DESIGN AND DRAWING OF STEEL STRUCTURES

UNIT I

CONNECTIONS

Objective:
To familiarize with the types of rolled steel sections, plastic analysis and IS Code provisions.

Syllabus:
Connections Structural steel – Rolled steel sections – Convention for member axes – Types of loads – Concept of Plastic Analysis – Idealized stress – strain curve, Full plasticity of beam section under bending – Plastic hinges, shape factors of rectangle, circle, triangle, T, channel and I sections – load factor – classification of cross – sections as per IS code - limit states of strength and serviceability. Riveted, bolted and pin connections – Strength of rivet / bolt welded connections – Introduction – Advantages and disadvantages of welding - Strength of welds – Butt and fillet welds; permissible stresses – IS code requirements Design of fillet weld subjected to moment acting in the plane and at right angles to the plane of the joints.

Learning Outcomes:
Students will be able to

- understand various structural steel sections.
- explain the concept of plastic analysis and shape factors.
- determine the strength of rivet and bolted connections.
- design the lintel beam and purlins in a roof truss.
- design the welded connections using is-800:2007.
- calculate the strength of the welded joint for eccentric loads.

LEARNING MATERIAL

A structure is a body composed of several elements so assembled that it can set up resistance against deformation caused due to application of external forces.

1. Various Structural Elements:

1. Tension Member
2. Compression Member
3. Flexure Member
4. Torsion Member
5. Foundation Elements

2.Structural Analysis deals with the determination of internal stresses in the elements of structure as well as determination of reaction components, when a structure is subjected to external forces.

3.Structural Design:

Two aspects:

1. Functional
2. Strength

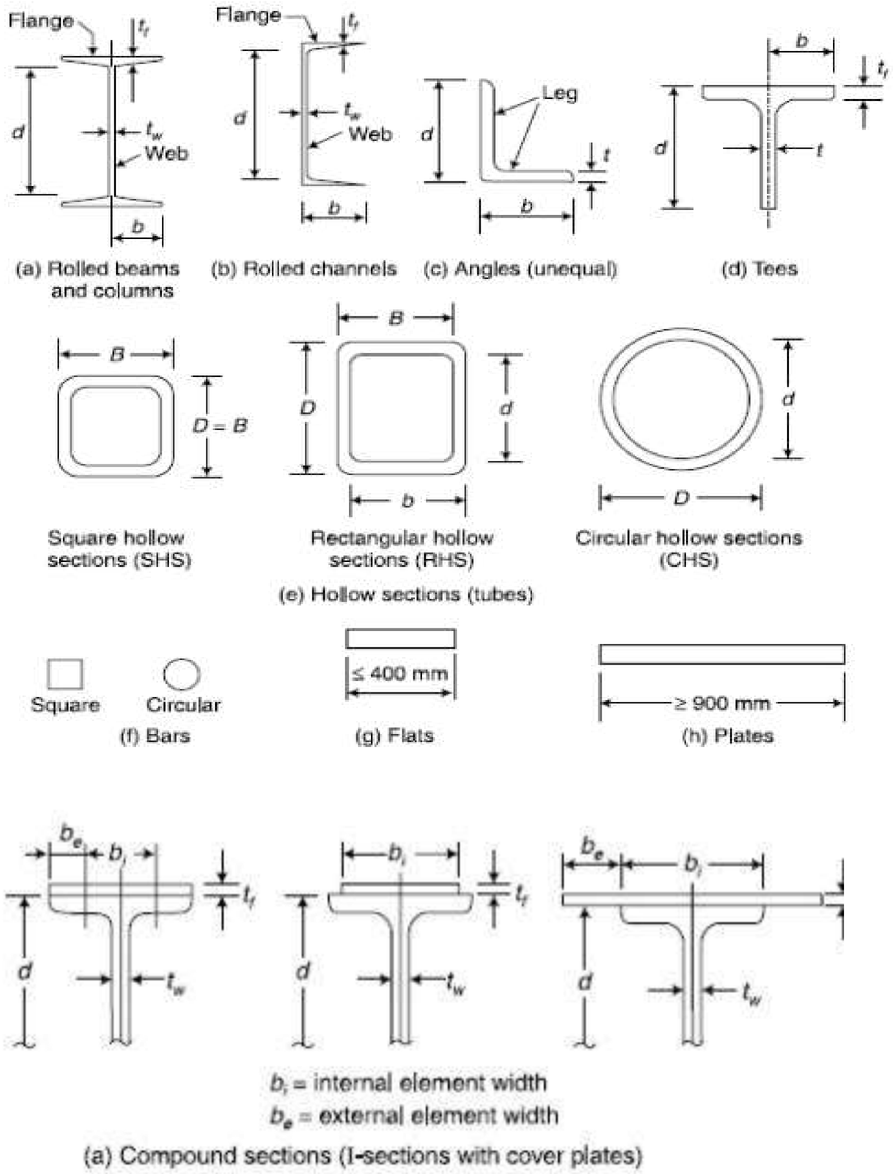
In functional design, a structure is so proportioned and constructed that it serves the needs effectively for which it is constructed.

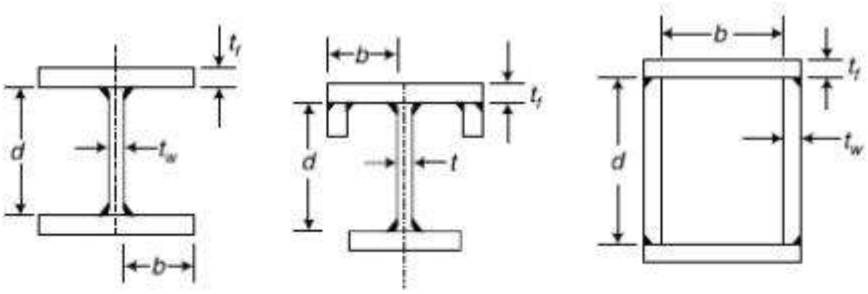
In structural design, the structure should be strong enough to resist external forces to which it is subjected during the entire period of service.

4. Structures are classified on the basis of materials and construction

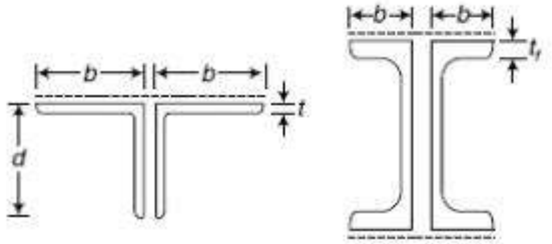
1. Steel
2. Aluminium
3. Timber
4. Plastic
5. Masonry
6. Concrete
7. Composite structures

5. Structural shapes & built-up sections





(b) Fabricated sections (welded girders)



Double angle section Double channel section

6. **Beams and Girders:**

Girders: A major beam which can also support small beams.
Joists: Closely spaced beams supporting the floor and roofs by building
Purlins: Roof beams usually supported by trusses
Rafters: Roof beams usually supported by purlins.
Lintel: Beams over window or door openings that support the wall above.
Girts: Horizontal wall beams used to support wall covering on side of an industrial building.
Spandrel beam: Beam around outside perimeter of a floor that support the exterior walls and outside edge of the floor.
Tension member: It is a member which is intended to resist axial tension. Tension members are also called as ties or hangers.
Compression Members: It is a member which is intended to resist compression stresses. Compressive members are also called as columns, struts, posts or stanchions etc. Compression members are also subjected to buckling in possible directions and bending. Hence slenderness ratio plays a vital role.

Slenderness ratio = l_e/r

Structural steel:

Steel is the most versatile commonly used structural material. The essential elements of steel are metallic iron and the element non-metallic carbon with small quantities of other elements such as silicon, nickel, manganese, chromium and copper.

Thus steel is an alloy.

Steel is usually more than 98 % iron, with other elements in present in small quantities and these other elements have pronounced effect on the properties of steel.

Types of steel:

1.Cast Iron: Low carbon content (brittle)

2.Wrought Iron: Has high carbon content (Ability to permit large deformations)

3.Steel: Has intermediate carbon content between cast iron and wrought iron.

IS: 800-2007 Code of practice for general construction in steel.

Other codes:

1. IS: 226-1975 Structural steel (standard quality)
2. IS: 1977-1975 Structural Steel (Ordinary quality)
3. IS: 2062-1984 Weldable structural steel
4. IS: 961- 1975 Structural Steel (High quality)
5. IS: 8500- 1977 Weldable structural steel (Medium and high strength qualities)

IS 226 is known as "mild steel" designated as ST 44-S mild steel is used for manufacture of rolled steel sections, rivets and both. It is suitable for all types of structures subjected to static, dynamic and cyclic loading and is suitable for welding up to 20 mm thick.

Chemical composition of steel as per IS 226-1975:

Constituents	Max. Percent
Carbon (upto 20 mm thick / dia)	0.23
Carbon (> 20 mm thick dia)	0.25
Sulphur	0.055
Phosphorus	0.055

Physical properties of mild steel:

1. Mass : 7.85 g/cc
2. E : 2.04×10^5 MPa (N/mm^2)
3. Modulus of rigidity, G = 0.785×10^5 MPa (N/mm^2)
4. Poisson's ratio, $\mu = 0.3$ (in elastic range)
5. Coefficient of thermal expansion, $\alpha_s = 12 \times 10^{-6}$ / ^oC or 6.7×10^{-6} / ^oF

Structural Steel Sections:

1.Rolled Steel Beam Sections (I Sections)

ISJB Indian Standard Junior Beam
ISLB Indian Standard Light Beam
ISMB Indian Standard Medium Weight Beam
ISWB Indian Standard Wide Flange Beam
ISHB Indian Standard Heavy Weight Beam
ISWB 600@133.7 Kg/m means 600mm overall depth and weight 133.7 Kg/m run of the beam

2.Rolled Steel Channel Sections:

ISJC Indian Standard Junior Channel
ISLC Indian Standard Light Channel
ISMC Indian Standard Medium Weight Channel
ISLC 400@47.5 Kg/m means light channel having total depth h= 400 mm and weight 47.5 Kg/m. Since it is not symmetrical about yy axis, it is subjected to twisting or torsion along with bending when used as a beam.

3.Rolled Steel Angle Sections:
i. Equal Angle Section:
ISA 65x65x8 indicates equal angle 8 mm thick with legs 65 mm each.
ii. Unequal angle section:
ISA 100x75x10 indicates unequal legs of size 100 mm x 75 mm with 10 mm thick each.

4.Rolled Steel T-Sections:
ISNT Normal T-Section
ISHT Wide Flange Tee
ISST Shoft Flange Tee
ISLT Right Weight Tee
ISNT 100 @15kg/m where depth is 100 mm.

5.Rolled Steel Bar Sections:
i.Round bars ISRO
ii. Square bars ISSQ

6.Rolled Steel Plates or Strips:
i. Plates- thicknesss ≥ 5mm and width ≥ 600 mm
ii. Strips -thicknesss < 5mm and width <600 mm

Loads on structures:

The basic requirement of any structure or structural component is that it should be strong enough to carry all possible types of loads to which it is likely to be subjected.

1.Dead Loads(IS:875(Part-1))

It consists of weight of all permanent constructions (walls, partitions, floors, beams, column etc.) including fixed equipment.

In case of bridges, dead load includes deck slab, wearing coat, track, side paths, railing, lighting, fixtures etc and main structural frame.

2.Live Loads(IS:875(Part-2))

a. Loads due to occupants
b. Movable machinery, equipment, furniture etc.
c. Snow load
d. Fluid pressure
e. Earth pressure
f. Earthquake forces
g. Blast forces
h. Thermal forces
i. Wind forces

Permissible stresses:

a. Direct tensile stresses
b. Direct compressive stress
c. Bending stress (tensile or compressive)
d. Shear stresses
e. Bearing stresses

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

[1] When action of different live loads is simultaneously considered, the leading live load shall be considered to be the one causing the higher load effects in the member/section.
[2] This value is to be considered when the dead load contributes to stability against overturning is critical or the dead load causes reduction in stress due to other loads.

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

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

Design philosophies:

Design of steel structures consists of the design of steel members and their connections so that they will safely and economically resist and transfer the acting loads.

Working stress method:

It is the elastic method of design. The worst combination of working loads is ascertained from code and the members are proportioned on the basis of working stresses. These stresses should never exceed the permissible stresses laid down by the code.

Permissible stress is the ratio of the yield stress to the factor of safety.

It is also defined as the ratio of strength of the member to the expected force.

Permissible stress = f_y/ F.S, where F.S is factor of safety.

Plastic Method:

Steel is the ductile material and from stress strain diagram, structure can take much higher loads than load at elastic limit. This is due to the fact that a major portion of the curve lies beyond elastic limit. This extra strength is termed as “Reserve strength”. This method is based on failure conditions rather than working load conditions.

In this method of design, failure occurs at extreme large deformations, and the structure fails at a much higher load called collapse load.

In plastic method, working loads are multiplied by load factors and dimensions of the section are when the entire cross-section becomes plastic, infinite rotations take place and the plastic hinge is formed. When sufficient plastic hinges are formed in the structure at the maximum stressed locations, a collapse mechanism is formed. Since the actual loads will be less than the collapse loads by a factor of safety, the members designed will be safe.

Limit State Method:

Limit state method is similar to plastic design which considers most critical stage of

1. Strength
2. Serviceability

The acceptable limit for the safety and serviceability requirements before failure occurs is called a limit state.

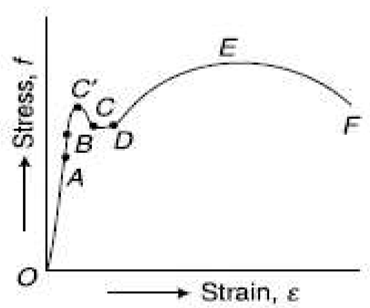
Objective:

To achieve a structure that it will not become unfit for used with an acceptable target reliability in general, structures should be designed on the basis of the most critical limit state and should be checked with other limit states.

The design should be based on characteristic values for material strengthening and applied loads, which take into account the probability of variations in the material strength and in the load to be supported.

The criterion to be satisfied in the selection of a member is Design action ≤ Design strength.

Stress- strain curve for mild steel under tension:

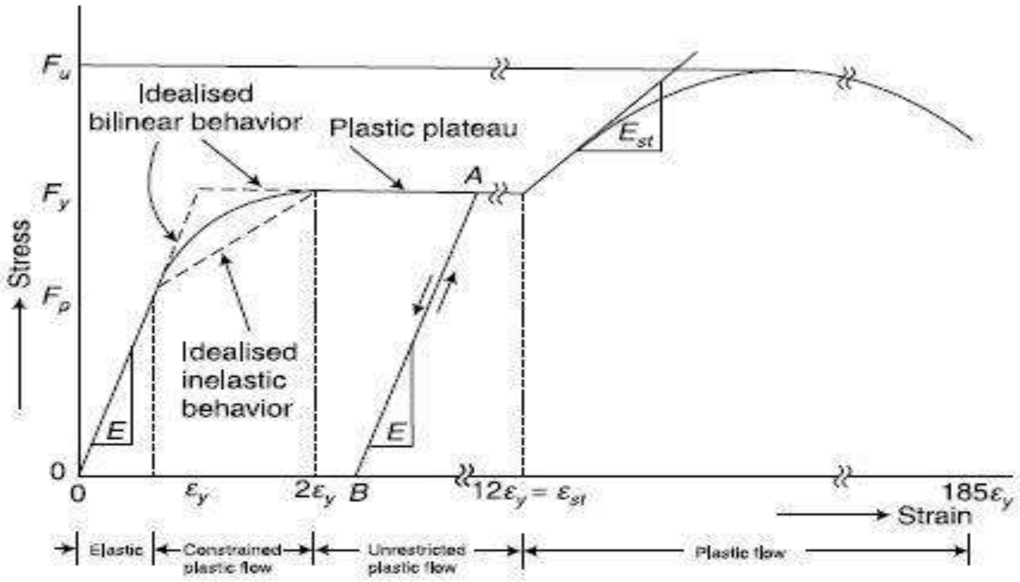


Engineering stress-strain curve

For most of the structural steels, point A & B are so close that limit of proportionality and upper yield points are almost same.
Two types of stress – strain curves:
Engineering stress – strain curve is based on the original C.S area and gauge length to find the stresses and strains respectively.
Trust stress – strain curve is based on the actual reduced area and increasing gauge length at the instant of recording.
Range CD is plastic range in which deformations continue to increase without any increase stress. Due to strain hardening, the material further gains capacity to absorb more stress.

ε_y – Strain correspond to yield stress
E_y – Modulus of elasticity within Elastic limit during strain hardening portion
ε_{st} – Strain at the onset of strain hardening
E_{st} – Tangent Modulus of steel at the onset of strain hardening
Normally E_{st} = 15 times E_y and E_{st} = $^1/_{30}$ th of E_y
CD – is known as plateau strain range
f_y – yield stress 250 Mpa (N/mm^2)
f_u – Unlimited stress 410 Mpa
E_y = 0.0012 i.e 0.12%
E_{st} = 0.015 i.e 1.5% (15 times E_y)
E = 2x10^5 N/mm^2 E_{st} =6700 N/mm^2 (30 times of E)

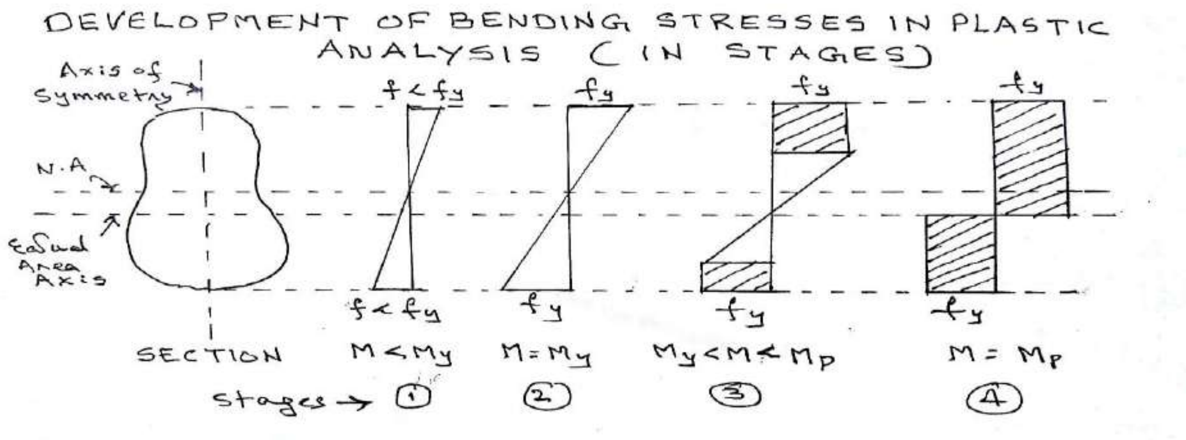
In plastic Analysis the stress – strain curve shown below is modified one and known as idealized curve



Idealised stress–strain curve

1. Proportional limit and lower limit area same
2. AB is plastic range also known as **" Plateau of yield"**
3. Effect of strain hardening is ignored and hence plastic flow in material due to yielding of C.S and producing excessive deformations is considered as indicative of failure

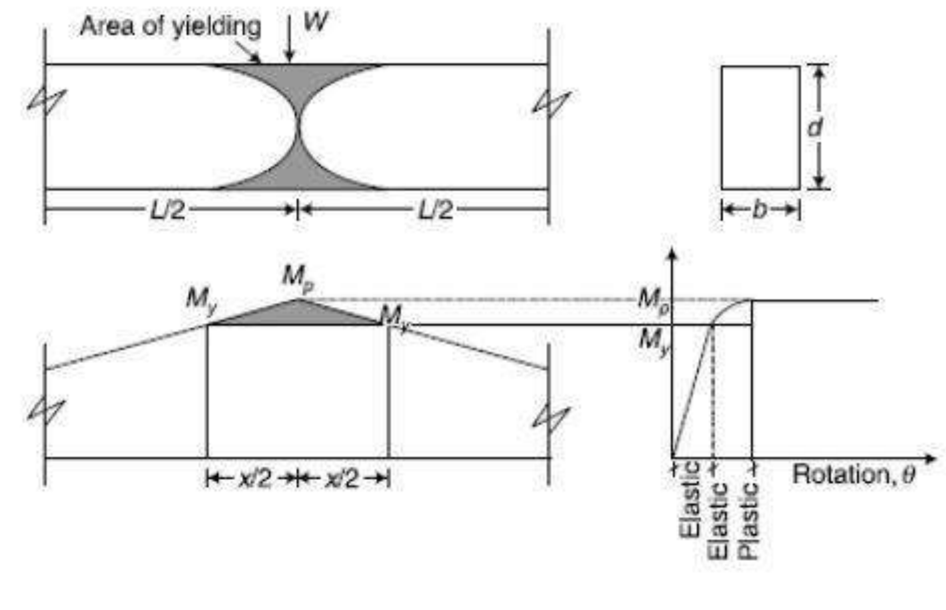
Plastic hinge:



Plastic hinge is a zone of yielding due to flexure in a structural member.

A structure can support the computed ultimate load due to formation of plastic hinges at certain critical sections (like at point loads, end of member meeting at a connection, change in geometry etc.). The member remains elastic until the moment reaches a value M_p (the maximum moment of resistance of a fully yielded cross-section)

Any additional moment will cause the beam to rotate with little increase in stress and this rotation occurs at constant moment (M_p). ∴ **"Plastic hinge"** is defined as an yielding zone due to flexure in a structure in which infinite rotation can take place at a constant restraining moment (M_p) of the section.



