PRESENTATION
ON
BOLTED CONNECTION

BY

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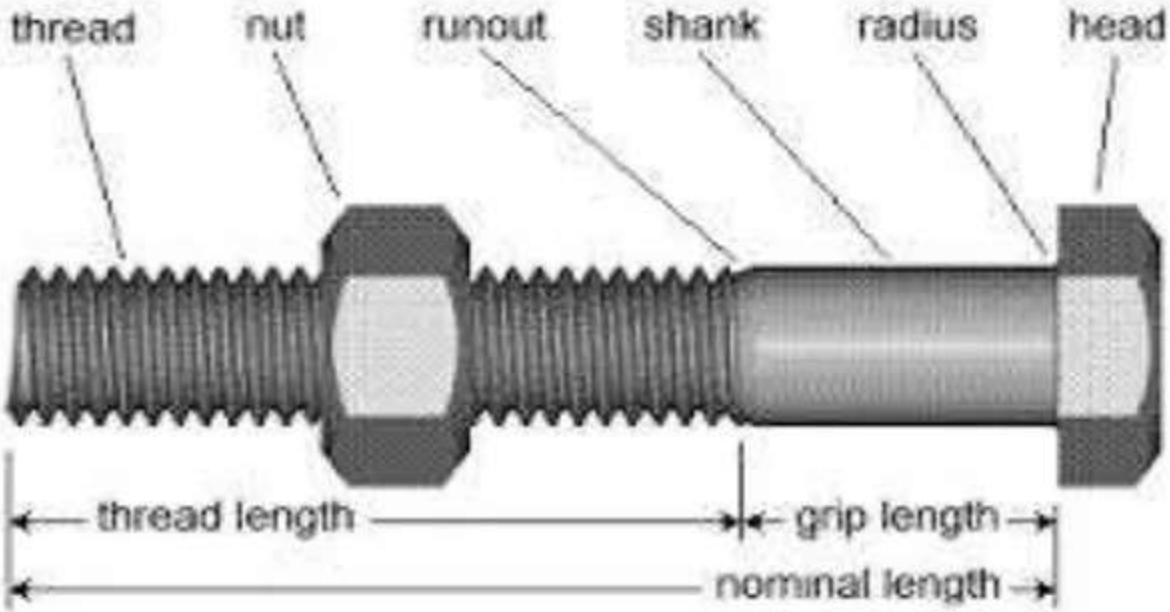
INTRODUCTION

- A bolt is a metal pin with a head formed at one end and shank threaded at the other in order to receive a nut.
- On the basis of load transfer in the connection bolts are classified as:
- (a) Bearing Type


thread
nut
runout
shank
radius
head
thread length
grip length
nominal length

Types of bearing bolts

- There are two types of bearing type bolts, namely,
- (i) Unfinished or Black Bolts.
- (ii) Finished or Turned Bolts.

Unfinished or Black Bolts.

- The shanks of black bolts are unfinished, i.e., rough as obtained at the time of rolling.
- For black bolts, diameter of bolt hole is larger and are used in most of the work.
- A black bolt is represented as M16, M20, etc. which means black bolt of nominal diameter

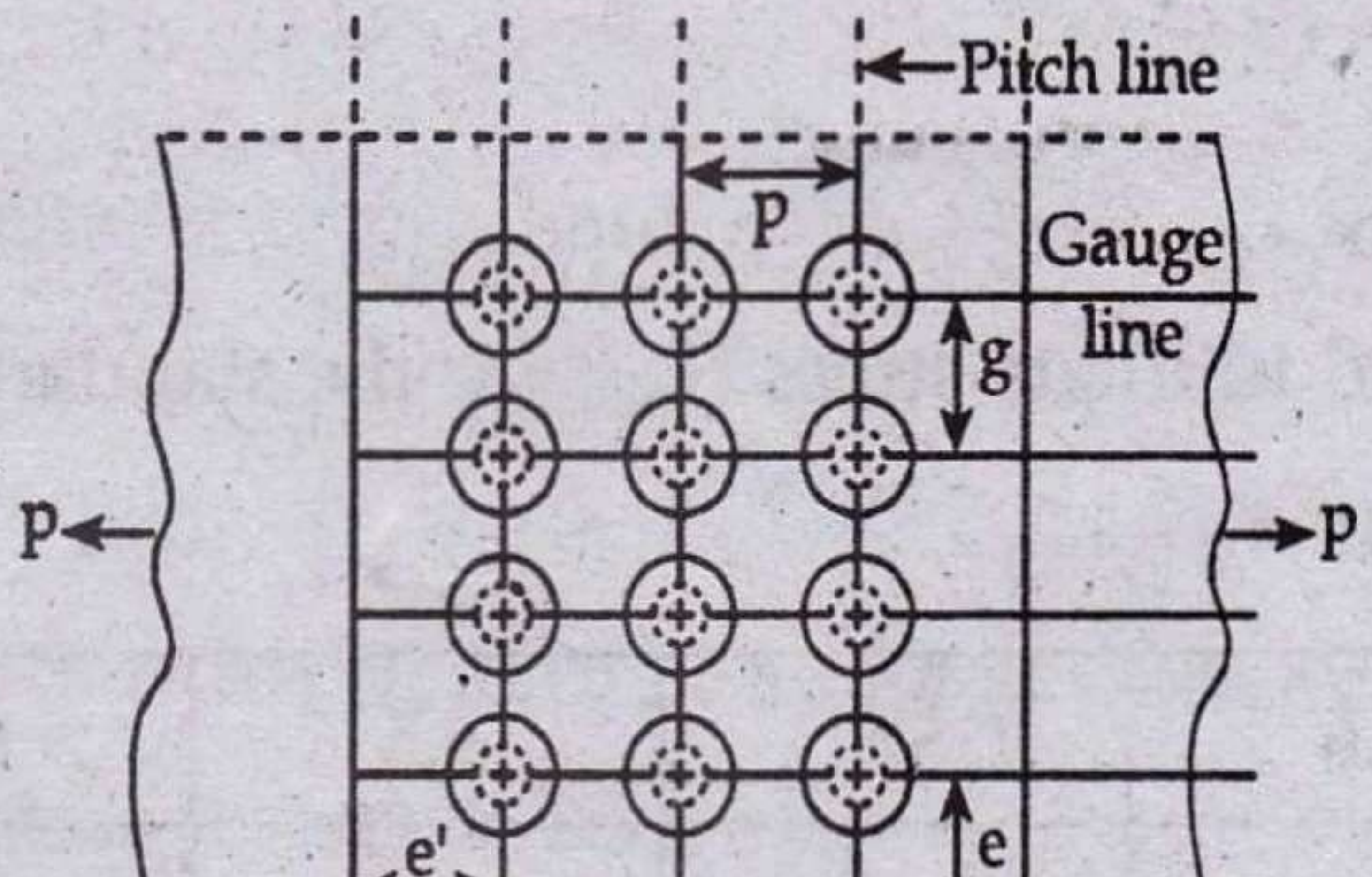
Finished or Turned Bolts.

- turned bolts are obtained by turning hexagonal shank to circular shape.
- The bolt hole diameter is only 1.5 mm larger than that of the shank in case of turned bolt.
- These bolts are used in special jobs like connecting machine parts subject to dynamic

TERMINOLOGY

The following terms used in bolted connection should be

1. ***Pitch of the Bolts (p)***: It is the centre-to-centre spacing of the bolts in a row, measured along the direction of load.
2. ***Gauge Distance (g)***: It is the distance between the two consecutive bolts of adjacent rows and is



Pitch line
p
Gauge line
g
P
P
e
e'

Grade Classification of Bolts

- The grade classification of bolt is indicative of the strength of the material of the bolt. The two grades of bolts commonly used are 4.6 and 8.8.
- For a 4.6 grade 4 indicates that the ultimate tensile strength of the bolt,

  fub = 4 x 100 = 400 N/mm2

  and, 0.6 indicates that the yield strength of the bolt,

  fyb= 0.6 x Ultimate strength

The yield stress (f_{yb}) and ultimate stress (f_{ub}) for the standard grades of bolts are given in the table below;

Grade of Bolt	4.6	5.6	6.5	6.8	8.8
Ultimate stress $(f_{ub})(N/mm^2)$	$= 4 \times 100$ $= 400$	$= 5 \times 100$ $= 500$	$= 6 \times 100$ $= 600$	$= 6 \times 100$ $= 600$	$= 8 \times 100$ $= 800$
Yield stress	$= 0.6 \times 400$	$= 0.6 \times 500$	$= 0.5 \times 600$	$= 0.8 \times 600$	$= 0.8 \times 800$

Specification

For connection, see section 10 code IS 800 : 2007.

1. Pitch 'p' shall not be less than 2.5d, where, 'd' is the nominal diameter of bolt (also see clause 10.2.2 of code IS 800 : 2007)
2. Pitch 'p' shall not be more than;

a) 16 t or 200 mm whichever is less, in case of tension members. (Also see clause 10.2.3.2 of code IS 800 : 2007).

c) In case of staggered pitch, pitch may be increased by 50 percent of values specified above provided gap distance is less than 75 mm. (See clause 10.2.3.4 of code IS 800 : 2007).

3. In case of butt joints maximum pitch is to be restricted to 4.5 d for a distance of 1.5 times the width of plate from the butting surface.

4. The gauge length 'g' should not be more than 100 + 4 t or 20 mm whichever is less.

5. Minimum edge distance shall not be;

a) Less than 1.7 x hole diameter in case of sheared or hand

6. Maximum edge distance (e) should not exceed

a) 12tE where, $E = \sqrt{\frac{250}{f_y}}$ and 't' is thickness of thinner outer plate (See clause 10.2.4.3 of code IS 800 : 2007).

b) 40 + 4 t where, 't' is the thickness of thinner connected plate, if exposed to corrosive influences.

Types of Bolted Connections

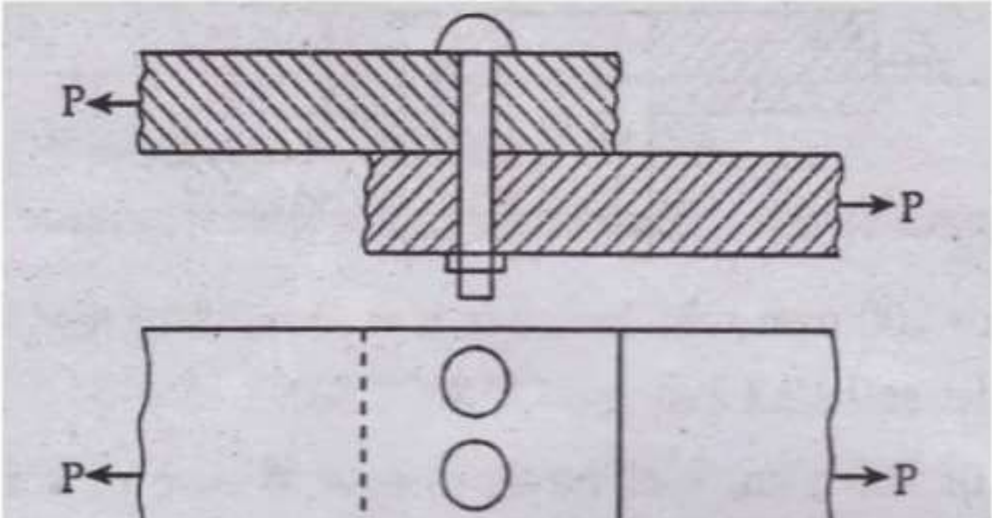
Types of joints may be grouped into the following two;

a) **Lap Joint:** It is simplest types of joints. In this, plates to be connected overlap one another.

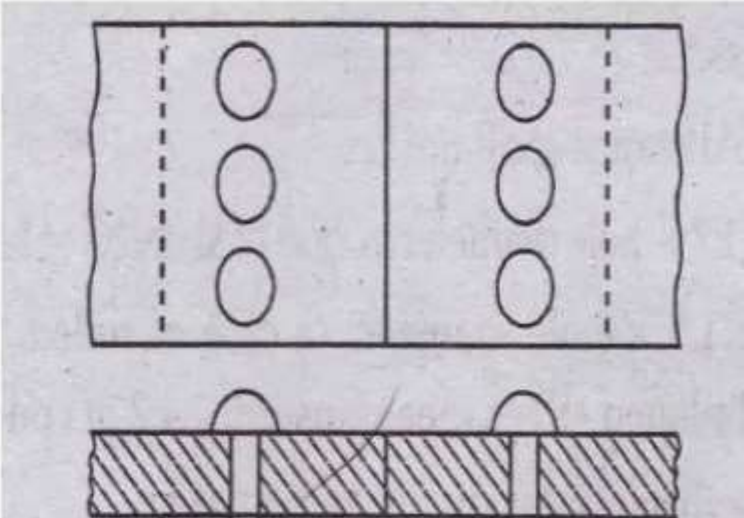
b) **Butt Joint:** In this type of connection, the two main plates about against each other and the connection is made by providing a single cover plate connected to the main plate or by double cover plates, one on either side connected to the main plates.

Bolted joint

lap joint



Butt joint



THANK YOU

Design of tension members

As per IS 800- 2007

Modes of failure

- ➢ *Gross section yielding*
- ➢ *Net section yielding*
- ➢ *Block shear failure*

❖ *Design strength of member is least of:-*

- *Strength due to yielding of gross section*

A. Design Strength due to Yielding of Gross Section

- The design strength of members under axial tension T_{dg},

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

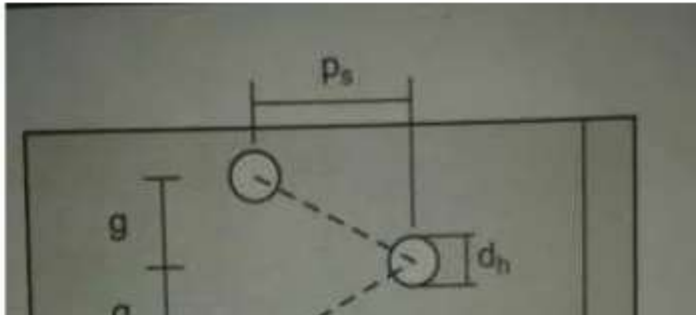
where $\gamma_{mo} = 1.1$

B. Design Strength due to Rupture of Critical Section

Plates – The design strength in tension of a plate, T_{dn},

$$T_{dn} = 0.9 f_u A_n / \gamma_{m1}, \quad \gamma_{m1} = 1.25$$

$$A_n = \left[b - nd_h + \sum \frac{p_i^2}{4 \ldots} \right] t$$



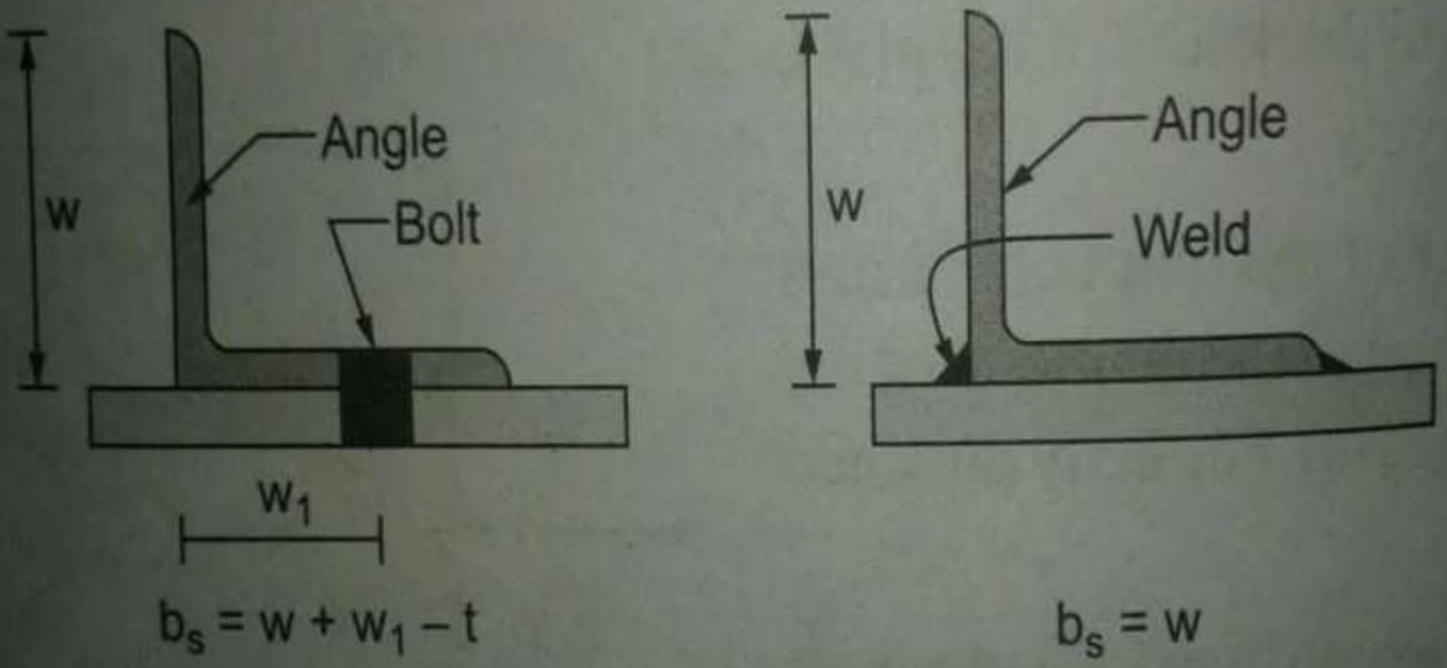


Fig. 4.4 Angles with single leg connections

- **Net section rupture in Threaded rods**

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

An = net root area at threaded section

- **Net section rupture in Single angles**

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

$\beta = 1.4 - 0.076\ (w/t)\ (f_u/f_y)\ (b_s/L) < (fy\ \gamma_{m0}/f_y\ \gamma_{m1})$

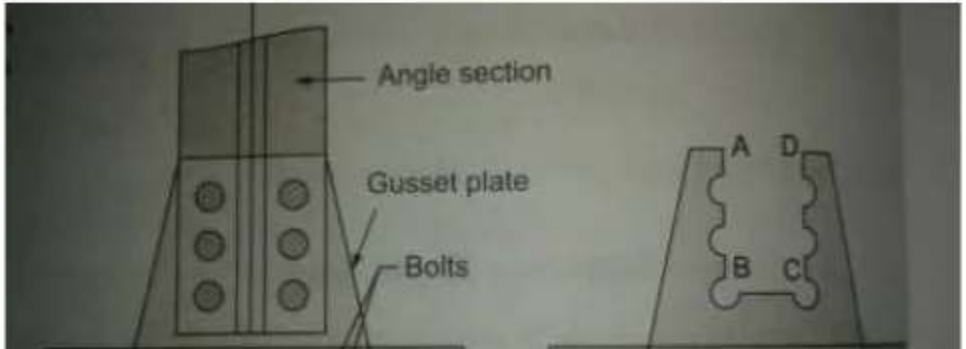
<0.7

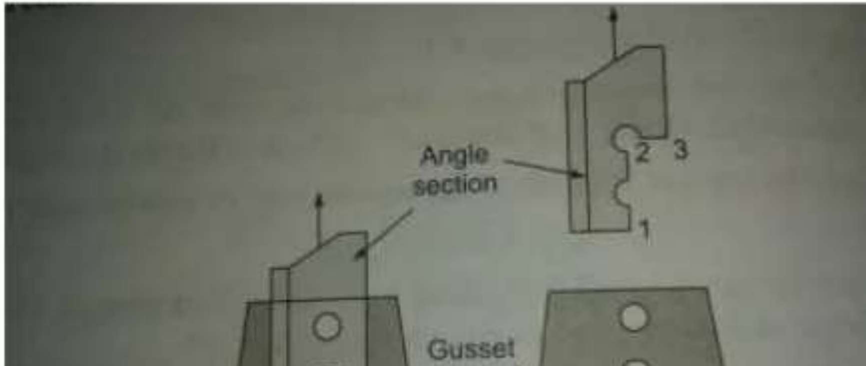
Legend refer code page 32: Design of Tension members

For preliminary sizing, the rupture strength of net section may be approximately taken as

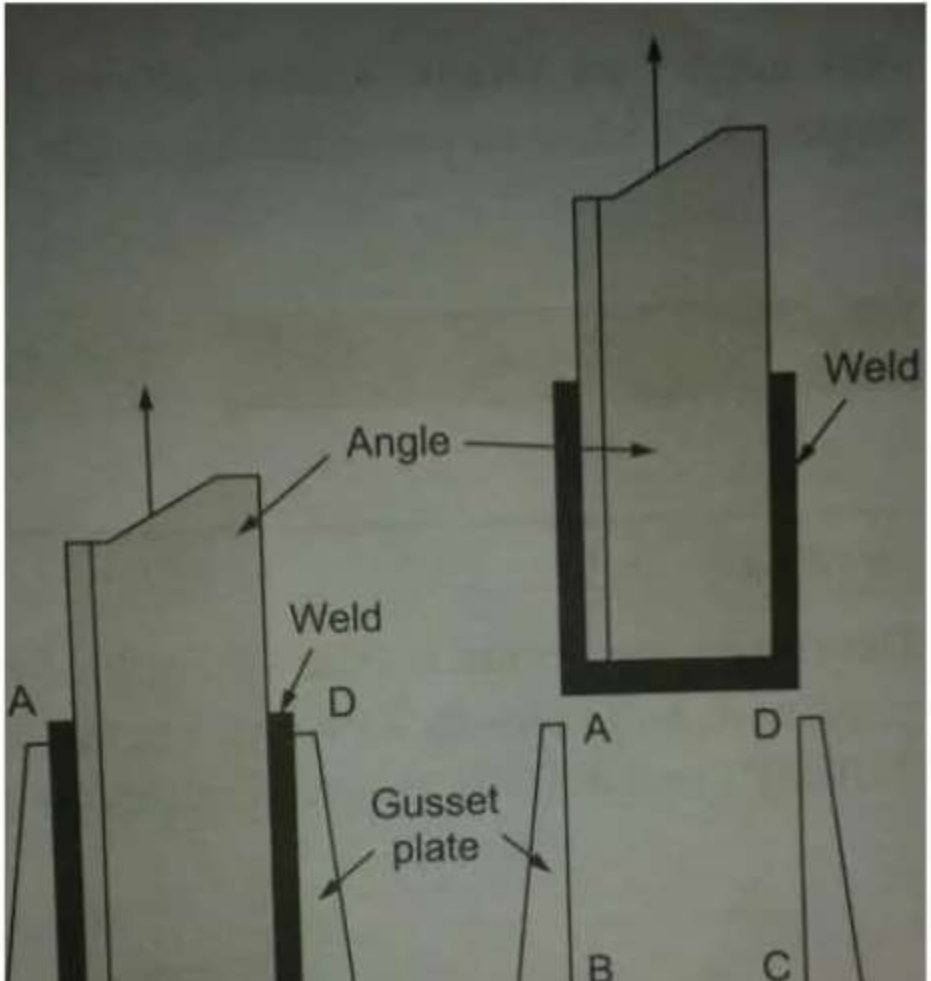
$$T_{dn} = \alpha A_n f_u / \gamma_m$$ (see page 33 of IS 800 : 2007)

BLOCK SHEAR FAILURE





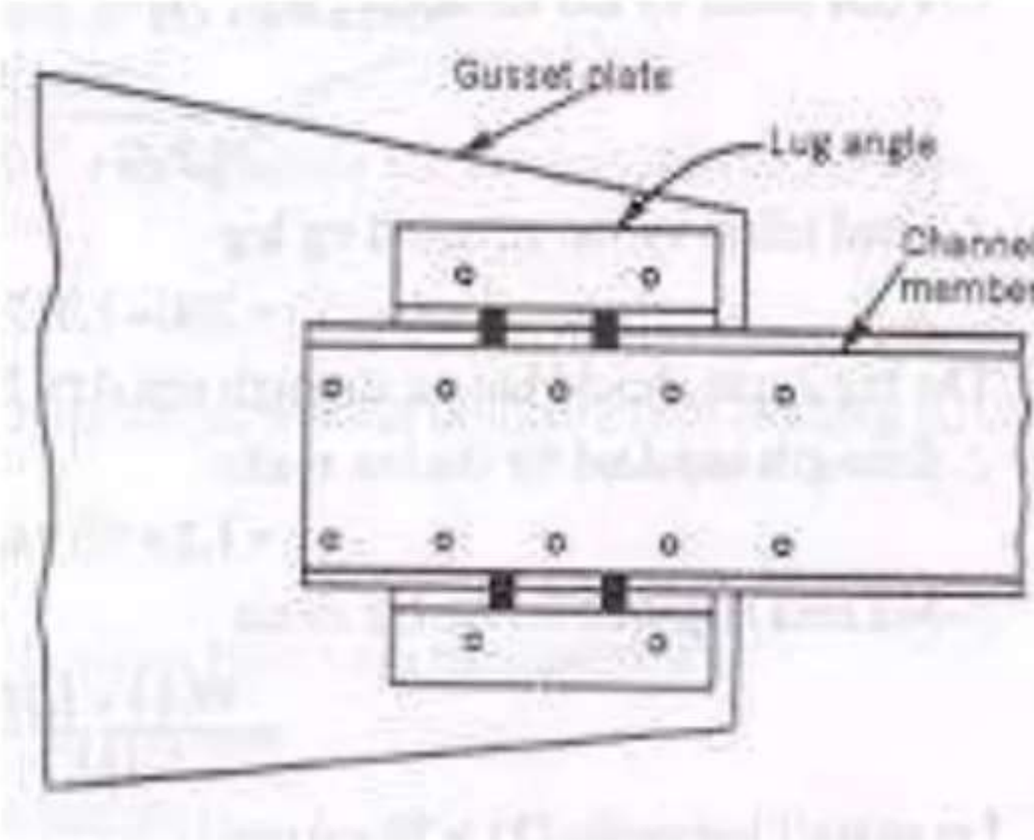
- Block shear failure is also seen in welded connections.
- A typical failure of a gusset in the welded connection is shown in the figure.
- The planes of failure are chosen around the weld
- Here plane B-C is under tension and planes A-B and C-D are in shear



Lug angles

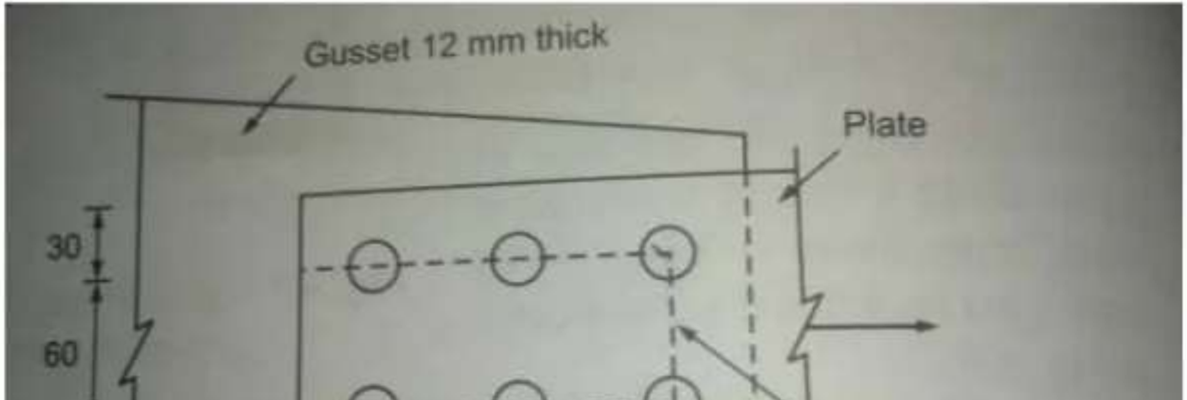
Short angles used to connect the gusset and outstanding leg of the main member as shown in figure

Refer Clause 10.12.2 of IS 800-2007



Problem

- Determine the tensile strength of the plate 120 mm x 8 mm connected to a 12 mm thick gusset plate with bolt holes as shown in the figure. The yield strength and ult. Strngth of the steel used are 250 MPa and 400 MPa. The diameter of bolt used is 16 mm.



