing and in which these walls form a cellular pattern in plan. Figure shows an example of such a wall layout. The second category includes simple cross-wall structures in which the main bearing walls are at right angles to the longitudinal axis of the building. The floor slabs span between the main cross-walls, and longitudinal stability is achieved by means of corridor walls, as shown in Fig. This type of structure is suitable for a hostel or hotel building having a large number of identical rooms. The outer walls may be clad in non-loadbearing masonry or with other materials. It will be observed that there is a limit to the depth of building which can be constructed on the cross-wall principle if the rooms are to have effective day-lighting. If a deeper block with a service core is required, a somewhat more complex system of cross-walls set parallel to both major axes of the building may be used, as in . All kinds of hybrids between cellular and cross-wall arrangements are possible, and these are included under the heading ‘complex’, a typical example being shown in Fig. Considerable attention has been devoted in recent years to the necessity for ensuring the ‘robustness’ of buildings. This has arisen from a number of building failures in which, although the individual members have been adequate in terms of resisting their normal service loads, the building as a whole has still suffered severe damage from abnormal loading, resulting for example from a gas explosion or from vehicle impact. It is impossible to quantify loads of this kind, and what is required is to construct buildings in such a way that an incident of this category does not result in catastrophic collapse, out of proportion to the initial forces. Meeting this requirement begins with the selection of wall layout since some arrangements are inherently more resistant to abnormal forces than others. This point is illustrated in Fig. 1.2: a building consisting only of floor slabs and cross-walls is obviously unstable and liable to collapse under the influence of small lateral forces acting parallel to its longer axis. This particular weakness could be removed by incorporating a lift shaft or stair well to provide resistance in the weak direction, as in Fig.). However, the flank or gable walls are still vulnerable, for example to vehicle impact, and limited damage to this wall on the lowermost storey would result in the collapse of a large section of the building. A building having a wall layout as in Fig. on the other hand is clearly much more resistant to all kinds of disturbing forces, having a high degree of lateral stability, and is unlikely to suffer extensive damage from failure of any particular wall. Robustness is not, however, purely a matter of wall layout. Thus a floor system consisting of unconnected precast planks will be much less resistant to damage than one which has cast-in-situ concrete floors with two-way reinforcement. Similarly, the detailing of elements and their connections is of great importance. For example, adequate bearing of beams and slabs on walls is essential in a gravity structure to prevent possible failure not only from local over-stressing but also from relative movement between walls and other elements. Such movement could result from foundation settlement, thermal or moisture movements. An extreme case occurs in seismic areas where positive tying together of walls and floors is essential. The above discussion relates to multi-storey, loadbearing masonry buildings, but similar considerations apply to low-rise buildings where there is the same requirement for essentially robust construction

LOAD BEARING WALL

What is load baring wall

A Load bearing wall (or bearing wall) is a wall that bears a load resting upon it by conducting its weight off to a foundation structure. The materials most often used to construct load-bearing walls in large buildings are concrete, block, or brick. A load bearing wall supports loads of a structure, such as floors, equipment, furniture, and people. At one-time building were constructed with very thick bricks walls carrying all floor and other loads. Design of these walls was not based on engineering data but only on well-intentioned but unscientific building codes. As buildings grew taller, the building code requirement for the thickness of a brick wall becomes economically prohibitive.

Depending on the type of building and the number of stories, load-bearing walls are engaged to the appropriate thickness to carry the weight above them, without doing so, it is possible that an outer wall could be becomes unstable if the load exceeds the strength of the materials used, potentially leading to the collapse of the structure.

Types of load bearing wall

Load-bearing walls may further be divided into the following types

- Solid masonry wall
- Cavity wall
- Faced wall
- Veneered wall

Types of load bearing wall

Solid masonry wall

Solid masonry walls are the most commonly use. These walls built of individual blocks of material, such as bricks, clay or concrete blocks, or stone, usually in horizontal courses, cemented together with suitable mortar.

A solid wall construct of the same type of building unties throughout its thickness. However, it may have an opening for doors, windows, etc.

Cavity wall

A Cavity wall is a wall comprising two leaves, each leaf being built of structural units and separated by a cavity and tied together with metal ties or bonding units to ensure that the two leaves act a sone structural unit.

The space between the leaves is either left as a continuous cavity or is filled with non-load bearing insulting and waterproofing material.

Cavity wall

Faced wall and veneer wall

A facing wall is a wall in which the facing and backing are of two different materials which bound together to ensure common action under load, Veneer walls are similar to non-load bearing walls in that they carry no weight except their own.The brick or tile fastens to a backing, but it does not exert a common action with the backing.

Perhaps the most common use brick veneer on wood frame dwelling. Other examples are architectural cotta and thin ceramic veneer on monumental buildings.

Non- load bearing wall

Non-load bearing walls carry only their own weigh. This type of wall used to close in a steel or concrete frame building. It is usually carried by supports, normally steel shelf angles at each floor, and is called a panel wall. When the wall is supported at the base only, it is called a curtain wall.

A partition wall is a thin internal wall which constructs to divide the space within the building into rooms or area. Generally, partition walls are non-load bearing.

A partition wall, separating two adjoining rooms must often provide a barrier to the passages of sound from one to another.

An additional requirement in all partitions walls is their capacity to support a surface suitable for decoration and which is able to withstand the casual damage by impact to which the occupation of the building is likely to subject them. On ground floors, partitions rest either on flooring concrete or on beams spanning between the main walls.

In multi-storeyed buildings, partitions support on concrete beams spanning between columns.

The total self-weight of partitions may considerably affect the total load carried on the framework and on the structural elements, and the building as a whole will become more economical, The thickness of partitions will affect the amount of usable floor space available in the building.

However, light and thin partitions often raise problems of sound insulation and fire resistance.

The partition wall should fulfill the following requirement.

- The amount wall should be strong enough to carry its own load.

- The partition wall should be strong enough to resist the impact in which the occupation of the building is likely to subject them.
- The partition wall should have the capacity to support a suitable decorative surface.
- A partition wall should be stable and strong enough to support some wall fixtures, wash-basins, etc.
- this wall should be as light as possible.
- A partition wall should be as thin as possible.
- A partition wall should act as a sound barrier, especially when it divides two rooms.

SLENDERNESS RATIO

The slenderness ratio of 200-mm-thick masonry wall specimens was maintained up **to 11.3 and 15 for 150-mm-thick masonry walls**. For the wall specimens of hollow concrete block masonry, an eccentricity of 0.1 t (t = thickness of wall) is chosen which is in the range of values specified by IS:1905-1987