Development of plastic hinge and hinge length

Limit state design of steel structures:

Limit state methods of design was developed to take account of all conditions that can make the structure unfit for use, considering actual behaviour of materials and structures. Limit state method of design also known as Load and Resistance Factor Method.

There are two limit states: 1. **Strength** 2.**Serviceability**

The acceptable limit for the safety and serviceability requirements before failure occurs is called a "**limit state**".

1. **Strength limit states** are based on load carrying capacity of structures including plastic strength, buckling, fracture, fatigue, overturning etc.
2. **Serviceability limit states** refer to the performance of the structures under service loads which include deflections, vibrations, deteriorations, corrosion etc.

In limit state design, basically statistical methods have been used for determination of loads and material properties with a small probability of structure (5%) reaching the limit states of strength and serviceability.

Characteristic Load and Characteristic Strength:

- In normal design calculations, we generally use a single value for each load and for each material property, with a margin to take care of all uncertainties. Such a value termed as **Characteristic Strength or Characteristic load**
- The **Characteristic yield Strength** of steel is the value of yield strength below which not more than 5% probability, during the life span of a structure.

Design criteria:

In limit state design, for ensuring the design objectives, the design is based on characteristic values of material strengths and applied loads (actions), which take into account the probability of variations in material strengths and the loads to be supported.

Partial safety factors are

1. γ_f - Partial safety factor applied to loads.
2. γ_m- Partial safety factor applied to material strength

The reliability of design is expressed as

Design action (Q_d) ≤ Design strength (S_d)

Q_d - The load effects on the structure (External)

S_d - Resistance or capacity of structure (Internal)

Design action $(Q_d) = \sum \gamma_f \, Q_{ck}$ ....(1)

Design strength $(S_d) = S_u / \gamma_m$ ....(2)

Where, Qck -Characteristic load (action)

S_u -Ultimate strength

<u>Types of connections:</u>

A steel structure is an assembly of various components which are fastened together through connections. If connections are not designed properly and fabricated with care, they may be the source of weakness or failure. Design of main members has already well defined and developed with established theories.

The following are the good requirements of connection:

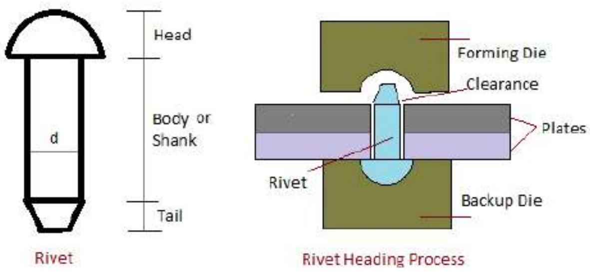
1. It should be rigid, to avoid fluctuating stresses which may cause fatigue failure.
2. There should be least possible weakening of the parts to be jointed.
3. It can be easily installed, inspected and maintained.

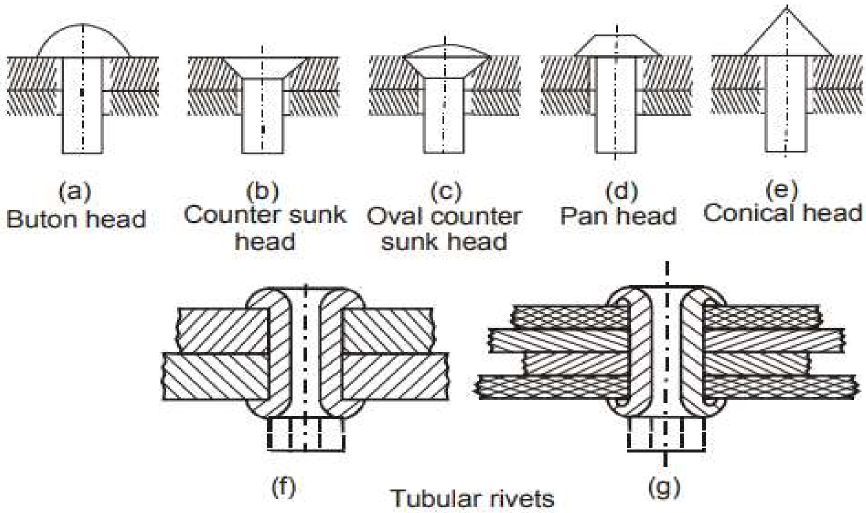
The following are the common connections used in structural steel work:

1) Riveted 2) Bolted 3) Welded

Definition of riveted joint:

Riveted joint is permanent joint with mainly two components (parts to be joined) which are held together by a rivet with the head at top and tail at the bottom.





Rivet Heading Process (Riveting):

Rivet heading process is done with the help of forming die and backup die which keep a rivet in between them and by application of force, rivet is set in the parts to be joined.

Equal and opposite force makes rivet to deform and tail part of rivet is converted to head at bottom so, complete rivet is seated in the plates.

In this riveting process, tail of rivet is converted to 'head' which is sometimes called 'shop head'.

For riveting parts to be joined are first drilled with the help of drilling machine. Clearance is taken into consideration while riveting because by pressing application diameter of rivet is somewhat increased.

Usually clearance is considered as per following:

If diameter of rivet, d = 12 to 24 mm, Clearance, C = 1.5 mm

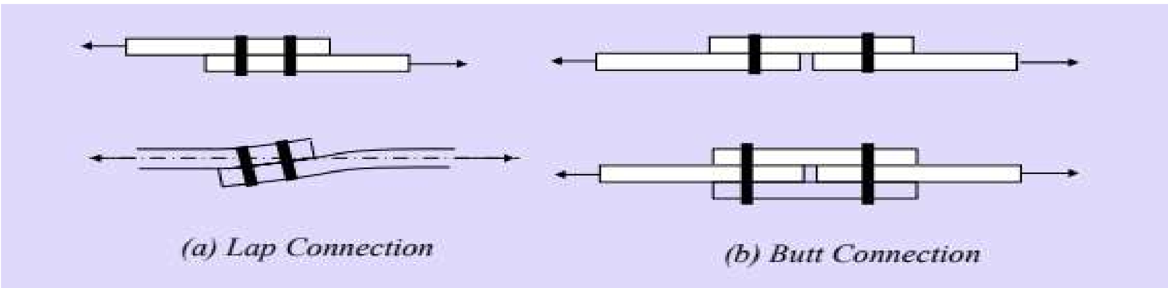
If diameter of rivet, d = 24 to 48 mm, Clearance, C = 2 mm

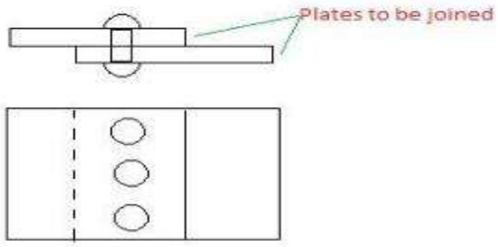
Types of riveted joints:

There are mainly two types of riveted joints, based on the rivet arrangement.

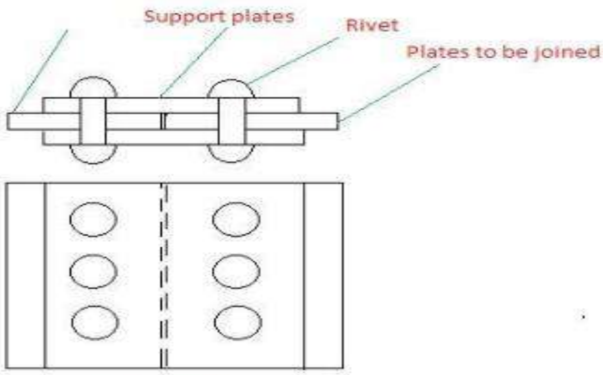
1.Lap joints

2.Butt joints





LAP JOINT



BUTT JOINT

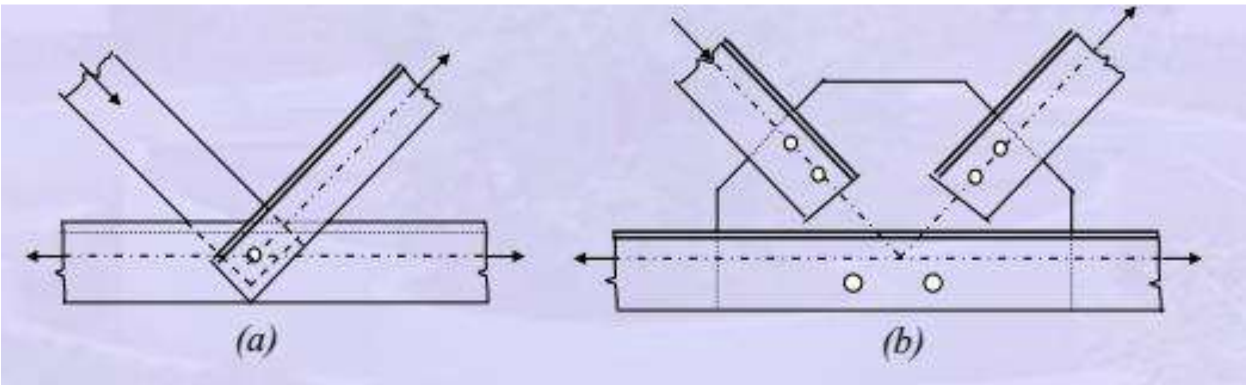
Both joints are also sub-classified into single riveted and double riveted. Sometimes based on joints strengths, triple riveted are also possible. Single riveted means one row of the rivet in joint

Advantages of riveted joints:
Cheaper fabrication cost
Low maintenance cost
Dissimilar metals can also be joined, even non-metallic joints are possible with riveted joints.
Ease of riveting process.

Disadvantages of riveted joints:
Skilled workers required
Leakage may be a problem for this type of joints, but this is overcome by special techniques.

Bolted Connections:
Bolting is the preferred method of connecting members on the site. Staggered bolt layout allows easier access for tightening with a pneumatic wrench when a connection is all bolted. High strength bolts may be snug-tightened or slip-critical. Snug-tightened connections are referred to as bearing connections. Bolts in a slip-critical connection act like clamps holding the plies of the material together. Bearing type connections may have threads included (Type N) or e xcluded (Type X) from the shear plane(s). Including the threads in the shear plane reduces the strength of the connection by approximately 25%. Loading along the length of the bolt puts the bolt in axial tension. If tension failure occurs, it usually takes place at the threaded section.

Concentric Connections
- The bolted connections are referred to as concentric connections (force transfer in tension and compression member)
- Eccentric connections (in reaction transferring brackets) or moment resisting connections (in beam to column connections in frames). This connection is found in moment resisting frames where the beam moment is transferred to the column. The connection is also used at the base of the column where a base plate is connected to the foundation by means of anchor bolts.



Shear connections:
Shear connections are the most prevalent type of connections in a steel frame building. Shear connections are called simple connections – Since they are assumed not to transfer bending moment, thus allowing end rotation of the member. Shear connections may be made to the web of the supported member while the flanges remain unconnected. Seat or hanger connections are the only type of shear connections that connect to the flange of the supported beam. Angles for shear connections may be attached to supporting members by bolting or welding. Although single plate connections, are the most economical, they must sometimes be evaluated for eccentricity. ƒ Single angle connections allow end-rotation for flexible connections. ƒ Single angle connections tend to have lower load capacities than double-angle connections.

Moment connections:
Moment connections are also called rigid connections. Moment connections carry a portion or the full moment capacity of the supported member thus preventing any end-rotation of the member. Moment connections are typically designed to also carry the shear component of the load.

Welded connections:
Welding consists of joining two pieces of metal by establishing a metallurgical bond between them. The elements to be connected are brought closer and the metal is melted by means of electric arc or oxyacetylene flame along with weld rod which acts metal to the joint. After cooling, the bond is established between two elements. Moment connections provide continuity between the supported and supporting members. ƒ Relative rotation between the supporting and supported members is negligible. ƒ The flanges of the supported member are attached to either a connection element or directly to the supporting member.

Advantages of welding
1. Due to absence of gusset plate, connecting angles etc., welded structures are lighter
2. The absence of making holes for riveting etc., makes welding process quicker
3. Welding is more adoptable than bolting or riveting (circular tubes can also be connected by welding)

4. It is possible to achieve 100% efficiency in welded joint, where as in bolted and riveted connection it can be up to 70-80% only
5. Noise produced in welding relatively less
6. Welded connections have good aesthetic appearance
7. Welded connection is air tight and water tight. Hence less danger for corrosion and leakages (particularly making water tanks).
8. Welded joints are rigid
9. mismatching of rivet holes creates considerable problem
10. Alterations in connections can be easily made in design of welded connections

Disadvantages of welding

1. Due to uneven heating and cooling, members are likely to distort in the process of welding.
2. There is a greater possibility of brittle fracture in welding.
3. A welded joint fails earlier than bolted joint, if the structure is under fatigue stresses.
4. Inspection of welded joint is difficult and expensive. It needs non-destructive testing.
5. Highly skilled person is required for welding.
6. Proper welding in field conditions is difficult.
7. Welded joints are rigid.

Design of stresses in weld:

Butt welds or Groove weld:

Butt welds shall be treated as parent metal with a thickness equal to the throat thickness, and the stresses shall not exceed the stresses permitted in the parent metal.

Design strength of groove weld in tension or compression:

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

Where f_y –Small or ultimate strength of weld or parent metal inMPa

L_w – Effective length of weld (mm)

t_e – Effective throat thickness of weld (mm)

γ_{mw} – Partial safety factor (table 5 of IS 800)

Design strength of groove weld in shear:

$$V_{dw} = \frac{f_{yw} L_w t_e}{\sqrt{3}\gamma_{mw}}$$

Where f_{yw} - characteristic yield stress of weld material

γ_{mw} – Partial safety factor (table 5 of IS 800)

Design strength of groove weld in bending:

$$F_{dw} = 1.2\frac{f_y Z_e}{\gamma_{mo}}$$

γ_{mo} - Partial safety factor against yielding (table 5 of IS 800)

Design strength of fillet weld

The design strength of fillet weld is based on its throat area and is given by

$$P_{dw} = \frac{f_u L_w t_e}{\sqrt{3}\gamma_{mw}}$$

(or)

$$P_{dw} = \frac{f_u L_w K * S}{\sqrt{3}\gamma_{mw}}$$

Where S- Size of the weld in mm

K – Reduction factor

f_u – ultimate strength of the weld (N/mm^2)

Length of the fillet weld

Assume size of the weld

Length of fillet weld= $(\sqrt{3}\gamma_{mw} \text{ x } P_{dw}) / (f_u t_e)$

DESIGN AND DRAWING OF STEEL STRUCTURES

UNIT-2

Tension and compression Members

Syllabus:

General Design principles of members subjected to direct tension - Design of tension members – Effective length of columns – Slenderness ratio – Permissible stresses – Design of compression members, struts & etc.

Learning Outcomes:

Students will be able to

- explain the concept of “effective net area”, “Net section” and “shear lag factor”.
- understand the failure modes of a tension member.
- design the members subjected to direct tension using I.S code.
- apply the design principles of compression members
- explain the effective length and slenderness ratio of column
- design the composite section of a column

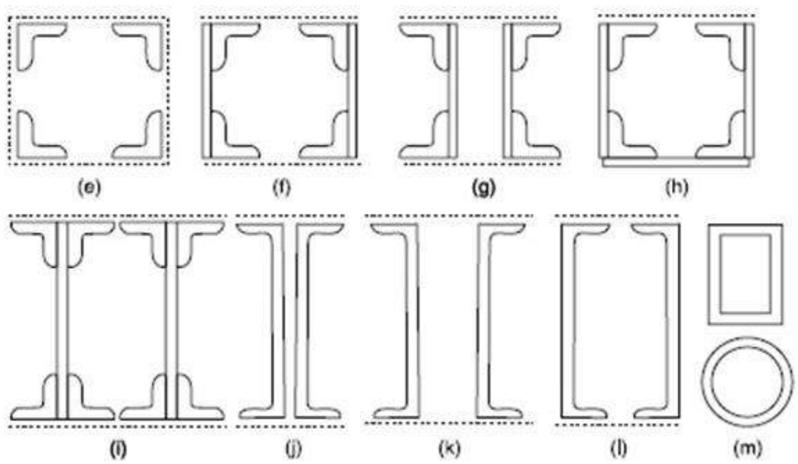
Learning Material

Tension Members

A Structural member subjected to two pulling forces applied at its ends is called a tension member. They are also known as tie members. Due to tension, bending stresses are minimized. However the efficiency of a tension member is affected due to end connections with rivets/bolts. For ductile materials, a uniform stress distribution assumption is reasonable when the material is loaded beyond the yield stress. Although the stress concentration near the holes/welds leads to the yielding of nearby fibres but the ductility of steel permits the redistribution of overstress in the adjoining sections till the fibres away from the holes/welds progressively reach yield stress.

Examples of tension members in some structural elements are

- in trusses, bridges, communication towers,
- guy wires in steel stacks,
- ties in lattice girders,
- bracing system in multi-storeyed buildings,
- main cables and suspended cables in suspension bridges,
- Hangers supporting floor beams etc.

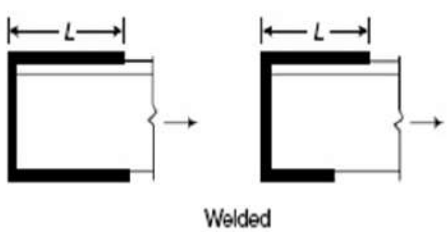


Shapes of tension members

Shear lag factor

The effectiveness of a tension member depends on the manner in which it is connected.

For members other than flat or bar, until fracture occurs across its critical section, its actual tensile stresses will be reduced since the stresses at connections are not transferred uniformly.



Types of Failure

Gross-Section Yielding: Considerable deformation of the member in longitudinal direction may take place before it fractures making the structure unserviceable.

Net-Section Rupture: The rupture of the member when net cross-section of the member reaches the ultimate stress.

Block-Shear Failure: A segment or block of material at the end of the member shears out due to high bearing strength of steel and high-strength bolts resulting in smaller connection length.

Design Criteria

$T \leq T_d$

T- Factored design tensile load

T_d-Design strength of the member

Design stepwise procedure of tension members

An initial estimate of the area is made from the following conditions (1) and (2) and the larger one is taken as the initial size estimate.

1. Gross section yielding:

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

2. Net section fracture:

   For plates and threaded rods $T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}}$

   For angles, etc. $T_{dn} = \frac{\alpha A_n f_u}{\gamma_{m1}}$

Once the trial shape is selected, the section is checked for slenderness ratio limit, gross section yielding, net section fracture, and block shear failure. The step-by-step procedure for the design of tension member subjected to axial load is as follows:

1. The net area required A_n to carry the factored load T is obtained by,

$$A_n = \frac{T}{0.9 f_u/\gamma_{m1}} \quad \text{or by} \quad \frac{T}{\alpha f_u/\gamma_{m1}}$$

as appropriate. Where T is the factored design load, f_u is the ultimate strength of the material, A_n is the net area of cross section, a = 0.6, 0.7 or 0.8 as appropriate and g_{m1} = 1.25.

2. The net area calculated thus is increased suitably (10% – 25%) to compute the tentative gross sectional area.
3. The trial gross area is also determined from its yield strength by

$$A_g = \frac{T}{f_y/\gamma_{m0}}$$

where f_y is the yield strength of the material and g_{m0} = 1.1.

4. From *IS Handbook No.1*, a suitable rolled section/built-up section providing a cross-sectional area matching with the computed gross-sectional area is selected.
5. The number of bolts (or weld) required to make the connection is calculated. These are arranged in a suitable pattern and the net area of the section provided is calculated. Thereafter, effective net area is determined if the section selected is not connected with all of its elements.
6. The design strength T_d of the trial section is calculated. This will be minimum of the strengths T_{dg} Eq. (6), T_{dn} Eq. (8)

The design strength of the member under axial tensile load should be the least of

Design strength due to yielding of gross-section (T_{dg}) – clause 6.2

Design strength due to Rupture of critical net – section (T_{dn}) – clause 6.3

Design strength due to Block shear (T_{db}) – clause 6.4

Compression Members

A compression member is a structural member which is straight and subjected to two equal and opposite compressive forces applied at its ends. Compression members are designated depending upon their position in structures. Columns, stanctions or posts are vertical compression members supporting floor or girders in a building. Strut is a compression member used in roof trusses and bracings. Principal rafter is the top chord member in roof trusses and boon is the principal compression member in a crane

Columns are classified as long, short or intermediate depending upon their slenderness ratios.

Excessive compression in long columns may cause yielding or buckling.

As the compression load on column increases, it causes some eccentricity and in turn causes some bending moment which causes further to deflect or buckle.

In general long columns fail by elastic buckling, intermediate columns fail by inelastic buckling and short columns fail by crushing or yielding.

Effective length

The form of curve into which a compression member tends to deflect depends upon the mode of end fixture. The effective length 'l' of a compression member is the distance between the two points of contra flexure. Since the precise determination of the effective length is difficult, Table-11 of IS code provides the effective length (KL) of columns and struts respectively for various end conditions.

Smaller the effective length, smaller is the danger of lateral buckling and the greater is its load carrying capacity.