Solution

- The design strength Td of the plate is calculated based on following criteria
- A) Gross section yielding:

The design strength Tdg of the plate limited to the yielding of gross cross section Ag is given by $$T_{dg} = f_y\, A_g\, /\gamma_{m0}$$

fy = 250 M pa

Step 1:-

Gross area of the tension member is obtained assuming that the section failed at yielding using equation

$$A_g = \frac{1.1 * T_{dg}}{f_y'}$$

Step 2:-

- **Taking into consideration reduction into the area due to holes and assuming efficiency 85%, area is increased by 15-20%.**

Assumed gross area= 1.15*Ag(required)

Step 3:-

- **Trial section is chosen by steel table by gros area.**

Step 4:-

Check as per I.S. Code 800:2007(Section 6)

A. **Design Strength due to Yielding of Gross Section**

- **The design strength of members under axial tension T_{dg},**

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

where $\gamma_{mo} = 1.1$

B. **Design Strength due to Rupture of Critical Section**

Plates – The design strength in tension of a plate, T_{dn},

$T_{dn} = 0.9 f_u A_n / \gamma_{m1}, \gamma_{m1} = 1.25$

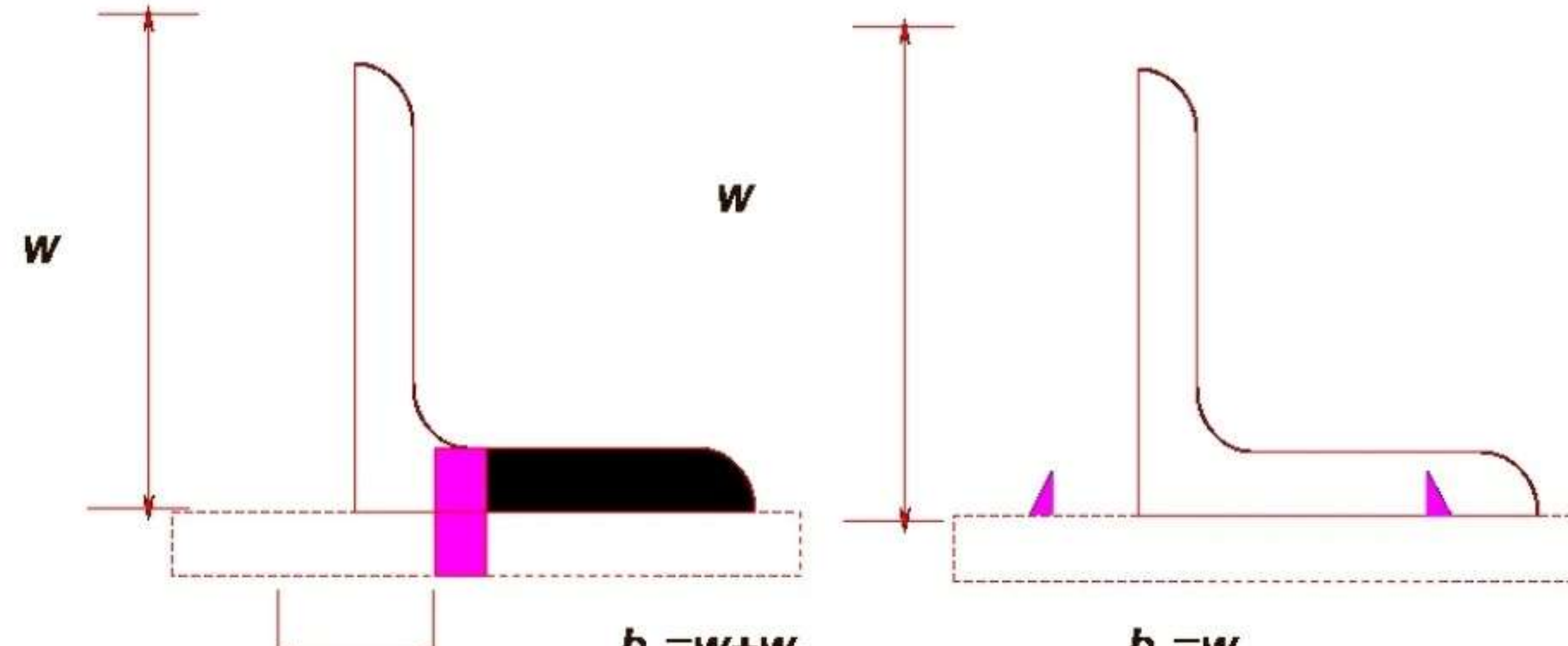
Single Angles – The design strength, T_{dn}, as governed by tearing

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

$$\beta = 1.4 - 0.076\, (w/t)\, (f_u/f_y)\, (b_s/L) < (fy\, \gamma_{m0} / f_y\, \gamma_{m1})$$

$$> 0.7$$

Alternatively, the tearing strength of net



w
w
b =w+w
b =w

ℂ. Design Strength due to Block Shear

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

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

or

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

Step 5:-

- **After checking design strength is less or too much in excess ,a second trial is selected and all design process are repeated**

Step 6:-

- **Slender ratio is also checked , it**

THANK YOU

Design of compression members

As per IS 800 : 2007

Compression Members

Compression members

- Structural Members subjected to axial compression/compressive forces
- Design governed by strength and buckling
- Columns are subjected to axial loads through the centroid.
- The stress in the column cross-section can be calculated as

$$f = \frac{P}{}$$

Failure modes of an axially loaded column

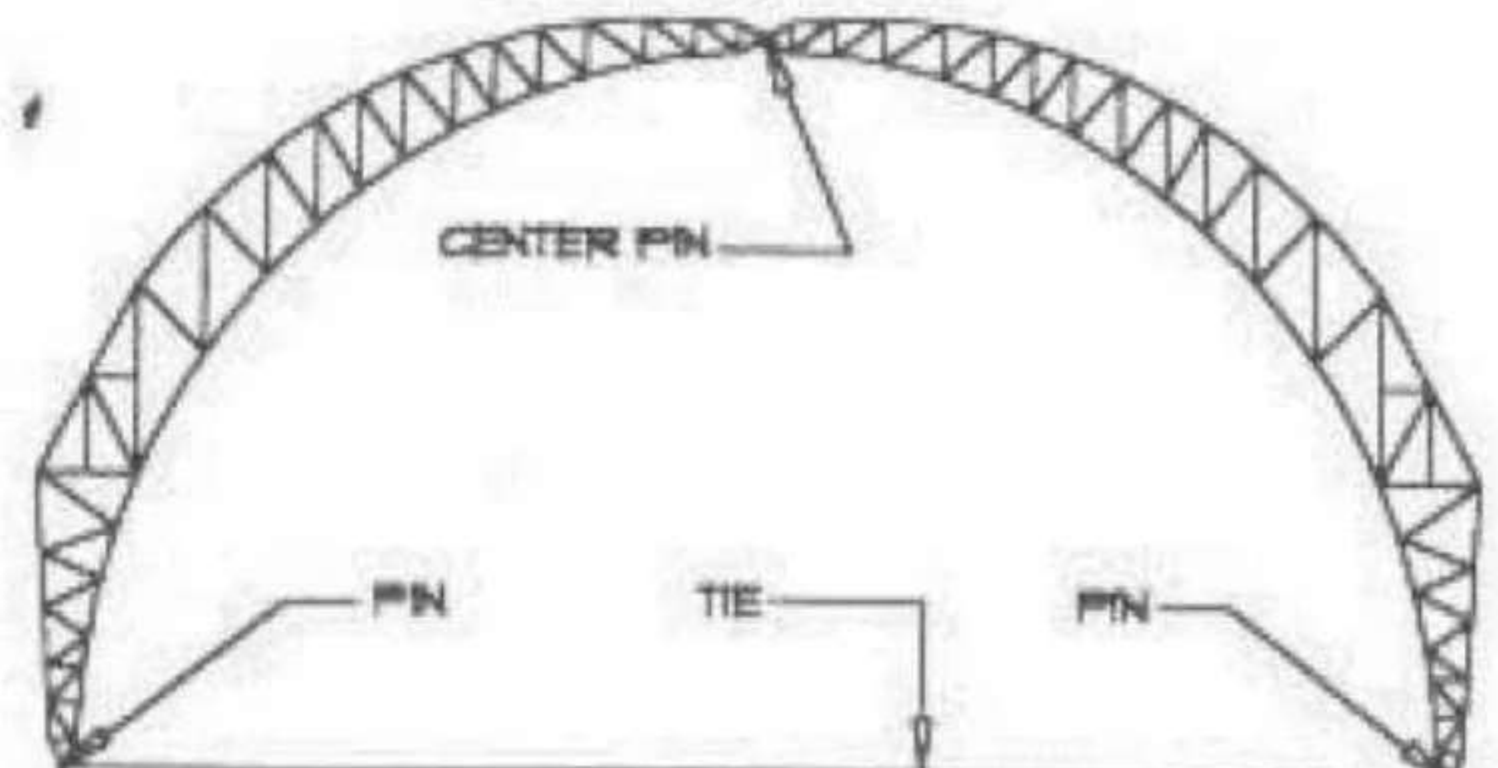
- Local buckling
- Squashing
- Overall flexure buckling
- Torsional buckling

- This ideal state is never reached. The stress-state will be non-uniform due to:
- Accidental eccentricity of loading with respect to the centroid
- Member out-of –straightness (crookedness), or
- Residual stresses in the member cross-section due to fabrication processes

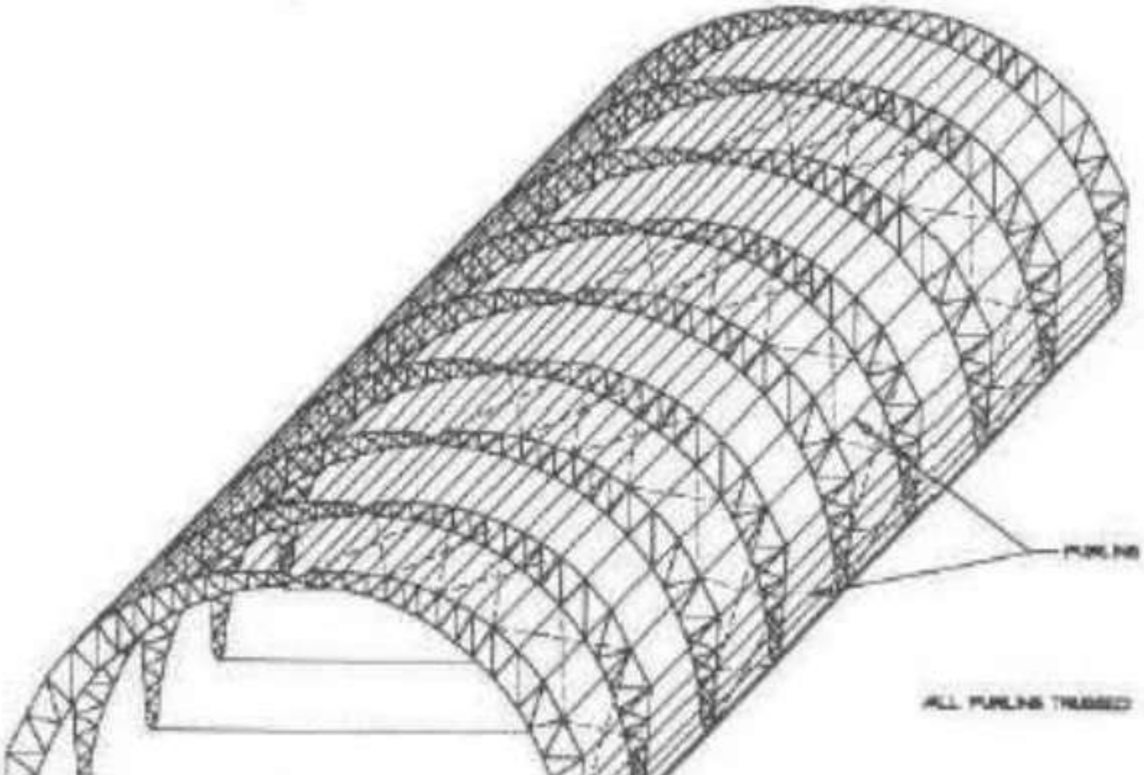
- In addition to most common type of compression members (vertical Members in structure),compression may include the
  - Arch ribs
  - Rigid frame members inclined or otherwise
  - Compression elements in trusses

Compression Members

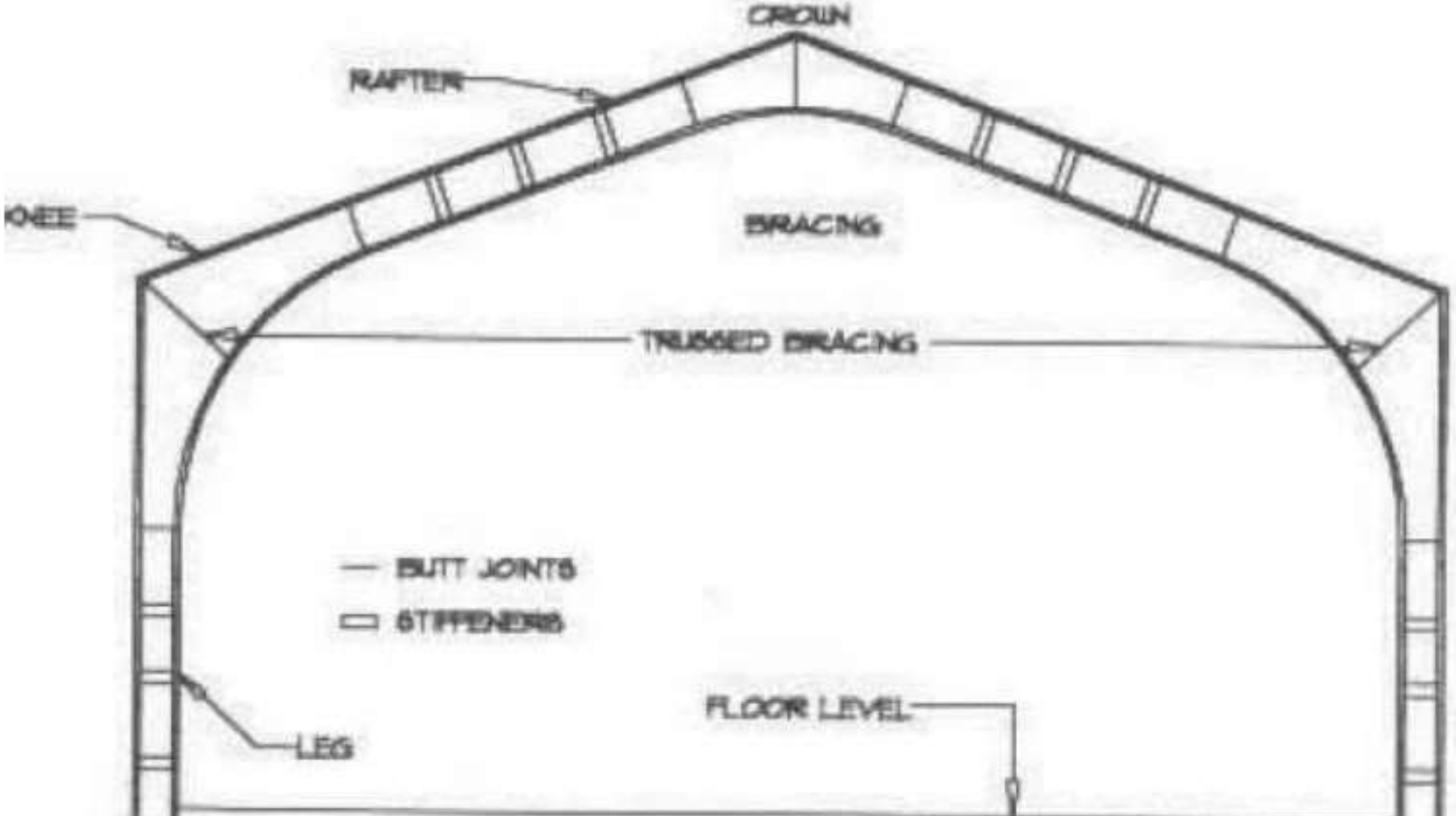
Compression Members



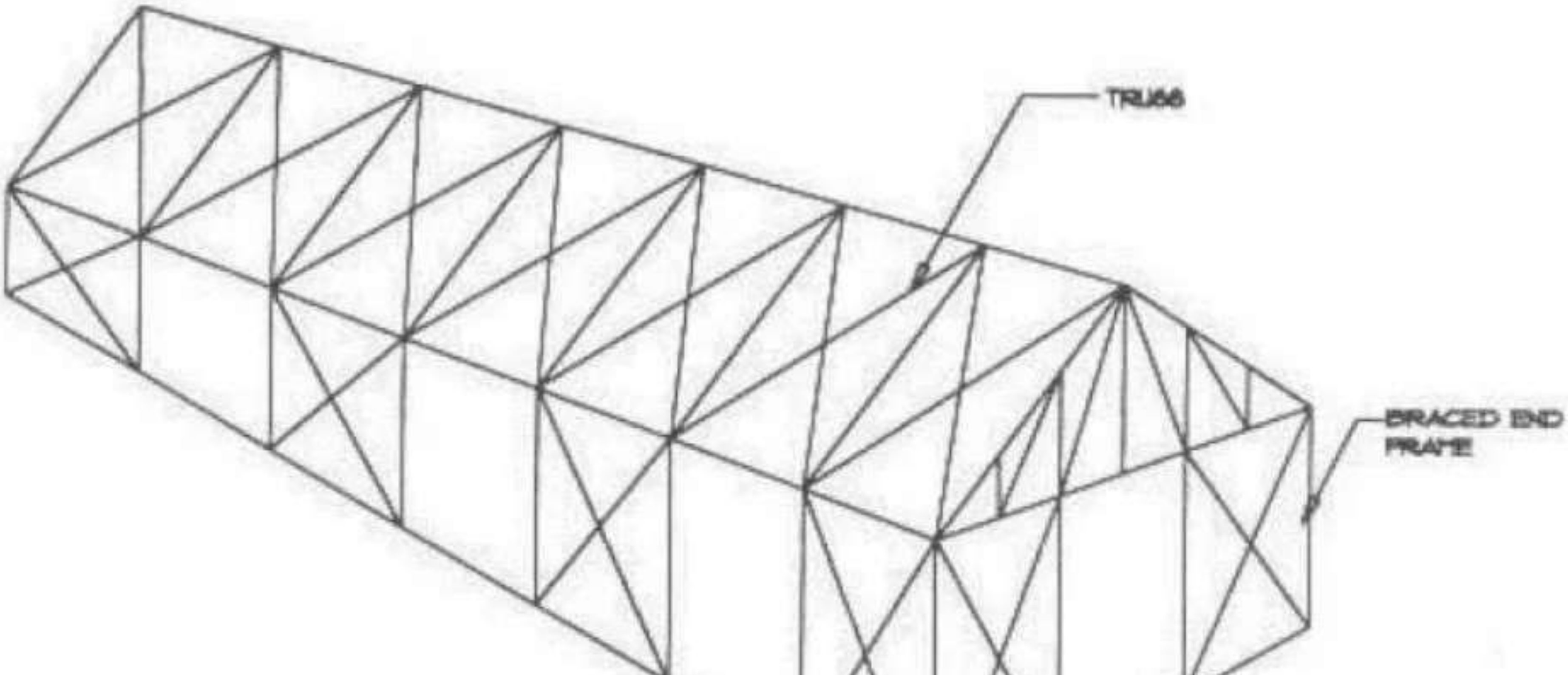
Compression Members



Compression Members



Compression Members



Compression Members

Elastic buckling of slender compression members

- Slender columns have low crippling load carrying capacity.
- Consider one such column having length 'L' and uniform cross section A hinged at both ends A and B. Let P be the crippling load at which the column has just buckled.

section.

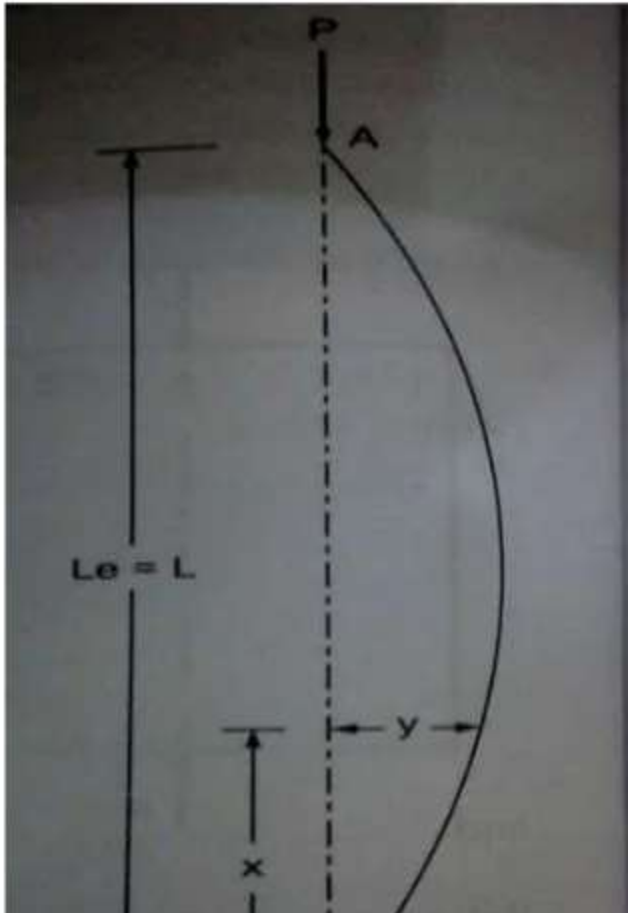
The bending moment at this sectional is given by (from slope and deflection concept)

$$M = EI \cdot \frac{d^2y}{dx^2}$$

or

$$EI \cdot \frac{d^2y}{dx^2} = -p \cdot y$$

(-ve sign indicates anticlock wise moment due to P at 'B')



Members

- The longer the column, for the same x-section, the greater becomes its tendency to buckle and smaller becomes its load carrying capacity.
- The tendency of column to buckle is usually measured by its slenderness ratio

$$\text{Slenderness Ratio} = \frac{L}{r}$$

Compression Members Vs Tension Members

Effect of material Imperfections and Flaws

- Slight imperfections in tension members are can be safely disregarded as they are of little consequence.
- On the other hand slight defects in columns are of great significance.
- A column that is slightly bent at the time it is put

Compression Members Vs Tension Members

- Tension in members causes lengthening of members.
- Compression beside compression forces causes buckling of member.

Compression Members Vs Tension Members

- Presence of holes in bolted connection reduce Gross area in tension members.
- Presence of bolts also contribute in taking load An = Ag

WHY column more critical than tension member?

- A column is more critical than a beam or tension member because minor imperfections in materials and dimensions mean a great deal.

WHY column more critical than tension member?

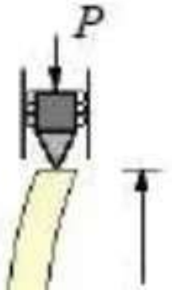
- The bending of tension members probably will not be serious as the tensile loads tends to straighten those members, but bending of compression members is serious because

Compression Member Failure

- There are three basic types of column failures.
- One, a compressive material failure(very short and fat).
- Two, a buckling failure,(very long and skinny).
- Three, a combination of both compressive and buckling failures.(length and width of a column is

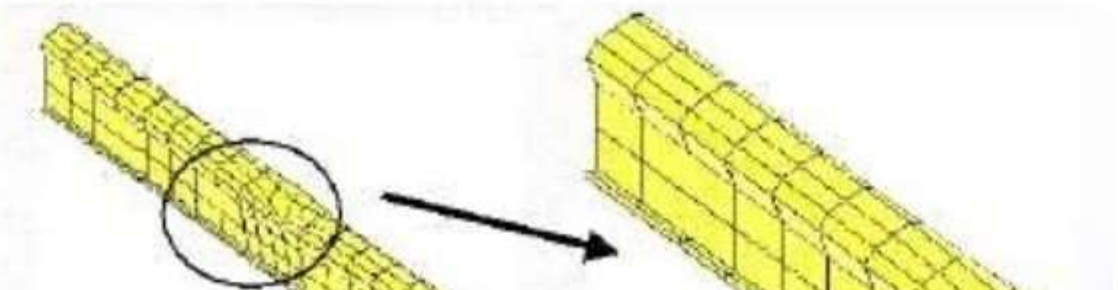
Compression Member Failure

- **<u>Flexural Buckling</u>** (also called Euler Buckling) is the primary type of buckling.members are subjected to bending or flexure when they become unstable



Compression Member Failure

- **Local Buckling** This occurs when some part or parts of x-section of a column are so thin that they buckle locally in compression before other modes of buckling can occur



Compression Member Failure

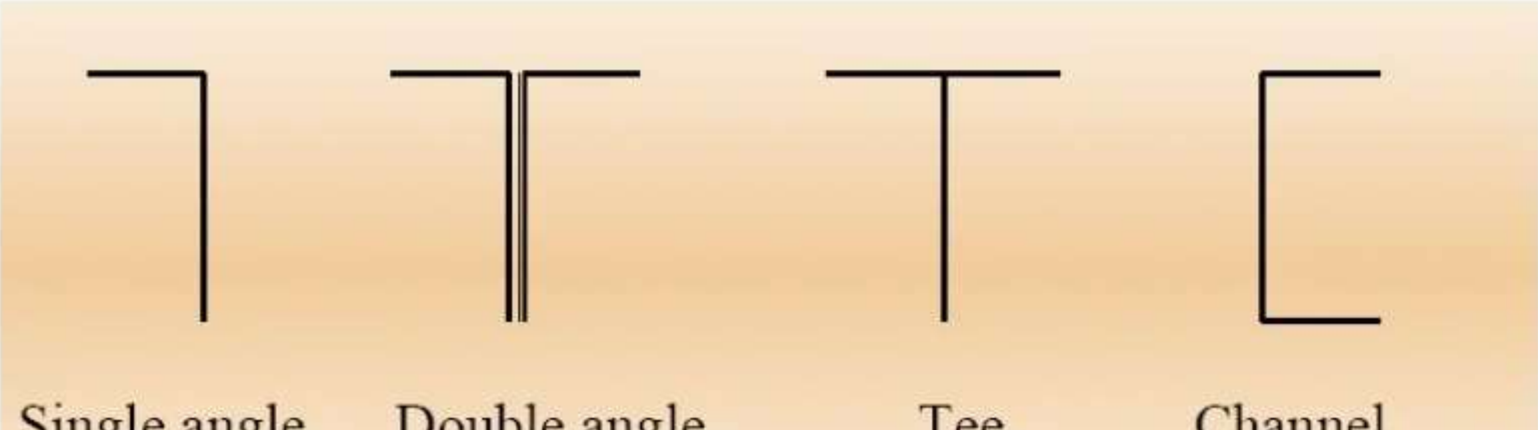
- **Torsional Buckling** These columns fail by twisting(torsion) or combined effect of torsional and flexural buckling.