Slenderness ratio(λ)

The tendency of the member to buckle is usually measured by slenderness ratio

$$\lambda = \frac{Effective\ length}{least\ redius\ of\ gyration} = \frac{le}{rzz\ or\ yy} = \frac{KL}{r}$$

For minimum steel requirements in column design, the slenderness ratio should be kept as smaller as possible.

This can be achieved by selecting a section which provides the largest value of minimum radius of gyration without providing more area of steel or by reducing the unsupported length of the member by some means.

Types of Buckling

Flexural Buckling:

Flexural Buckling also called as Euler's buckling. It is a deflection caused by bending about the axis corresponding to the largest slenderness ratio. This is usually the minor principal axis i.e., the one with smaller radius of gyration.

Torsional buckling

Thin wall members with open cross-section shapes are sometimes weak in torsion and hence may buckle or twist rather than bending. Torsional buckling occurs when torsional rigidity is smaller than the bending rigidity.

Flexural-torsional buckling

It causes by combination of both flexural buckling and torsional buckling. This failure can occur only with unsymmetrical cross-sections such as channels, structural tees, double angle shapes, equal single angles etc.

Design of axially loaded compression members

Step-1: As trial and error method, for an average column height of 3 to m, the slenderness ratio will fall between 40 and 60.

If the column is a longer one, little higher than 60 may be assumed.

Further if the column containing is very heavy factored loads, a smaller value of slenderness ratio should be assumed.

Such columns will require very large radii of gyration.

Step-2: For assumed slenderness ratio above, the design compressive stress (fcd) is determined from table- 9 (a) to (d) for appropriate curve as given in table 10.

Step 3: The cross-sectional area required to carry factored load at the assumed compressive stress is computed as

$$Ag = \frac{Pu}{assumed\ compressive\ stress}$$

Step4: A section that provides the required Ag is selected from steel tables.The section is choosen that r_{min} should be as large as possible.

Step5 : The effective length of the member is calculated on the basis of end conditions from table-11 and slenderness ratio is computed which should be less than the permissible slenderness ratio.

Step6: For estimated slenderness ratio, f_{cd} is calculated from table-9(a) to (d)

Step-7: For single angle section loaded concentrically the design strength is determined using the eqn.

$f_{cd}=\frac{fy/\gamma_{mo}}{\phi+[\phi 2-\lambda 2]^{0.5}} = \frac{\chi fy}{\gamma_{mo}} \leq \frac{fy}{\gamma_{mo}}$

where

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

λ= non-dimensional effective slenderness ratio

However, a single angle, when loaded through one of its leg is subjected to flexural torsional buckling, the equivalent λ is such as a case

$\lambda_e= \sqrt{(k1 + k2\lambda vv^2 + k3\lambda_{\phi}^2)}$

where k1,k2,k3 = constants depending upon the end conditions as given in table-11

- IS: 800-2007 proposes multiple column curves (a to d) fig-8 pg. 35 based on Perry- Robertson approach. These curves are for different cross-sections of columns to account for initial imperfection of their geometry.
- Alternatively, the code also recommends the following equation for estimating the design compressive stress, fcd of axially loaded compression members. It considers the residual stresses, initial bow and accidental eccentricities of the load.

$f_{cd}=\frac{fy/\gamma_{mo}}{\phi+[\phi 2-\lambda 2]^{0.5}} = \frac{\chi fy}{\gamma_{mo}} \leq \frac{fy}{\gamma_{mo}}$

where

ϕ=0.5[1+α(λ-0.2)+ λ^2]

λ= non-dimensional effective slenderness ratio

f_{cc}= Euler buckling stress = $\frac{\Pi^2 E}{(\frac{KL}{r})^2}$

$\lambda = \sqrt{\frac{fy}{fcc}} = \sqrt{fy(\frac{KL}{r})^2/\Pi^2 E}$

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

α= imperfection factor given in table 8.5

$\chi = \frac{1}{\phi+[\phi 2-\lambda 2]^{0.5}}$ stress reduction factor (given in table 8.7 for different buckling curves)

γ_{mo}= Partial safety factor for material strength.

Design strength of column

The design strength of member subjected to axial compression are classified as buckling cuves a,b,c & d.

The design compressive strength P_d= $A_e f_{cd}$

Where A_e = effective sectional area

f_{cd}= design stress in compression

The stress reduction factor (χ) and the design compressive stress f_{cd} for different buckling curves, yield stress and the effective slenderness ratio are given in table-8(a) to(d) and 9(a) to (d) from pg-36 to 43 of IS-800:2007 code. $\lambda vv = \frac{\frac{l}{r_v}}{\epsilon\sqrt{\frac{\Pi^2 E}{250}}}$ and $\lambda\phi = \frac{(b1+b2)}{\epsilon\sqrt{\frac{\Pi^2 E}{250}} * 2t}$

l=centre to centre length of the supporting member

r_v= radius of gyration about the minor axis

b1,b2 = width of the two legs of the angle

t=thickness of the leg

ϵ=yield stress ratio $({}^{250}/{}_{fy})^{0.5}$

The design strength of the member is computed by multiplying the design compressive stress thus obtained with the effective C.S area provided.

The value of the design compressive strength Pd > Pu

DESIGN AND DRAWING OF STEEL STRUCTURES

Unit-3

Syllabus: Design of Column Foundations

Built-up compression members, design of lacing and battens, design of eccentrically loaded columns, splicing of columns, design of column foundations, types of column bases- design of slab base and gusset base, column bases subjected to moment.

Learning Outcomes:

Students will be able to

- explain the importance of lacing/battened systems to built-up columns.
- design the built-up compression members with lattice systems like lacing and battens.
- explain the concepts of splicing of columns.
- apply the knowledge of design concepts of eccentrically loaded columns.
- apply the principles of design to column bases like slab base and gusset base.

Learning Material

Built-up columns:

When the size and shape of the standard rolled steel sections are limited and the available sections cannot accommodate the required sectional area and when a large radius of gyration is required in two difficult directions, built-up sections are preferred.

For economical design of heavily loaded columns, the least radius of gyration of column section is increased to maximum ($r_y \geq r_z$). To achieve this condition, the rolled sections are kept away from the centroidal axis of the column and are connected by connecting system known as "Lattice System."

Lattice systems are

i. Lacing
ii. Batten plates

Lacing:

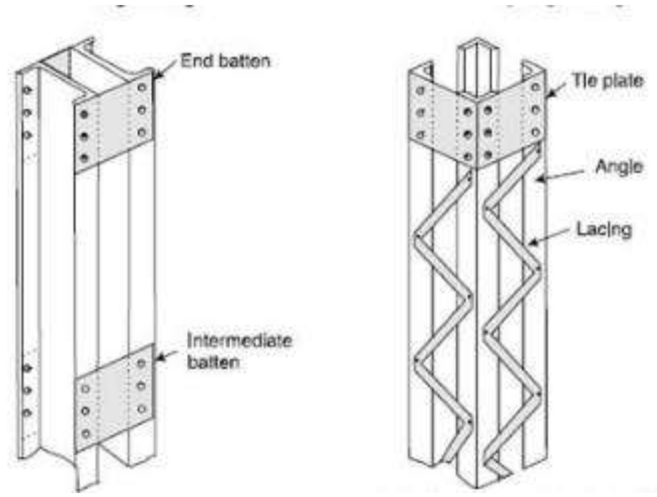
When lacings are provided with battens, the shear component of the axial load tends to expand the column laterally.

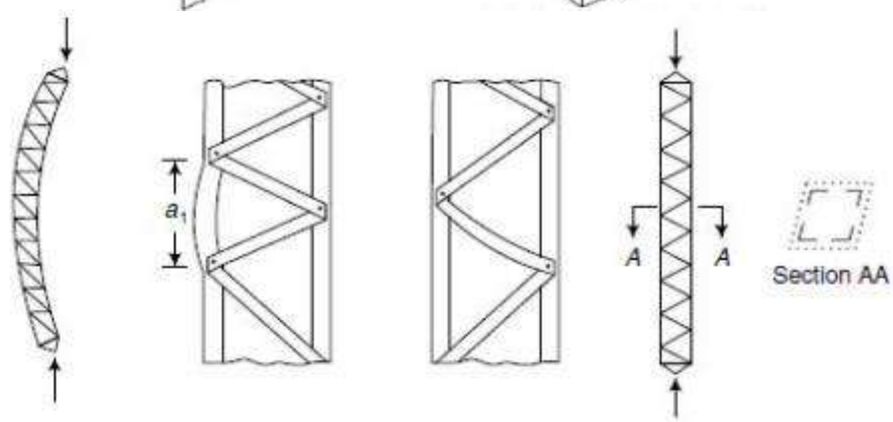
Because of lacing, the lateral expansion of the columns must accompany shortening under load, if there is shortening of lacing bars.

The introduction of battens restrain this action and the lacing system with battens behaves like a truss which may result in large stresses leading to failure; the lacings are forced to share the axial compressive load in the strut.

Since lacings and battens are not designed as load carrying elements, and since their primary function is to hold and keep the main members of built-up column in position and equalize the stress distribution between them, a system comprising of lacing with battens is not recommended.

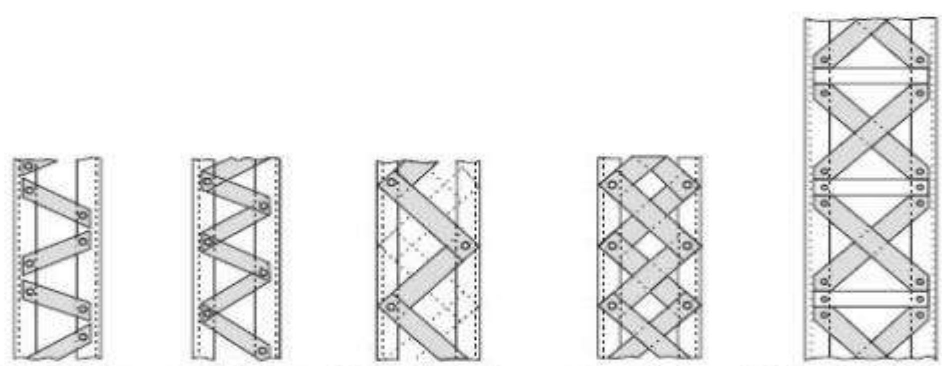
In the design of built-up columns, the following conditions are considered by four angle lattice column.





(a) Failure of column as a whole (b) Failure of main component (c) Failure of lacing (d) Distortion of cross section

Flat or angles are normally used as lacings. Usually they are connected with single rivet or bolt at the ends.



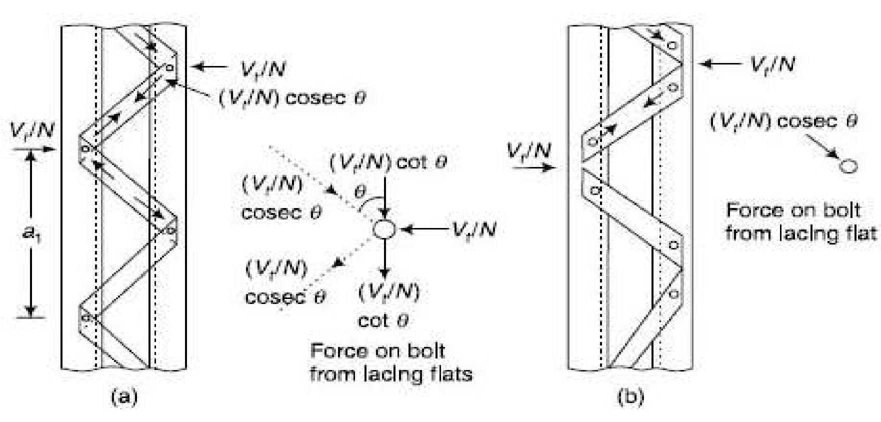
(a) Single flat lacing non-intersecting (b) Single flat lacing inter-secting (c) Single flat lacing, non-mirror image (not recommended) (d) Double flat lacing intersecting (e) Double flat lacing with batten (not recommended)

Design of lacing columns:

1. The design compressive stress (f_{cd})is assumed between 125 and 175 MPa.
2. Required cross-sectional area to carry P_u is assumed as A= $\frac{p_u}{\text{fcd}}$
3. A suitable section comprising of two channels or four angles or two I-sections with or without extra plates as required may be selected (from steel tables).
4. The sections are spaced such that the radius of gyration of section about the axis perpendicular to the plane of lacing is not less than the radius of gyration about the axis in plane of lacing.(r_{xx})
   This can be achieved by making r_y=r_z
5. Estimate effective length (KL) and slenderness ratio (KL/r) from IS:800
   The effective slenderness ratio of the laced column should be taken as 1.05 times the $\left(\frac{KL}{r}\right)$ actual maximum slenderness ratio in order to account for shear deformation effect.
6. Obtain the f_{cd} from buckling curves IS:800
   Design capacity of member is A_e* $f_{cd} > p_u$
7. Angle of inclination of lacing bar with longitudinal axis of the component member should be kept between 40-70 degrees. Initially it may be assumed as 45^o and the spacing of the lacing bars a_1 is calculated. The maximum spacing of the lacing bars should be such that the minimum slenderness ratio of the compound member $\left(\frac{a_1}{r_y}\right)$ is not greater than 50 or 0.7 times the slenderness ratio of the member as a whole.
   Where a_1–length of compression member.
   r_y= radius of gyration about y axis.
8. The lacing for compression member should be proportioned to resist a total transverse shear V_t equals to 2.5% of axial force of the column, V_t is divided equally in all parallel planes 'n' in which there are shear resisting elements such as lacing.
   V_t / n is the shear in each lacing.
   Normally for channel sections,N=2
   For four angle sections, N=4
   Compressive force on lacing bar = $\frac{v_t}{n}\text{cosec}\,\theta$ for single lacing system

   $\frac{v_t}{2n}\text{cosec}\,\theta$ for double lacing system

9.
10.
11.



12. Section of lacing bar

i. Initially lacing flat size is assumed

The minimum width of the flat should not be less than 3 times the dia of the connector assumed. (dia of bolt)

ii. The thickness of lacing t $\nless \frac{1}{40}$ of its effective length

For double lacing, t $\nless \frac{1}{60}$ of its effective length

iii. The minimum 'r' of the flat is $\frac{t}{\sqrt{12}}$

iv. The slenderness ratio of the lacing bar should be < 145

<u>In welded connection,</u> the effective length of the lacing bar is 0.7 times the distance between the inner ends of weld. Assume the size of the weld.

13. The design compressive strength of the lacing is then computed for λ of lacing should be > Pu of flat.

The tensile strength of the flat is also calculate > Pu of the flat.

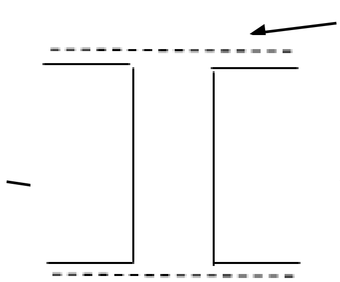
14. In case of welding of lacing bars, the welding should be done along the each side of lacing bar for the full length of the lap.

Overlap of the lacing bar $\nless$ 4 times the thickness of the bar or the member whichever is less.

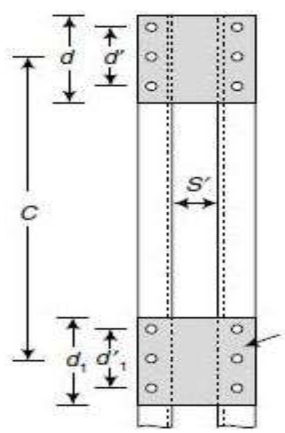
15. To check the distortion of the column section, at the column ends, tie plates are provided at the ends of the lacing system and should be able to resist the forces to which the lacing flat is subjected. This tie plate is designed as batten plate.

Design of battened column

Battens



Channel section



Battens are the plates connected to the main components of the compression members.

They are placed in the opposite sides to each other on the two parallel faces of the compression member. Battens are provided at ends and also at the intermediate levels.

Design Steps:

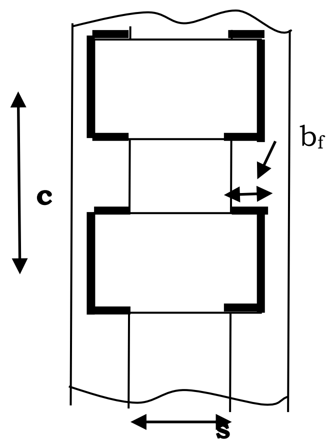
1. Column sections with battens are designed similar to a laced system. Assume the design compressive strength 120-150 MPa.
2. The effective slenderness ratio $\lambda_e = 1.1\left(\frac{KL}{r}\right)$
   Where KL/r is the actual maximum slenderness ratio.
3. The spacing of battens 'c' should be such that the minimum slenderness ratio of the component member $\left(\frac{C}{r_y}\right)$ is not greater than 50 or 0.7 times the slenderness ratio of the member as a whole, about the axis parallel to battens.
4. The effective depth of end battens should not be less than the distance between C.G of component members and should be more than twice the width of one component member. The depth of intermediate batten is taken as ¾ of effective depth of end batten and should be more than twice the width of one component member.
   Where two channels are placed back to back, spaced at S apart and connected by battens.
   Effective depth of end battens, $d' = S' + 2\ C_{yy}$
   Overall depth of end battens, d = S' + 2 edge distance
   Effective depth of intermediate battens, $d_1' = \frac{3}{4}d'$
   Overall depth of intermediate batten, $d_1 = d_1'$ + 2 edge distance
5. The thickness t of battens should be not less than 1/50 of distance between the inner most connecting lines of bolts or welds perpendicular to the main member.
   $$t = \frac{1}{50}\left[s' + 2g\right]$$
   Where g- gauge distance from IS code.
6. Battens should be designed to carry B.M and shear due to transverse shear force V_t which is 2.5% of total axial force on the main compression member. This V_t shall be divided equally in all parallel planes 'n' in which there is shear resisting elements like battens or continuous plates.
7. Battens should also resist the longitudinal shear and moment arising from the transverse shear V_t.
   $V_b = \left(\frac{V_t C}{N.S}\right)$ and $M = \left(\frac{V_t C}{2N}\right)$ for each connection ; $V_b .s = V_t .c$
   Where V_b- Longitudinal shear force
   V_t – Transverse shear force
   c- cross-section distance of battens
   S- Minimum transverse distance between centroid of bolts / welds.
8. Shear stress $\left(\frac{V_b}{A1}\right)$ is calculated in the section of battens and it should be less than $\left(\frac{f_y}{\sqrt{3}\gamma_{m1}}\right)$
   Where A_1- Area of the cross section of batten

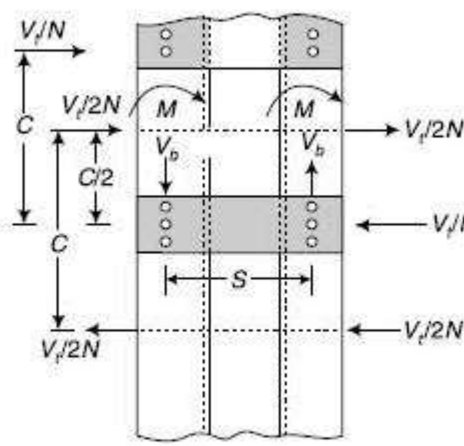
γ_{m1} - Partial safety factor
t- thickness of batten
d – Overall depth of batten

9. Bending stresses in batten should be less than $\left(\frac{f_y}{\gamma_{m1}}\right)$

$\sigma_{bc,cal} = \frac{M}{Z} = \frac{M}{\frac{1}{6}td^2} = \frac{6M}{td^2} < \frac{f_y}{\gamma_{m1}}$

10. The connection bolted or welded is designed to resist the longitudinal shear and B.M in bolted connections, not less than two bolts shall be used.





Forces on battens

Welded Connection:

i. Where tie or battens overlap the main members, the amount of overlap shall not be less than 4t (thickness of plate).
ii. The length of weld connecting each edge of the batten plate to the member shall in aggregate be not less than half the depth of the batten plate.
iii. At least $1/3^{rd}$ of the weld shall be placed at each end of this edge.
iv. The length of weld and depth of batten plate shall be measured along the longitudinal axis of the main member.
v. In addition the welding shall be returned along the other two edges of the plates transversely to the axis of the main member for a length not less than the minimum lap specified.(4t)

Column Splice:

Connecting two pieces of column sections to get the required length of the column is called column splice.

Specifications for design of splices:

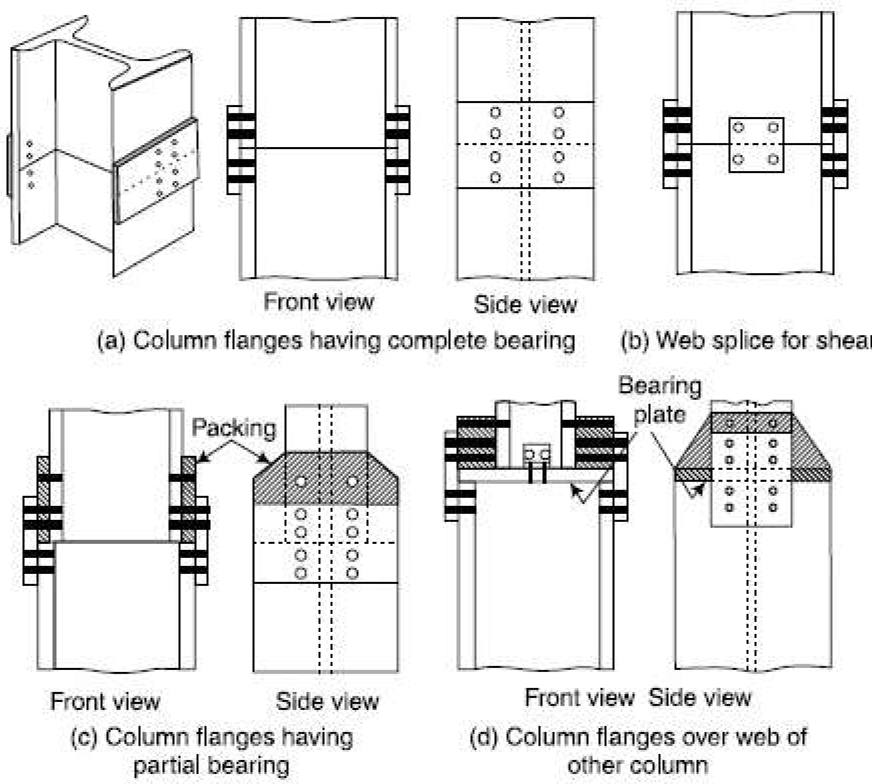
1. Where the ends of the compression members are faced for complete bearing over the whole area, these should be spliced to hold the connecting members accurately in position to resist any tension when bending is present.

2. Where such members are not faced for complete bearing, splices should be designed to transmit all the forces to which these are subjected.
3. Splices are designed as short columns.

Normally splices are located at the point of contra-flexure of the column.

When columns are restrained in direction and position this point will be at the middle of the column due to wind stresses.

In common practice to design a column of two storeys length, splice it at about 30-150 cm above first floor.



(a) Column flanges having complete bearing (b) Web splice for shear
(c) Column flanges having partial bearing (d) Column flanges over web of other column

Design:

1. Ascertain the nature of loads the splice is subjected to
   i. Axial compressive load ii. Bending moment iii. Shear
2. For axial compressive load the splice plates are provided on the flanges of two column sections to be spliced.

i) If the column has machine ends, the splice is designed to keep the column in position and to carry tension due to bending moment. Then the column splice plate and connections should be designed to carry 50 % of axial load and tension due to bending.

ii) If the ends of the column are not machined, the splice and connections are designed to resist 100 % axial load.

The load P_{u1} for design of splice and connections due to axial factored load P_u on column.

$P_{u1} = P_u / 4$ for machine ends

$= P_u / 2$ for non- machine ends

Load due to B.M, P_{u2} = Mu/ Lever arm

Where lever arm is c/c distance of two splices plates.

3. Splice plates are assumed to act as short columns (zero slenderness ratio) Plates will be subjected to yield stresses.
4. C.S area of splice plates = (Pu coming over the splices / yield stress) Width of splice plate is equal to the width of column flanges. Thickness of splice plate = t = C.S Area/ width.
5. Nominal dia of bolts for connection is assumed and strength of bolt is computed. No. of bolts, N= (Load coming over splices / Strength of each bolt)
6. In case of bearing plate is to be designed between two column sections, the length and width of the plate are kept equal to the size of the lower – storey column and thickness is computed, t= (Mu due to factored load / Moment resistance of the plate)

Note:

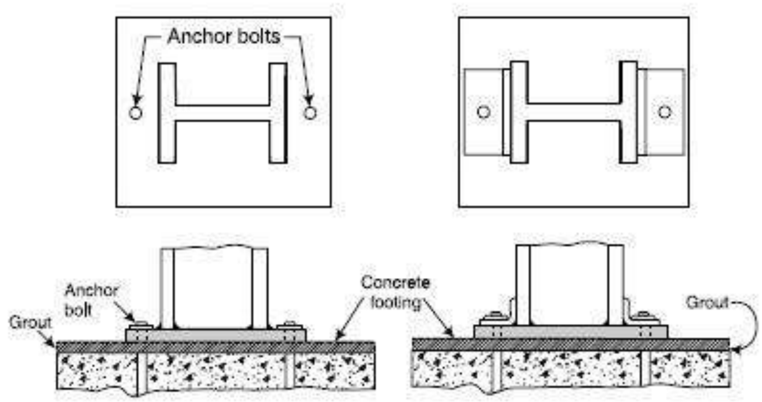
a. If the joint is subjected to both axial load and B.M, the splice plate is provided on flanges. The total load to which splice and connections are designed will be (P_{u1} + P_{u2}) Bolts are checked in tension.
b. When shear acts in addition to gravity loads, a splice plate will be provided on web (web takes the shear)
c. Splice plates should be so placed that the centroidal axis of the splice coincides with the centroidal axis of the members joined to avoid eccentricity.
d. If packings are provided, additional bolts are provided for packings.

Design of Column Bases:

A column base consists of a horizontal steel plate which is provided at the bottom most end of a column and placed on concrete pedestal.

The column base ensures that the bearing pressure between the column end and the concrete pedestal is within the permissible limit so that no crushing of concrete occurs. The base plate is usually butt welded at the column and is connected to the concrete pedestal using anchor bolt. The simplest of column bases is slab base which may be directly butt welded to the column as shown in the figure.

The slab base is connected by two anchor bolts in case of an axially loaded column and 4 anchor bolts in case of a beam-column as shown in fig.



(a) Column subjected to axial load

(b) Column subjected to axial load and bending moment

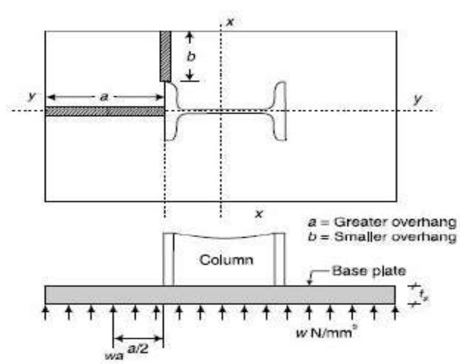
1. Slab Base:

When the column is subjected to only axial load, the base can be designed assuming a uniform bearing pressure from concrete below. For small loads, a steel plate alone, shop welded to the column, can be used to transmit the loads the column pedestal such a base plate is called "slab base"

When columns are subjected to bending moments along with axial loads, angle sections are attached to the flanges of the column. These angles are anchored with foundation bolts to the concrete pedestal. Slab bases are most suitable and economical for lightly loaded columns only.

Thickness of slab base is designed from consideration of bending of the portions of the base slab that extend beyond the column profile.

Theoretical Considerations for design:



It is assumed that bending moment occurs at the edge of the column. Also slab bends simultaneously about the two principal axes of the base slab.

Since a>b and assuming μ = 0.3 ,

For 1 mm strip of slab projection

Maximum bending moment = w x1x a x $\frac{a}{2}$ = $\frac{wa^2}{2}$

Where w- intensity of bearing pressure from concrete below.

Taking 1 mm strip slab projection along y-y axis

Max B.M = $\frac{wb^2}{2}$

Net bending moment = =$\frac{wa^2}{2} - 0.3\frac{wb^2}{2} = \frac{w}{2}(a^2 - 0.3b^2)$

Moment capacity of base plate
$M_p = 1.2f_y z_e$ --------------- 1

Where z_e- elastic section modulus

Moment capacity of 1 mm strip of plate

$M_{ps} = 1.2f_y\frac{t_s^2}{6}$ ------------- 2

Equating 1 & 2 and Apply partial safety factor for material

$1.2\frac{f_y}{\gamma_{mc}}\frac{t_s^2}{6} = \frac{w}{2}(a^2 - 0.3b^2)$

$$t_s = \sqrt{2.5w(a^2 - 0.3b^2)\frac{\gamma_{mc}}{f_y}}$$

Design Steps:

1. Assume suitable grade of concrete and bearing strength of concrete using 0.6fck.
2. Area of steel base =
$$\frac{P}{breaking\ strength\ of\ concrete} = A$$
   Where A = required area of the slab in mm².
   P= factored load from column in Newtons
3. A square plate is generally provided and the size of the base plate may be worked as L=B=$\sqrt{A}$
   Let a & b - greater and smaller projections of slab beyond column face.

$(D + 2a)(b_f + 2b) = A$

Where L- length of base plate

B- width of base plate

D- Depth of column section

b_f- width of column flange

4. Actual W=$\frac{P}{A_1}$

   Where A = area provided

5. Calculate the thickness of base plate (ts)

$$t_s = \sqrt{2.5w(a^2 - 0.3b^2)\frac{\gamma_{mo}}{f_y}} \geq t_f$$

6. Holding down (anchor) bolts 2 or4 nos and 20 mm dia are usually provided

   n=2 - when the base is subjected to only axial force

   n=4 – when it is subjected to both axial and B.M

7. Welding joint between the column and the base plate is designed.

Gusset Base Connection:

When load on the column section is too large and when the axial load is accompanied by bending moments also, gusset base is preferred.

It consists of base plate, two gusset plates and two gusset angles when bolted connection are made.

In case of welded connections are made, gusset angles are not required.

Gusset plates & gusset angles increase the bearing and consequently reduces the thickness of the base plate. Also gusset material supports the base plate against bending resulting less base plate thickness.

To check against bending of gusset plates due to upward pressure below the base plate, the width to thickness ratio for gusset plate is limited as follows.

D ≤ 29.3 ϵt_g for portion of gusset plate welded to column flange

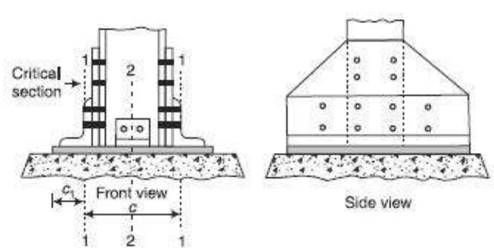
S_0 ≤ 13.6 ϵt_g for the outstand of gusset plate from the edge of the column flange.

Where $\epsilon = \sqrt{\frac{250}{f_{yg}}}$

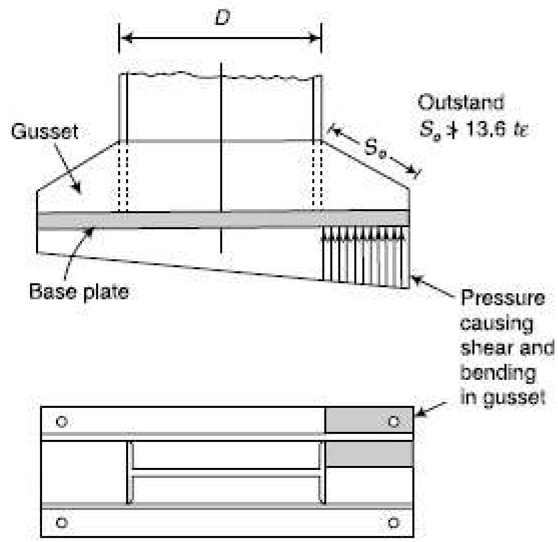
f_{yg} = design yield stress of gusset plate

t_g = thickness of gusset plate

Welded Gusseted Base Plates:



Bolted Gusset Plate



(a)

Welded Gusset Base Plate

Gusset plates are designed to resist shear and bending. The moment in gusset should not exceed the bending strength of the gusset plate.

Bending strength of gusset plate $M_{dg} = \frac{f_{yg} z_e}{\gamma_{m0}}$

Where Z_e = elastic section modulus

γ_{m0} = partial safety factor = 1.1

For 1 mm length $Z_e = (1/6) x 1 x\ t^2 = t^2/6$

Design Steps:

1. Assume suitable grade of concrete if not given
   Bearing strength of concrete = 0.45 fck
   (This reduced strength 0.45fck is used against max of 0.6fck as recommended by the code)
2. Area of base plate, $A = \frac{factored\ load\ (P)}{Bearing\ strength of\ concrete}$
3. Size of gusset material is assumed
   a. Gusset plate should not be less than 16 mm thick for bolted gusset plate
   b. Gusset angle is chosen so as to accommodate two rows of bolts in vertical leg and one row of bolts in horizontal leg. Unequal angle is preferred.

Thickness of gusset angle is almost same as gusset plate.

c. The length of gusset material is normally kept equal to the length of the base plate, parallel to the column flanges
d. For welded gusset plate, gusset angles are not required

Dimension of the base plate parallel to the web

For bolted connections

L= depth of the section+ 2(thickness of gusset plate + leg length of the angle+ over hang)

For welded connections

L= depth of the section + 2 (thickness of gusset plate + over hang)

B is obtained, B=$\frac{A}{L}$

Provided area A_1 =L x B

4. Actual intensity of bearing pressure 'w' from concrete below the base plate

$$w= \frac{P}{A_1}$$

5. Thickness of base plate 't_b' is computed by equating the moment at the critical section to the moment of resistance of the gusset at that section. The critical section 1-1 is assumed to lie at the root of the fillet of angle for bolted connections (as the load from flanges of the column is transferred to the base plate through gusset material)
Thickness of base plate, t_b = computed thickness (t) – thickness of gusset plate
B.M at critical section per unit width of the gusset plate

$$M= \frac{wc_1{}^2}{2}$$

Moment of resistance as critical section 1-1, $\text{Md} = \frac{1.2 f_y z_e}{\gamma_{m0}}$

$$M_d = M$$

$$\frac{1.2 f_y}{\gamma_{m0}} * \frac{1}{6} * 1 * t^2 = \frac{\text{wc}_1{}^2}{2}$$

$$\frac{0.4}{1.1} f_y t^2 = \text{wc}_1{}^2$$

$$\text{t}=\sqrt{\frac{1.1 * \text{wc}_1{}^2}{0.4 f_y}}$$

$$\text{t}=\text{c}_1\sqrt{\frac{1.1 * \text{w}}{0.4 f_y}} \qquad \text{--------------------} \ 3$$

Where t- aggregate thickness of base plate & gusset angle
t_b= thickness of base plate
t= thickness of angle
C1= portion of the base plate acting as cantilever

Also t_b can be checked against the B.M about section 2-2, $M=\frac{wl^2}{8}$

(Portion of the base plate between the column flanges may be assumed as simply supported between section 1-1)

6. Use 2 or 4 nos of bolts and provide 20 mm dia bolts.
7. Design the welded joint between the column end and the base plate
   For bolted gusset plate, the strength of the bolt is determined in single shear

   No of bolts = $\frac{Total\ factored\ shear}{strength\ of\ bolt}$

8. Determine the gusset plate size
   Length of the gusset plate is kept equal to the side of the base plate parallel to which it is provided.
   Height of gusset plate is governed by no. of rows of bolts or the length of weld to be accommodated.
9. Thickness of gusset plate may be assumed and buckling of the top edge should be checked by
   $D \leq 29.3\ \epsilon t_g$
   $S_0 \leq 13.6\ \epsilon t_g$
   Also $M_{dg}= f_{yg}\ Z_e\ /\ \gamma_{m0}$

DESIGN AND DRAWING OF STEEL STRUCTURES

UNIT-4

Syllabus: Beams

Introduction, types of sections, classification of cross-sections, beams with symmetrical cross-sections, behavior of beams in flexure, lateral stability of beams, lateral torsional buckling, design of laterally supported beams, shear centre, shear lag effect, shear buckling, web buckling, web crippling, design of laterally unsupported beams, design of purlins in roof trusses and lintels, check for shear, buckling and deflection.

Learning Outcomes:
Students will be able to

- Classify the types of beams depending upon its configuration as an element in the structure.
- Distinguish between beam and beam-column.
- Explain lateral stability and local buckling of flexural member.
- Understand the behavior of web buckling and web crippling of rolled steel beam sections.
- Design laterally supported beams using IS-800:2007 code provisions.
- Explain the concept of shear centre and shear flow.
- Design the lintel beam and purlins in a roof truss.

Learning Material

Introduction:

Structural member subjected to transverse loads (loads perpendicular to its longitudinal axis) is called a beam. Beams provided in the buildings to support roofs are known as "Joists". Large beams supporting the joists are known as "Girder". Exterior beams at floor level of buildings which carry part of the floor load and that of exterior walls are called "spandrels". Beams which carry roof loads in trusses are called "Purlins". Beams which support loads from masonry over opening are called "Lintels". A horizontal beam spanning the wall columns of industrial buildings used to support wall covering is called a "Girt".

Structural members subjected to bending, when accompanied by large axial compressive loads are known "Beam- column"

Beams are subjected to

- Simple bending
- Unsymmetrical bending
- Bi-axial bending

A. Simple bending:

The loading plane must coincide with one of the principle planes of doubly symmetrical section (I- Section) and for singly symmetrical section (channel), it must pass through the shear centre.

B. Unsymmetrical bending:

When the plane of loading does not pass through the shear centre, then the bending is called unsymmetrical bending. In such a case the bending is coupled with torque.

C. Bi-axial bending:

Bi-axial loading is characterized by simple bending (with no torsion) occurring about both the principle planes.

Beams with symmetrical cross-sections:

- Beams are primarily subjected to bending accompanied by shear in the loading plane, with no external torsion and axial force.
- However due to instability caused by compressive stresses, torsional stresses will produce.
- This instability is characterized as lateral buckling (when it is of a general nature involving entire span) or local bucking (involving only local components) such as compression flange, compression part of the web etc. of a beam

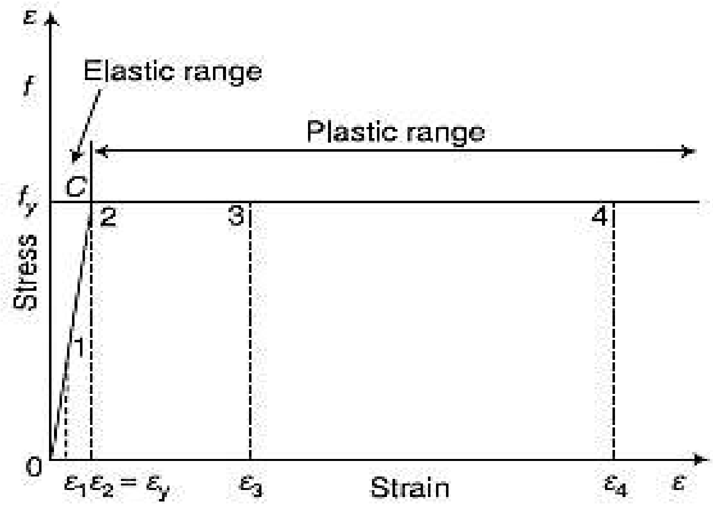
Design Considerations:

Beams are supposed to be most critical members of a structure.

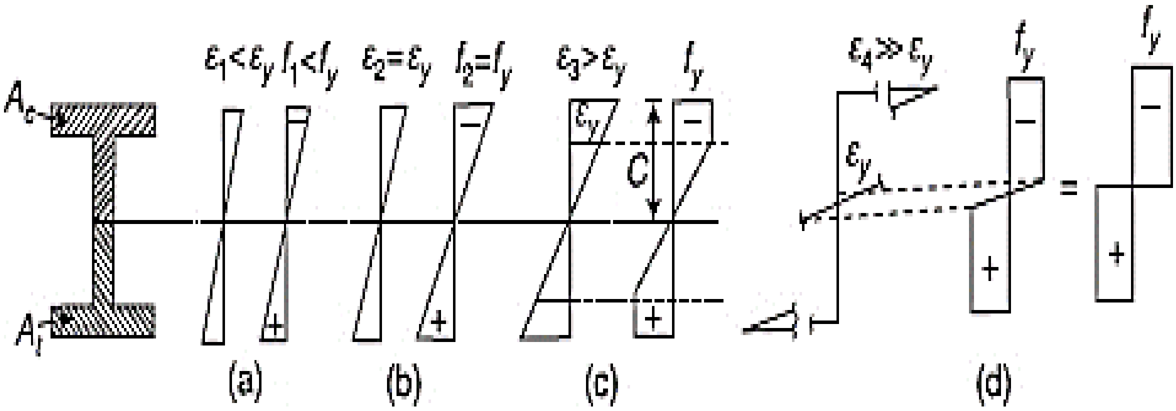
The main considerations in design of a beam are

1. It should be proportioned for bending keeping in view the lateral and local stability of compression flange and capacity of section against shear and local bearing.
2. They should be proportioned for stiffness for keeping the deformations (deflections) under service conditions.
3. They should be designed for economy in size and grade of steel to yield the most economical section.