Sections used for Compression Member

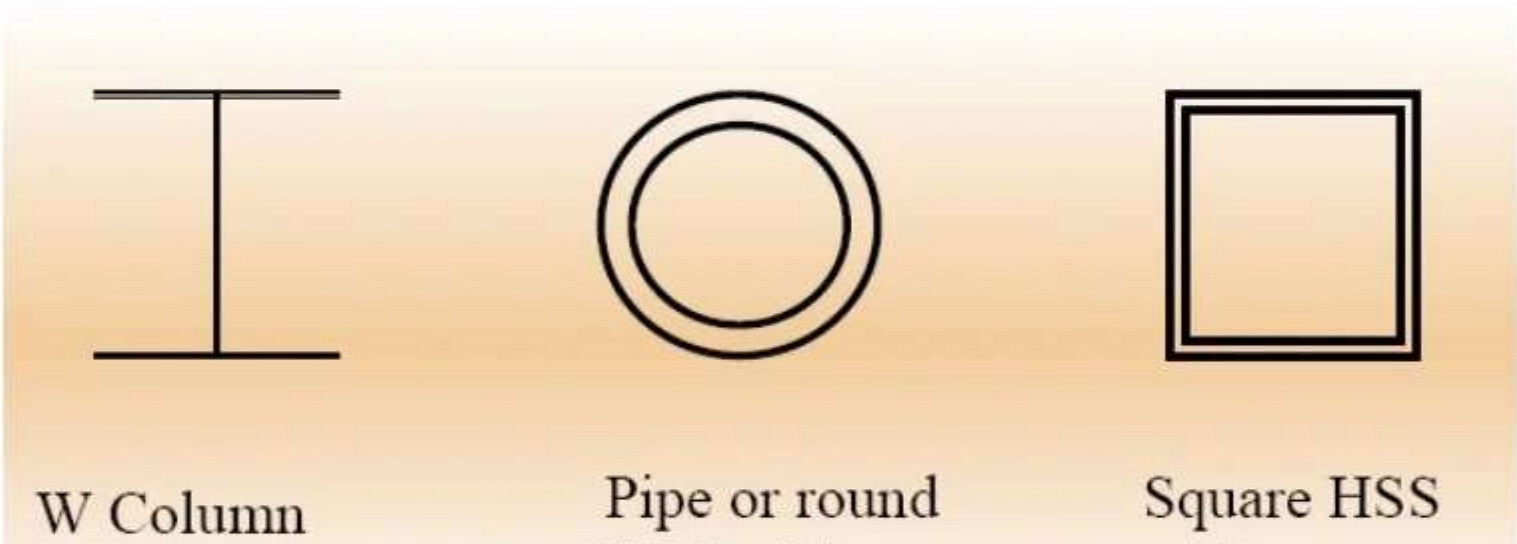
- In theory numerous shapes can be used for columns to resist given loads.
- However, from practical point of view, the number of possible solutions is severely limited by section availability, connection problems, and type of

Sections used for Compression Member

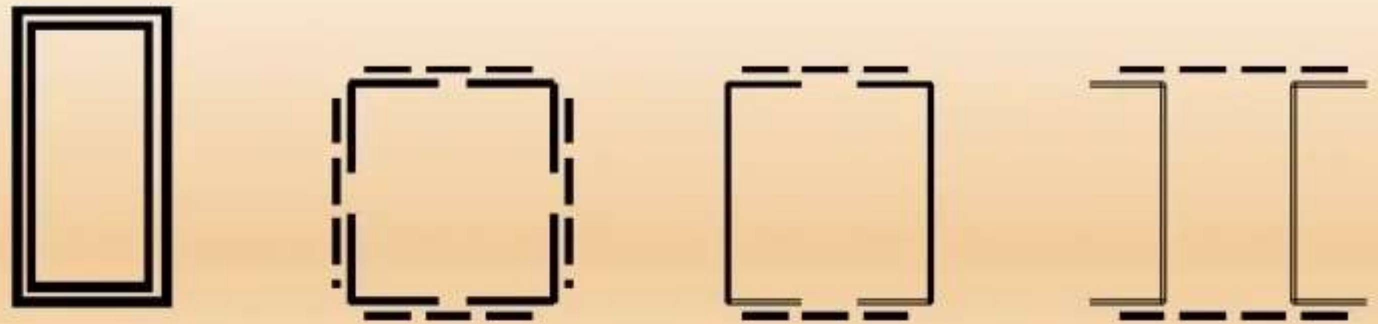
Figure 1. Types of Compression Members



Sections used for Compression Member

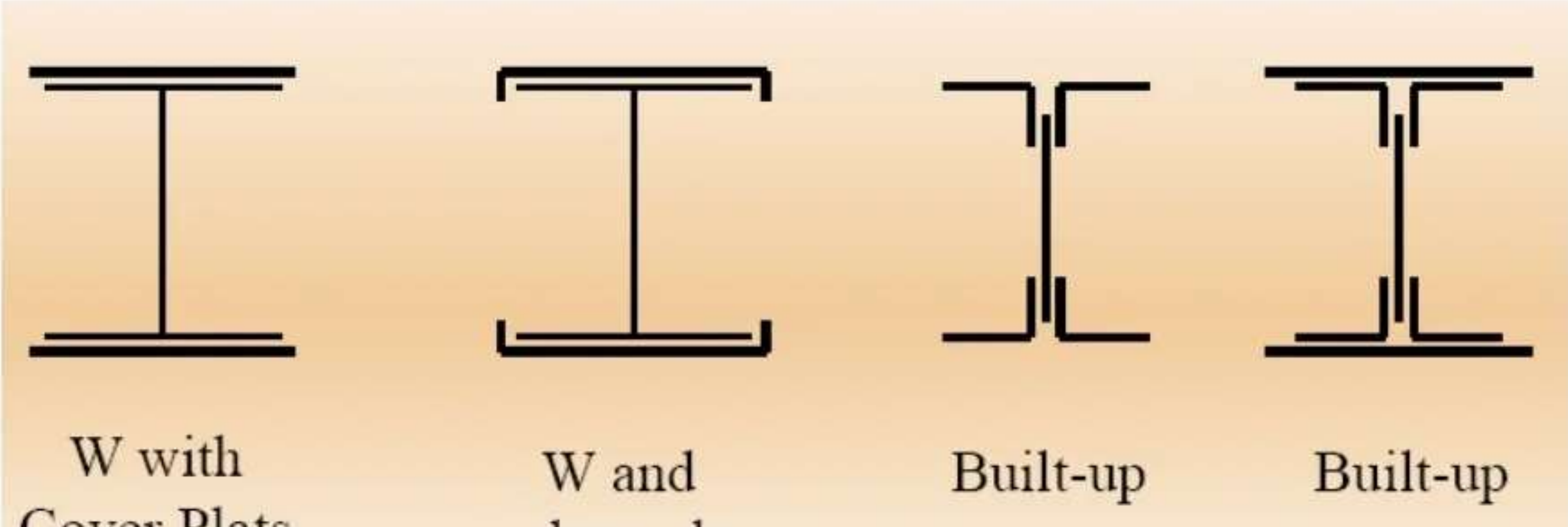


Sections used for Compression Member



Rectangular HSS tubing | Four angle box section | Box section | Box section

Sections used for Compression Member



Column Buckling

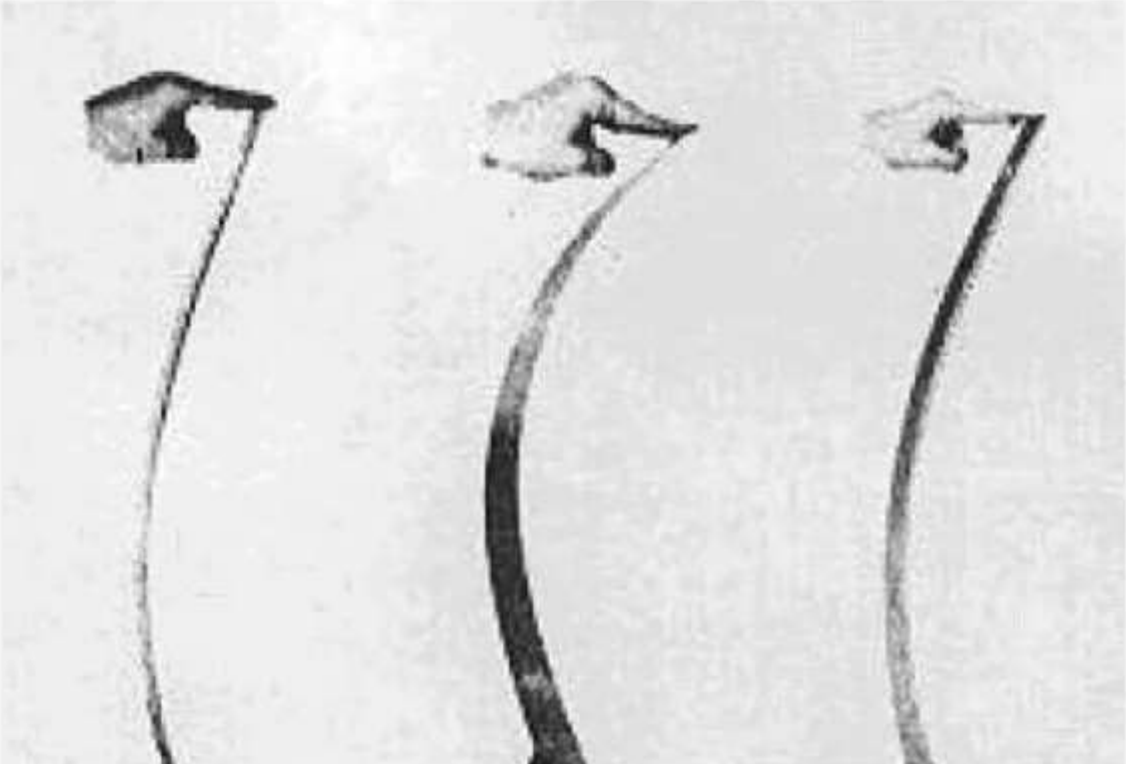
- Buckling
- Elastic Buckling
- Inelastic Buckling

Column Buckling

- Buckling is a mode of failure generally resulting from structural instability due to compressive action on the structural member or element involved.
- Examples of commonly seen and used tools are provided.

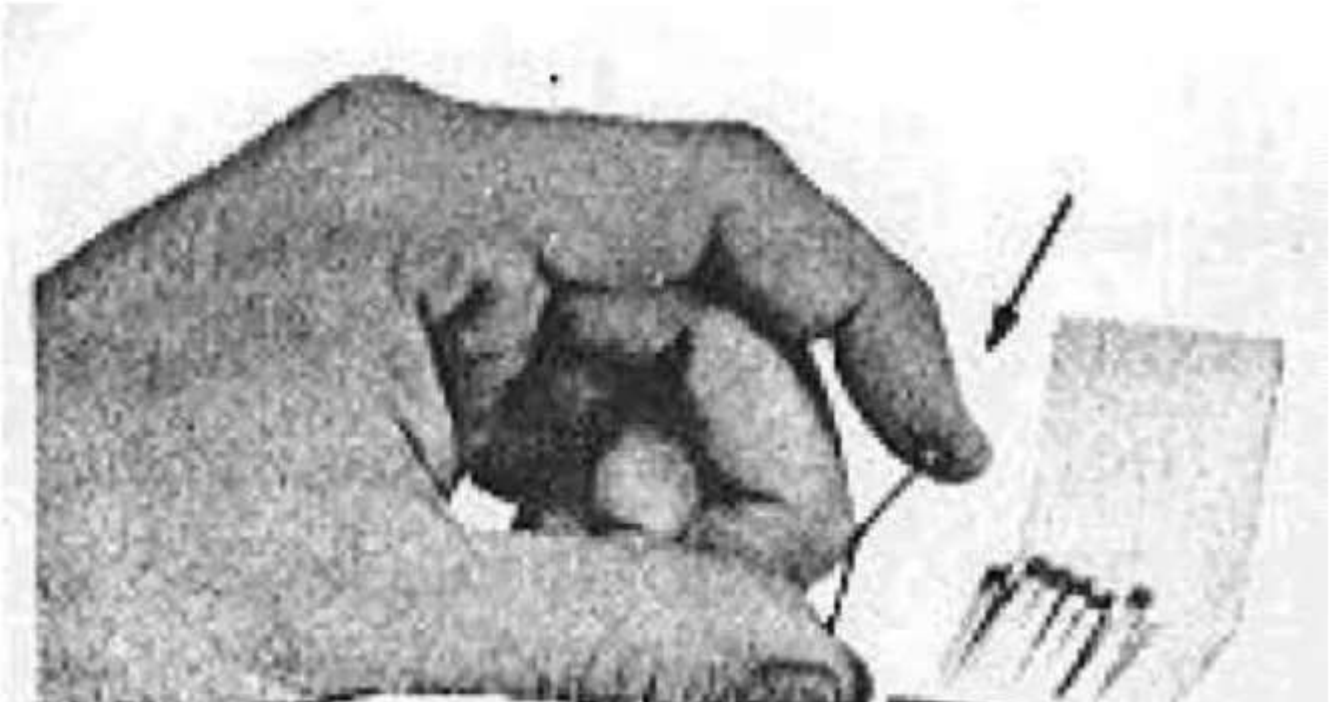
Buckling

Example



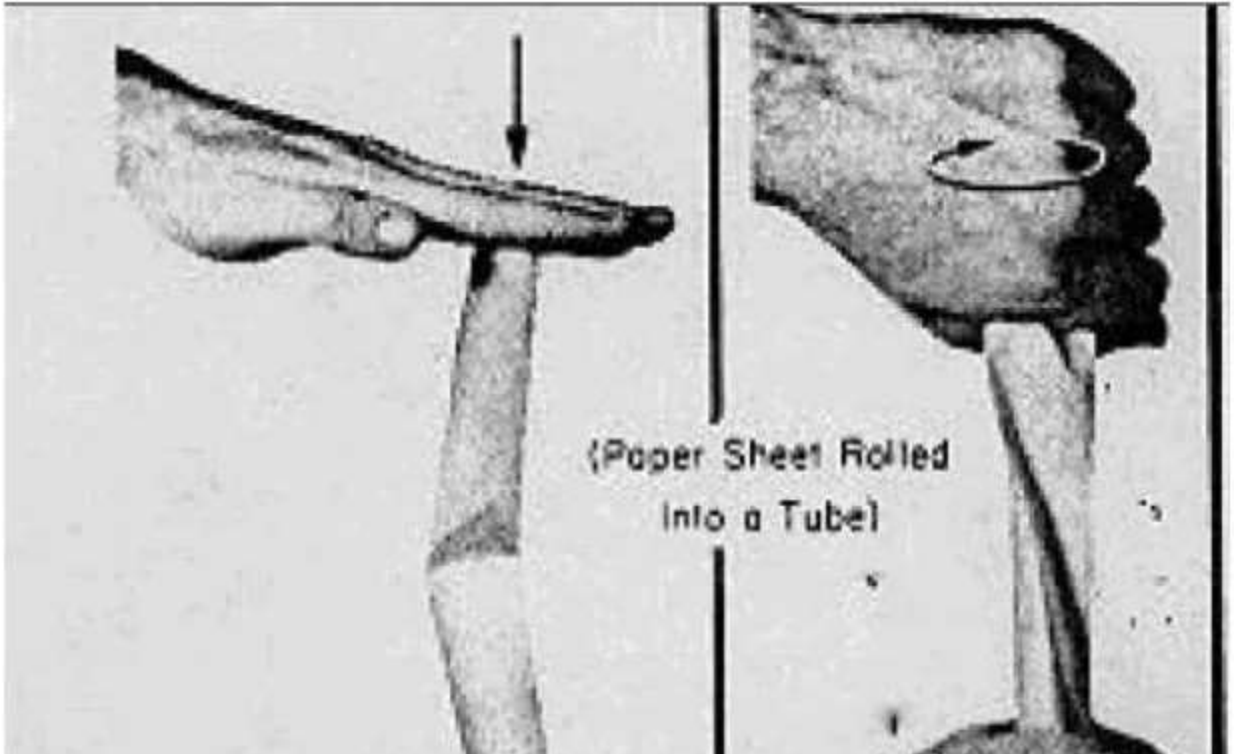
Buckling

Example



Buckling

Example



Buckling

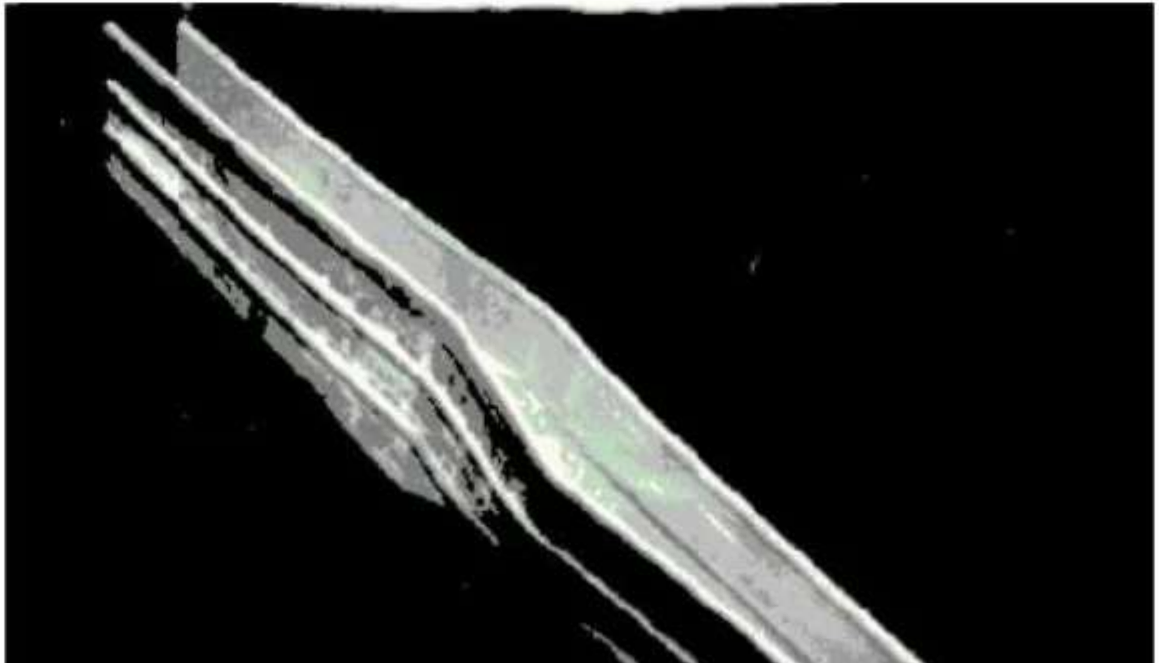
Example

Buckling

- Example (a) is temporary or elastic buckling.
- Example (b,c,d) are examples of plastic buckling.

Column Buckling

- Steel column buckling

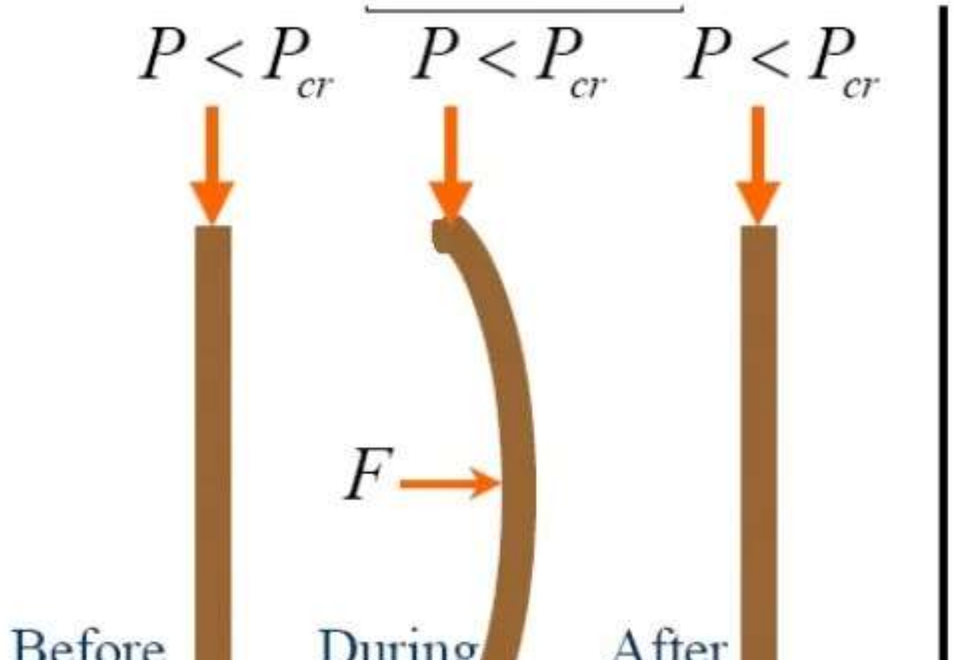


Mechanism of Buckling

- Let us consider Fig 1, 2, 3 and study them carefully.
- In fig1 some axial load P is applied to the column
- The column is then given a small deflection by giving a small force F.
- If the force P is sufficiently small, when the force F is removed, the column will go back to its

Mechanism of Buckling

Fig 1



Mechanism of Buckling

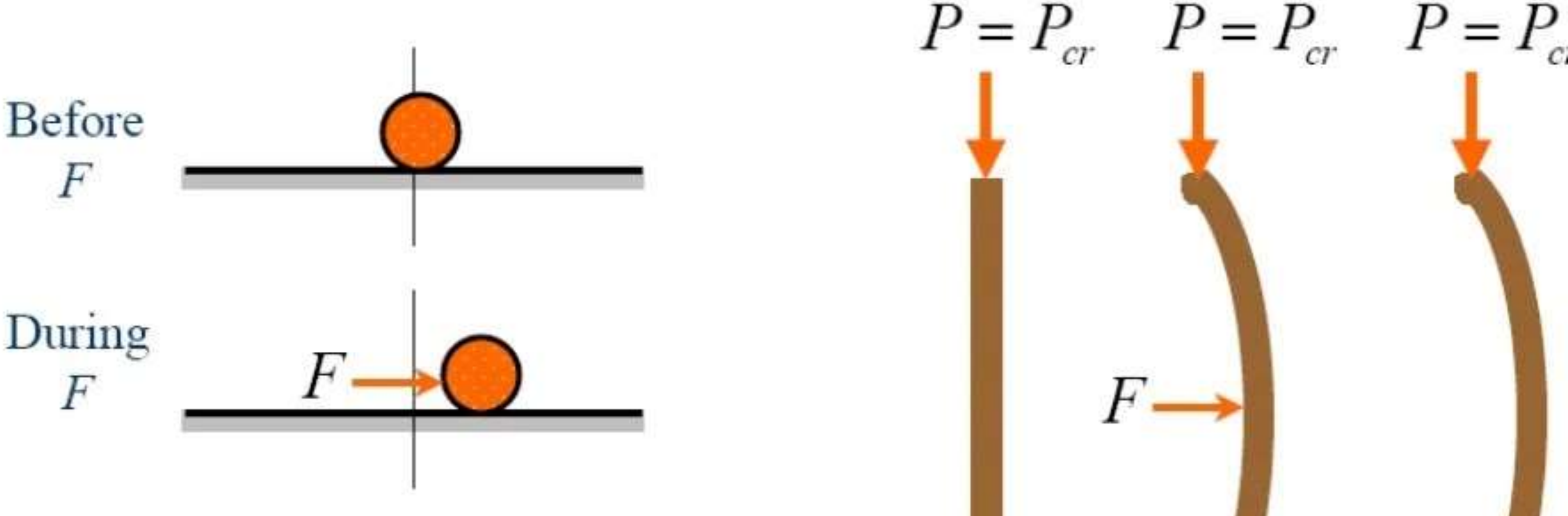
- The column will go back to its original straight position. Just as the ball returns to the bottom of the container.
- Gravity tends to restore the ball to its original position while in columns elasticity of column itself acts as a restoring force.

Mechanism of Buckling

- The same procedure can be repeated with increased load untill some critical value is reached.

Mechanism of Buckling

Fig 2

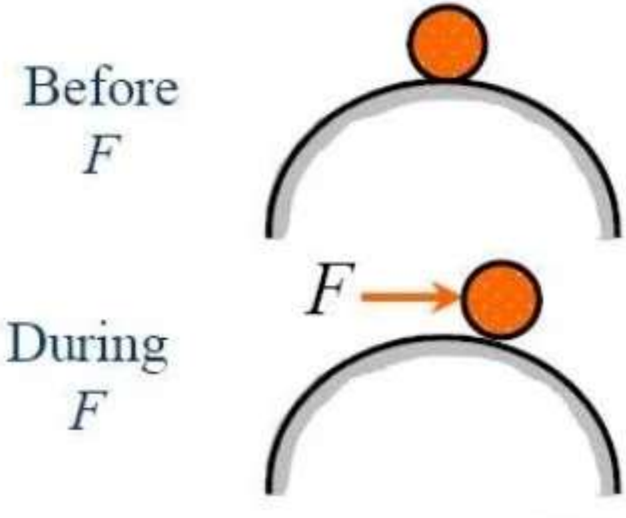


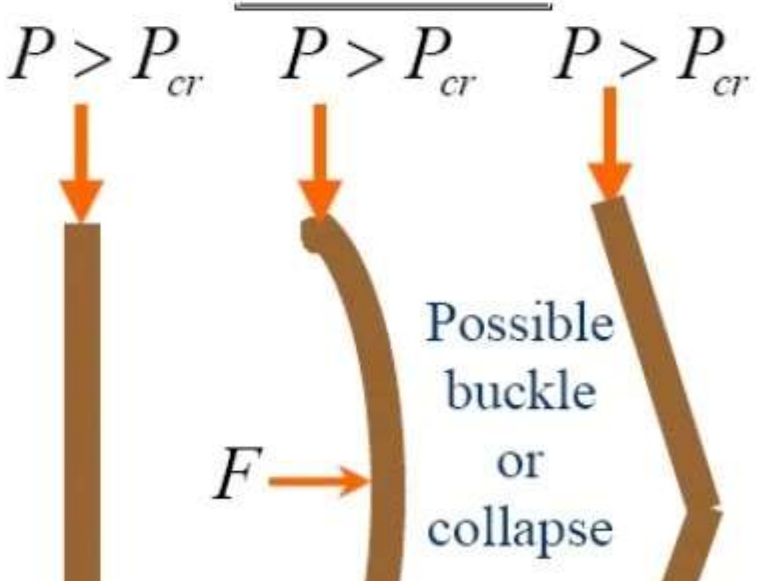
Mechanism of Buckling

- The amount of deflection depends on amount of force F.
- The column can be in equilibrium in an infinite number of bent position.

Mechanism of Buckling

Fig 3





Mechanism of Buckling

- The elastic restoring force was not enough to prevent small disturbance growing into an excessively large deflection.
- Depending on magnitude of load P,

Mechanism of Buckling

Conclusions

- This type of behavior indicates that for axial loads greater than P_{cr} the straight position of column is one of unstable equilibrium in that a small disturbance will tend to grow into an excessive deformation.
- Buckling is unique from our other structural elements considerations in that it results from state of unstable equilibrium

Mechanism of Buckling

Conclusions

- Buckling of long columns is not caused by failure of material of which column is composed but by determination of what was stable state of equilibrium to an unstable one.

Compression member Buckling

- Buckling occurs when a straight, homogeneous, centrally loaded column subjected to axial compression suddenly undergoes bending.
- Buckling is identified as a failure limit-state for columns.

Compression member Buckling

- The value of P at which a straight column becomes unstable is called the Critical Load.
- When column bends at critical load, it is said to have buckled.

Column buckling curves

- Classification of different sections under different buckling class a, b,c and d are given in Table 10 of IS 800: 2007 (page 44).
- The stress reduction factor χ , and the design compressive stress fcd, for different buckling class, yield stress and effective slenderness ratio is given in table 8 (page 37)
- Table 9(page 40) shows the design compressive stress, fcd for different buckling class a to d.