Behaviour of Beam in Flexure:



Idealised elasto-plastic stress-strain curve



$$M_1 = f_1 Z_e$$

$$M_2 = M_y = f_2 Z_e = f_y Z_e$$

$$M_3 = f_y(b_f t_f + d_0 t_w)\ (y_c + y_t) + f_y\ \frac{t_w d_1^2}{6} \qquad (7)$$

Where d_0 = depth up to which the web is plastified,

d_1 = depth of the web which is not plastified

$$M_p = f_y Z_p = f_y \left[b_f t_f (d - t_f) + t_w \left(\frac{d}{2} - t_f \right)^2 \right]$$

Classification of Cross-sections:

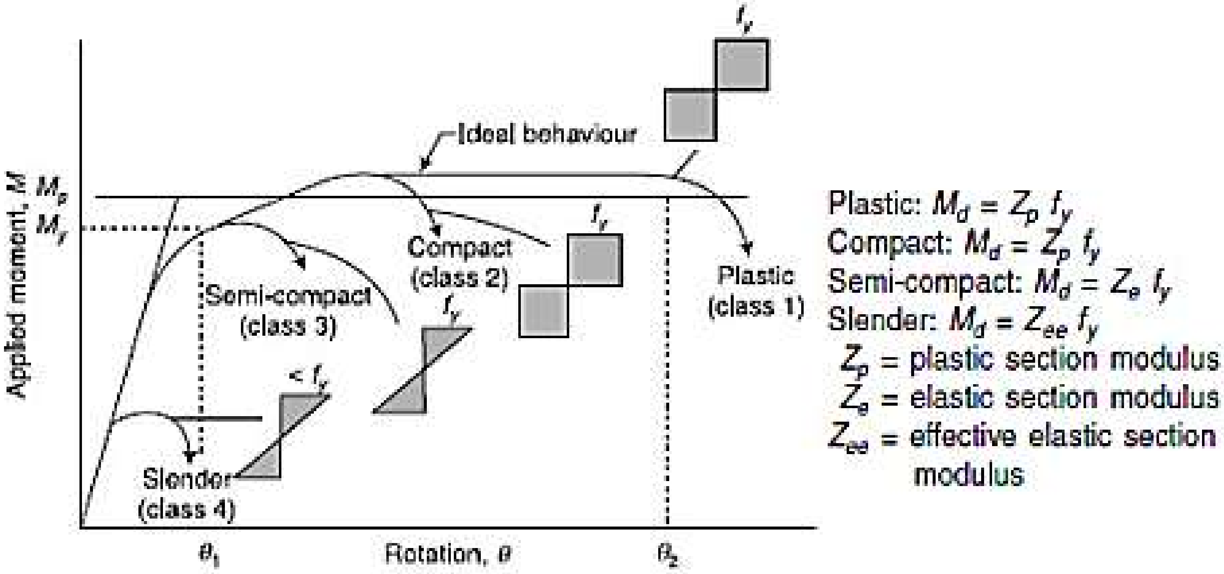


Fig-Moment-rotation behavior of four classes of cross-sections

1. **Class-1 Plastic cross-sections:**
   These sections can develop plastic hinges and have rotation capacity required for failure of the structure by formation of plastic mechanism.
   b/t_f < specified under class I
2. **Class-2 Compact cross-sections:**
   Such sections can develop plastic moment of resistance but have inadequate plastic hinge rotation capacity for formation of plastic mechanism, due to local buckling.
   b/t_f > specified under class I < class II
3. **Class-3 Semi-Compact cross-sections:**
   These are the sections in which the extreme fibres in compression can reach yield stress but cannot develop the plastic moment of resistance due to local buckling.
   b/t_f > specified under class II < Class III
4. **Slender Sections:**
   Cross-sections in which the sections buckle laterally even before attainment of yield stress are called as slender sections.

Lateral stability of Beams:

- A beam is subjected to bending stresses under applied loads.
- In case of simply supported beams of the flanges will be subjected to compression and the other to tension.
- For more strength, the sections are so proportioned that I_{zz} about the principal axis normal tot the web is considerably larger than I_{yy} about the principal axis parallel to the web.
  $I_{zz} > I_{yy}$
- Such a situation is relatively weak in bending resistance in the plane normal to the web and therefore, the compression flange of the beam tends to buckle in the only direction in which it is free to move i.e horizontally.
- This buckling increases with increase of I_{zz} to that of I_{yy}.
- Since the bottom flange being in tension, tends to remain straight, buckling of top flange will take place as shown in fig.

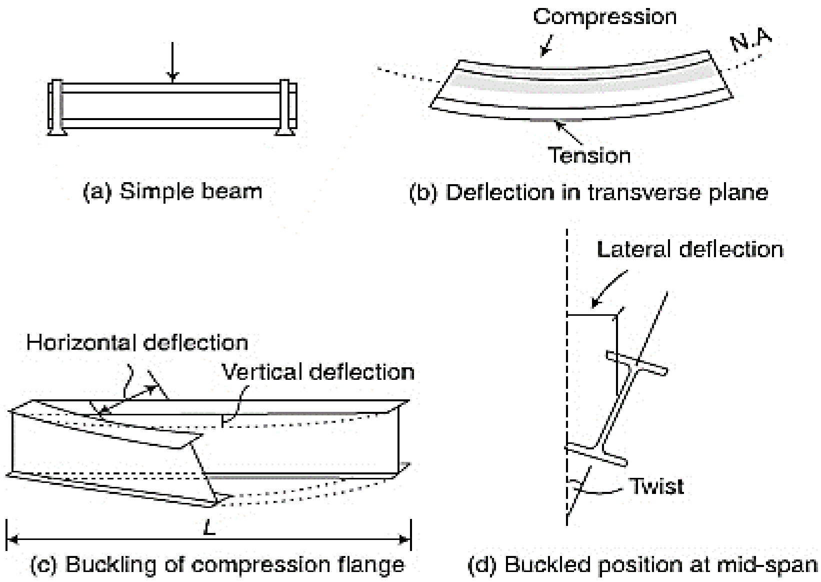


Fig- lateral torsional buckling of I-section

Thus, the lateral buckling of compression flange is also accompanied by twisting.

- When the beam deflects laterally, the applied moment exerts a torque about the deflected longitudinal area.
- The bending moment at which a beam fails by lateral buckling when subjected to a uniform moment is called its "Elastic Critical Moment".
- A beam in which the compression flange is restrained laterally is called a "Laterally restrained Beam".

Lateral Torsional Buckling:

The compression flange of an I-Beam acts like a column and will buckle sideway if the beam is not sufficiently stiff or the flange is not restrained laterally.

The buckling of a beam loaded in the plane of its strong axis and buckle about its weak axis and is accompanied by twisting moment is characterized as "Lateral Torsional Buckling"

The load at which such a beam buckles can be much less than that causing moment capacity to develop.

$$M_{cr} = \frac{\pi}{L}\sqrt{EI_y GI_t + \left(\frac{\pi E}{L}\right)^2 I_w I_y} \quad \ldots\ldots \quad \boxed{1}$$

where EI_y = flexural rigidity (minor axis)

GI_t = torsional rigidity

I_t, I_w = St-Venant torsion constant and warping constant.

L = unbraced length of the beam subjected to constant

The above equation is applicable to compact doubly symmetrical I-shaped beams, I – sections loaded in plane of their web and I-sections with compression flanges larger than their tension flanges.

However, if the I_{xx} about bending axis is equal or less than I_{yy} about out of the plane, lateral torsional buckling will not occur. For shapes bent about minor axis (yy axis) with $I_{xx} < I_{yy}$ or for circular or square shapes equation -1 can be modified as

$$M_{cr} = \frac{\pi}{L}\sqrt{EI_y GI_t}\sqrt{1 + \frac{\pi^2 EI_w}{L^2 GI_t}} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad \boxed{2}$$

$$= \frac{\pi}{L}\sqrt{EI_y GI_t}\sqrt{1 + \frac{EI_y}{GI_t}\left(\frac{\pi}{2}\frac{h}{L}\right)^2}$$

This is applicable for sections of semi-compact.

The first term of equation relates the capacity of beam shape to resist St.Venant torsion and the second term is the measure of the contribution of warping to the torsional resistance of the beam.

Short and deep beams demonstrate very large warping resistance, while long and shallow girders have low warping stiffness on resistance

For $I_w = 0$, eqn-2 becomes

$$M_{cr} = \frac{\pi}{L}\sqrt{EI_y GI_t} \quad \text{................} \quad \boxed{3}$$

On account of different loading and support conditions, a factor c_1 can be introduced and eq-2 may be rewritten as

$\boxed{4}$

$\boxed{5}$

$\boxed{6}$

$$M_{cr} = c_1\frac{\pi}{L}\sqrt{EI_y GI_t}\sqrt{1+\frac{\pi^2 EI_w}{L^2 GI_t}} \quad \boxed{7}$$

$$M_{cr} = c_1\sqrt{EI_y GI_t}\left[\frac{\pi}{L}\sqrt{1+\frac{\pi^2 EI_w}{L^2 GI_t}}\right]$$

$$M_{cr} = c_1\sqrt{EI_y GI_t}\left[\frac{\pi}{L}\sqrt{1+\frac{\pi^2}{B^2}}\right] \qquad \left(\text{Putting } \frac{L^2 GI_t}{EI_w}\right.$$

$$M_{cr} = c_1\sqrt{EI_y GI_t}\,\gamma \qquad \left(\text{Putting } \frac{\pi}{L}\sqrt{1+\frac{\pi^2}{B^2}} = \gamma\right) \qquad (8)$$

Eq 7 is the basic equation for lateral torsional buckling of beams. The factor c_1 taken into account the level of application of load as well as type of loading which may influence the elastic critical moment value of a beam.

c_1 is called “equivalent uniform moment factor or coefficient”

The boundary conditions at lateral supports have two components

1. Torsional restraint:
   Where the cross-section is prevented from rotation about shear centre.
2. Warping restraint:
   Where the flanges are prevented from rotating in their own plane about an axis perpendicular to the flange.

The M_{cr} corresponding to the torsional buckling of a doubly symmetrical prismatic beam subjected to uniform moment in the unsupported length and torsional restraining lateral supports is given by

$$M_{cr} = \frac{\pi^2 EI_y}{(L_{LT})^2}\sqrt{\frac{I_w}{I_y} + \frac{GI_t (L_{LT})^2}{\pi^2 EI_y}}$$

where I_y, I_w, I_t = moment of inertia about the minor axis, warping constant, and St. Venant’s torsion constant of the cross section, respectively

G = Shear Modulus

L_{LT} = effective length against lateral torsional buckling.

Bending strength of beams:

In general a beam may fail

i. Due to lateral torsional buckling
ii. Flange local buckling
iii. Web local buckling

Either in elastic range or in inelastic range, the strength corresponds to each of the three limit states must be computed and the smallest value will control.

In beam design, the following aspects are to be considered; moments, shears, deflections, crippling, buckling and lateral support.

For laterally supported beams, the design of bending strength of a beam is governed by the yield stress, while for a laterally unsupported beam, the lateral torsional buckling controls the design.

Design of laterally supported Beams

Beams with plastic or compact shapes and with lateral support throughout have nominal flexural strength equal to the full plastic moment 'M' at any of its section due to external action should be less than or equal to the design bending strength 'M_d'

$$M \leq M_d \qquad \text{for } \frac{d}{t_w} \leq 67\epsilon$$

Where ϵ - yield stress ratio $=\left(\frac{250}{f_y}\right)^{\frac{1}{2}}$, normally = 1

d – Depth of the web

t_w- Thickness of web

If $\frac{d}{t_w} \geq 67\epsilon$, The web is susceptible to shear buckling before yielding. The design for bending strength may be calculated using one of the following methods

a. Bending moment and axial force acting on the section may be assumed to be resisted by flange and web to resist shear only.
b. Bending moment and axial force acting on the section may be assumed to be resisted by the entire section and web to resist combined normal and shear stresses using elastic theory.

CASE- 1

$V \leq 0.6\ V_d$

V- Factored design shear force

V_d - Design shear strength of the section

Then, the shear force does not influence on bending moment by low shear force

Design bending moment

$$M_d = \beta_b Z_p \frac{f_y}{\gamma_{mo}} \qquad \text{....................(1)}$$

$$\leq 1.2\ Z_e \frac{f_y}{\gamma_{mo}} \text{(for simply supported beams)}$$

$\leq 1.5\ Z_e \frac{f_y}{\gamma_{mo}}$(for cantilever beams)

β_b= 1.0 for plastic and compact sections

$=\frac{Z_e}{Z_p}$ for semi- compact section

Where Z_e and Z_p = elastic and plastic section moduli

f_y – yield stress to material

γ_{mo}- 1.1 Partial safety factor in yielding

<u>For slender section</u>

$M_d = Z_e f_y$ ....(2)

Note: 1. For most of I – sections and channel sections the ratio $\frac{Z_p}{Z_e}$ is less than 1.2 and plastic moment capacity equation (1) governs the design.

2. For slender having $\frac{Z_p}{Z_e}$ >1.2, the constant 1.2 may replaced by the ratio of $\frac{factored\ load}{service\ load}$ i.e by γ_f

Thus 1.2 $Z_e f_y$ becomes γ_f $Z_e f_y$

CASE- 2

When V>0.6 V_d

When factored design force V is greater than the design shear strength V_d the design bending strength will be less than the moment capacity of the section due to interaction between moment and shear.

Then shears are characterized by higher shear force. This reduced moment capacity (M_{dv}) may be determined as follows

a) <u>For Plastic and compact Sections:</u>

M_{dv}= M_d-$\beta(M_d - M_{fd}) \leq 1.2\ Ze \frac{f_y}{\gamma_{mo}}$

Where M_d – Plastic Design moment of entire section disregarding higher shear force effect, but considering web buckling effect.

M_{fd}-Plastic section strength of c.s area exclude the shear area considering γ_{mo}

β= $\left[2\frac{V}{V_d} - 1\right]^2$

V_d- Design shear strength as governed by web yielding or web buckling

V- Factored applied shear force governing by web yielding or web buckling

Z_e- Elastic section modulus of the whole section

b) <u>For semi- compact section:</u>

Most standard I & Channel sections are compact a few are semi-compact became flange width – thickness ratio, but none are slender.

M_d = $Ze \frac{f_y}{\gamma_{mo}}$

Shear centre:

It is the point of intersection of the bending axis and the plane of transverse section. The bending axis of the beam is its longitudinal axis through which transverse loads causing the bending must pass in order that bending of the beam is not accompanied by its twisting.

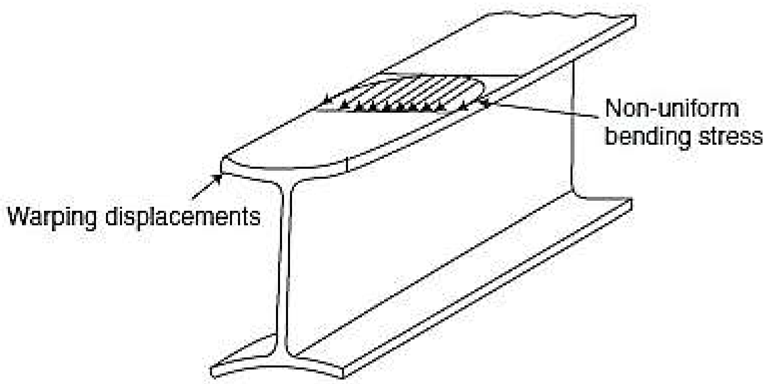
For a cross-section with two-axes of symmetry, the shear centre will fall at the intersection of the two axes, thus coinciding with the centroid of the section. For sections with one axis of symmetry, the shear centre will fall somewhere on that axis not necessarily at the centroid of the beam i.e to avoid torsion in some of the beams the line of action of the load should not pass through the centroid of the section. The shear centre is of particular importance for beams whose cross-sections are composed of thin parts which provide considerable bending resistance but little resistance to torsion. Channels, angle sections which are made up of thin plates fall into this class.

Shear-lag effect:

One of the assumptions in theory of simple bending is that plane sections before bending remain plane after bending.

However the shear strains induced in flanges cause the sections wrap and modifies the bending stresses result in higher stresses near the junction of web to flange with the stress dropping as the distance from beam web increases.

Hence, the resultant stress distribution across the flange is non-uniform and is shown in fig. This phenomenon is known as "shear lag"



Shear lag effect

For rolled I-sections this effect is negligible, While for flanges in built in sections and for wide flanges beams, This effect is considerable. Shear lag effect depends on width- to-span ratio, Beam end restraints and type of load. Point load causes more Shear lag than u.d.l

As per IS 800- 2007, the shear lag in flanges may be neglected provided

a) $b_o \leq \frac{L_o}{20}$ (for outstand elements)
b) $b_i = \frac{L_o}{10}$ (for internal elements)\

where Lo- length between points of zero moment (in flexure) in span

b_o- width of flange outstand

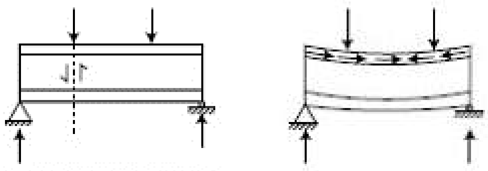
b_i- width of internal element

Shear strength of beams:

A beam is subjected to two types of shears

I. Transverse (vertical) shear
II. Longitudinal shear

- **Transverse shear** failure does not usually occur as the beam will fail due to web crippling prior to transverse shear failure, but it may occur if the beam is short and heavily loaded.



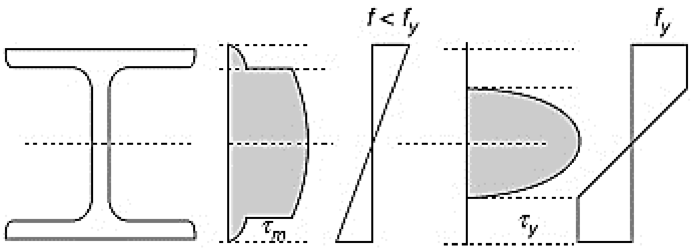
- **Longitudinal (Horizontal) shear** occurs due to bending of the beam. The fibres above N.A Shorten in length and below NA will elongate under sagging B.M Hence the fibres tend to slip over each other and this effect is maximum at NA.
- The tendency of slip is resisted by shear strength of the material.
  Longitudinal shear $\tau = \frac{VA\bar{y}}{I_z t}$ ....(1)

Where V- longitudinal shear force

I_z- M.I of entire c.s about z-z axis

$A\bar{y}$ - static moment of area (A) of c.s above the fibre under consideration about neutral axis

t- Thickness of section at layer under consideration.



(a) Elastic range (b) Plastic range (shear yielding of web)

It can be seen that flanges resist a very small portion of shear and web resist significant portion of shear.

Yield of web represents one of the shear limit states

Average shear stress $\tau_{ava} = \frac{V}{dt_w} = \frac{V}{A_v}$ ....(2)

Where d- depth of web, t_w – Thickness of web and A_v – shear area

The nominal shear yield strength of the web based on V_{on} – Mises yield criterion can be represented a

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

$$\boxed{V_n = \frac{A_v f_{yw}}{\sqrt{3}}}$$ —— (3) ——

where f_{yw}- yield strength of the web

V_n – Nominal shear resistance

Two cases of shear resistance

1) **Nominal shear resistance of a cross-section due to plastic shear resistance**

$V_n = V_p$

where $V_p = \frac{A_v f_{yw}}{\sqrt{3}}$ ———————————— (4)

As per IS 800-2007, the shear area of different cases are for I- section

For I- section:

Under major axis bending, Hot rolled $A_v = h\ t_w$

Welded, $A_v = d\ t_w$

Under minor axis bending, Hot rolled or welded, $A_v = 2.0\ b\ t_f$

For rectangular hollow sections of uniform thickness

When loaded parallel to depth (h), $A_v = \frac{Ah}{(b+h)}$

Loaded parallel to width(b), $A_v = \frac{Ab}{(b+h)}$

For circular hallow tubes of uniform thickness

$A_v = \frac{2A}{\Pi}$

For plates and solid bars, $A_v = A$

Where A – Area of crass-section

b- Overall breadth of tubular section or breadth of I=section flanges

d- clear depth of web between flanges

h- overall depth of the section

t_f- Thickness of the section

t_w- thickness of web

Design shear strength under combined high shear & bending

$V_d = \frac{V_n}{\gamma_{mo}}$ Where γ_{mo}- Partial safety factor against Shear = 1.1

2) **Resistance to shear Buckling**

The web in a rolled steel section behave like a column when placed under concentrated loads Since the web is thin, it is subjected to a vertical buckling.

Generally the resistance to shear buckling is verified

When $\frac{d}{t_w} > 67 \in$ for web without stiffener

And $\frac{d}{t_w} > 67 \in \sqrt{\frac{k_v}{5.35}}$ for web with stiffener

Where k_v is the shear buckling coefficient

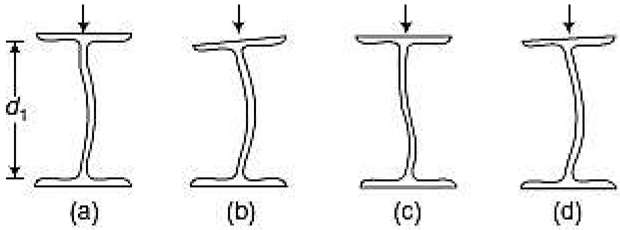
$$\epsilon = \sqrt{\left(\frac{250}{f_y}\right)} \text{ normally } 1.0$$

Note:

1. For rolled I –section $\frac{d}{t_w}$ is < 67 ∈ hence shear buckling can be neglected.
2. Compressive bending stresses in the web may cause local buckling in the form of longitudinal waves and so shear buckling can be neglected.
3. For I- section, design is for flexure and check against shear only.

Web Buckling:

The web in a rolled steel section behaves like a column when placed under concentrated loads. Web is thin and therefore subjected to buckling (vertical buckling). The buckling of the column web is influenced by the restraints provided for the flanges.



Buckling failure from vertical loading

In all cases, bottom flange is assumed to be restrained against later deflection and rotation. For the top flange, the end restraints and the effective depth of the web to be considered are as below.