- The curve corresponding to different buckling class are presented in non-dimensional form as shown in the figure below. Using this curve one can find the value of fcd (design compressive stress) corresponding to non- dimensional effective slenderness ratio λ (page 35)

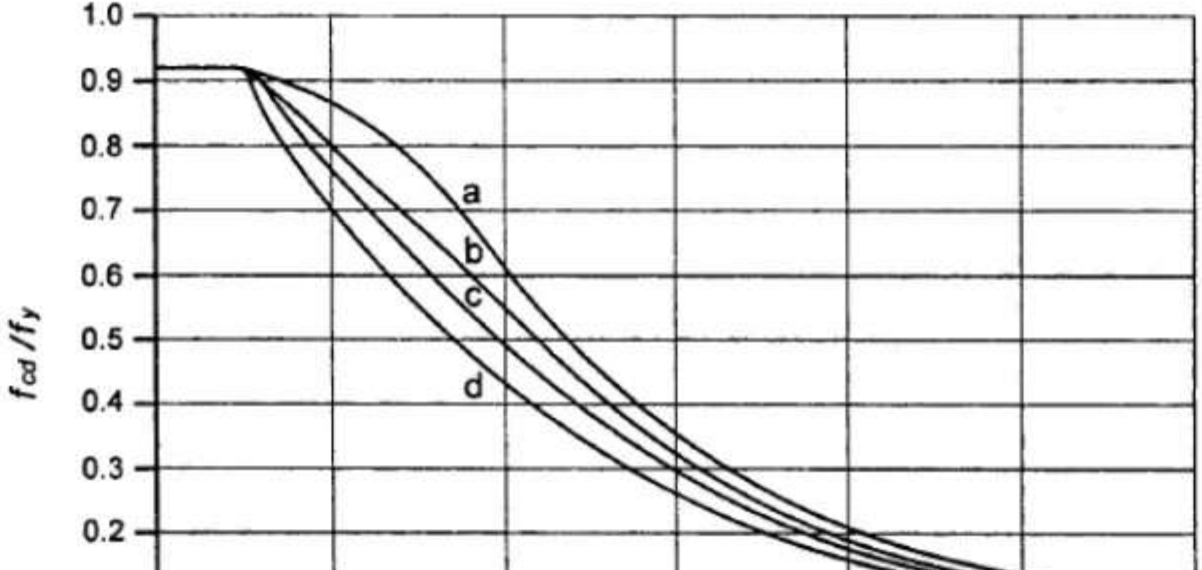
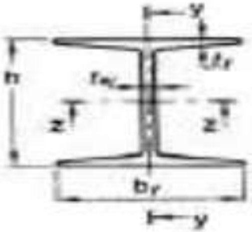
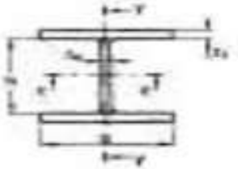
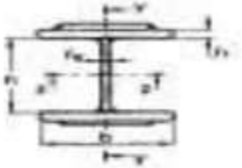
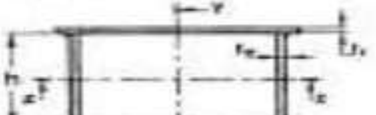


Table 10 Buckling Class of Cross-Sections

(*Clause* 7.1.2.2)

Cross-Section	Limits	Buckling About Axis	Buckling Class
(1)	(2)	(3)	(4)
Rolled I-Sections 	$h/b_f > 1.2$: $t_f \leq 40$ mm	z-z y-y	a b
	$40 \leq$ mm $< t_f \leq 100$ mm	z-z y-y	b c
	$h/b_f \leq 1.2$: $t_f \leq 100$ mm	z-z y-y	b c
	$t_f > 100$ mm	z-z y-y	d d
Welded I-Section  	$t_f \leq 40$ mm	z-z y-y	b c
	$t_f > 40$ mm	z-z y-y	c d
Hollow Section 	Hot rolled	Any	a
	Cold formed	Any	b
Welded Box Section 	Generally (except as below)	Any	b
	Thick welds and $b/t_f < 30$	z-z	c

Design compressive strength

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

$$P < P_d$$

where

$$P_d = A_e f_{cd}$$

where

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

Clause 7.3.2

7.3.2 *Effective Sectional Area, A_e*

Except as modified in **3.7.2** (Class 4), the gross sectional area shall be taken as the effective sectional area for all compression members fabricated by welding, bolting and riveting so long as the section is semi-compact or better. Holes not fitted with rivets, bolts or pins shall be deducted from gross area to

IS 800: 2007 Clause 7.1.2.1

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

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

where

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

λ = non-dimensional effective slenderness ratio

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

where

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

α = imperfection factor given i Table 7;

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

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

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

Design of compression members

- Assumptions made
- The column is assumed to be absolutely straight.
- The modulus of elasticity is assumed to be constant in a built- up column
- Secondary stresses are neglected

Design steps

- For beginners , for an average column size of 3-5 m the slenderness ratio of 40 to 60 is selected. For very long column a λ of 60 may be assumed. For column with very heavy factored load a smaller value of slenderness ratio should be assumed.
- Choose a trial section by assuming an appropriate slenderness ratio from following table

Type of member	slenderness ratio
Single angle	100-50
Single channel	90-110
Double angles	80-120
Double channels	40-80
Single I- Section	80-100

- Select a trial section by referring the table above and from steel tables
- Calculate KL/r for the section selected. The calculated value of slenderness ratio should be within the max limiting value given by IS 800-2007 (page 20)

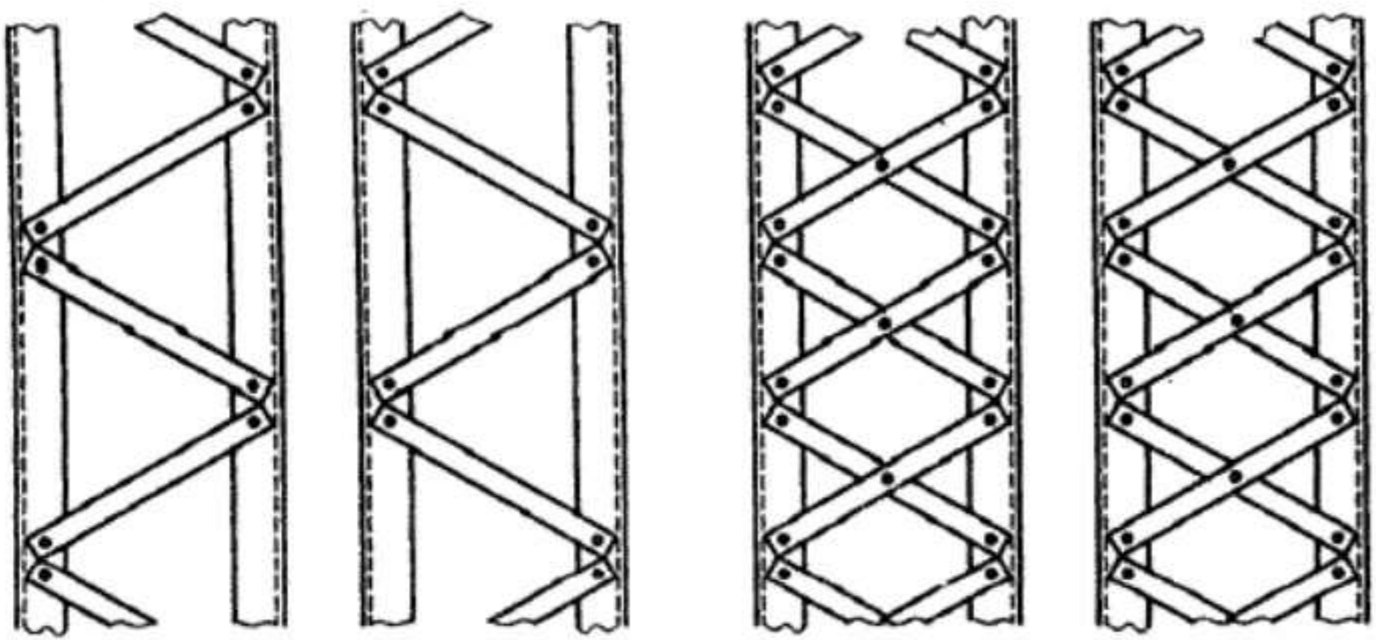
- Calculate fcd and the design strength Pd = A. fcd
- For the estimated value of slenderness ratio, calculate the design compressive stress (fcd) , by any method i.e. using buckling curve or by using equations given by IS 800: 2007 (refer Cl. 7.1.2)
- The design strength of member is calculated as

Sl No.	Member	Maximum Effective Slenderness Ratio (KL/r)
(1)	(2)	(3)
i)	A member carrying compressive loads resulting from dead loads and imposed loads	180
ii)	A tension member in which a reversal of direct stress occurs due to loads other than wind or seismic forces	180
iii)	A member subjected to compression forces resulting only from combination with wind/earthquake actions, provided the deformation of such member does not adversely affect the stress in any part of the structure	250
iv)	Compression flange of a beam against lateral torsional buckling	300
v)	A member normally acting as a tie in a roof truss or a bracing system not considered effective when subject to possible reversal of stress into	350

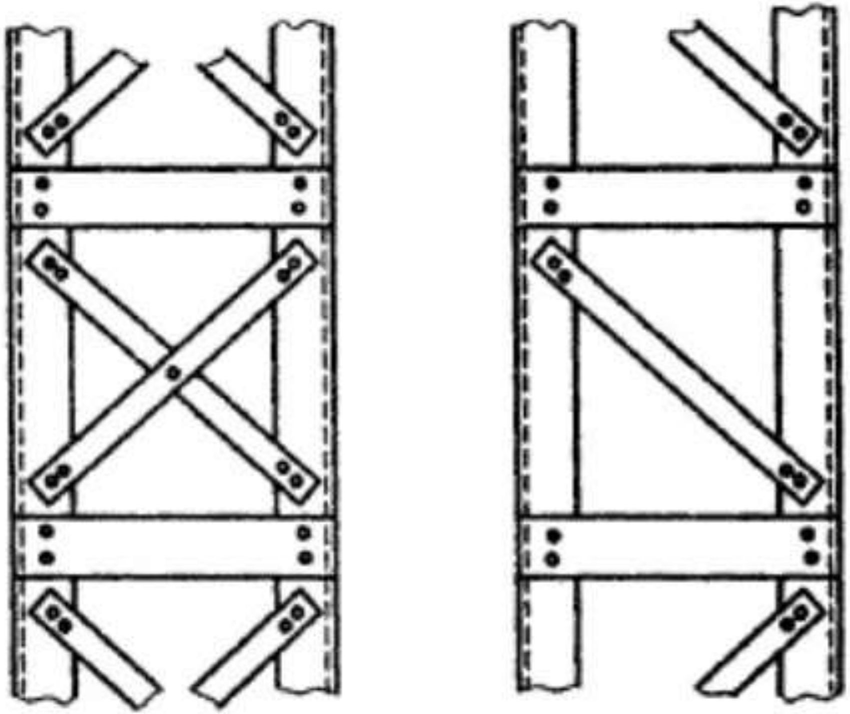
Built-up Column members

- Laced member
- Struts with batten plates
- Battened struts
- Members with perforated cover plates

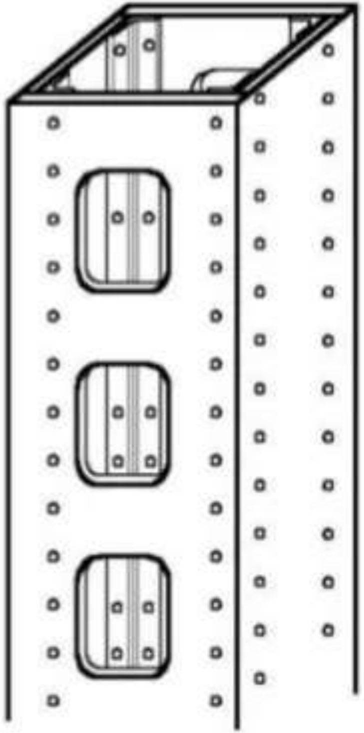
Built- up compression member



LACING ON FACE A | LACING ON FACE B | LACING ON FACE A | LACING ON FACE B



10C Double Laced and Single Laced System Combined with Cross Numbers



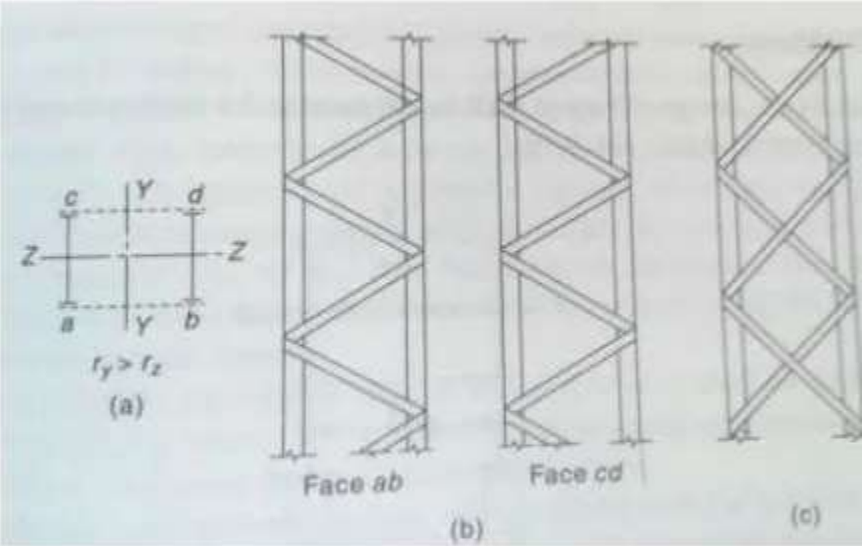
Perforated plate column

Column with single lacing

Note that lacings and batten plates are not designed to carry any load. Their primary function is to hold the main

Lacings : rules specified in IS 800: 2007

- Radius of Gyration of combined column @ axis perpendicular to plane of lacing > radius of gyration @ axis parallel to plane of lacing (i.e. ry > rz)figure (a)
- Lacing system should be uniform throughout the length of column
- Single and double laced



The lacing shown in figure b for fac
cd is thus not recommended

- Lacing shall be designed to resist a total transverse shear Vt at any point in the member, equal to 2.5% of the axial force in the member, and this shear shall be divided among the lacing systems in parallel planes.
- Lacings in addition should be designed to resist any shear due to bending moment or lateral load on the member.
- Slenderness ratio of lacing shall not exceed 145
- Effective length shall be taken as the length between inner end bolts/rivets of the bar for single lacings and 0.7 times the length for double lacings effectively connected at intersections. For welded bars the effective length is taken as 0.7 times the distance between the inner ends of the welds connecting the single bars to the members.
- Min width of lacing bar shall not be < than app 3 times dia of the

- When welded lacing bars overlap the main members the amount of lap should not be < than 4 times the thickness of the bar and the welding is to be provided along each side of bar for the full length of the lap. Where lacing bars are fitted between main members, they should not be connected by fillet weld or by full penetration butt weld.
- Plates shall be provided at the ends of laced compression members and shall be designed as battens.
- Flats, angles, channels or tubes may be used as lacings
- Whether double or single the angle of inclination shall be between 40deg to 70deg to axis of the built-up member.
- The eff slenderness ratio (KL/r)e of the laced column shall be taken

50)

- No of battens shall be such that the member is divided into not < than three bays.(i.e there should be min of three bays)
- Battens are designed to resist simultaneously;

Longitudinal shear

$$V_b = V_t.L_o / ns$$

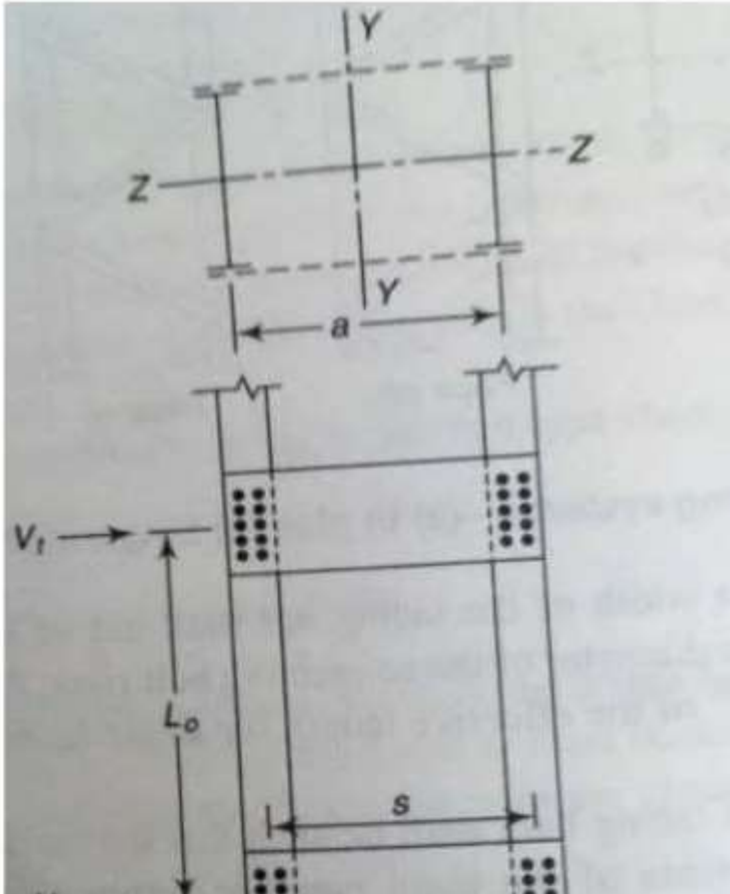
And moment

$$M = V_t.L_o / 2n$$

Where

L_o = distance bet c/c of battens, longitudinally

n= no of parallel planes of battens



Built up columns

- Used when large loads are expected and for efficient use of member.
- Consists of two or more individual members
- For economic design of heavily loaded long columns the least radius of gyration of column section is increased to maximum (ry > = rz).
- To achieve this the rolled steel sections are kept away from centroidal axis of column.

- When plates are used for battens, the eff. depth between end bolts/rivets or welds shall not be less than twice the width of one member in the plane of battens; nor less than 3/4th of perp. distance between centroids of the main members for intermediate battens; and not less than the perp. distance between the centroids of main members for the end battens. Refer figure to right.
- Eff depth of end batten

$$d' = S' + 2cyy$$

Overall depth of end batten

$$d = d' + 2 \times \text{edge distance}$$

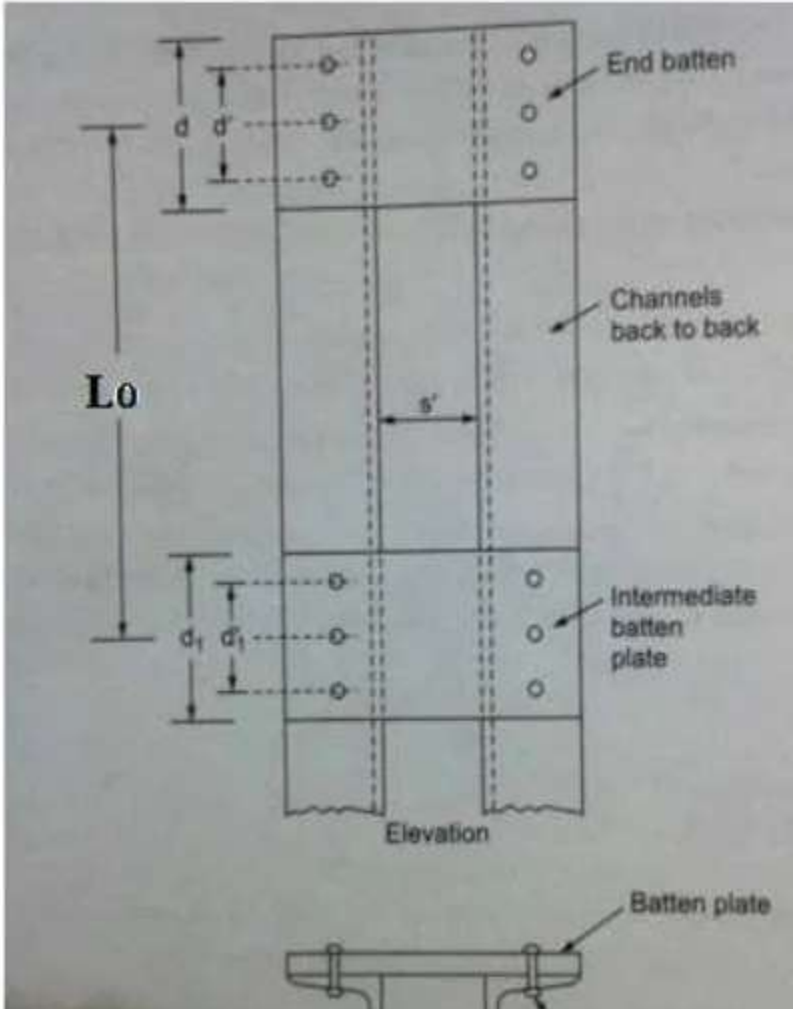
Effective depth of intermeddiate batten

$$d1' = 3/4^{th}\ d'$$

Overall depth of intermediate batten

$$d1 = d1' + 2 \times \text{edge distance}$$

Where cyy = the distance taken from steel table for



- Shear stress calculated in the battens

  = (Vb/ A1)

This should be less than

$$\frac{Vb}{A1} \langle \frac{fy}{\sqrt{3}.\gamma mo}$$

Where A1 = cross sectional area of batten = t.d

γm0= partial safety factor= 1.1

t = thickness of batten

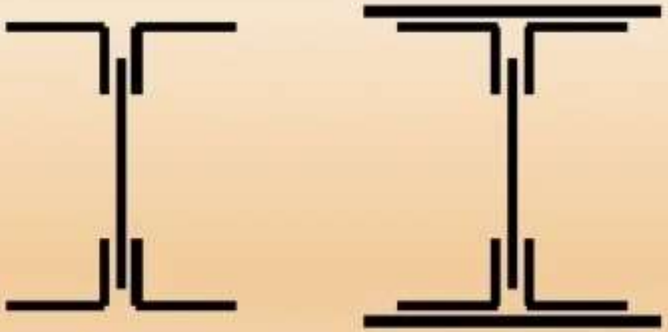
d = overall depth of the batten

- The bending stress in the section is calculated and it should be < fy/ γm0 as

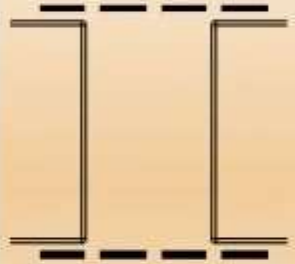
- Requirements of size not required when other rolled sections are used for battens with their legs or flanges perp. to the main member.
- When connected to main members by welds, the length of weld connecting each end of batten shall not < ½ the depth of the batten plate; atleast 1/3rd of its length should b placed at each end of the edge; in addition the weld shall be returned along the other two edges for a length not < the min lap (i.e not < 4 times thicknes of the plate. The length of the weld and the depth of batten shall be measured along the longitudinal axis of the member

- The effective slenderness ratio of the battened column shall be taken as 1.1 tines (KL/r)o, where (KL/r)o is the max actual slenderness ratio of the column, to account for the shear deformation effects.

Compression members composed of two members back to back



Built-up Built-up



Box section

- The slenderness ratio of each member between the two connections should not be > than 40 or 0.6 times min slenderness ratio of the strut as a whole.
- The ends of strut should be connected with a minimum of two bolts/rivets or equivalent weld length (weld length must not be less than the maximum width of the member) and there should be two additional connections in between, spaced equidistant along the length of member.
- Where there is small spacing between the members washers (in case of bolts) and packing (in case of welding) should be provided to make the connections.
- Where the legs of angles or T's are more than 125 mm wide , or where web of channels is 150 mm wide, a min of two bolts/rivets

- The bolts/rivets should be 16 mm or more in dia for a member <= 10 mm thick and 20 mm in dia for a member <= 16 mm thick and 22 mm in dia for members > than 16 mm thick
- Such members connected by bolts/welding should not be subjected to transverse loading in a plane perp. to the riveted/bolted or welded surfaces.
- When placed back to back, the spacing of bolts/rivets should not exceed 12t or 200 mm and the longitudinal spacing between the intermittent welds should not be more than 16 t, where t is thickness of the thinner section.

Problems

- An ISHB 400 @ 806.4 N/m is to be used as column 3.5 m long with both ends restrained against rotation and translation. Determine the design axial load on the column section. Also assume the following data : fy = 250 N/mm2, fu = 410 n/mm2 and E= 2x 10e5 N/mm2.

Solution:

Method 1

Properties of ISHB 400 from handbook/steel table:

A= 10466 mm2	bf= 250 mm	tw = 10.6 mm	h= 400 mm
tf= 12.7 mm	rz = 166.1 mm	ry = 51.6 mm	L = 3.5 m = 3500mm

Refer table 10 page 44

h/bf = if > 1.2 and tf = < 40mm

λy=40 and fy = 250 fcd =206 Mpa and

y= KL/ ry

=(0.65x 3500) /51.6

= 44.09 for both ends fixed k= 0.65

For buckling class b and fy=250MPa, design compressive stress=200 Mpa= fcd (refer table 9(b) of IS code)This is obtained by interpolating between the tow values of λy

λy=40 and fy = 250 fcd =206 Mpa and so (50-40)→ (194-206)

λy=50 and fy = 250 fcd =194Mpa (50 – 44.09) →(194 –x)

Cross multiplying 10 x (194-x) = (- 12)x (5.91)

x= 200 MPa

Method 2

we can use the formulae

Non-dimensional slenderness ratio

λ = non-dimensional effective slenderness ratio

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

- **Method 3**

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

where

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

λ = non-dimensional effective slendernes

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

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

- Imperfection factor

$\alpha = 0.34$ (for buckling class b)

Find $\phi = 0.5[1+0.34(0.5-0.2)+0.5^2]$

$= 0.676$

$\chi = 1/ \{ 0.676+ [0.676^2 - 0.5^2]^{½}\}$

$=0.884$

Fcd $= (250 \times 0.884)/ 1.1$

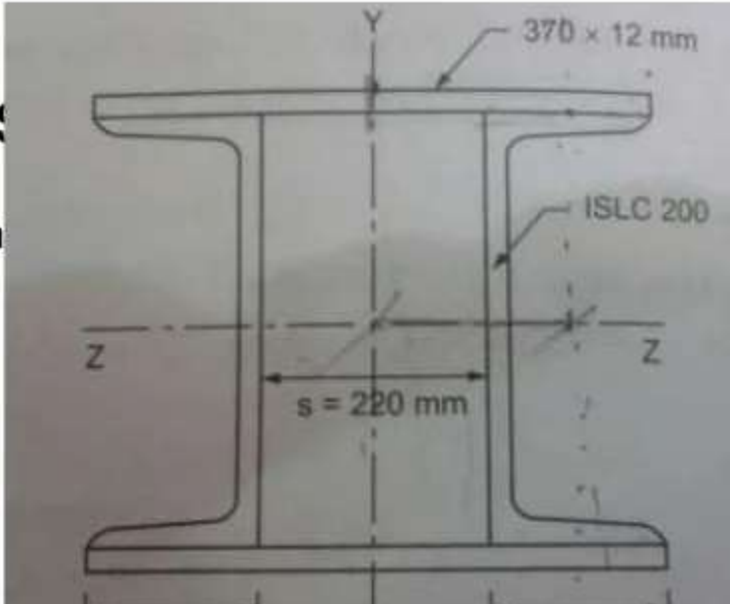
Figure shows a built up column section. The column has effective length of 4.75 m. find the design compressive load for the column. Take fy = 250 N/mm2 E = 2x 10e5 N/mm2

Solution

For fcd what is required

- Need to know buckling clas
- Slenderness ratio $\lambda = L_{eff} / r_{m}$

$$= KL / r_{min}$$



Refer Steel table to get details of section

- IS SP.1.1964.pdf

Properties of two ISLC 200(back to back)

A= Izz= Iyy= Zyy

ryy=

Properties of built up section

Total area = Area of Cs + area of cover plates

Izz = I zz of channels (back to back) + I z of cover plates@ Z
Z

I yy = I yy of channels (back to back) + I yof cover plates @

- Buckling class=
- Calculate least radius of gyration =

- Corresponding to slendernes ratio and buckling class and for fy= 250 n/mm2
- Find design compressive strength from any of the methods given in IS code is.800.2007- code of practice for gener steel.pdf

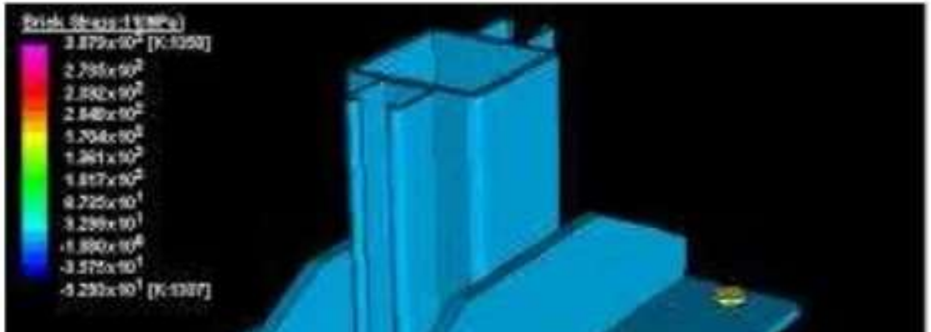
Column bases and Caps (Part of Module III)

- Column bases like base plates are used to transmit load from columns to foundations
- It reduces intensity of loading and distributes it over the foundations.
- Area of base plate chosen is so chosen such that the intensity of load distributed is less than the bearing capacity of concrete on which it rests.

In case of steel columns, safety of a column and thus a structure depend mainly upon;

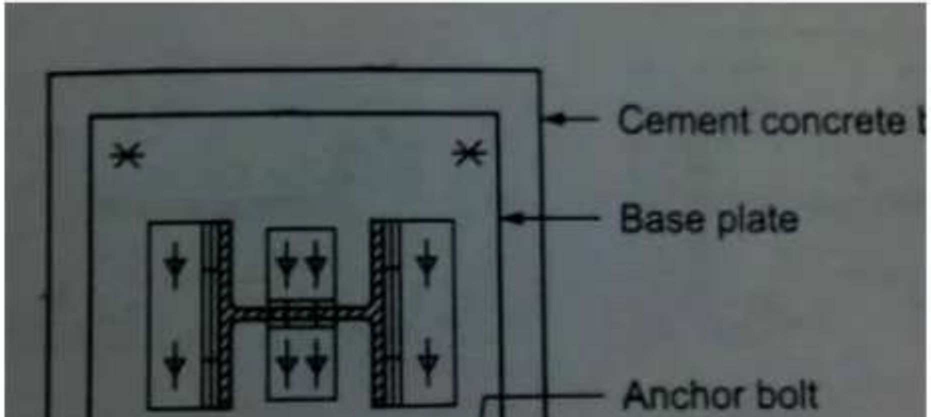
- Stability of foundations and consequently on the bases.

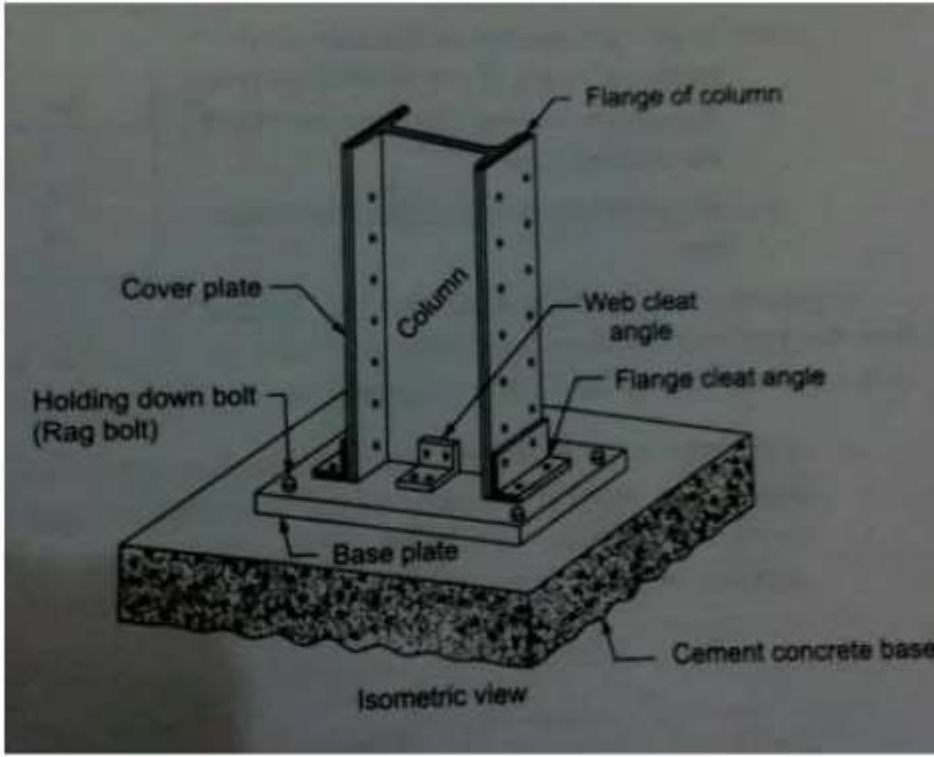
The main types of bases used are shown in figure. These are as



Slab base

- For columns carrying small loads, slab bases are used.
- Consists of base plate and cleat angles





- The machined column end transfers the load to the slab base by direct bearing

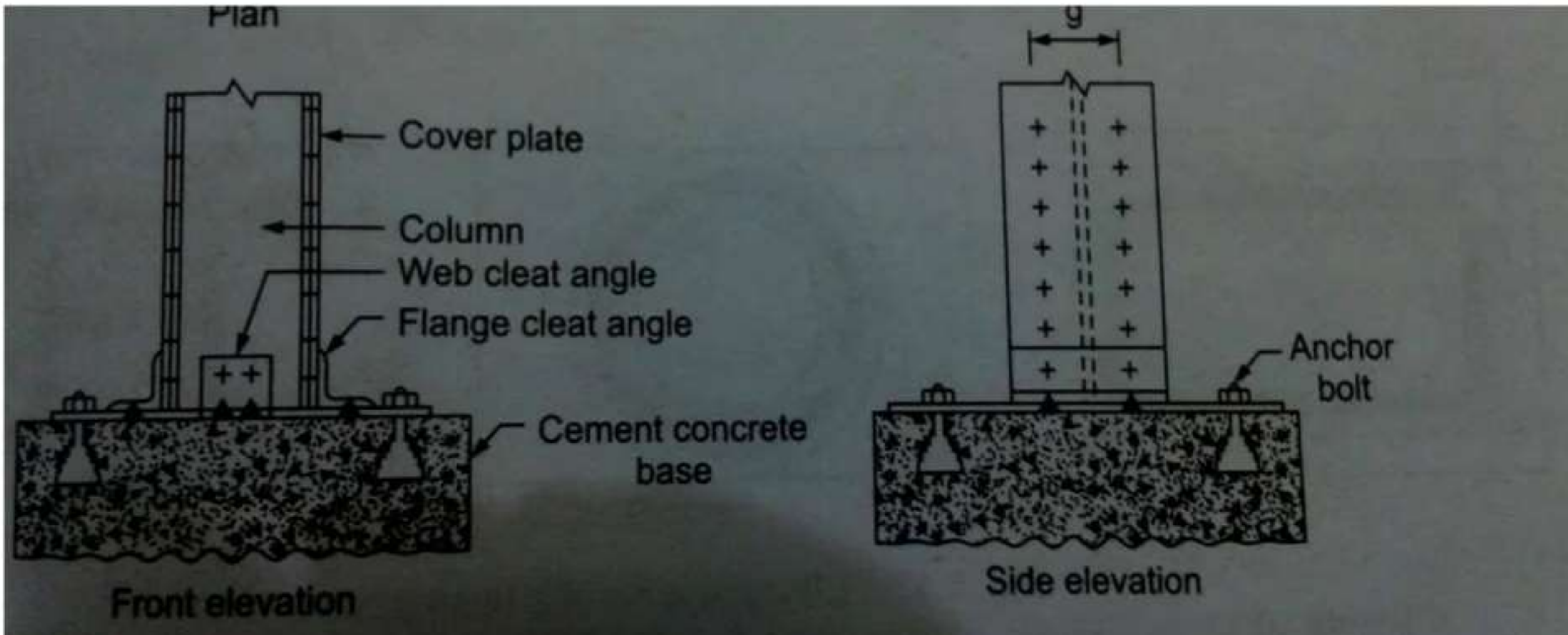


Fig. 5.15 Slab base

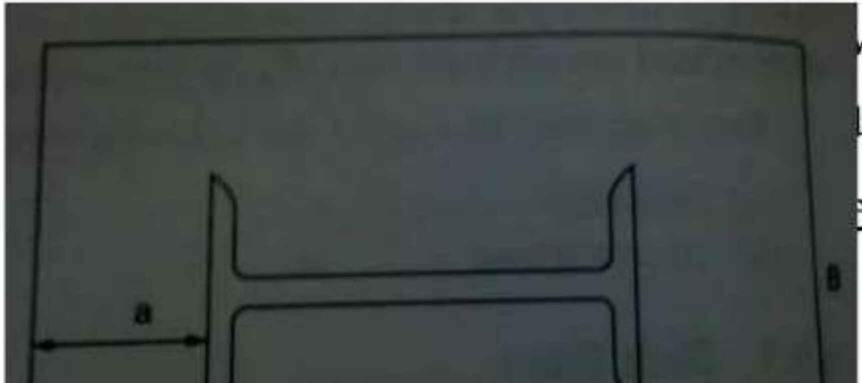
- Base plate is subjected to

Specifications by IS Code

- Minimum thickness ts of rectangular slab bases supporting columns under axial compression shall be

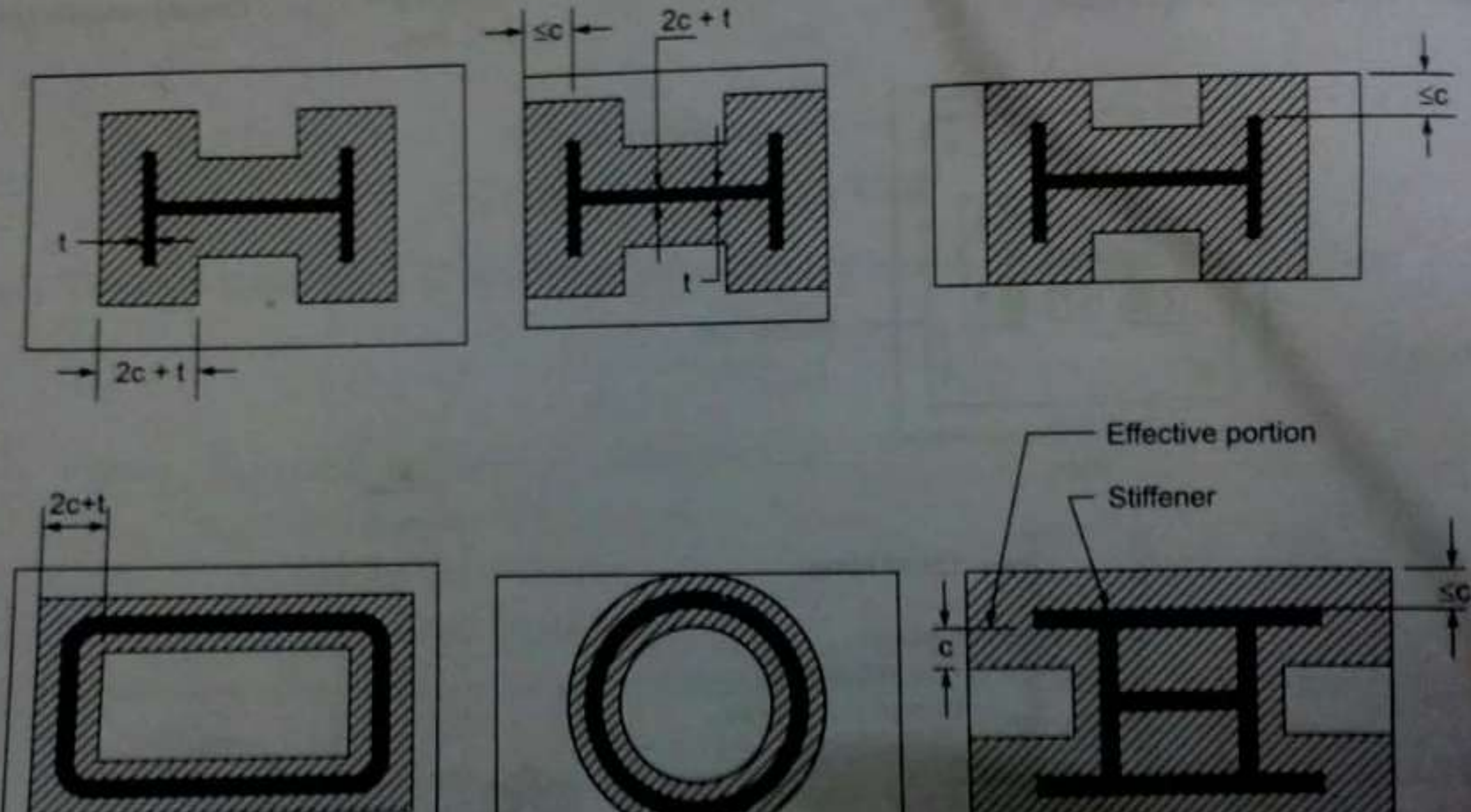
$$ts = \sqrt{2.5w(a^2 - 0.3b^2).\gamma_{mo} / fy} < tf$$

Where w = uniform press from below slab base
a,b = larger and smaller projection,
vely as shown in figure beyond the
lar circumscribing the column
e thickness of compression member



Sometimes a base plate of dim greater than the requirement may have been provided. In such cases the IS recommendation is as follows;

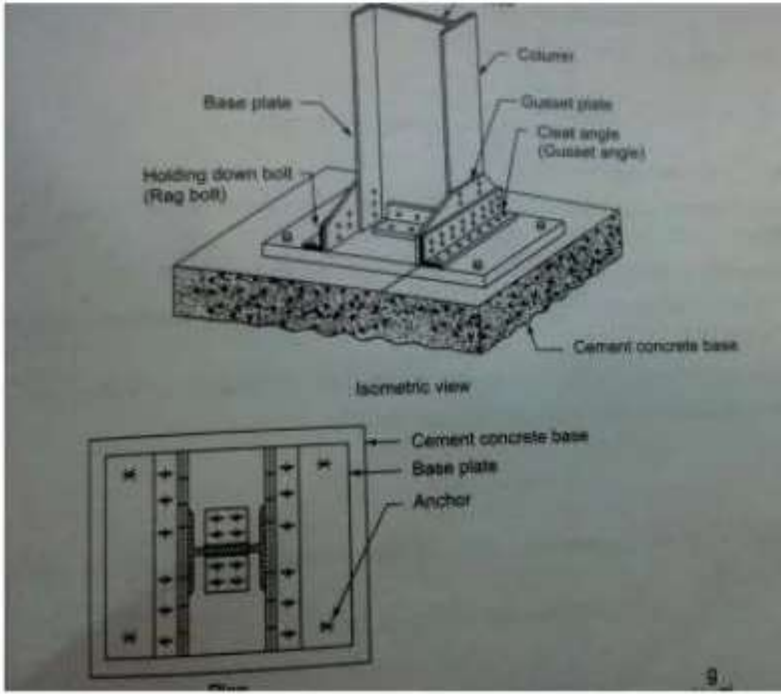
- If size of the base plate is larger than required to limit the bearing pressure on the base support, an equal projection c of the base plate beyond the face of the column and gusset may be taken as effective in transferring the column load as given in figure such that the bearing pressure of the effective area does not exceed the bearing capacity of the concrete base. Shown in following slide



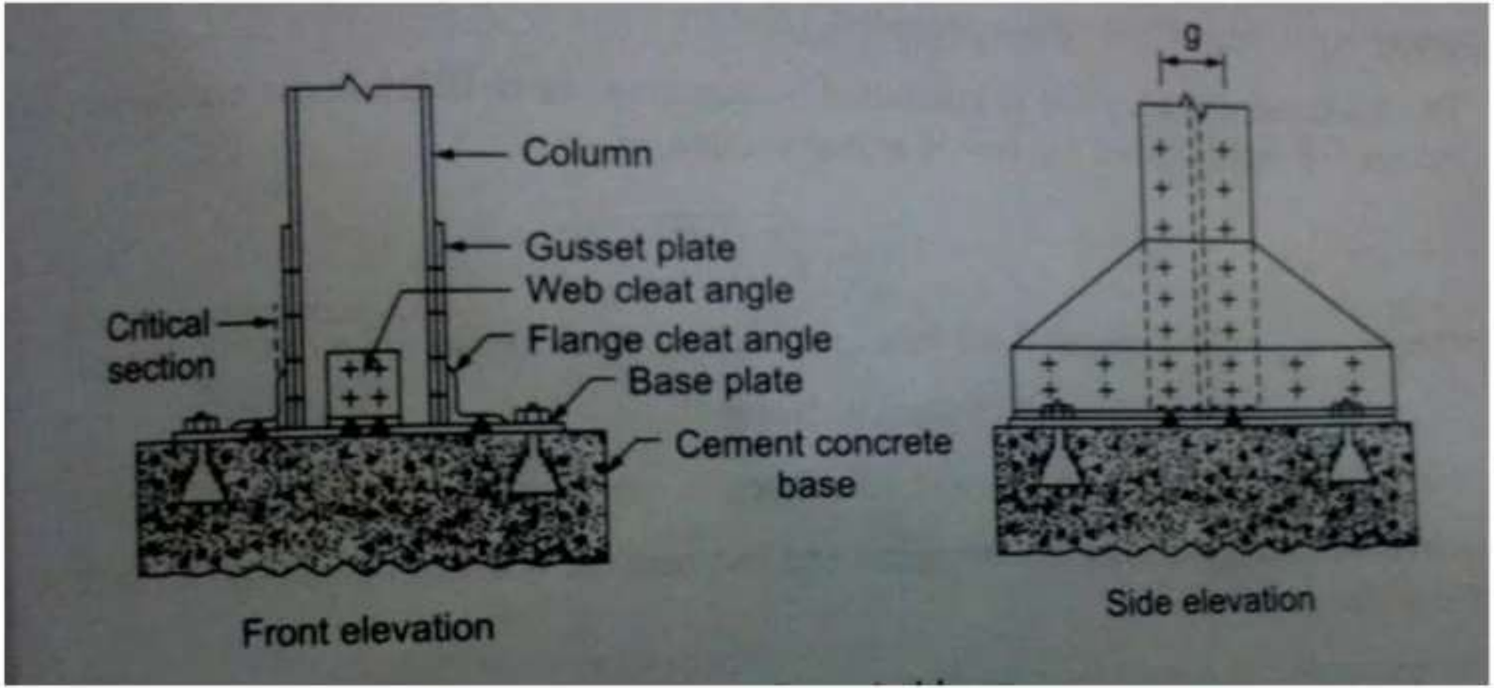
≤c
2c + t
t
t
2c + t
≤c
Effective portion
Stiffener
2c+t
c
≤c

Gusseted base

- For columns carrying heavy loads gusseted bases are used.
- Here load is transmitted to base plate through gusset plates attached to flanges of the column by means of angle iron cleats (also gusset angles)



- Here the thickness of the base plate will be less than slab base for the same axial load as the bearing area of



- The base plate is anchored at the four corners to the foundation with bolts to check the lateral movement

Design steps

1. Assume a suitable grade of concrete if not given in numerical. Based on the characteristic strength of concrete (fck) the bearing strength of concrete can be determined by 0.45fck
2. The area required of base plate is computed by

$$A\ (\text{plate}) = \frac{Pu}{\textbf{Bearing strength of concrete}}$$

where Pu = factored load on column

3. The size of plate is calculated from A(plate). The gusset plate should not be less than 16 mm in thickness for the bolted base plate.

The dimension of base plate parallel to the web can be calculated as

L= depth of section(d) + 2 (thickness of gusset plate + leg length of angle + overhang) (for bolted plate)

L= depth of section (d) + 2(thickness of gusset plate) + overhang (for welded plate)

the other dimension B can now be calculated as

B = A(plate) / L

4. The intensity of bearing pressure w from base concrete is calculate using expression

5. The thickness of the base plate is computed by equating the moment at the critical section to the moment of resistance of the gusset at the section

$$t = C1 \sqrt{2.7\, w/fy}$$

where C1 = the portion of the base plate acting as cantilever in mm

fy = yield strength of the steel in N/mm2

w = intensity of pressure calculated in step (4)

6. Bolted welded connection are designed and increase the number of bolts so it can be provided in regular pattern

Elastic Buckling of Columns

- The critical buckling load P_{cr} for columns is theoretically given by

$$P_E = \frac{\pi^2 E I}{L^2}$$

$$F_E = \frac{\pi^2 E I}{A L^2}$$

$$F_E = \frac{\pi^2 E}{(L/r)^2}$$

Effective lengths in different directions

Elastic Buckling of Columns

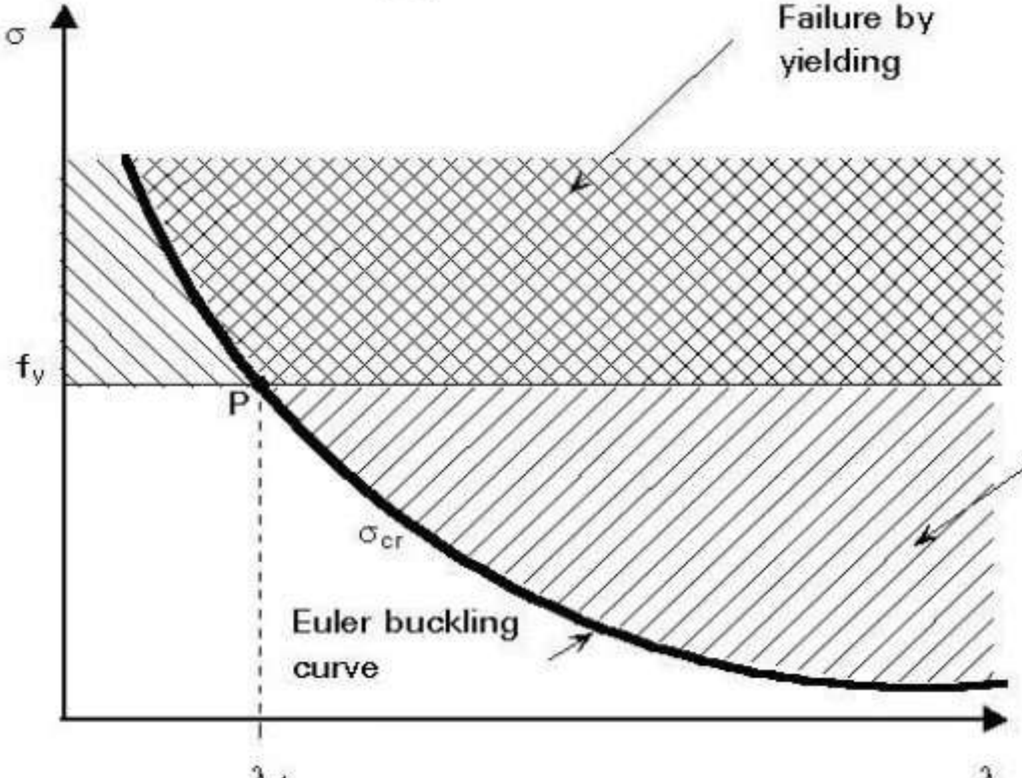


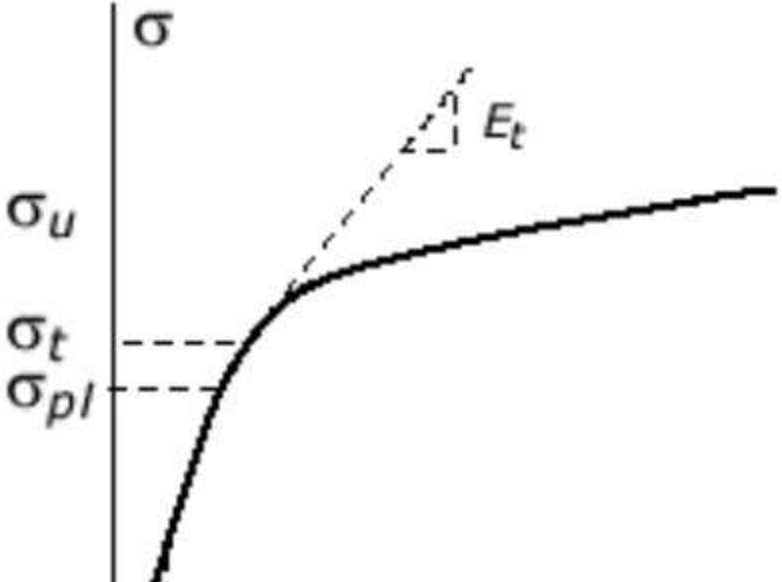
Figure 9 Local buckling curve and modes of failure

Inelastic Buckling of Columns

- In elastic buckling, it was assumed that a column made of a metal whose stress-strain curve is linear until a yield plateau reached.
- For a column with intermediate length, when buckling occurs after the stress in the column exceeds the proportional limit of the column material and before the stress reaches the

Inelastic Buckling of Columns

Tangent-Modulus Theory



$$\sigma_t = \frac{E_t \pi^2}{\left(L_{\text{eff}}/r\right)^2}$$

Inelastic Buckling of Columns

Tangent-Modulus Theory: Drawbacks

- Engesser's Conclusion was challenged with the basis that buckling begins with no increase in load.
- The tangent-modulus theory oversimplifies the inelastic buckling by using only one tangent modulus. In reality, the tangent

Inelastic Buckling of Columns

Tangent-Modulus Theory: Drawbacks

- The tangent-modulus theory tends to underestimate the strength of the column, since it uses the tangent modulus once the stress on the concave side exceeds the proportional limit while the convex

Inelastic Buckling of Columns

Reduced Modulus Theory

- Engesser presented a second solution to the inelastic-buckling, in which the bending stiffness of the x-section is expressed in terms of double modulus E_r to compensate for the underestimation given by the

Inelastic Buckling of Columns

Reduced Modulus Theory

- For a column with rectangular cross section, the reduced modulus is defined by:

$$E_r = \frac{4EE_t}{\left(\sqrt{E}+\sqrt{E_t}\right)^2}$$

The corresponding critical stress is,

Inelastic Buckling of Columns

Reduced Modulus Theory: Drawbacks

- The reduced-modulus theory tends to overestimate the strength of the column, since it is based on stiffness reversal on the convex side of the column.

Inelastic Buckling of Columns

Reduced Modulus Theory: Drawbacks

- The reduced-modulus theory oversimplifies the inelastic buckling by using only one tangent modulus. In reality, the tangent modulus depends on the stress which is a

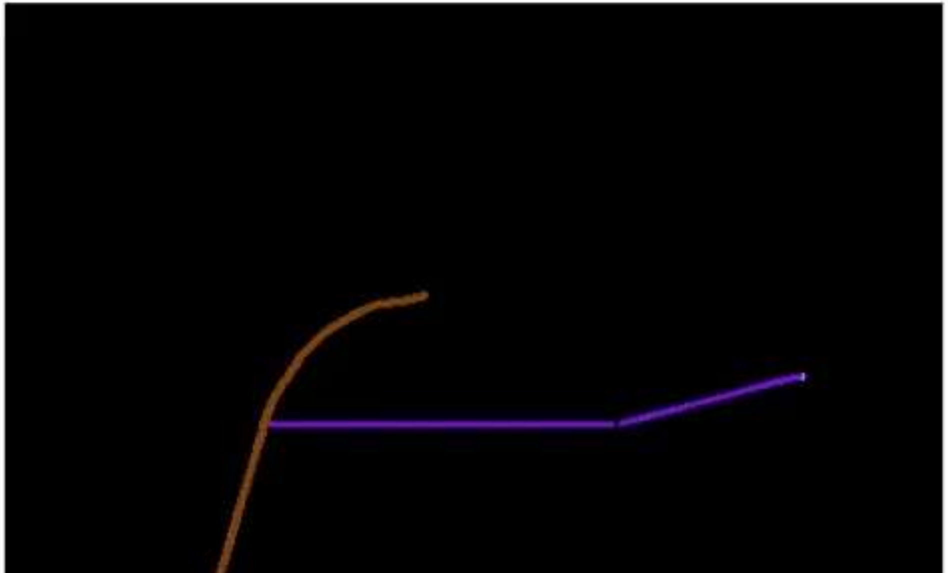
Inelastic Buckling of Columns

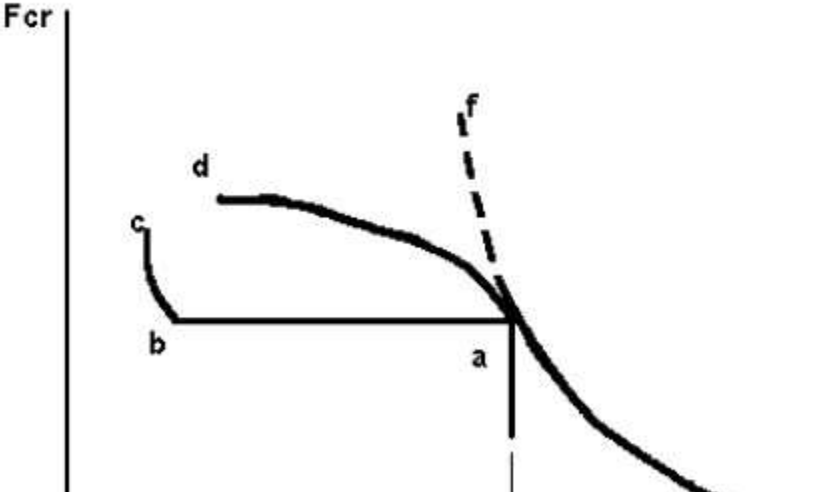
Shanley's Theory

- The critical load of inelastic buckling is in fact a function of the transverse displacement w
- Practically there are manufacturing defects in mass production and geometric inaccuracies in assembly.

Inelastic Buckling of Columns

Shanley's Theory





Factors effecting Buckling

1. End Connections
2. Eccentricity of loads/Crookedness
3. Residual stresses

Factors effecting Buckling

1. End Connections

- Rotation of ends of columns in building frames is usually limited by beams connecting to them.

Factors effecting Buckling

1. End Connections: Effective length

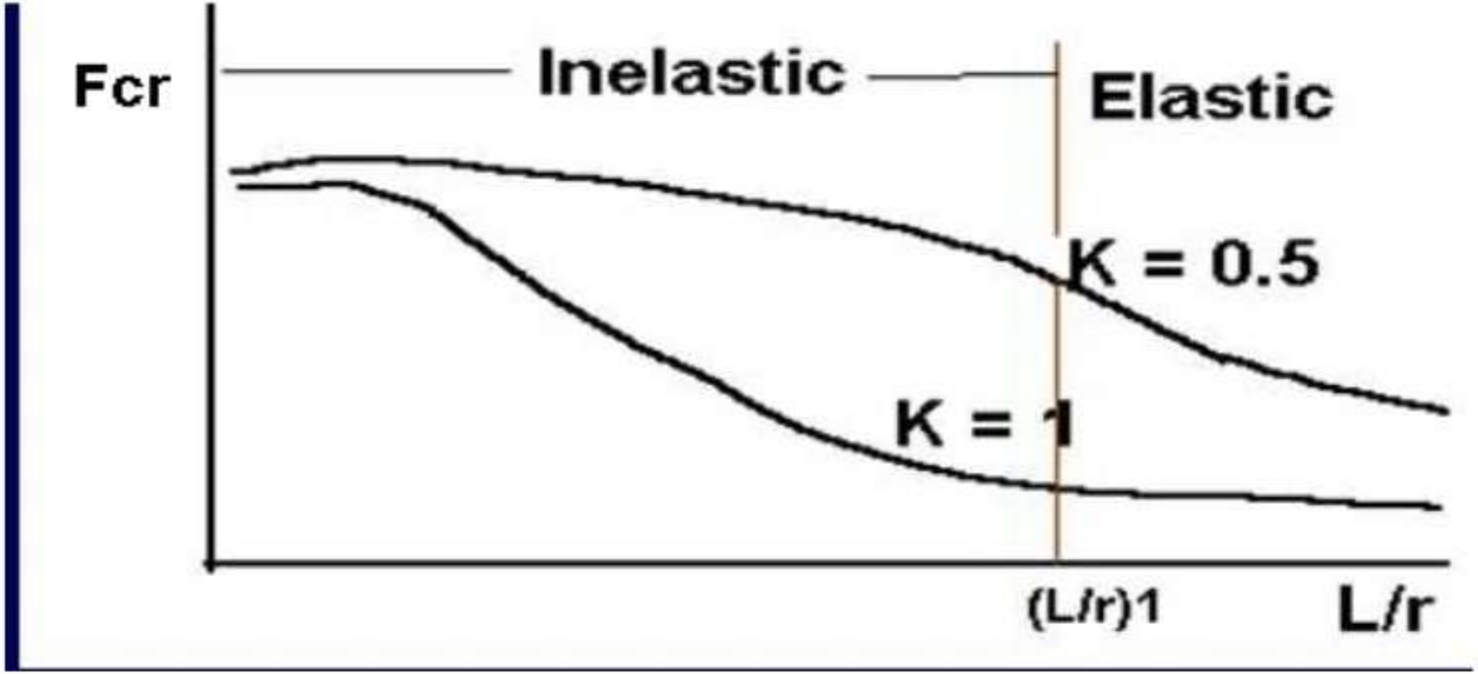
$$P = \frac{\pi^2 E I}{(2L)^2}$$

$$P_E = \frac{\pi^2 E I}{L^2}$$

$$P = \frac{\pi^2 E I}{(KL)^2}$$

Factors effecting Buckling

1. End Connections: Effective length



- *A column with fixed ends can support four times as much load as a column wi*

Factors effecting Buckling

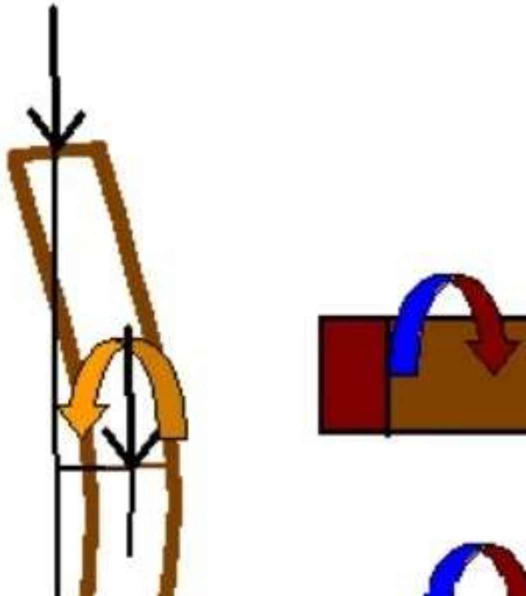
2. Effect of initial crookedness

- The initial out-of-straightness is also termed "initia crookedness" or "initial curvature".
- It causes a secondary bending moment as soon as any compression load is applied, which in turn leads to further bending deflection.