1. Restrained against lateral deflection and rotation.(Fig.a)
   effective depth $= d_1/2$
2. Restrained against lateral deflection but not against rotation. (Fig.b)
   effective depth $= (2/3)\ d_1$
3. Restrained against rotation but not against lateral deflection(Fig.c)
   effective depth $= d_1$
4. Not restrained against rotation and lateral deflection(Fig.d)
   effective depth $= 2d_1$

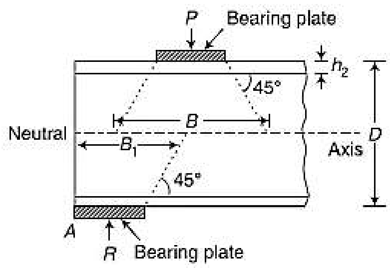


Fig. 9.9 Bearing length for buckling consideration

The maximum di mpression occurs at the neutral axis and will be inclined at 45°

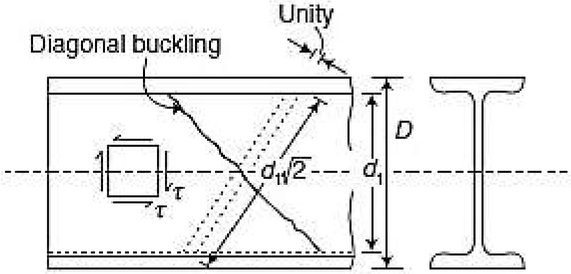
$$F_{wb} = B_1 t_w f_{cd}$$

to it.

Web buckling strength at the support

Where f_{cd} – allowable compressive stress corresponding to the assumed web strut

B- length of stiff portion of the bearing + additional length given by dispersion at 45^0 to the level of neutral axis.



Diagonal compression

Consider a column of unit thickness and of length equal to $d_1\sqrt{2}$ (as inclined at 45° with NA).

The effective length will be $(d_1\sqrt{2})/2$ if ends are assumed as fixed

The effective length will be $d_1\sqrt{2}$ if ends are assumed as hinged

Where ends are fixed,

$$\text{Effective length} = \frac{d_1\sqrt{2}}{2} = \frac{d_1}{\sqrt{2}}$$

$$\text{Minimum radius of gyration} = \frac{t}{\sqrt{12}}$$

$$\text{Slenderness ratio, } \lambda = \frac{d_1/\sqrt{2}}{t/\sqrt{12}} = \frac{d_1\sqrt{6}}{t} = 2.45\frac{d_1}{t}$$

Where $\lambda = l_e / r$

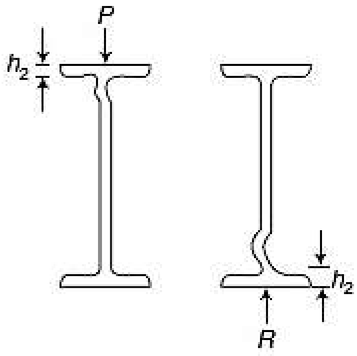
Web Crippling:

Loads and reactions concentrated along a short length of flange of beams are restrained by compressive stresses in the web which vary with the distance from the load. The web of rolled steel sections are therefore subjected to a large amount of stresses just below the concentrated loads and above the reactions from the support.

Stress concentration occurs at the junction of web and the flange. As a result large bearing stresses are developed below the concentrated loads.

Consequently, the web near the portion of stress concentration tends to fold over the flange.

This type of local buckling phenomenon is called **crippling** or **crimpling of web.**



Web failure by local crippling

The bearing $F_w = A_e \frac{f_{yw}}{\gamma_{m0}}$ strength, ,

Where f_{yw} – design yield strength of web

A_e - effective area of web (b_1 t_w)

Where b_1 – bearing length = b + 2n_1 (under concentrated loads)

= b + n_1 (under reaction supports)

P - concentrated load in Newtons

Deflection:

In addition to being safe against strength, the structure must be serviceable. Excessive deflections may lead to crack in the plaster or ceiling and may damage the material attached to or supported by the beam.

Limit of maximum deflection is span/300

In general beam deflection is a function of loading, span, modulus of elasticity and the geometry of the cross-section. For regular loads, usual deflection formulae may be used.

Procedure for laterally supported beams:

1. Service loads expected on the beam are ascertained. Service loads are multiplied with γ_f to determine the factored loads
2. Calculate maximum B.M (M) and max. shear (V). These forces M& V are referred as design forces.
3. A trail plastic section modulus for the beam is worked out by

$$Z_{pz,\,req} = \frac{M\gamma_{m0}}{f_y}$$

   Where M- design moment
   f_y- yield stress of the material
   γ_{mo} - Partial safety factor = 1.1
4. Suitable section may be selected having section modulus equal to or more than $Z_{pz\ required}$.
5. Classification of section is checked as plastic, compact or semi-compact.
6. Trail section is checked for shear.

(i) The design shear force V should be less than the design shear capacity V_d.

$$V_d = \frac{f_y}{\sqrt{3}\gamma_{m0}} ht_w$$

where h = overall depth of the section

t_w = thickness of the web

(ii) The beam is checked for high/low shear case.

If the beam is checked for high/ low shear force

If $V \le 0.6\ V_d$, the case is of low shear

if $V > 0.6\ V_d$, it is high shear case.

7. Check for design of bending strength.

i. For lower shear case,

$M_d > M$

Where M_d – Design bending strength

M – Design bending moment

Design bending strength, $M_d = \beta_b Z_p \frac{f_y}{\gamma_{mo}}$

$\le 1.2\ Z_e \frac{f_y}{\gamma_{mo}}$ (for simply supported beams)

$\le 1.5\ Z_e \frac{f_y}{\gamma_{mo}}$ (for cantilever beams)

ii. For higher shear case

$M_{dv} \ge M$ (Plastic & compact section)

$M_d \ge M$ (For semi-compact section)

Where M_{dv} – Design bending strength for plastic and semi - compact section

M_d – Design bending strength for semi-compact section

$M_d = Z_e \frac{f_y}{\gamma_{mo}}$ or $\beta\ Z_p f_{bd}$

Where $\beta = 1$

$\beta = \frac{Z_e}{Z_p}$

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

and $M_{dv} = M_d - \beta(M_d - M_{fd}) \le 1.2\ Ze \frac{f_y}{\gamma_{mo}}$

where M_{fd} - Plastic design strength of area of whole c.s excluding the shear area.

$M_{fd} = Z_{jd} \frac{f_y}{\gamma_{mo}}$

Where $\boxed{Z_{jd} = Z_p - A_w\, y_w}$

Where A_w- Area of web = ht_w

Y_w -$\frac{h}{4}$(For Symmetrical I, Channel section)

8. The trail section is checked against web buckling
   If $\frac{d}{t_w}$ is $\leq 67 \in$ (for webs without stiffeners)
   For web buckling
   Web buckling strength > Design shear force
   Capacity of the section = $A_b f_{cd}$
   A_b- Area of web at NA of the beam
   f_{cd}- Design compressive stress

9. Check for web bearing, $F_w > V$
   Where F_w – web bearing strength = $A_e \frac{f_{yw}}{\gamma_{mo}}$
   $A_e = \left[b + 2.5(t_f + R_1)\right] t_w$
   f_{yw} is the yield stress of web of beam section

10. Check for deflection
    Permissible deflection is $\frac{L}{300}$

Design of laterally unsupported beams:

Beams with bending about major axis and compression flanges not restrained against lateral bending fail by lateral torsional buckling before attaining its bending strength.

The effect of lateral torsional buckling need not be considered if $\lambda_{LT} \leq 0.4$

Where λ_{LT} is non-dimensional slenderness ratio for lateral torsional buckling.

Buckling strength for laterally unsupported beams is given by

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

where $\beta_b = 1.0$ (for plastic and compact sections)

$= Z_e/Z_p$ (for semi-compact sections)

where Z_e = elastic section modulus

Z_p = plastic section modulus

f_{bd} = design bending compressive stress

$$= \chi_{LT} \frac{f_y}{\gamma_{m0}}$$

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

χ_{LT} = bending stress reduction factor to account for lateral torsional buckling

$$= \frac{1}{\phi_{LT} + (\phi_{LT}^2 - \lambda_{LT}^2)^{0.5}} \leq 1.0$$

$$\phi_{LT} = 0.5\,[1 + \alpha_{LT}(\lambda_{LT} - 0.2) + \lambda_{LT}^2]$$

α_{LT} = imperfection factor

= 0.21 (for rolled section)

= 0.49 (for welded section)

λ_{LT} = non-dimensional slenderness ratio

$$= \sqrt{\beta_b Z_p f_y / M_{cr}} \leq \sqrt{1.2\, Z_e f_y / M_{cr}}$$

$$= \sqrt{f_y / f_{cr,b}}$$

$$M_{cr} = \sqrt{\frac{\pi^2 EI_y}{(L_{LT})^2}\left[GI_t + \frac{\pi^2 EI_w}{(L_{LT})^2}\right]} = \beta_b Z_p f_{cr,b}$$

where M_{cr} = elastic critical moment corresponding to lateral-torsional buckling of beam

I_y = moment of inertia about minor axis

I_w = warping constant = $(1 - \beta_f)\,\beta_f I_y h^2{}_f$

I_t = St.Venant's constant

$= \Sigma \frac{b_i t_i^3}{3} = 2\frac{b_f t_f^3}{3} + \frac{b_f t_\omega^3}{3}$ for open section (e.g. I section)

(In the second term h_f, c/c distance between flanges = $h - t_f$, has been used in place of $h - 2t_f$ to account for added stiffness of web-to-flange junction and fillets.)

G = shear modulus

where r_y = radius of gyration about the weaker axis

L_{LT} = effective length for lateral-torsional buckling (Tables 9.1 and 9.2)

β_f = ratio of moment of inertia of compression flange to the sum of moments of inertia of compression and tension flanges

t_f = thickness of the flange

$f_{cr,b}$ = extreme fiber compressive elastic lateral buckling stress

$f_{cr,}$ b

$$= \frac{1.1\pi^2 E}{(L_{LT}/r_y)^2}\left[1 + \frac{1}{20}\left(\frac{L_{LT}/r_y}{h_f/t_f}\right)^2\right]^{0.5} \qquad (24)$$

The values of can also be obtained using table 14 of IS-800: 2007 corresponding to different values of

$$\frac{KL}{r}\left(\text{or, } \frac{L_{LT}}{r}\right) \text{ and } \frac{h}{t_f}.$$

Design of lintel beams:

Beams provided over openings in masonry walls to support the masonry above the openings are called lintels.

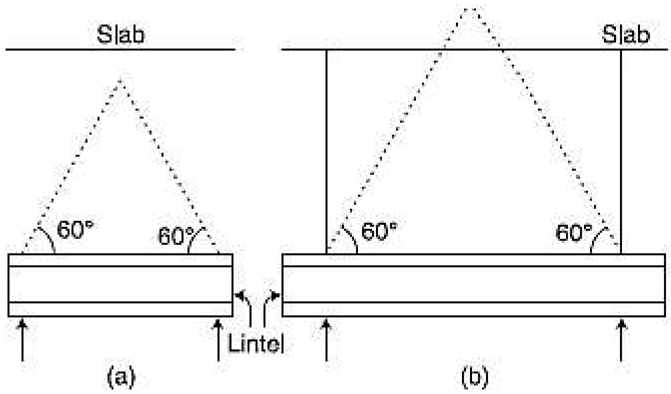
Flat or plate sections can be used for small openings.

Angles back to back or inverted T-sections can be used for moderate section.

Channels or I-sections or built-up sections can be used for large openings.

It is assumed that after setting of mortar, the load from the masonry is distributed by arch action.

The following are the two cases for distribution of arch action loading



Loads from masonry over lintels

When slab over the lintel is above the apex of equilateral triangle formed over the lintel, thus a triangular load is distributed over the lintel

When the slab over the lintel is below the apex of equilateral triangle formed over the lintel, the load of masonry in the rectangular length equal to the span of the lintel and the height of slab above the lintel, is assumed to act on the lintel.

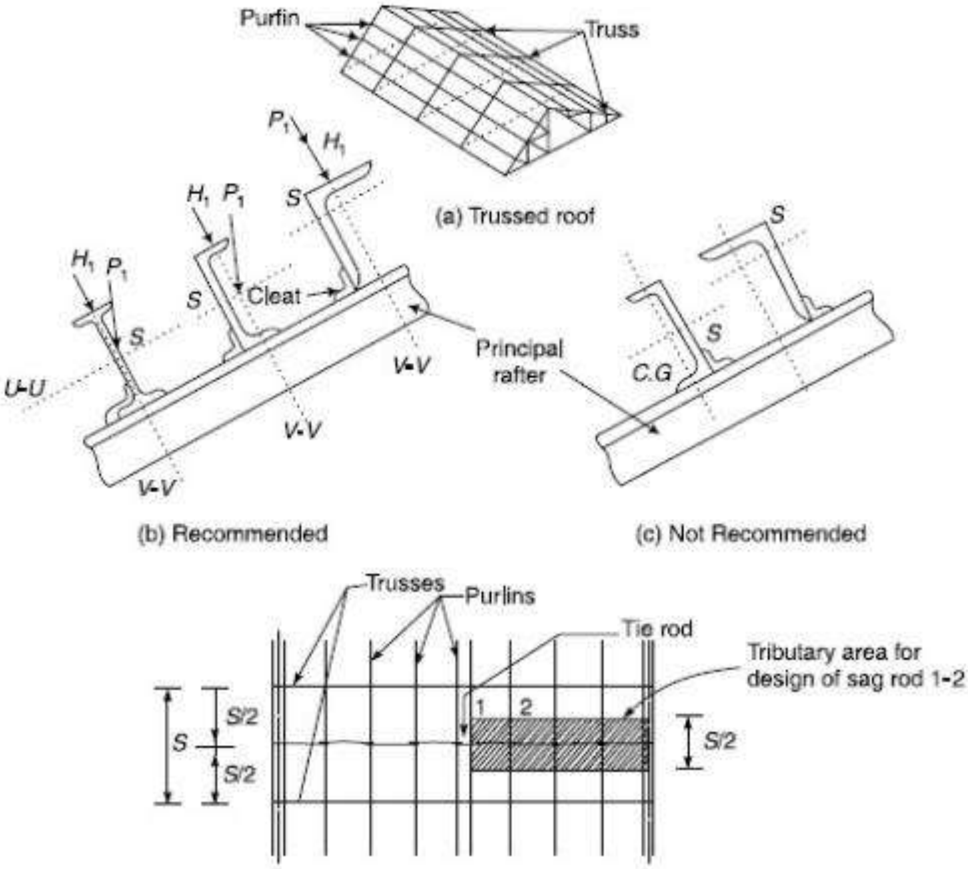
<u>Design of Purlins in Roof Trusses:</u>

Purlins are beams provided over trusses to support roofing material. Channel and angle sections are commonly used as purlins. Two axes u-u and v-v replace the axes z-z and y-y as shown in the fig.

The wind force is assumed to act normal to the roof trusses and gravity loads pass through the C.G of purlin section. Hence purlins are subjected to twisting in addition to bending. Such a bending is known as "Unsymmetrical bending".

Purlins are designed as continuous beams as per I.S Code.

Hence B.M = $wL^2 / 10$



(d) Plan of trussed roof system showing load on the roof area tributary to the top sag rod

<u>Design procedure for purlin having Channel/ I-Section :</u>

1. Calculate gravity loads p_1 and wind load H_1 (gravity loads are due to sheeting and live loads) and multiply with γ_m to obtain factored loads
   $P = \gamma_f \, p_1$ and $H = \gamma_f \, H_1$
2. Calculate maximum $M_{uu} = \frac{PL}{10}$ and $M_{vv} = \frac{HL}{10}$
   P- Factored load along uu axis (in Newton's)
   H- Factored load along vv axis (in Newton's)
   L- span of the beam(Purlin)
   Now change the notation as $M_{uu} = M_z$ and $M_{vv} = M_y$
3. Since purlins are subjected to biaxial bending, trial and error method is adopted.

A trail section is selected from IS tables and the properties b and d are noted

$$Z_{pz} = M_z \frac{\gamma_{mo}}{f_y} + 2.5\left(\frac{d}{b}\right) \frac{M_y \, \gamma_{mo}}{f_y}$$

γ_{mo}- 1.1,

d- Depth of trail section

b- Width of the trail section

For the trail section, knowing b & d values, Check the adequacy of the section

4. The design capacity of the section Mdz & Mdy are given by

$$M_{dz} = Z_{pz} \frac{f_y}{\gamma_{mo}} \leq 1.2 \, Z_{ez} \frac{f_y}{\gamma_{mo}}$$

And $M_{dy} = Z_{py}\frac{f_y}{\gamma_{mo}} \quad \leq \gamma_f Z_{ey}\frac{f_y}{\gamma_{mo}}$

For safety, $M_{dz} > M_z$ and $M_{dy} > M_y$

5. Local capacity of the section is checked by

$$\frac{M_z}{Z_{dz}} + \frac{M_y}{Z_{dy}} \leq 1$$

6. Check for deflection of purlin and should be less than $\frac{L}{180}$
7. The section effect of wind may cause the lower flange of the purlin in compression and since lower flange is unrestrained, the purlin will be subjected to lateral torsional buckling. Then the buckling resistance of the section M_{dz} about *zz*-axis is calculated.
   Buckling resistance about yy- axis is not needed as the pulins are restrained in Z-plane by roofing and will be stable for moments about y-y axis.
   $M_{dz} = \beta_b Z_p f_{bd}$
   And the design is checked by using equation
   $\frac{M_z}{Z_{dz}} + \frac{M_y}{Z_{dy}} \leq 1$

Design Procedure for Angle Purlins:

Angle purlins are unsymmetrical about both the axis. Normally angle purlins are used when the slope of the roof is less than 30^0.

Procedure

1. Gravity and wind loads are calculated. Both the loads are assumed normal to the roof truss.
2. Calculate Max. B.M $\frac{wL^2}{10}$ or $\frac{WL}{10}$
3. Obtain $Z = \frac{M}{1.33*0.66*f_y}$ Where f_y-is yield stress.
4. Assume Angle Section by taking depth of angle section $\frac{1}{45}$ of span and width of angle as $\frac{1}{60}$ of span.
5. Procedure is as per the I.S code.

DESIGN AND DRAWING OF STEEL STRUCTURES

UNIT-5

Syllabus:

Design of welded plate girders- flexure strength of flanges, shear strength of web, shear buckling-intermediate web stiffeners, longitudinal stiffeners, bearing stiffeners and torsional stiffeners, design of welded plate girder- curtailment of flange plates.

Learning Outcomes:

Students will be able to

- understand various concepts of a plate girder.
- explain the design considerations of plate girder.
- design the flexural strength of flanges and shear strength of web of a plate girder.
- describe the need of introducing the transverse, longitudinal and bearing stiffeners to the web of a plate.
- analyze the curtailment of cover plates in plate girder design,

Learning Material

A flexure member (beam) is usually designed for bending and shear.

When spans are large and loads are heavy, even I- section with cover plates may be sometimes insufficient. In such a case the following sections may be provided.

1. Two or more I- sections, connected appropriately (Uneconomical)
2. Plate girder
3. Truss girder

Plate girders can be built to any designed proportion truss girders are higher cost of fabrication and erection welded plate girders are preferred compared to riveted or bolted girders.

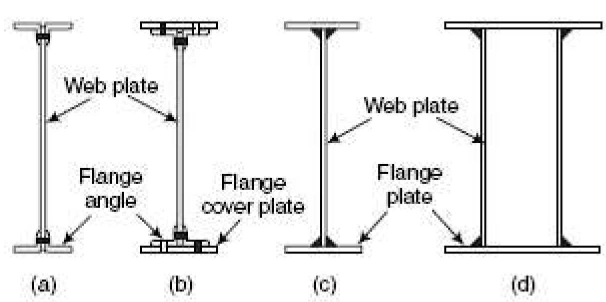


Plate girder arrangements

Elements of plate girder

1. Web 2. Flanges 3.Stiffeners

Design concept:

- Flexural strength of plate girder is governed by flanges.
- Limit sates considered are yielding of tension flanges and buckling of compression flanges.
- Vertical buckling into web and flange local buckling.
- Flange buckling due to, local torsional buckling.
- Total strength consists of strength prior to buckling plus the post-buckling strength derived from tension-field action.

If the unstiffened web is incapable of resisting the applied shear, spaced stiffener and intermediate stiffeners are used.

Additional stiffeners are used at points of concentrated loads called bearing stiffeners.

End bearing stiffeners are used to carry the reactions Shear flow,

F = $\frac{V\,\overline{Ay}}{I}$

Web thickness:

Initially the thickness of web is assumed as

1) 6mm if painted
2) 8mm if unpainted

Minimum web thickness:

Thickness of web should meet the following serviceability and compression flange buckling criterion.

a) Serviceability criterion: (Pg- 63-65 of IS:800-2007)

1) When transverse stiffeners are not provided

   $\frac{d}{t_w} \leq 200 \in$ (Web connected to flanges along both longitudinal edges)

   $\frac{d}{t_w} \leq 90 \in$ (Web connected to flanges along one longitudinal edge only)

2) When only transverse stiffeners are provided

   $\frac{d}{t_w} \leq 200 \in_w$ For 3d$\geq c \geq d$

   $\frac{c}{t_w} \leq 200 \in_w$ For 0.74d $\leq c \leq d$

   $\frac{d}{t_w} \leq 270 \in_w$ For c $< 0.74d$

   For c > 3d, Web is considered unstiffened.

3) When transverse stiffeners and longitudinal stiffeners are provided at one level only, at 0.2d from the compression flange.