Factors effecting Buckling

2. Effect of initial crookedness

- A stable deflected shape is possible as long as the external moment, i.e. the product of the load and the lateral deflection, does not exceed the internal moment resistance of any section.



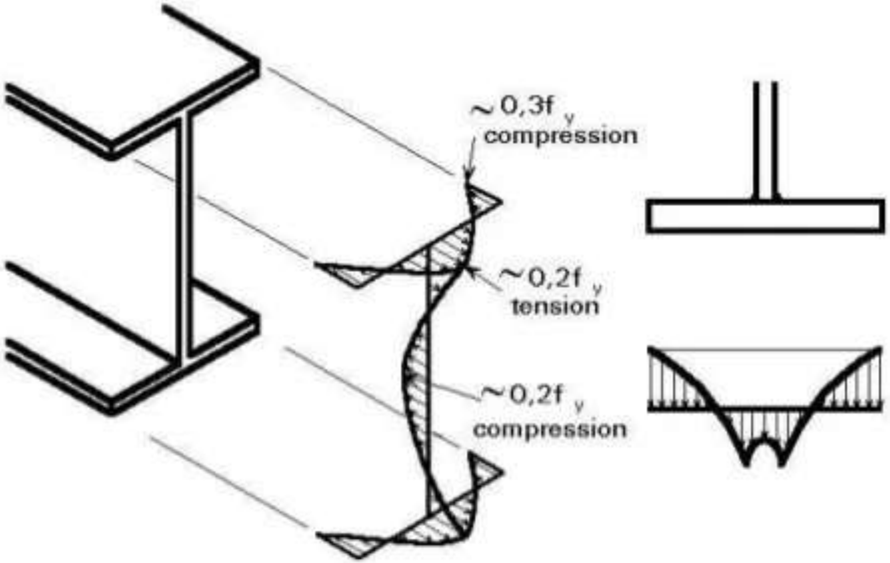
Factors effecting Buckling

3. Effect of Residual Stresses

- Complete yielding of x-section did not occur until applied strain equals the yield strain of base material.
- The residual stresses does not affect the load corresponding to full yield of x-section.

Factors effecting Buckling

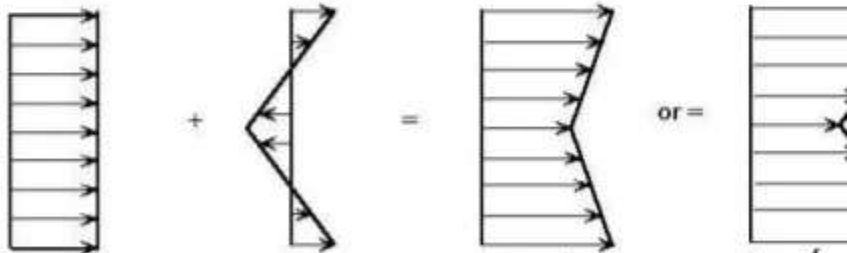
3. Effect of Residual Stresses



Example of residual stresses due to hot-rolling

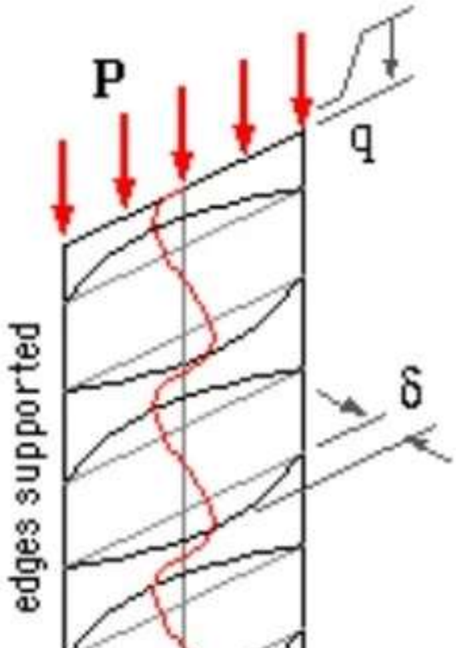
Example of residual stresses due to welding

(a)

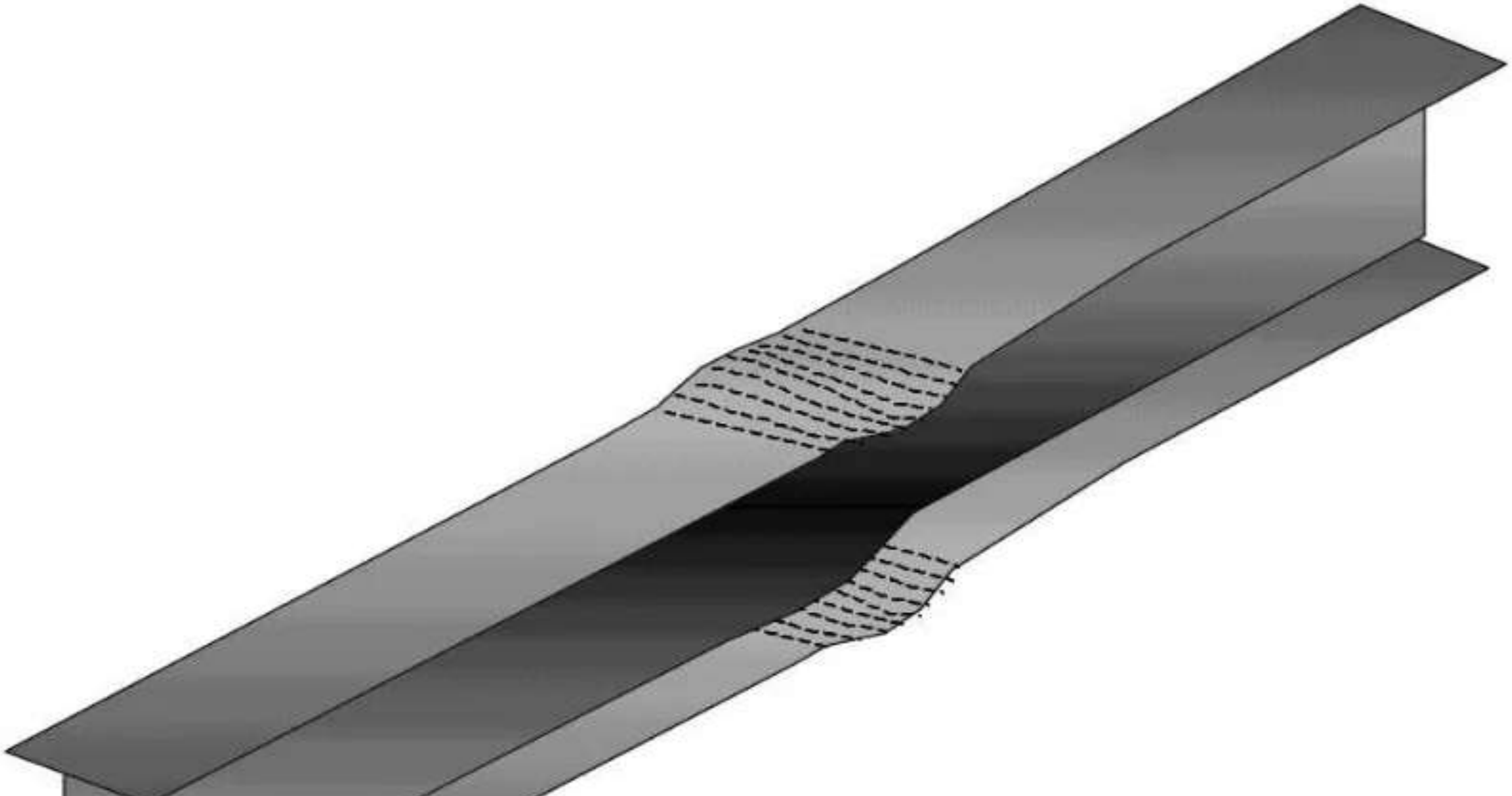


Local Buckling

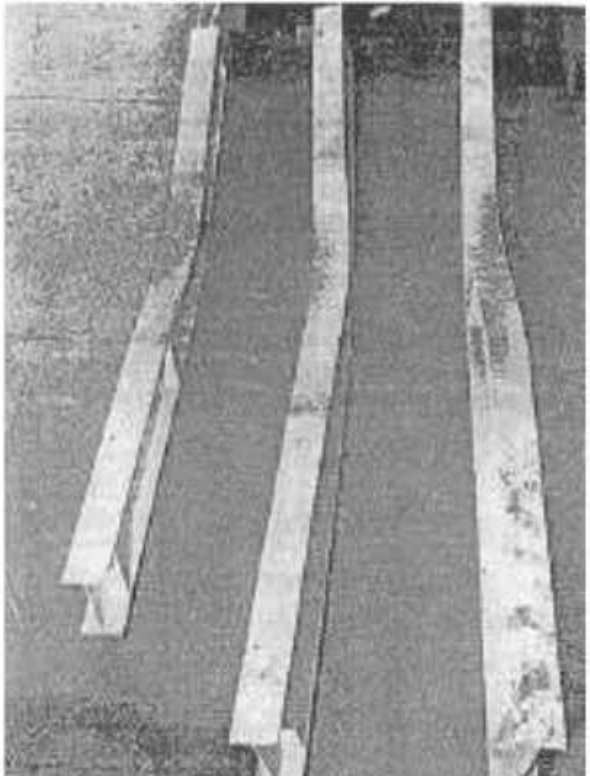
- If the column section is made of thin (slender) plate elements, then failure can occur due to *local buckling* of the flanges or the webs in compression well before the calculated buckling strength of the whole member is reached.
- When thin plates are used to carry compressive stresses they are particularly susceptible to buckling about their weak axis due small moment of Inertia.



Local Buckling



Local Buckling



Flange Buckling

Local Buckling

Local Buckling

- If *local buckling* of the individual plate elements occurs, then the column may not be able to develop its buckling strength.
- Therefore, the local buckling limit state must be prevented from controlling the column strength.

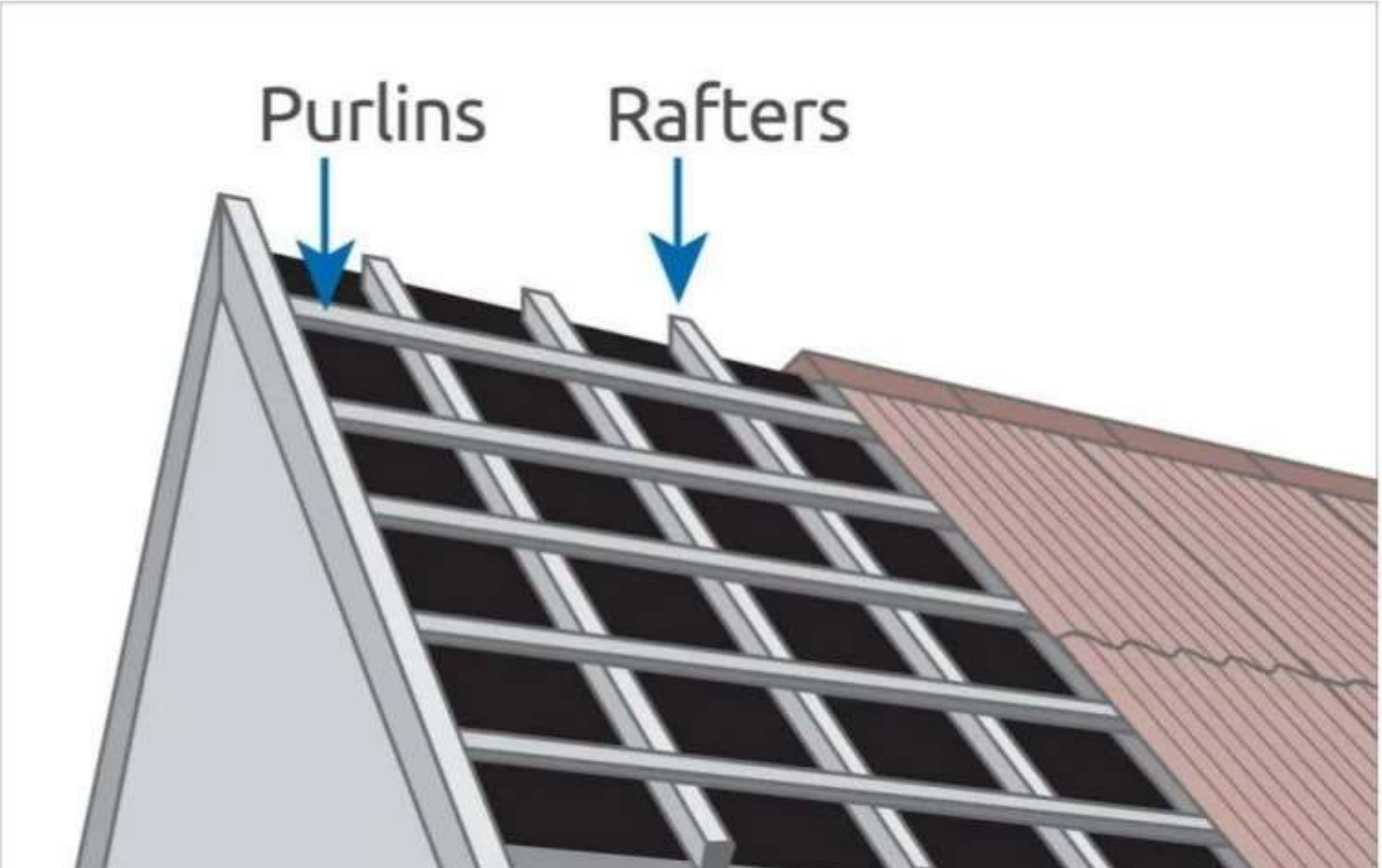
Design of beam

IS 800- 2007

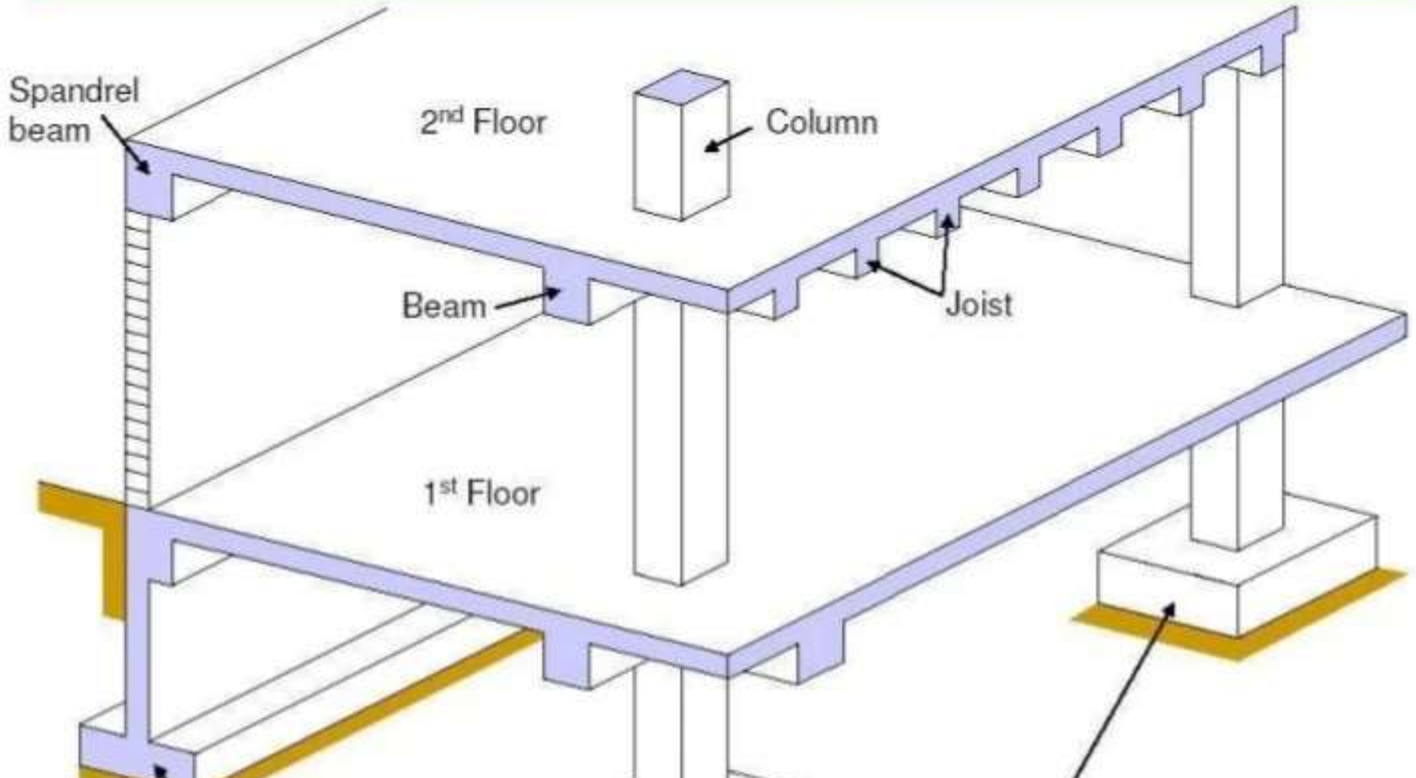
Beams

- Horizontal member seen in a structure spannin between columns.
- Support loads which are resisted by bending an shear
- Supports floors, roof sheeting as purlins, sid cladding.

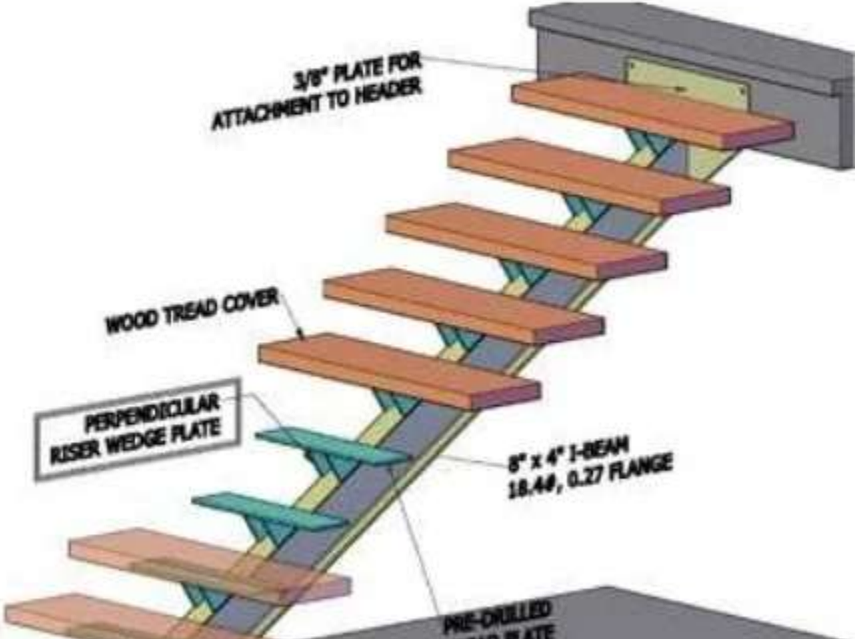
- Floor beams- major beam supporting the secondar beams or joists
- Girder- floor beam in buildings
- Lintel –beam used to carry wall load over openings, i. doors, windows etc
- Purlin- roof beam supported by roof trusses
- Rafter- roof beam supported by purlins
- Spandrel beam- beam at outter most wall of buildings


Purlins
Rafters

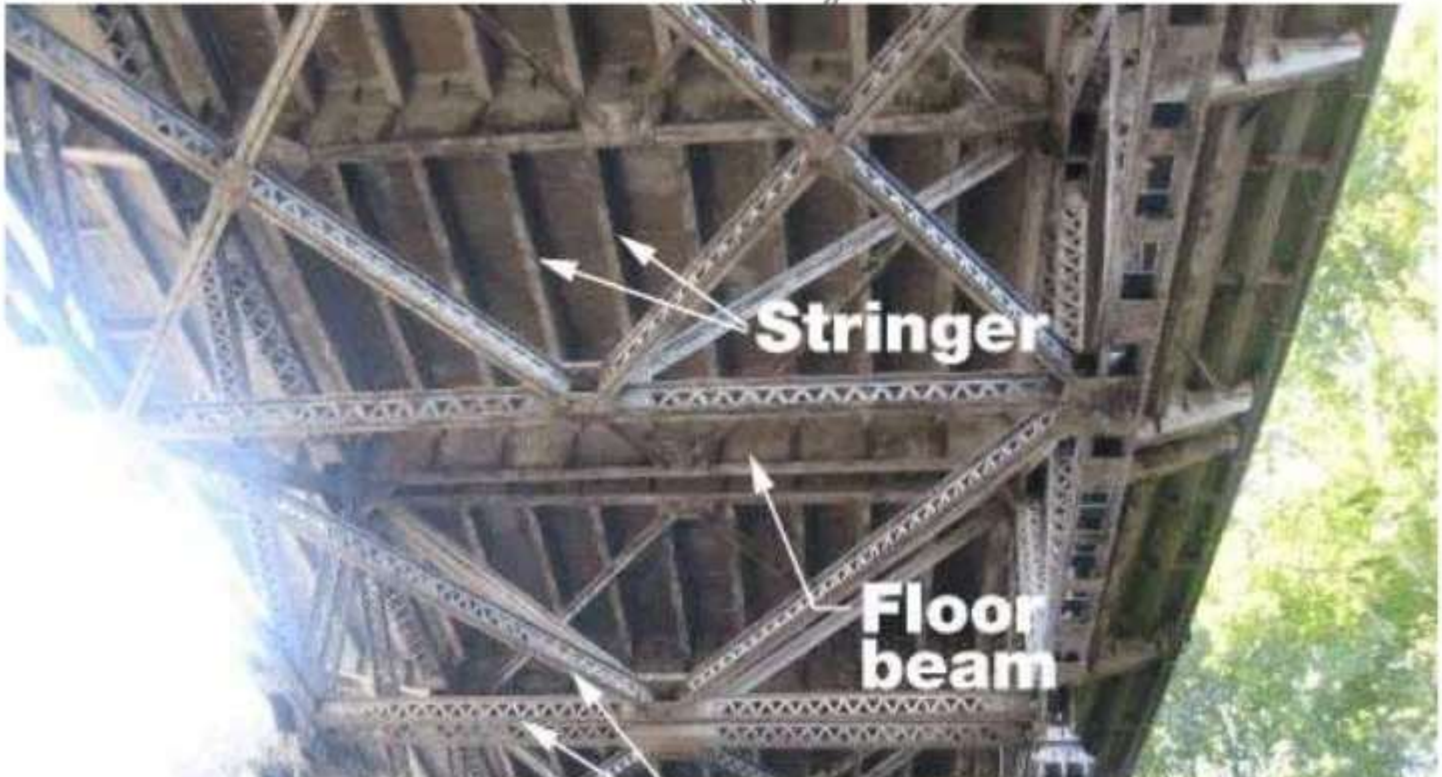
Typical Structure (1)



Stringer beam under stairs



Stringer beam in bridge


Stringer
Floor
beam

- Based on how beam is supported
- Simply supported beams
- Cantilever beams
- Fixed beams
- Continuous beams

Commonly used type beam sections

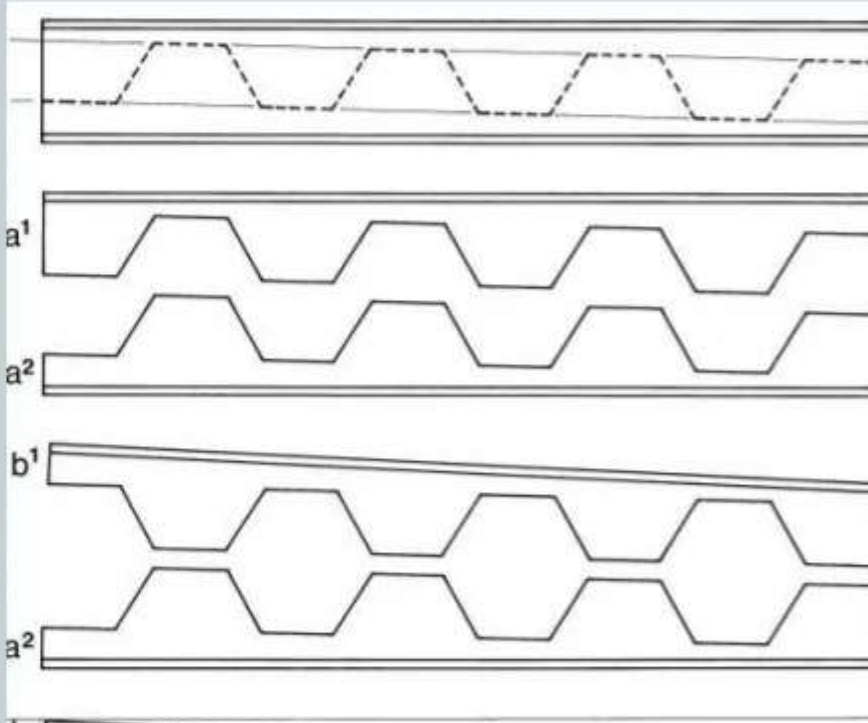
- *Universal beams (rolled sections*): in this material i
concentrated in the flanges and very efficient in uni
axial bending
- *Compound beam*: universal beam strengthened by
flange plates. Resist bending in vertical as well a
horizontal direction.
- *Composite beam*: universal beam with roof slal

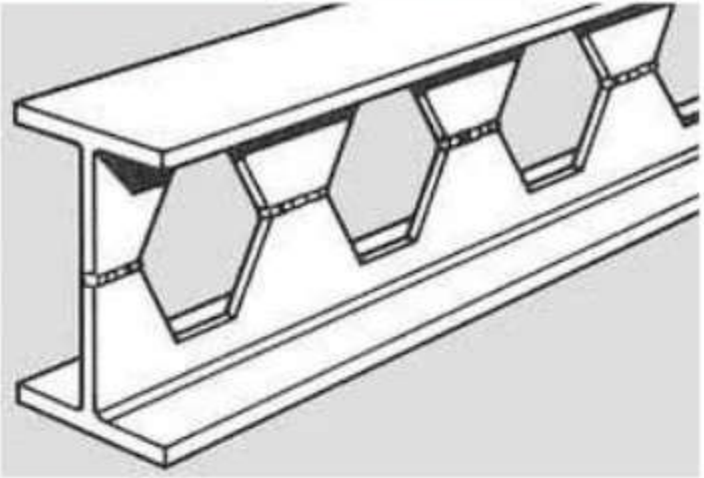
- Castellated beams: beams made by applying a specia technique to wide flange I-beam. This technique consists of making a cut in the web of a wide flange beam in a corrugated pattern. The cut parts are separated and the lower and upper parts are shifted and welded as shown in the next slide

What is advantage of castellation

- Light, strong and cheap
- Easy to assemble at construction site
- Openings simplify the work of installer and electrician, since taking pipes cross the beams do not pose a problem
- Constructon elements such as ceiling systems can be installed easily.

- depth can be determined at will by changing the cutting pattern
- combining of a lighter upper half with a heavier lower half






TWO WAY TRAFFIC / KEEP RIGHT

Classification of beam section

- Bending strength of a beam depends upon how well the section performs in bending
- Thin projecting flange of an I-beam is likely to buckl
prematurely
- Web of an I-section can buckle under compressive stress due to bending and shear
- In order to prevent such local buckling it is necessar

The sectional dimensions should be such that the following conditions are satisfied:-

- When the design is made using elastic analysis the member should be able to reach the yield stress under compression without buckling.
- When design is done by plastic analysis the member should be able to form plastic hinges with sufficient rotation capacity(i.e ductility) without local buckling, so as to permit redistribution of bending moments needed before reaching collaps

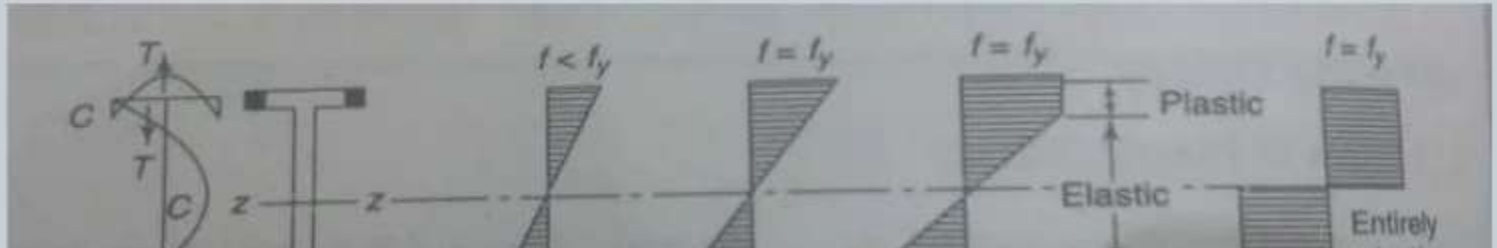
Plastic analysis

- The transition from elastic to plastic analysis
- In elastic design method, the member capacity is based on the attainment of yield stress.
- Steel's unique property of ductility is not utilised
- Ductility enables the material to absorb large deformations beyond the elastic limit without fracture, due to which steel possesses reserve

Concepts of plastic analysi

- Plastic analysis makes the design more rational, since level of safety is related to collapse load of the structure and not to apparent failure at one point.

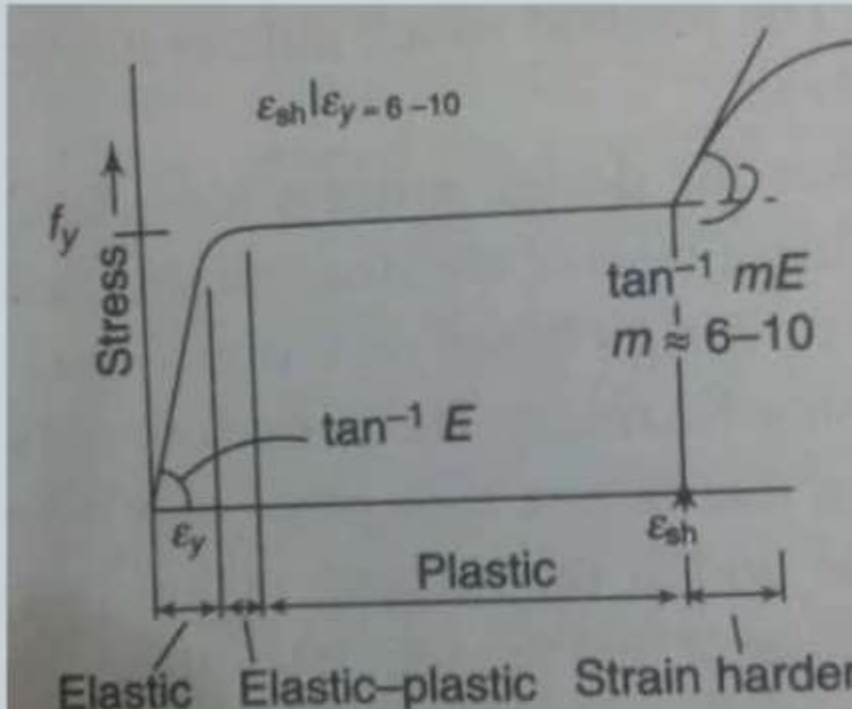
Consider I-beam subjected to steadily increasing BM 'M' as shown in figure.



- When yield stress reaches the extreme fibre as shown in figure (b) the nominal moment strength Mn of the beam is referred to as the yield moment My and is given by Mn= My = Ze.fy

  Where Ze is the elastic section modulus
- Further increase inBM causes the yield to spread inwards from the outter upper surfaces of the beam as shown in figure (C)

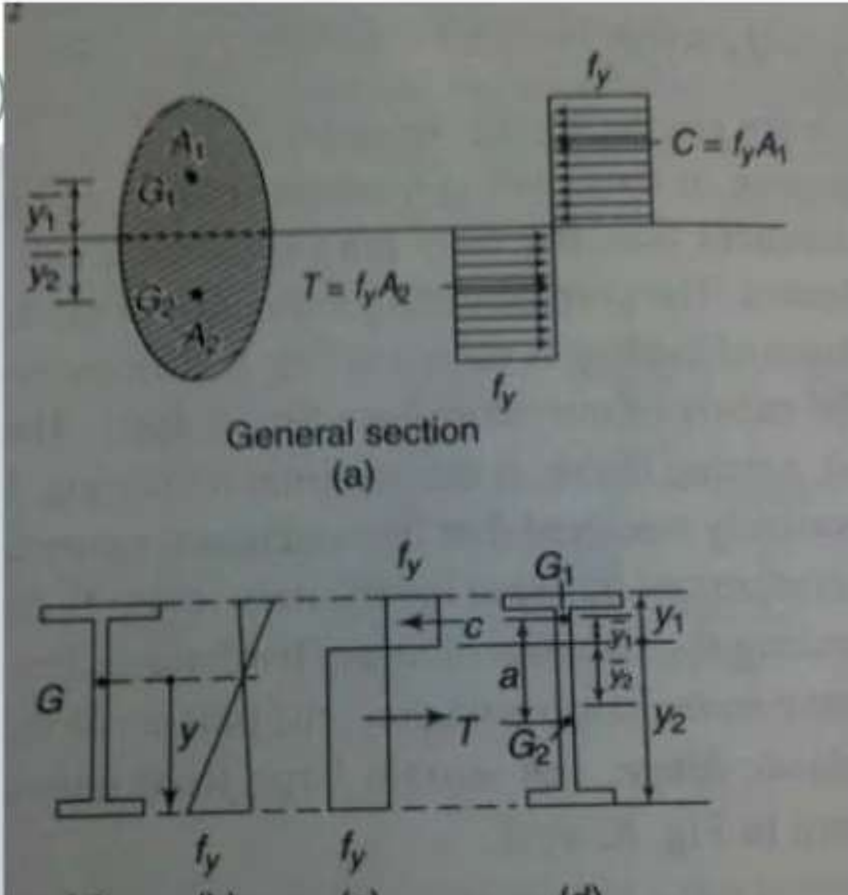
- This stage of partial plasticity occurs because of the yielding of the outer fibres without increase of stresses as shown by the horizontal line of the idealised stress strain diagram shown in figure
- Upon increasing the BM further, the whole section yields as shown in figure d. When this condition is reached every fibre has a strain equal to or greater than εy = fy/Es.The nominal moment strength Mn at this



- Any further increase in BM results only in rotation, since no greater resisting moment than the fully plastic moment can be developed until strain hardening occurs.
- The maximum moment Mp is called the *Plastic moment of resistance*, the portion of the member where Mp occurs is termed as *plastic hinge*.

- For equilibrium of normal forces, the tensile and compressive forces should be equal. In elastic stage, when bending varies from zero at neutral axis to a max a the extreme fibres, this condition is achieved when the neutral axis passes through the centroid of the section.
- In fully plastic stage, because the stress is uniformly equal to the yield stress, equilibrium is achieved when the neutral axis divides the section into two equal areas.

- Considering the general cross section in fig. and equating the compressive and tensile forces we get.
- Fy .A1 = fy. A2
- Since A1= A2= A/2
- A= A1+A2
- Plastic moment of resistance
- Mp = fy.A1. y1 + fy. A2.y2
- =fy. A/2 (y1+y2)

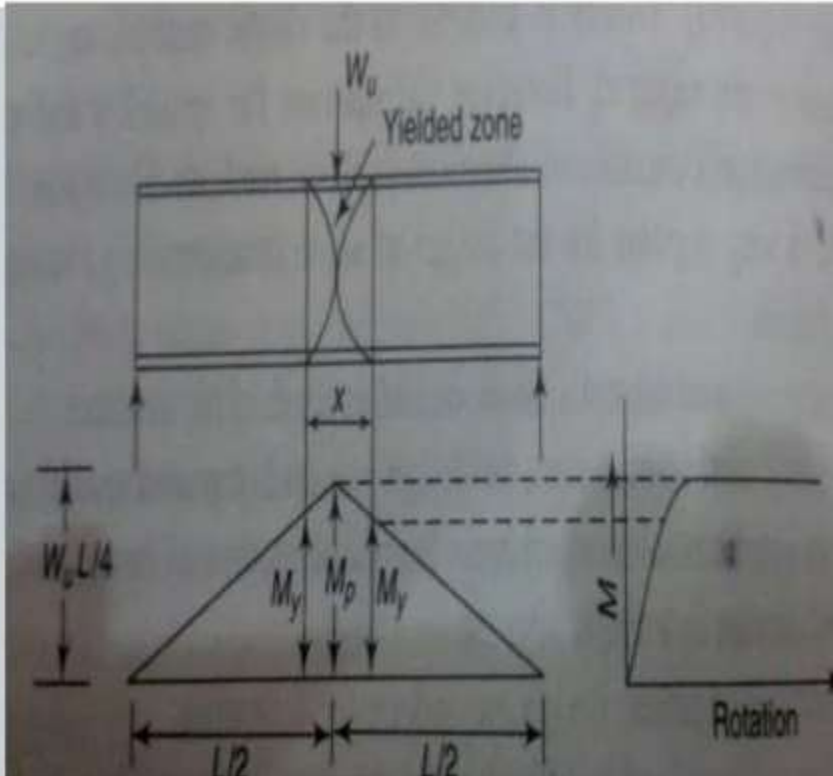


Shape factor

- The ratio Mp/My is a property of the cross sectional shape and is independent of the material properties. This ratio is known as the shape facot v and is given by v = Mp/My = Zp/ Ze
- For wide flange I – sections in flexure about the strong axis (Z-Z) the shape factor ranges from1.09 to about 1.18 with average value being about 1.14.
- One may conservatively take the plastic moment strength of I-sections bent about their strong axis to be atleast 15% greater than the strength My when the extreme fibre reaches the yield stress fy.
- On the other hand the shape factor for I bent about their minor axis is

Plastic hinge concept

- As external load increases, thus BM increases, rotation at a section increases proportionally up to the yield point.
- Further increase in moment will generate a non linear relation between the stress and strain and the curvature increases rapidly to reach an unbounded value as moment tends to reach the value of Mp.
- The part on the moment rotation curve where there is plasticity may be projected onto the moment diagram which can then be projected to the



- Thus the plastic hinge can be defined as a yielded zone. In thi zone the bending of a structural member can cause an infinit rotation to take place at constant plastic moment Mp of th section.

Plastic hinges in a member are formed at:-

- Max moment locations,
- At intersections of two members where BM is same; in weake section
- Restrained ends,

- Considering the figure again
- Let Wu be the load on a simply supported beam as shown

Mp= Wu.L/4

also Mp= fy. Zp= fy.bh²/4---(1)

And My = fy.Ze = fy. bh²/6---(2)

We can write Eq. 2 as

My = fy. (bh²/4)(2/3)

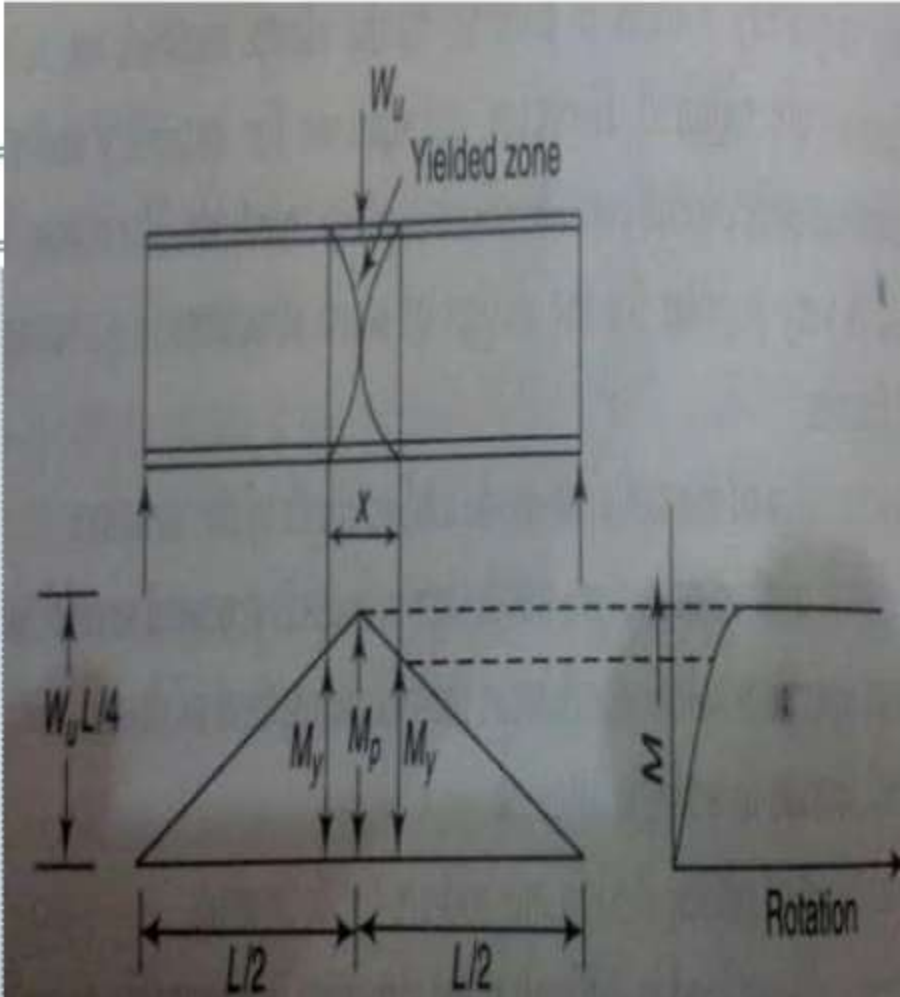
= Mp . (2/3)-------(3)

Let x be the length of plasticity zone. From similar triangles

Mp/(L/2)= My /(L/2 – x/2)

Mp/L = My / (L-x)

My/Mp = (L-x)/L



- For uniformly loaded fixed-end beam, Bowles(1980) showed that the hinge length at the ends is given by

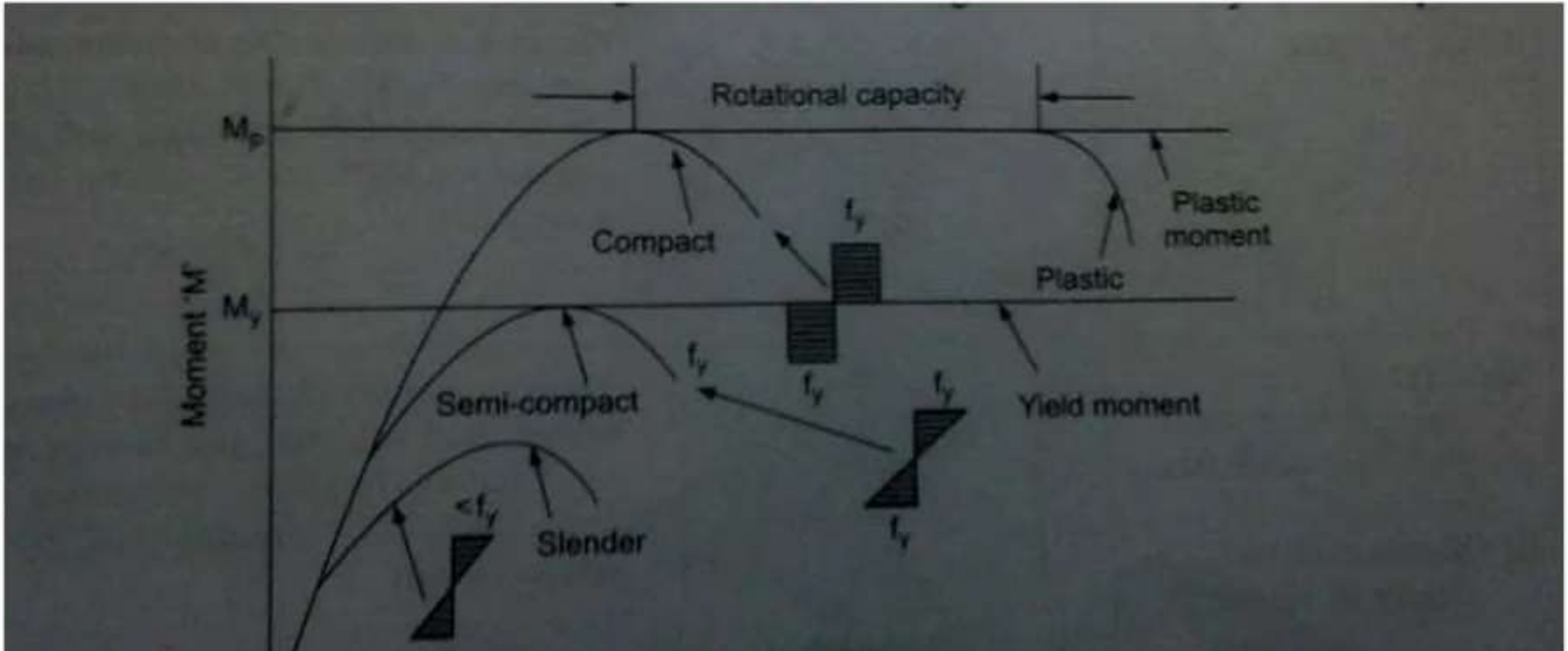
$$x = \left(\frac{L}{2.83}\right)\sqrt{1 - \frac{1}{v}}$$

- For an I-section, the length of the plastic hinge at the centre of the beam is (taking v=1.12)

Based on above the beam sections are classified as follows as per IS 800-2007

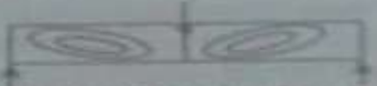
- Class 1(Plastic): cross section which can develop plastic hinges and have rotation capacity required for failure of the structure by formation of plastic mechanism. The section having width to thickness ratio of plate element less than that specified under class 1 as shown in table 2 (page 18)..\is.800.2007- code of practice for gener steel.pdf
- Class 2 (compact section): cross section which can develop plastic moment of resistance, but have inadequate plastic hinge rotation capacity for formation of plastic mechanism, due to local

- Class 3(Semi compact section):cross section in which extreme fibre in compression can reach yield stress but cannot develop plastic moment of resistance, due to local buckling. The width to thickness ratio of plate shall be less than that specified under class 3,but greater than that specified under class 2 as shown in the table 2is.800.2007- code of practice for gener steel.pdf
- Class 4 (slender): cross-section in which the elements buckle locally even befor reaching yield stress. The width to thickness ratio of plate shall be greater than that specified under class 3. IS


Rotational capacity
M_p
M_y
Moment 'M'
Compact
f_y
f_y
Plastic moment
Plastic
Yield moment
Semi-compact
f_y
f_y
f_y
f_y
<f_y
Slender

Failure modes of beams

Table 10.2 Main failure modes of hot-rolled beams

Mode		Comments
Excessive bending triggering collapse Category 1		This is the basic failure mode provided (a) the beam is prevented from buckling laterally and (b) the component elements are at least compact, so that they do not buckle locally. Such 'stocky' beams will collapse by plastic hinge formation.
Lateral torsional buckling of long beams which are not suitably braced in the lateral direction (i.e., 'unrestrained' beams) Category 2		Failure occurs by a combination of lateral deflection and twist. The proportions of the beam, support conditions, and the way the load is applied are all factors which affect failure by lateral torsional buckling.
Failure by local buckling of a flange in compression or web due to shear or web under compression due to concentrated loads Category 3	  	Unlikely for hot-rolled sections, which are generally stocky. Fabricated box sections may require flange stiffening to prevent premature collapse. Web stiffening may be required for plate girders to prevent shear buckling. Load bearing stiffeners are sometimes needed under point loads to resist web buckling.
Local failure by (a) shear yield of web, (b) local crushing of web, and (c)		Shear yield can only occur in very short spans and suitable web stiffeners will have to be

Design strength in bending (flexure)

- Design bending strength (bs) of beam, supported against lateral torsional buckling(laterally supporte beam) is governed by the yield stress.
- The factored design moment, M at any section, in a beam due to external actions, shall satisfy the relationship M<=Md where Md is the design bending strength of the section.

Laterally supported beam

- A beam may be assumed to be adequately supported at the supports provided the compression flange has full lateral restraint and nominal torsional restraint at support supplied by web cleats, partial depth of plates etc.
- Full lateral restraint to cmpression flange may be assumed to exist if the frictional or other positive restraint of a floor connection to the compression flange of the member is capable of resisting a lateral force not

Classification of laterally supported beams

- Laterally supported beams of plastic,compact or semicompact sections are classified into the following cases;
- Case i: Web of section susceptible to shear buckling before yielding
- Case ii : Web of section not susceptible to shear buckling before yielding

Case I

- Web of section susceptible to shear buckling before yielding
- When the flanges are plastic, compact, semi-compact but the web is susceptible to shear buckling before yielding ($d/tw <= 67\,\varepsilon$) the design yielding stress may be calc using one of the following methods:
- i) the BM and axial force acting on the section may be assumed to be resisted by flanges only and web is designed only to resist shear.
- ii) the whole section resist the BM and axial force acting on the section and therefore the web has to be designed for combined shear and its share of normal stresses. This is done by using simple elastic theory in case of semi-compact webs and simple plastic theory in case of compac

Case II

- Web of section not susceptible to buckling under shear before yielding (page 59)
- d/tw >= 67 ε

Beams in this case are stoky beams where ε is given by (see right)

For these beams the factored SF V does not exceed 0.6Vd, where Vd is the design shear strength given by

8.4.2 *Resistance to Shear Buckling*

8.4.2.1 Resistance to shear buckling shall be v as specified, when

$d/t_w > 67\varepsilon$ for a web without stiffeners, a

$> 67\varepsilon\sqrt{\frac{K_v}{5.35}}$ for a web with stiffeners

where

K_v = shear buckling coefficient (*see* **8.4.2.**

$\varepsilon = \sqrt{250/f_y}$

A_v = shear area, and

- when factored design SF does not exceed *0.6 Vd*, the design bending strength *Md* shall be taken as

$$Md = \frac{\beta b.zp.fy}{\gamma m0} \leq \frac{1.2.ze.fy}{\gamma m0} in.case.of.simply sported beam$$

$$Md = \frac{\beta b.zp.fy}{\gamma m0} \leq \frac{1.5ze.fy}{\gamma m0} for.cantilever.beam$$

Where βb = 1.0 for plastic and compact section

- When factored SF V exceeds 0.6 Vd, the design strength Md will be taken as Md = Mdv
- Where Mdv = design bending strength under high shear
- As per IS code this is calculate as follows:-

$$Mdv = Md - \beta(Md - Mfd) \leq \frac{1.2Ze.fy}{\gamma mo} (forplastiandcompactsect$$

- Where $\beta = \left[2\frac{V}{Vd} - 1 \right]^2$

$$Mfd = \frac{Zfd.fy}{\gamma m}$$

$$where--Zfd = Zp - \frac{tw.h^2}{4}$$

$$Mdv = \frac{Ze.fy}{} \; for.semi-compact section$$

Effect of holes in the tension zone(page 53)

8.2.1.4 *Holes in the tension zone*

a) The effect of holes in the tension flange, on the design bending strength need not be considered if

$$(A_{nf}/A_{gf}) \geq (f_y/f_u)(\gamma_{m1}/\gamma_{m0})/0.9$$

where

A_{nf}/A_{gf} = ratio of net to gross area of the flange in tension,

f_y/f_u = ratio of yield and ultimate stress of the material, and

γ_{m1}/γ_{m0} = ratio of partial safety factors against ultimate to yield stress (*see* **5.4.1**).

b) The effect of holes in the tension region the web on the design flexural strength ne not be considered, if the limit given in (above is satisfied for the complete tensi zone of the cross-section, comprising t tension flange and tension region of t web.

c) Fastener holes in the compression zone of t cross-section need not be considered in desi bending strength calculation, except f oversize and slotted holes or holes witho any fastener.

Shear lag effects

- Simple theory of bending is based on the assumption that plane sections remain plane after bending. But presence of shear strain causes section to warp. Its effect is to modify the bending stresses obtained by simple theory, producing higher stresses near junction of a web and lower stresses at points far away from it as shown in the figure. This effect is called shear lag.
- the effect is minimal in rolled sections, which have narrow and thick flanges and more pronounced in plate girder sections having wide

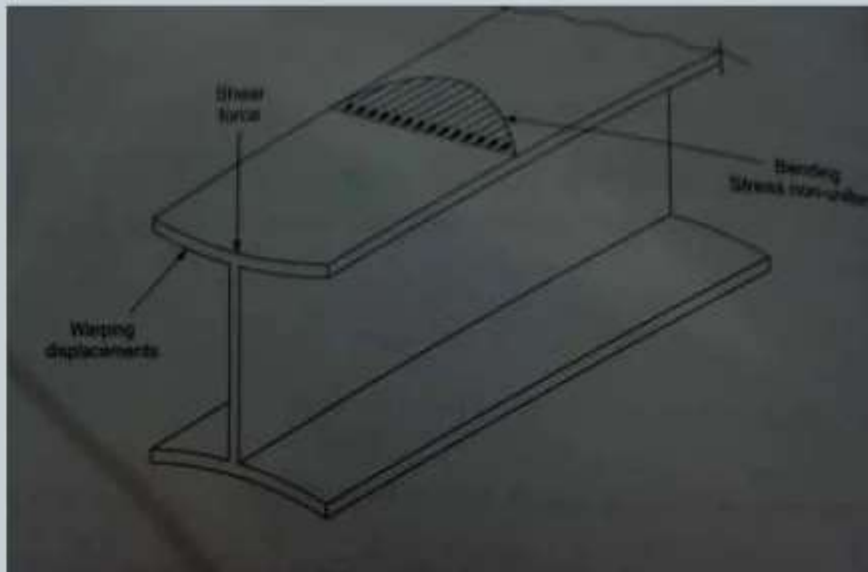


Fig. 6.4 Shear lag effect for an I-section.

8.2.1.5 *Shear lag effects*

The shear lag effects in flanges may be disregarded provided:

a) For outstand elements (supported along one edge), $b_o \leq L_o / 20$; and

b) For internal elements (supported along two edges), $b_i \leq L_o / 10$.

where

L_o = length between points of zero moment (inflection) in the span,

b_o = width of the flange with outstand, and

Design of laterally unsupported beams

- Under increasing tranverse loads, a beam shoul attain its full plastic moment capacity.
- This type of behaviour in laterally supported beams have been already covered
- Two imp assump made to achieve the ideal behaviour are
- i) the compression flange is restrained from moving laterally
- Any form of local buckling is prevented.
- A beam experiencing bending about major axis and its compression flange not restrained against buckling may not attain its material capacity. If the laterally unrestrained length of a beam is relatively long

- Resistance to lateral buckling need not be checked separately fo the following cases:-
- i) bending is about minor axis of the section
- ii) section is hollow(rect/tubular) or solid bars
- iii) in case of major axis bending, $\lambda LT <= 0.4$ where λLT is th non dimensional slenderness ratio for torsional buckling.
- The design bending strength of laterally unsupported beam i given in [is.800.2007- code of practice for gener steel.pdf] (pag 54)

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

where

β_b = 1.0 for plastic and compact sections.
= Z_e/Z_p for semi-compact sections.

Z_p, Z_e = plastic section modulus and elastic section modulus with respect to extreme compression fibre.

f_{bd} = design bending compressive stress, obtained as given below [*see* Tables 13(a) and 13(b)]

$f_{bd} = \chi_{LT} f_y / \gamma_{m0}$

χ_{LT} = bending stress reduction factor to account for lateral torisonal buckling, given by:

$$\chi_{LT} = \frac{1}{\left\{\phi_{LT} + \left[\phi_{LT}^2 - \lambda_{LT}^2\right]^{0.5}\right\}} \leq 1.0$$

α_{LT}, the imperfection parameter is given by:

α_{LT} = 0.21 for rolled steel section
α_{LT} = 0.49 for welded steel section

The non-dimensional slenderness ratio, λ_{LT}, is gi
by

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

where

M_{cr} = elastic critical moment calculated accordance with **8.2.2.1**, and

$f_{cr,b}$ = extreme fibre bending compressive st

corresponding to elastic lateral buc moment (*see* **8.2.2.1** and Table 14).