   $\frac{d}{t_w} \leq 250 \in_w$ For 2.4 d $\geq c \geq d$

   $\frac{c}{t_w} \leq 250 \in_w$ For 0.74d $\leq c \leq d$

   $\frac{d}{t_w} \leq 340 \in_w$ For c $< 0.74d$

4) When there is a second longitudinal stiffeners provided at neutral axis

$\frac{d}{t_w} \leq 400 \in_w$

Where d=depth of the web

t_w= thickness of the web

$\in_w = \sqrt{\frac{250}{f_{yw}}}$,

f_{yw} = Yielding stress of the web

The above criteria are to ensure

i) Web will not buckle under normal service conditions
ii) Web is strong enough so that flange will not buckle into web.
iii) if c/d <1.0 that web panel has higher strength.

b) <u>Compression Flange buckling criterion:</u>

- When a plate girder bends, its curvature gives vertical compression in the web due to downward vertical component of compression flange bending stress and upward vertical component of tension flange bending stress.
- The web must have sufficient vertical buckling strength to withstand this squeezing effect.
- To prevent vertical buckling of flange into web before the yield stress is reached into the flange, the maximum permissible web slenderness ratio $\left(\frac{d}{t_w}\right)$ which depend on stiffener spacing $[\left(\frac{c}{d}\right) ratio - aspect\ ratio]$ is limited by the code.
- for trail $\left(\frac{d}{t_w}\right)$ may be assumed between 135 and 240.

<u>To avoid buckling of compression flange: (Pg- 64 of IS:800-2007)</u>

1) When transverse stiffeners are not provided $\frac{d}{t_w} < 345 \in_t{}^2$
2) When only transverse stiffeners are provided
   i) $\frac{d}{t_w} \leq 345 \in_t{}^2$ for $c \geq 1.5d$
   ii) $\frac{d}{t_w} \leq 345 \in_t$ foe c< 1.5ds

   Where c- spacing of transverse stiffeners.

<u>Depth of web:</u>

Depth of plate girders vary of $\frac{1}{8}$ to $\frac{1}{12}$ of their spans for different spans and loading condition

Depth of plate girders (d) for which area of steel used is minimum, will have minimum weight and is called optimum depth.

Optimum depth (d):

Resting moment of I-section is resisted by flanges.

$M_z = f_y b_f t_f d$ ______________(1)

f_y- Design strength of flanges

b_f-$width\ of\ flange$

t_f- Thickness of the flange

d- depth of the web

The gross c.s area of the beam,

$A=2b_f t_f+dt_w$ ______________________(2)

using (1) and (2)

$A= \frac{2M_z}{df_y} + dt_w$______________________(3)

Let k-Slenderness ratio of the web, $k = \frac{d}{t_w}$

then,

$A= \frac{2M_z}{kf_y t_w} + kt_w^2$ ___________________(4)

Optimum value of t_w may be obtained by $\frac{dA}{dt_w} = 0$

$$\frac{dA}{dt_w} = \frac{-2M_z}{kf_y t_w^2} + 2kt_w = 0$$

$$\frac{2M_z}{kf_y t_w^2} = 2kt_w$$

$$t_w^3 = \frac{M_z}{k^2 f_y}$$

$t_w = \sqrt[3]{\frac{M_z}{k^2 f_y}}$_______________________(5)

Equation 3 may be rewritten as

$A=\frac{2M_z}{df_y} + \frac{d^2}{k}$_____________________(6)

To obtain optimum d, $\frac{dA}{dd} = 0$

$$\frac{dA}{dd} = \frac{-2M_z}{d^2 f_y} + \frac{2d}{k} = 0$$

$$d^3 = \frac{kM_z}{f_y}$$

$$d = \sqrt[3]{\frac{kM_z}{f_y}} \quad \text{\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_} (7)$$

After establishing the depth of girder, it is assumed that shear taken by web and moment as taken by flange.

Proportioning the flanges:

It is desirable to maximize the lever arm of internal flange forces for taking move moment and minimize the flange material.

For riveted or bolted plate girder:

Flanges are with pairs of angles with or without cover plates unequal angles with long leg horizontal are preferred as M.I will be more and longer length is available for connections with the flange plate. Normally riveted/bolted plate girder without flange cover plates is economical.

For welded plate girder:

However in a welded plate girder, the flange consists of one plate only and variation in flange area is achieved by varying the width or thickness of flange if more than one flange plate is provided, they are connected together by longitudinal welds.

Flange area:

For a non-composite plate girder the ratio of width of flange and depth of section may be chosen as 0.3 and for a composite section it may be kept as 0.2.

$\frac{b_f}{t_f}$ ratio should be such that the section is either plastic or compact or semi-compact to avoid local buckling before reaching yield stress.

Flange size may be selected that it will be overloaded in bending.

Assuming that flanges carry the bending moment

$$M_z = A_f \frac{f_y}{\gamma_{mo}} d \qquad (d \simeq d_1)$$

$$A_f = \frac{M\gamma_{mo}}{f_d d}$$

Where d_1= distance between centroids of flanges (initially depth of the girder)

If $d/t_w < 67\,\epsilon$, the web is not susceptible to local buckling and moment capacity is determined similar to restrained beams.

Flexural strength: (Pg- 59 of IS:800-2007)

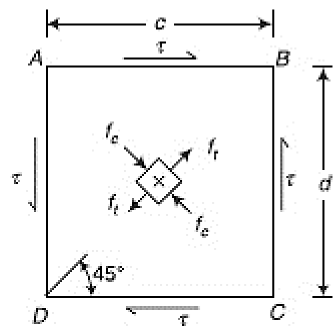
Flexural strength of plate girder is based on the tension flange yielding or compression flange buckling. The buckling of compression flange is governed by local buckling or lateral – torsional buckling of the flange.

If $\frac{d}{t_w} > 67 \epsilon_w$ and plastic, compact or semi-compact, the web is susceptible to shear buckling.

Shear strength of web: (Pg- 59 of IS:800-2007)

Shear buckling of web occurs because of diagonal compression and depends upon $\frac{d}{t_w}$ and spacing of intermediate stiffeners. Shear capacity of the web comprises of the strength before the onset of buckling and post-buckling strength.

Pre-buckling behavior:



The fig shows a square web plate between two vertical stiffeners subjected to vertical and complimentary shear stress. Consider an element x in equilibrium the element is subjected to principal compression along diagonal AC and principal tension along BD . On increasing the applied loads gradually, the shear stress also increases in consequence and plate buckles along the comp diagonal AC the shear stress beyond which the plate cannot take the compressive stress is called "Elastic critical shear" ($\tau_{cr.e}$)

$$\tau_{cr,e} = k_v \frac{\pi^2 E}{12\,(1-\mu^2)\left(\frac{d}{t_w}\right)^2}$$

As per IS:800, shear buckling should be verified when

$$\frac{d}{t_w} > 67\,\varepsilon_w \qquad \text{(for unstiffened web)}$$

$$\frac{d}{t_w} > 67\,\varepsilon_w \sqrt{\frac{k_v}{5.35}} \qquad \text{(for stiffened web)}$$

k_v - shear buckling coefficient

Shear Buckling Design Methods: **(Pg- 59-60 of IS:800-2007)**

i. **Simple Post-Critical Method:**
This method, based on the shear buckling strength, can be used for web of the I-section girders with or without intermediate stiffeners, provided the web has transverse stiffeners at the supports.

$$V_n = V_{cr}$$

where V_{cr} = shear force corresponding to web buckling $= A_v \tau_b = d t_w \tau_b$

τ_b = shear stress corresponding to web buckling determined as follows:

$= \dfrac{f_{yw}}{\sqrt{3}}$ for $\lambda_w \leq 0.8$ (shear yielding)

$= [1 - 0.8(\lambda_w - 0.8)](f_{yw}/\sqrt{3})$ for $0.8 < \lambda_w < 1.2$ (shear buckling)

$= \dfrac{f_{yw}}{\sqrt{3}\lambda_w^2}$ for $\lambda_w \geq 1.2$ (shear buckling plus tension field action)

λ_w = non-dimensional web slenderness ratio for shear buckling stress

$= \sqrt{f_{yw}/(\sqrt{3}\,\tau_{cr,e})}$

$\tau_{cr,e}$ = the elastic critical shear stress of the web

$$= \frac{k_v \pi^2 E}{12(1-\mu^2)(d/t_w)^2}$$

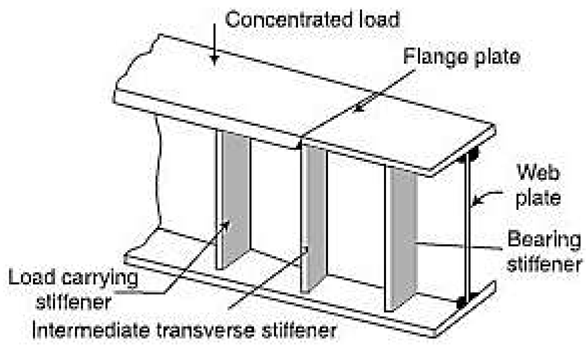
where μ = Poisson's ratio

k_v = 5.35 when transverse stiffeners are provided only at supports

$= 4.0 + \dfrac{5.35}{(c/d)^2}$ for $c/d < 1.0$

$= 5.35 + \dfrac{4.0}{(c/d)^2}$ for $c/d \geq 1.0$

Where c, d are spacing of transverse stiffeners and depth of the web.



Welded plate girder

Stiffeners: (Pg- 60 of IS:800-2007)

1. Intermediate stiffeners:

   These web stiffeners are used to prevent web buckling due to shear.

2. Bearing stiffeners:
   These are used to prevent crushing of web under concentrated loads or reactions and checked against local buckling (crippling) of web.

Intermediate stiffeners:

1. Transverse stiffeners(Vertical stiffeners):

These increase the buckling resistance of web due to shear. They have to satisfy

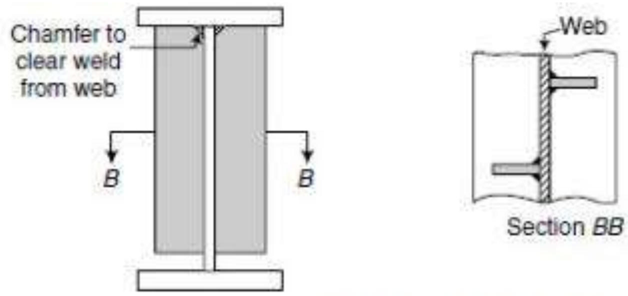
i. Sufficiently stiff so as not to deform appreciably as web tends to buckle.
ii. Sufficiently strong to withstand the shear transmitted by the web.

Transverse stiffeners are generally used only on one side of the web plate.

Longitudinal stiffeners, when necessary may be located on the opposite side of transverse stiffeners.

Web stiffeners are welded continuously to the web plate.

Normally c/d ratio ranges from 1.2 to 1.6.



(d) Welded intermediate stiffener

Near support at end panel c/d ratio is in the range of 0.6 to 1.0.

Spacing of intermediate stiffeners are specified previously as given in the IS. Code.

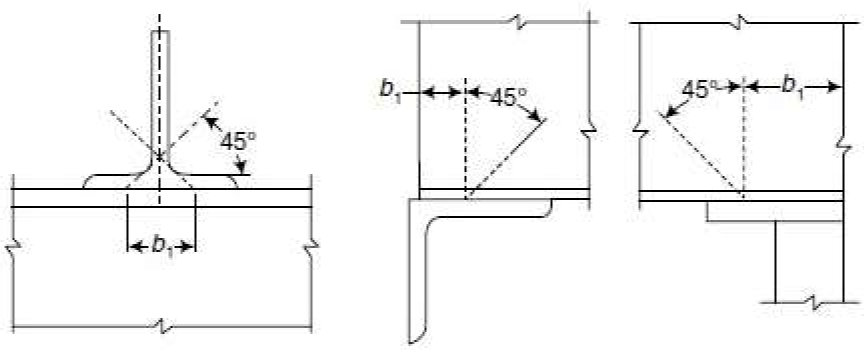
Outstand of web stiffeners:

Outstand of stiffener from face of web should not exceed $20t_q\epsilon$

where t_q – thickness of stiffener, When outstand is between 14 $t_q\epsilon$ & 20 $t_q\epsilon$, the design is on the basis of a core section with an outstand of $14t_q\epsilon$.

Stiff bearing length (b_1):

b_1 should be fixed such that it cannot deform appreciably in bending.



Minimum stiffeners:

Transverse web stiffeners not subjected to external loads or moments should have a second moment of area, I_s about centre line of the web, if stiffeners are on both sides of the web.

$$\text{if } c/d \geq \sqrt{2}, \qquad I_s \geq 0.75\, d t_w^3$$

$$\text{and} \quad \text{if } c/d < \sqrt{2}, \qquad I_s \geq \frac{1.5\, d t_w^3}{c^2}$$

Longitudinal stiffeners (Horizontal stiffeners):

They increase the buckling resistance considerably compared to transverse stiffeners when the web is subjected to bending.

They consist of angle section for a riveted/bolted plate girder and plate section for a welded plate girder and provided in compression zone at 1/5 distance from compression flange.

If required another horizontal stiffener is provided at neutral axis.

Required M.I for horizontal stiffener $I = ct_w^3$

Where c- spacing of vertical stiffeners

t_w - thickness of web.

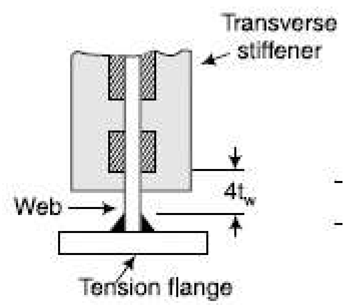
Required M.I for horizontal stiffener at N.A , $I = d_2 t_w^3$

d_2 = twice the clear distance between compression flange plate to the neutral axis.

Connection of intermediate stiffeners:

Intermediate stiffeners are to withstand a shear between each component of the stiffener and the web and not less than $t_w^2/ 5b_s$ in (kN/mm)

Where t_w – web thickness
b_s- outstand width of the stiffener in mm.



Intermediate web stiffener:

Load carrying stiffeners:

Bearing Stiffeners:

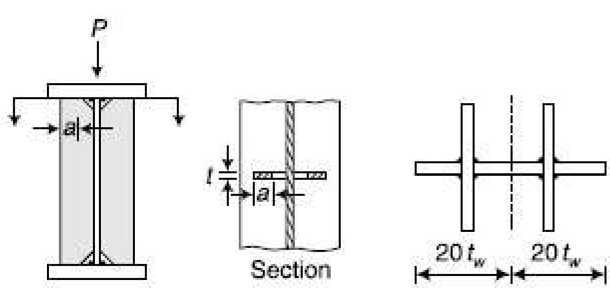
Bearing stiffeners are used to transfer concentrated loads on the girder and heavy reactions at supports to the full depth of the web.

These are required when web is insufficient strength for any of the limit states of web yielding, web crippling or sideways web buckling.

A bearing stiffener at support is called "End bearing stiffener".

They are placed

1. Tight with the web
2. Straight (it is not crimpled or joggled)
3. In pairs of two or four plate for welded girder.



(e) Welded bearing stiffener

Bearing stiffeners should be designed for the applied loads or reaction less than the local capacity of the web, F_w is given below

$$F_w = \frac{(b_1 + n_2)\, t_w f_{yw}}{\gamma_{m0}}$$

Where b_1 - stiff bearing length

n_2 – length obtained by dispersion through flange to the web junction at slope 1:2.5 to the

plane of the flange.

t_w – web thickness

f_{yw} – yield stress of web.

Bearing stiffeners are required to prevent sideway web buckling only under limited number of circumstances.

Torsional stiffeners:

When bearing stiffeners are required to provide torsional restraint at the supports of the beam, they should meet the following criteria.

M.I of stiffener $I_s \geq 0.34\, \alpha_s D^3 T_{cf}$

$$\text{where} \quad \alpha_s = 0.006 \quad \text{for} \quad \frac{L_{LT}}{r_y} \leq 50$$

$$= \frac{0.3}{(L_{LT}/r_y)} \quad \text{for} \quad 50 < \frac{L_{LT}}{r_y} \leq 100$$

$$= \frac{30}{(L_{LT}/r_y)^2} \quad \text{for} \quad \frac{L_{LT}}{r_y} > 100$$

D- overall depth of the beam at support.

T_{cf} - maximum thickness of compression flange in the span under consideration.

L_{LT} – laterally unsupported effective length of compression flange of beam.

r_y – radius of gyration of beam about minor axis.

Curtailment of flange:

For welded plate girder, the curtailment of flange plate consists in reducing the thickness of flange plate in proportion to the moments along the span.

Design Procedure for Plate Girder:

1. Determine whether the girder is to be designed assuming that it is laterally restrained throughout. For cases where compression flange is not restrained, lateral torsional buckling may occur and will have to be designed.
2. Self-weight of plate girder is to be assumed.
   Imposed loads are estimated and critical moment and shear force in the girder are computed. The factored forces are determined by multiplying with γ_f.
3. Optimum thickness and depth of plate girder are determined by using

$$t_w = \left(\frac{M_z}{f_y k^2} \right)^{0.33}$$

$$d = \left(\frac{M_z k}{f_y} \right)^{0.33}$$

Where k is assumed suitably.

4. Flange area is computed $A_f = \dfrac{M_z \gamma_{m0}}{f_y d}$

   Outstand of flange plate is worked out from serviceability and buckling criteria. Once flange width is fixed, flange thickness = A_f / b_f
5. The flange is classified, Plastic flanges are preferred.

   $$\frac{b}{t_f} < 8.4\, \varepsilon_f$$

   where b = outstand of flange $= \dfrac{b_f - t_w}{2}$
6. The trial section is checked for bending. Moment capacity is checked.

   $$M_d = \beta_b Z_p f_y / \gamma_{m0}$$
   $$\leq 1.2\, Z_e f_y / \gamma_{m0}$$

   M_d > M actual (Pg- 53 of IS:800-2007)
7. Flanges are curtailed in length to achieve an economical design.
8. Girder is checked for maximum shear.
   -This shear strength is a function of d/t_w ratio and aspect ratio c/d. Ascertain what type of stiffener are required. Web panel is checked by using either post critical method or tension field method.
9. Design the stiffeners.
10. The plate girder is checked for web buckling.
11. The plate girder is also checked for web bearing.
12. Connection of web to flange and web to stiffeners are designed.
13. All web and flange connections are made using fillet weld connections.

UNIT-6

ROOF TRUSSES

Syllabus:

Roof Trusses: Types of roof trusses- design loads- load considerations- IS Code recommendations, structural details- design of simple roof trusses- purlins, members and joints with welding- tubular trusses.

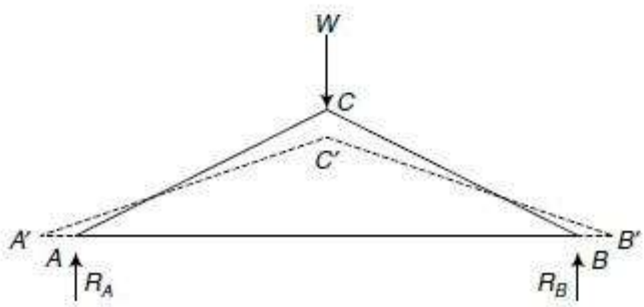
Learning Outcomes:

The student will be able to

- understand different types of trusses used in industrial buildings and the nature of the loads acting.
- determine the wind pressure on trusses using IS 875 (Part 3).
- design the connections in a roof truss.
- design tension and compression members using IS 800: 2007 code.

Learning Material:

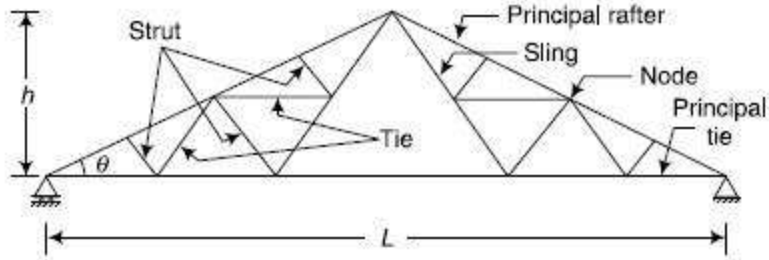
Roof trusses are integral part of an industrial building. Other structures where roof trusses are the choice are for long span floors and roofs of cinema halls, commercial complexes, stadiums etc. Trusses in the form of bracings are used to resist lateral loads and to provide lateral stability.
A pin jointed truss consists of triangular network of compression and tension members.



Let ABC is a triangular truss under the action of gravity load W at C, the truss will deflect to shape A'B'C' as shown in fig.

A& B move outwards putting member AB in tension and C will move downwards putting the members AC & BC in compression.

The points of intersection of truss members are "node" points and the distance between nodes is called "Panel Length".

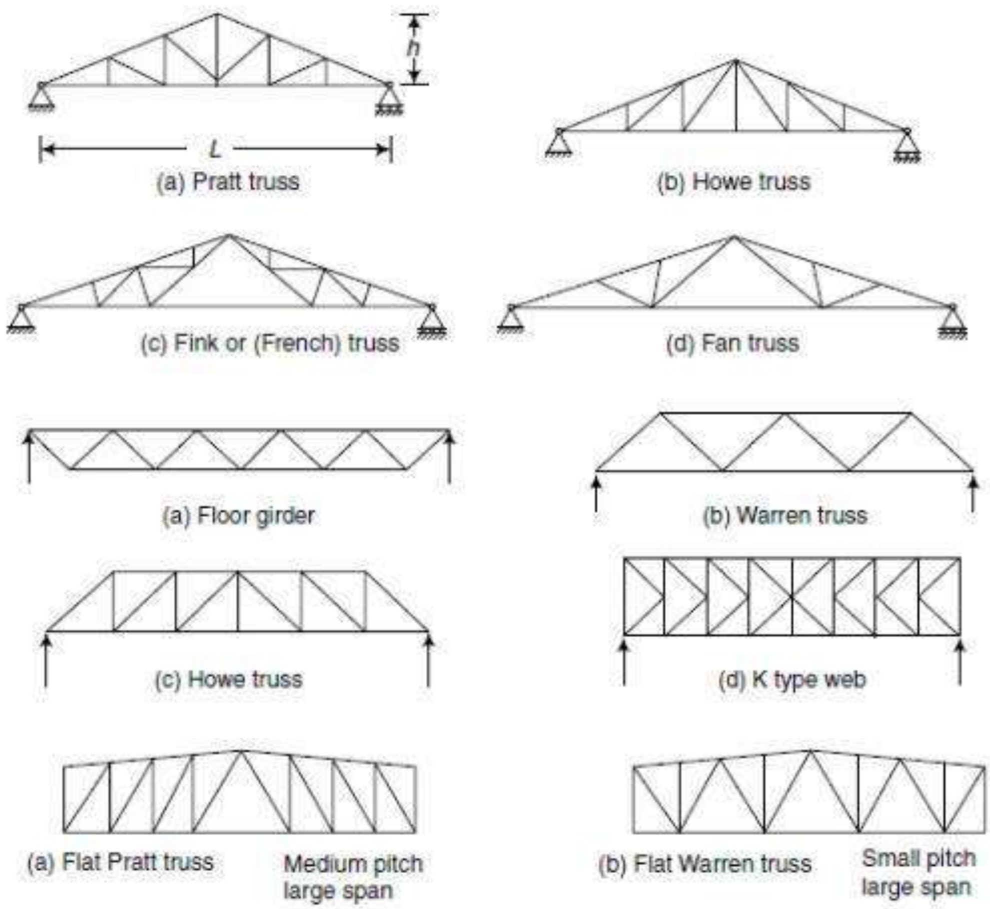


Nomenclature of truss members

Principal rafters and struts are compression members.

Principal ties and sag ties are tension members.

Types of trusses:



Spacing of trusses:

- The spacing between trusses is determined by the required column spacing and by considerations of minimum cost for the structure as a whole.
- The usual economical spacing ranges from 4 m to 8 m, with the lower limit for short truss spans and the higher limit for long spans about 30m or over.
- If the spacing of the trusses is small, the cost of the trusses per unit area increases

The economic spacing of roof trusses:

$$t = 2p + r$$

Where

- t – cost of the trusses per unit area
- p – cost of the purlins per unit area
- r – cost of the roof covering per unit area.

Panel length:

- It is economical to place purlins at panel points to avoid bending in the top chords.
- The framing of the truss member depends upon the spacing of the purlins.

Loads on Roof Truss:

Roof trusses are subjected to DL, LL, snow loads and wind loads.

Dead Load: Dead loads of he truss include the DL of roof material, purlins, trusses and bracing system.

The self weight of roofing material is described in IS-1911.

Asbestos Cement(A.C)Sheets - 120 to 156 N/m^2

Galvanized Iron(G.I) Sheets - 52.2 to 138.1 N/m^2

Roofing tiles - 350 to 850 N/m^2

Self-weight of purlin is known in advance as these are designed prior to truss.

Dead weight of truss = 10% of the loads on truss.

Dead weight of roof truss in N/m^2 = ((Span/3) + 5)*10.

Weight of bracing may be assumed 12-15 N/m^2 of plan area.

Live loads as per IS 875

Roof Slope	Access	Live load
$\leq 10^o$	Provided	1.5 kN/m^2 of plan area
$> 10^o$	Not provided	0.75 kN/m^2 of plan area

Note:

For roof sheets on purlins, 0.75 kN/m^2 less 0.01 kN/m^2 for every degree increase in slope upto 20^o and 0.02 kN/m^2 for every degree increase > 20^o

Snow load: If the structure is situated where the roof is subjected to snow anytime, the load considered for design should be max of the live load or snow load. Snow load also depends upon the pitch of the roof, the shape and roof material.

Snow load = 2.5 N/m^2 per mm length of snow. For slopes > 50^o , the snow load is neglected.

Wind loads:

The most critical loads on roofs of industrial buildings is wind load.(Ref IS 875 (Part III))

As wind blows against a structure, its surface (outer) experiences the effect of wind force.

Wind exerts pressure or suction on exterior surfaces of a building.

For low height structures, it is not that important, but for tall structures its effect is significant and requires serious consideration.

Wind pressure intensity at any height of a structure depends upon the velocity, density of air, shape & height of structure, topography and angle of wind track.

Design Wind Pressure:

Let V_b – Basic max. wind pressure as per IS-875:1987, it depends on a short interval of time of 3 seconds, with a 50 yrs return period, for different zones of the country.

This wind speed is then modified to obtain the design wind velocity (V_z) as follows:

$V_z = V_b \, . \, k_1.k_2.k_3$

Where V_z = design wind velocity at any height 'z' in m/s.

k_1 = probability factor or risk coefficient.

k_2 = terrain, height and structure size factor.

k_3 = topography factor.

Terrain factor (k_2):

It is grouped into four categories

1. When height < 1.5 m – open sea coasts, flat treeless plains.
2. Height < 1.5 – 10 m - parks, air fields, open lands.
3. Height upto 10 m – wooded areas, towns, industrial areas.
4. Height > 25 m - large city centresm well developed industrial complexes.

On the basis of category (1,2,3 or 4) structures are grouped as

Class – A – Max.dimension of structure 20 m. Class – B- Max.dimension of structure 20-50 m. Class – C- Max.dimension of structure > 50 m

<u>k_3 Factor:</u>

For ground level where wind slope is less than 3^0

$k_3 = 1$

For wind slope greater than 3^0 , k_3 = 1.0 to 1.36

For hill or ridge , k_3 = 1+ C s

Where

C = 1.2(Z/L) for upwind slope 3^0 - 17^0

= 0.36 for upwind slope >17^0

Z- Height of crest of hill

L- Projected length of upwind zone from average ground level of crest in wind direction

S- A factor from figure of hill or ridge

Design wind pressure (in N/m^2), at height z $p_z = 0.6\, V_z^2$ ______________(2)

Wind load in roof:

To calculate wind load on individual structural element, like roofs, wall and individual cladding units, it is essential to take account of the pressure difference between opposite faces of such elements. For clad structures, it is necessary to know the internal and external pressures.

Wind load 'F' acting normal to the individual structural element or cladding unit is given by

$$\boxed{F = (C_{pe} - C_{pi})\, A\, p_d}$$

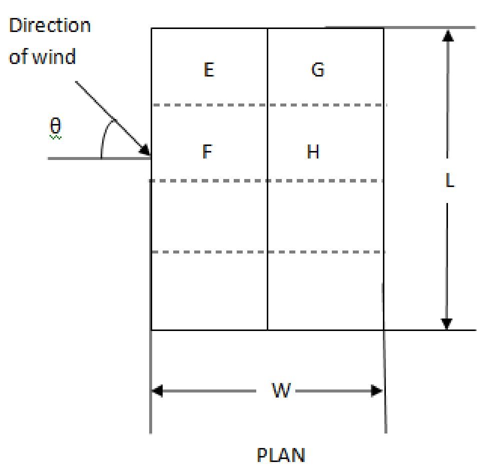
Where C_{pe} – External pressure coefficient

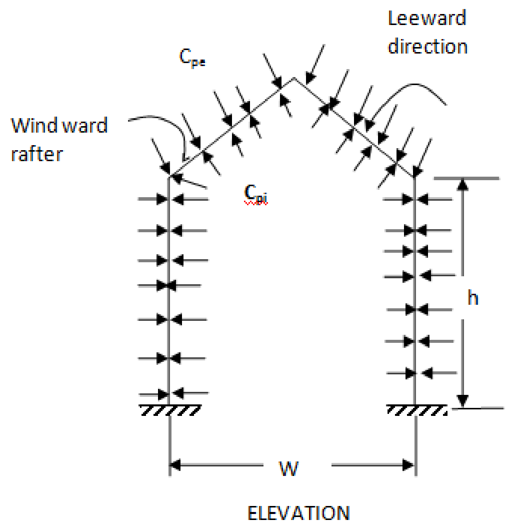
C_{pi} – Internal pressure coefficient

A – Surface Area of element under consideration

p_d - Design wind pressure.

- Internal air pressure may be positive or negative depending upon the direction of flow of air with respect to openings in the building.
- When the air pressure is less than the atmospheric pressure, it is negative known as "suction".
- Openings in relation to the wall area define as "permeability of the building".





External pressure coefficients (C_{pe}) for pitched and rectangular clad building From IS 875 Part(3)

Building height ratio	Roof angle (α)	Wind angle θ = 0		Wind angle θ = 90^0	
		EF	GH	EG	FH
$\frac{h}{w} \leq \frac{1}{2}$	10	-1.2	-0.4	-0.8	-0.6
	20	-0.4	-0.4	-0.7	-0.6
	30	0	-0.4	-0.7	-0.6
$\frac{1}{2} < \frac{h}{w} < \frac{3}{2}$	10	-1.1	-0.6	-0.8	-0.6
	20	-0.7	-0.5	-0.8	-0.6
	30	-0.2	-0.5	-0.8	-0.8
$\frac{3}{2} < \frac{h}{w} < 6$	10	-0.7	-0.6	-0.8	-0.8
	20	-0.8	-0.6	-0.8	-0.8
	30	-1.0	-0.5	-0.8	-0.7

Internal Pressure Coefficients (C_{pi})

It depend upon the permeability of cladding to flow air

Permeability	Opening in relation to wall area (%)	Internal Air pressure Coefficient C_{pi}
Zero	0	±0.00
Normal	5	±0.20
Medium	5 -20	±0.50
Large	>20	±0.70

Note:

- Positive wind load indicates that the force acting towards structural element while negative indicates away from it.

Load combinations:

As per IS code various combinations of loads on roof trusses are considered and the critical condition is taken for the design.

1. DL + Snow load
2. DL + partial or full live load (which ever causes the maximum stress in the member)
3. DL + WL + Internal positive air pressure
4. DL + WL + Internal negative air pressure
5. DL + LL+ WL

Note:

a) 3 & 4 load combinations should be considered with wind direction normal or parallel to ridge, whichever is more.
b) The above load combinations should be used with appropriate partial load factors.

Example Problem:

Estimate the design wind pressure for a 100 m high lattice tower located on the out skirts of plain region of Visakhapatnam. Also estimate the wind load in terms of effective front area (A_e) of the tower.

Sol: From IS 875, V_b=50 m/s at Visakhapatnam

For 100m, k_1=1.05, k_2 for category 2 and class C=1.17

k_3=1.0 level ground

$V_z = V_b\, k_1\, k_2\, k_3$ = 50*1.07*1.17*1.0=62.6m/s

p_z= wind pressure = 0.6 V_z^2 =0.6*62.5^2 = 2343.75 N/m^2

Let solidarity ratio Φ=0.2, C_f = 2.7

Wind load, F = $C_f\, A_e\, p_d$ = 2.7* A_e *2343.75 = 6328.12 A_e Newtons

Example Problem:

Determine the design forces in the members of a Fink type roof truss for an industrial building for the following data. Also find the reactions.

Overall length of the building	48 m
Overall width of the building	16.5 m
Width (c/c of roof columns)	16 m
c/c spacing of trusses	8 m
Rise of truss	1/4 of span
Self weight of purlins	318 N/m
Height of columns	11 m
Roofing and side coverings	Asbestos cement sheets (dead weight = 171 N/m^2)

The building is located in industrial area Naini, Allahabad. Both the ends of the truss are hinged.

Use steel of grade Fe 410.

Solution

Truss configuration

Let α be the inclination of the roof with the horizontal.

$$\tan\alpha = \frac{4}{8} = \frac{1}{2}$$

$$\Rightarrow \quad \alpha = 26°34' = 26.566°$$

Length of rafter $= \sqrt{(16/2)^2 + 4^2} = 8.94$ m

Length of each panel L_0U_1, U_1U_2, U_2U_3, $U_3U_4 = 8.94/4 = 2.235$ m

Loads on panel points

(i) Dead Load

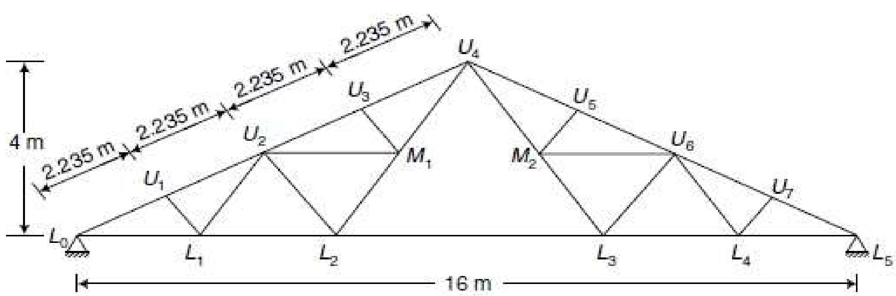
Assume weight of bracing = 12 N/m^2

Dead weight of AC sheets = 171 N/m^2

$$\text{Self-weight of roof truss} = \left(\frac{\text{span}}{3} + 5\right) \times 10$$

$$= \left(\frac{16}{3} + 5\right) \times 10 = 103.33 \text{ N/m}^2$$

$$\simeq 110 \text{ N/m}^2$$



Truss configuration

Self-weight of purlin $= 318$ N/m

$= 318 \times 8 = 2544$ N

Panel length $= 2.235$ m

The panel length in plan $= 2.235 \cos 26°34' = 2.00$ m

Load on each intermediate panel due to dead load

$$= (12 + 171 + 110) \times (8 \times 2) + 2544 = 7232 \text{ N} \simeq 7.4 \text{ kN}$$

Load on end panel points of the rafter = 7.4/2 = 3.7 kN

(ii) Live load

$$\alpha = 26°\,34' = 26.566°$$

Let us assume that no access is provided to the roof. The live load is reduced by 20 N/m^2 for each one degree above 10° slope.

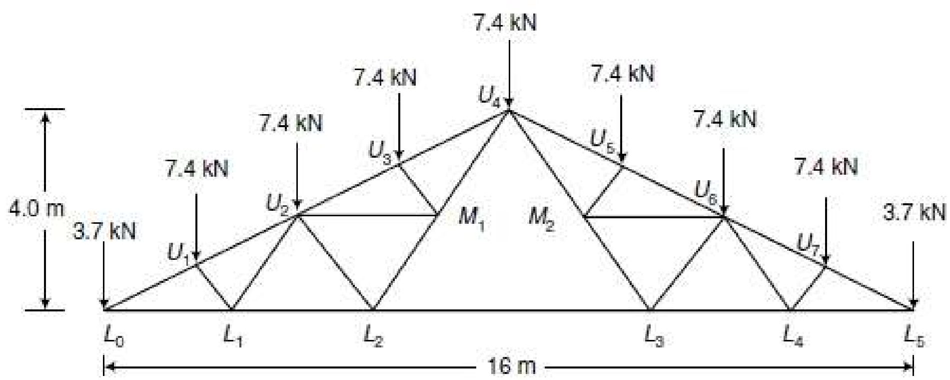
$\therefore$ Live load = 750 − 20 × (26.566 − 10) = 418.68 N/m^2

The load on each intermediate panel $= 418.68 \times 8 \times 2$

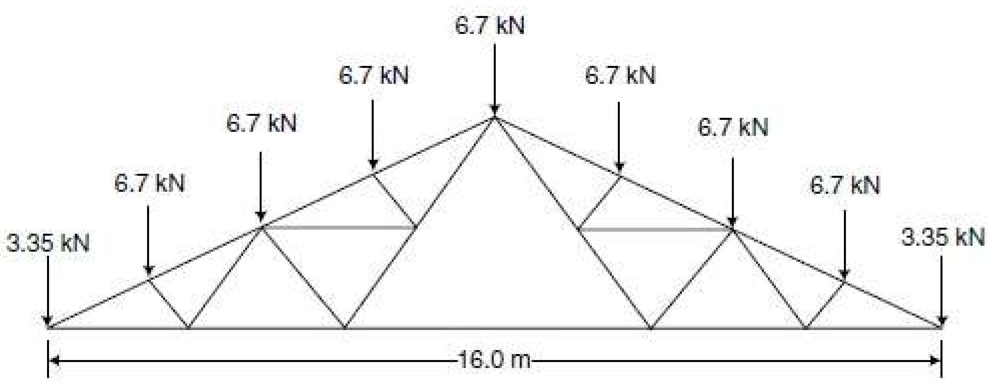
$= 6698.88 \text{ N} = 6700 \text{ N} = 6.7 \text{ kN}$

The load on each end panel point $= \dfrac{6700}{2} = 3350 \text{ N} = 3.35 \text{ kN}$

The resultant panel point loads due to dead and live loads are shown in



Dead loads at panel points



Live loads at panel points

Wind Loads

Let us assume the life of the industrial building to be 50 years and the land to be plain and surrounded by small buildings.

$k_1 = 1.0$	(for 50 years)
$k_2 = 0.89$	(for terrain category 3, building height = 11 m)
$k_3 = 1.0$	(for plain land)
$V_b = 47$ m/s	

Design wind speed, $V_z = k_1 k_2 k_3 V_b$

$= 1.0 \times 0.89 \times 1.0 \times 47 = 41.83$ m/s

Design wind pressure, $p_d = 0.6\, V_z^2$

$$= 0.6 \times 41.83^2 = 1049.8 \text{ N/m}^2 = 1.05 \text{ kN/m}^2$$

Height of building column above ground level, $h = 11$ m

Width of building,

$w = 16$ m

$$\frac{h}{w} = \frac{11}{16} = 0.6875 \qquad \left(\frac{1}{2} < \frac{h}{w} < \frac{3}{2}\right)$$

Let us assume the building to have normal permeability. For such condition, the internal pressure coefficient is ± 0.2, plus sign indicates pressure while minus sign indicates suction.

enlists external pressure coefficients for the condition $\frac{1}{2} < \frac{h}{w} < \frac{3}{2}$ and for the wind angle θ. When wind angle is 0°, the wind is normal to the eave strut and for wind angle of 90° the wind is normal to the ridge.

In the present example the roof angle α is 26.566° for which the coefficients are tabulated below. The wind force is given by

$$F = (C_{pe} - C_{pi})\, p_d A$$

The table also shows, wind force for various conditions.

Wind Angle $\alpha = 26.566$ Case	*Pressure Coefficients C_{pe} Windward*	*Pressure Coefficients C_{pe} Leeward*	*C_{pi}*	*$C_{pe} - C_{pi}$ Windward*	*$C_{pe} - C_{pi}$ Leeward*	*Wind Load (F) Windward*	*Wind Load (F) Leeward*
Normal to eave strut(0°)	−0.371	−0.5	0.2	−0.571	−0.70	−10.72	−13.14
	−0.371	−0.5	−0.2	−0.171	−0.30	−3.21	−5.63
Normal to ridge strut (90°)	−0.8	−0.731	0.2	−1.00	−0.931	−18.77	−17.48
	−0.8	−0.731	−0.2	−0.60	−0.531	−11.26	−9.97

1. The values of coefficient C_{pe} for various conditions in the table have been calculated by interpolation from
2. The critical wind load on panel points from the above table for windward sides are $-18.77 kN \simeq -18.8$ kN and on leeward side are $-17.48 \simeq -17.50$ kN
3. The calculation of critical wind loads on panel points are as follows.

   (i) *Windward side*

   $F_1 = (C_{pe} - C_{pi}) p_d A = (-0.8 - 0.2) \times 1.05 \times (8 \times 2.235) = -18.77 \simeq -18.8 kN$

(Intermediate panel points)

$$F_2 = -18.8/2 = -9.4 \text{ kN}$$

   (ii) *Leeward side*

(End panel points)

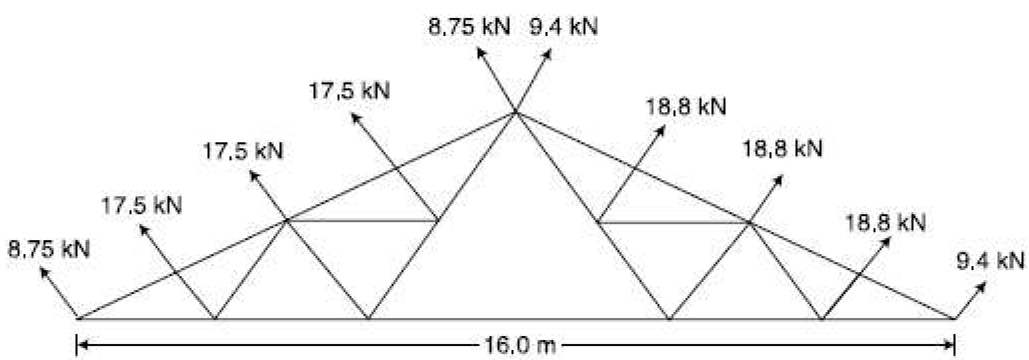
$F_3 = (C_{pe} - C_{pi}) p_d A = (-0.731 - 0.2) \times 1.05 \times (8 \times 2.235) = -17.48 \simeq -17.5$ kN

(Intermediate panel points)

$F_4 = -17.5/2 = -8.75$ kN

(End panel points)

The resultant panel point loads are shown in



Wind loads at panel points (wind from right)