8.2.2.1 *Elastic lateral torsional buckling moment*

In case of simply supported, prismatic members with symmetric cross-section, the elastic lateral buckling moment, $M_{cr,}$ can be determined from:

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

$f_{cr,b}$ of non-slender rolled steel sections in the above equation may be approximately calculated from the values given in Table 14, which has been prepared using the following equation:

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

The following simplified equation may be used in the case of prismatic members made of standard rolled I-sections and welded doubly symmetric I-sections, for calculating the elastic lateral buckling moment, M_{cr} (*see* Table 14):

$$M_{cr} = \frac{\pi^2 EI_y h_f}{2 L_{LT}^2}\left[1 + \frac{1}{20}\left(\frac{L_{LT}/r_y}{h_f/t_f}\right)^2\right]^{0.5}$$

where

I_t = torsional constant $= \sum b_i t_i^3 / 3$ for open section;

I_w = warping constant;

I_y, r_y = moment of inertia and radius of gyration, respectively about the weaker axis;

L_{LT} = effective length for lateral torsional buckling (*see* **8.3**);

h_f = centre-to-centre distance between flanges; and

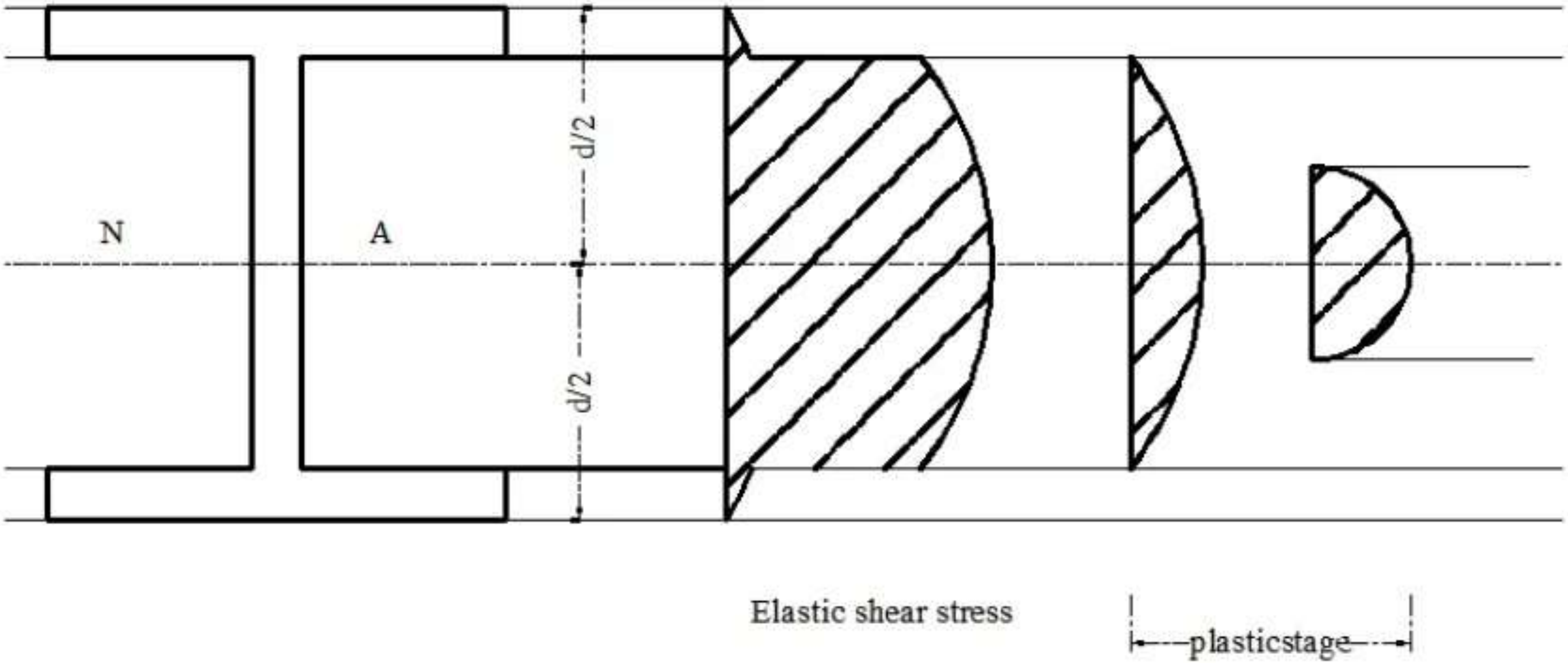
t_f = thickness of the flange.

Shear strength of beams

- Consider an I-beam subjected to max SF (at supp of SSB). The external shear 'V' varies along the longitudinal axis 'x' of the beam with BM as
- V= dM/dx, while beam is in the elastic stage, the internal stresses τ,which resist external shear V1' and can be as :

$$\tau = \frac{VQ}{Iz.t}$$

- V=SF under consideration



Pattern of shear stress distribution

8.4 Shear

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

$$V \leq V_d$$

where

V_d = design strength

= V_n / γ_{m0}

where

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

The nominal shear strength of a cross-section, V_n, may be governed by plastic shear resistance (*see* **8.4.1**) or strength of the web as governed by shear buckling (*see* **8.4.2**).

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

$$V_n = V_p$$

The nominal shear strength of a cross-section, V_n, may be governed by plastic shear resistance (*see* **8.4.1**) or strength of the web as governed by shear buckling (*see* **8.4.2**).

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

$$V_n = V_p$$

where

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

A_v = shear area, and

f_{yw} = yield strength of the web.

8.4.1.1 The shear area may be calculated as given below:

I and channel sections:

Major Axis Bending:

Hot-Rolled — $h\, t_w$

Welded — $d\, t_w$

Minor Axis Bending:

8.4.2 *Resistance to Shear Buckling*

8.4.2.1 Resistance to shear buckling shall be verified as specified, when

$$d/t_w > 67\varepsilon \text{ for a web without stiffeners, and}$$

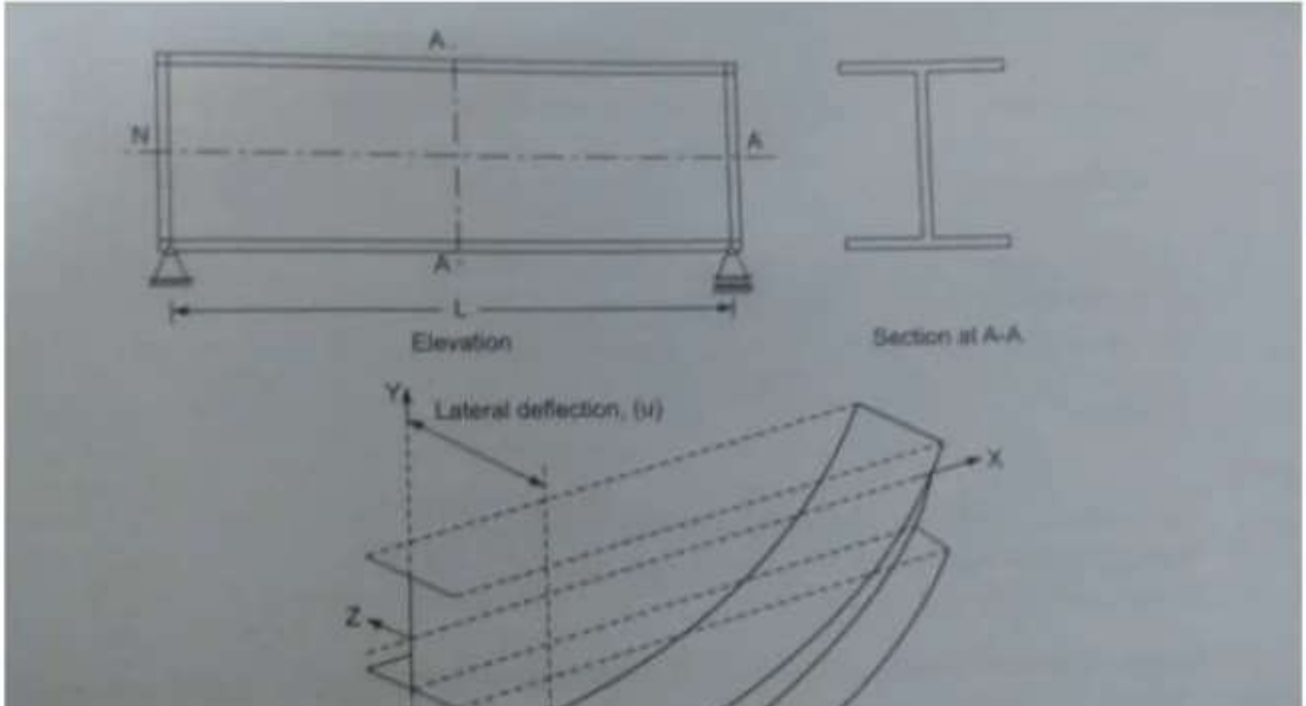
$$> 67\varepsilon\sqrt{\frac{K_v}{5.35}} \text{ for a web with stiffeners}$$

where

K_v = shear buckling coefficient (*see* **8.4.2.2**), and

LATERAL TORSIONAL LOADING

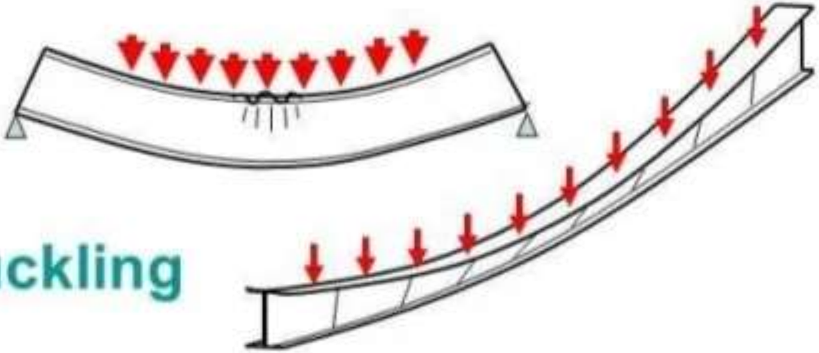
When a beam fails by lateral torsional buckling, it buckles about it weak axis, even though it is loaded in the strong plane. The beam bends about its strong axis up to the critical load at which it buckles laterally, refer figure.



What can go wrong ?

STEEL BEAMS:

- Bending failure

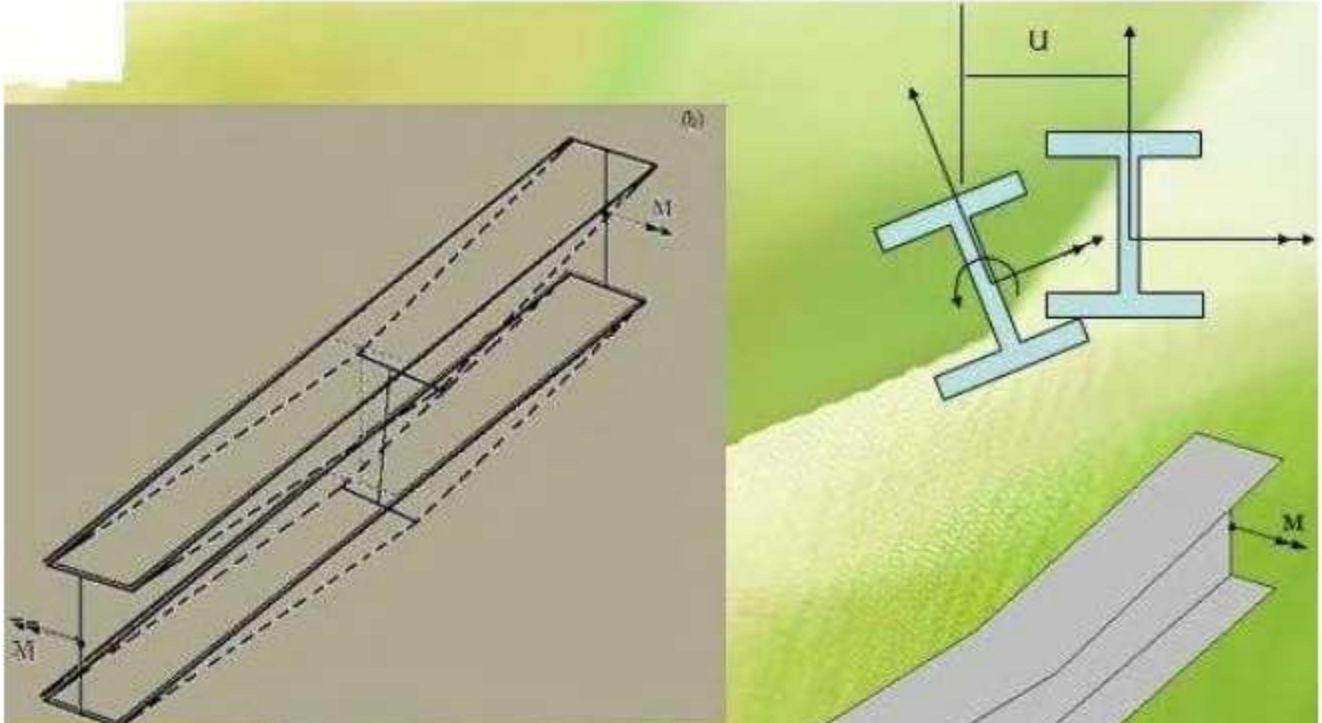


- Lateral torsional buckling

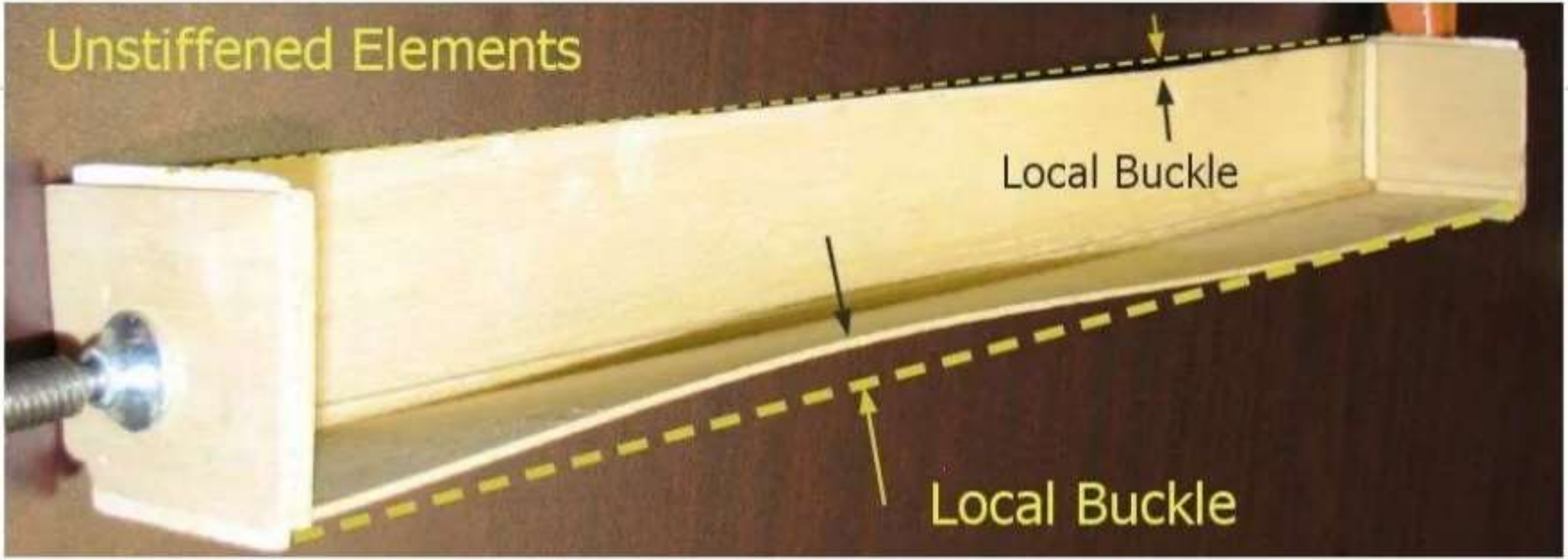
- Shear failure

- Bearing failure (web crippling)

Lateral buckling



Local buckling



The lateral torsional bucking of an I-section is considered with the following assumptions

- The beam is initially undistorted.
- Its behaviour is elastic.
- It is loaded by equal and opposite end moment in the plane of the web.
- The load acts in the plane of web only.

Effective length lateral torsional buckling

- For SS beams and girders for span length L, where no lateral restraint to the compressive flanges are provided, but where each end of beam is restrained agains torsion, the effective length L_{LT} of the lateral buckling can be taken as given in the table as per IS 800 (page 58)

Table 15 Effective Length for Simply Supported Beams, L_{LT}

(*Clause* 8.3.1)

Sl No.	Conditions of Restraint at Supports		Loading Condition	
	Torsional Restraint	Warping Restraint	Normal	Destabilizing
(1)	(2)	(3)	(4)	(5)
i)	Fully restrained	Both flanges fully restrained	0.70 L	0.85 L
ii)	Fully restrained	Compression flange fully restrained	0.75 L	0.90 L
iii)	Fully restrained	Both flanges fully restrained	0.80 L	0.95 L
iv)	Fully restrained	Compression flange partially restrained	0.85 L	1.00 L
v)	Fully restrained	Warping not restrained in both flanges	1.00 L	1.20 L
vi)	Partially restrained by bottom flange support connection	Warping not restrained in both flanges	$1.0\,L + 2\,D$	$1.2\,L + 2\,D$
vii)	Partially restrained by bottom flange bearing support	Warping not restrained in both flanges	$1.2\,L + 2\,D$	$1.4\,L + 2\,D$

NOTES

1 Torsional restraint prevents rotation about the longitudinal axis.

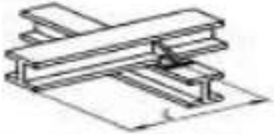
2 Warping restraint prevents rotation of the flange in its plane.

3 D is the overall depth of the beam.

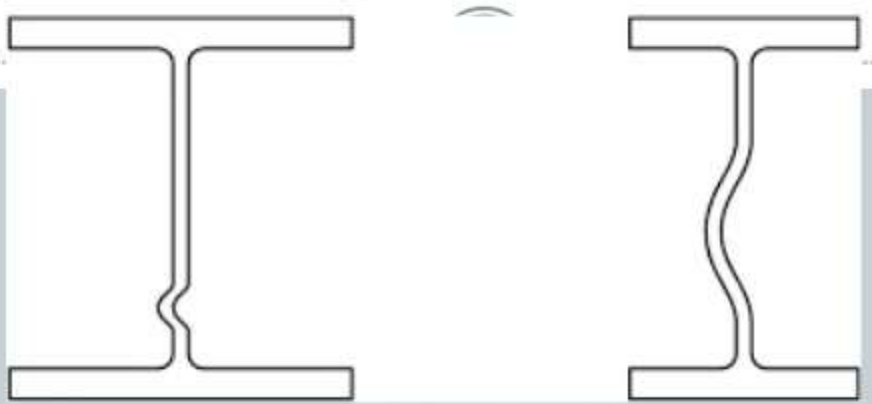
In SS beams with intermediate lateral restraints against torsional buckling the effective length for lateral torsional buckling should be equal to 1.2 times the

For cantilever beams for projecting length L, the effective length LLt to be used shal
given in table cl 8.3.3 page 61 of is.800.2007- code of practice for gener steel.pdf

Table 16 Effective Length, L_{LT} for Cantilever of Length, L
(*Clause* 8.3.3)

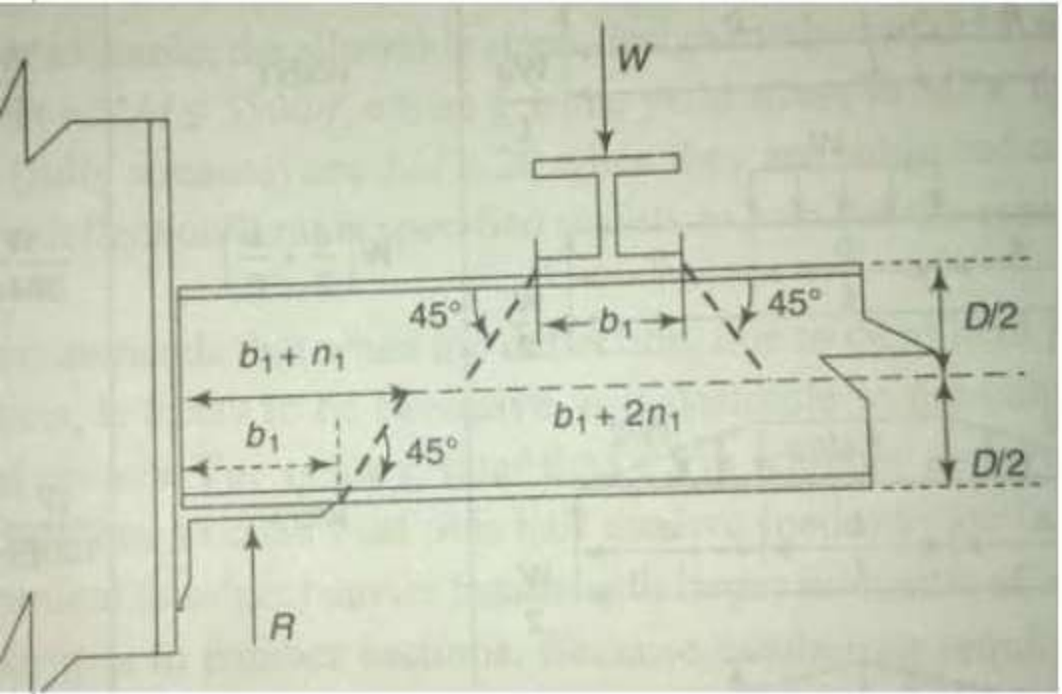
Restraint Condition		Loading Condition	
At Support	At Top	Normal	Destabilizing
(1)	(2)	(3)	(4)
a) Continuous, with lateral restraint to top flange 	i) Free ii) Lateral restraint to top flange iii) Torsional restraint iv) Lateral and torsional restraint	3.0L 2.7L 2.4L 2.1L	7.5L 7.5L 4.5L 3.6L
b) Continuous, with partial torsional restraint 	i) Free ii) Lateral restraint to top flange iii) Torsional restraint iv) Lateral and torsional restraint	2.0L 1.8L 1.6L 1.4L	5.0L 5.0L 3.0L 2.4L
c) Continuous, with lateral and torsional restraint 	i) Free ii) Lateral restraint to top flange iii) Torsional restraint iv) Lateral and torsional restraint	1.0L 0.9L 0.8L 0.7L	2.5L 2.5L 1.5L 1.2L
d) Restrained laterally, torsionally and against rotation on plan 	i) Free ii) Lateral restraint to top flange	0.8L 0.7L 0.6L	1.4L 1.4L 0.6L

Web buckling and web crippling



Web crippling Web buckling

- A heavy load or reaction conc. on a short length produces a region of high compressive stresses in the vertical elements of the web either under the load of at the support. The web under a load or above buckling as shown in figure (right) and a web reaction point, may cause web failures such as web crippling or crushing as shown in figure(left) above.
- Web buckling occurs when the intensity of vertical compressive stress near the



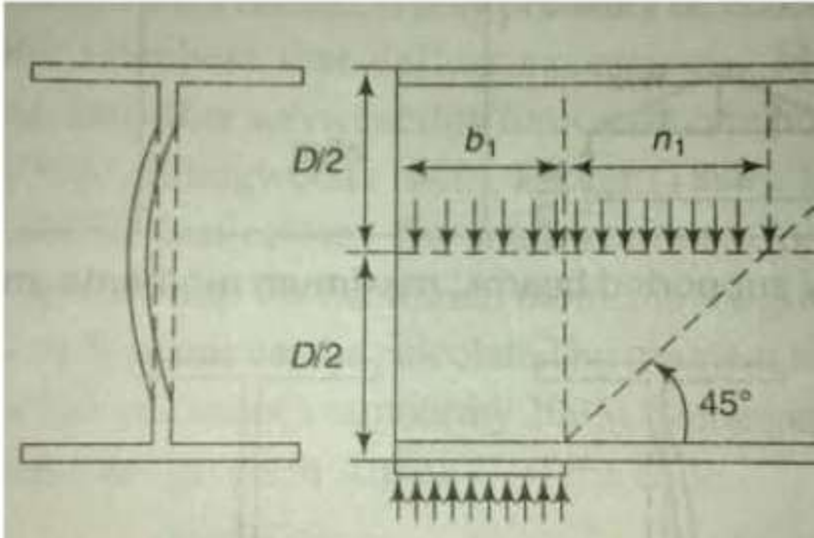


Fig. 10.34 Effective width for web buckl

Dispersion of concentrated load for evaluation of web buckling

But for built up beams having greater rations of depth to thickness of web, failure

The web buckling strength at support will be

$$Fwb = (b1+n1).tw.\ fc$$

Where (b1+n1) is the length of the stiff portion of the bearing plus the additional length given by the dispersion at 45 to the level of NA, fc is the allowable compressive stress corresponding to the assumed web strut according to buckling curve 'c' , tw is the thickness of web plate.

Effective length = $(d1\sqrt{2})/2$
Mini radius of gyration = $t/\sqrt{12}$

Slenderness ratio = le/ry

$$= [(d1\sqrt{2})/2]/[t/\sqrt{12}] \text{ or } d1\ \sqrt{6}\ /\ t$$

$$\lambda = 2.45\ d1/t$$

Hence the slenderness ratio of the idealised compression strut is taken as $\lambda = 2.45$ d1/t

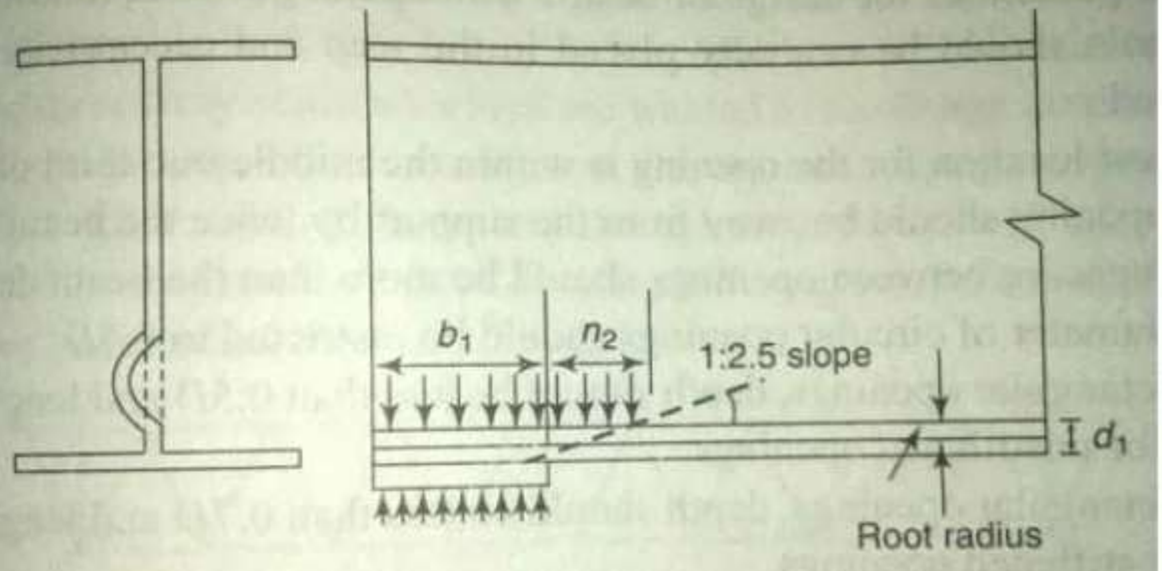


Fig. 10.35 Effective width of web bearing

Similarly in case of web crippling the crippling strength can also be calculated assuming an empirical dispersion length = b1 + n2

The dispersion length is **b1= b +n2**


Shear Studs
Top Flange
Web stiffener
Flange to web fillet weld
Web
Bottom Flange

Deflection

Table (Page 31)IS 800 gives recommended limits for deflection for certain structural members

- reasons for limiting deflections:

i. Excessive deflection creates problem for floors or roof drainage called ponding which leads to corrosion of steel reinforcement inside floor

ii. In case of beams framed together are of different sections must deflect in the same way

Designer can reduce deflections by

i. Increasing depth of the beam section

Purlins

- Beams used on trusses to support sloping roof systems bet adjacent trusses
- Channels, angle sections, cold formed C or Z sections
- Placed in inclined position over the main rafters
- Purlins may be designed as simple,continuous,cantilever beams
- Simple beams yields largest moments and deflections(BM max = $WL^2/8$)
- Continuous (moment = $WL^2/10$)

Design procedure of Channel/I-section purlins

- Design of purlin is by trial and error procedure and various steps involved in design are as follows

1. The span of purlin is taken as c/c distance between adjacent trusses.
2. Gravity loads P, due to sheeting and LL, and the horizontal load H due to wind are computed.
3. The components of these loads in directions perp. and parallel to sheeting are determined. These loads are multiplied by p.s.f

4. The BM max Muu and Mvv are calculated by

Muu = Pl/10 and Mvv = Hl/10

Purlins are subjected to biaxial bending and requir trial and error method for design.

The required value of section modulus may b determined from the following expression given b Gaylord

$$Zpz = Mz.\frac{\gamma mo}{fy} + 2.5\frac{d}{p}\left[My.\frac{\gamma mo}{fy}\right]$$

5. The deign capacities of the section Mdz and Mdy are given by:

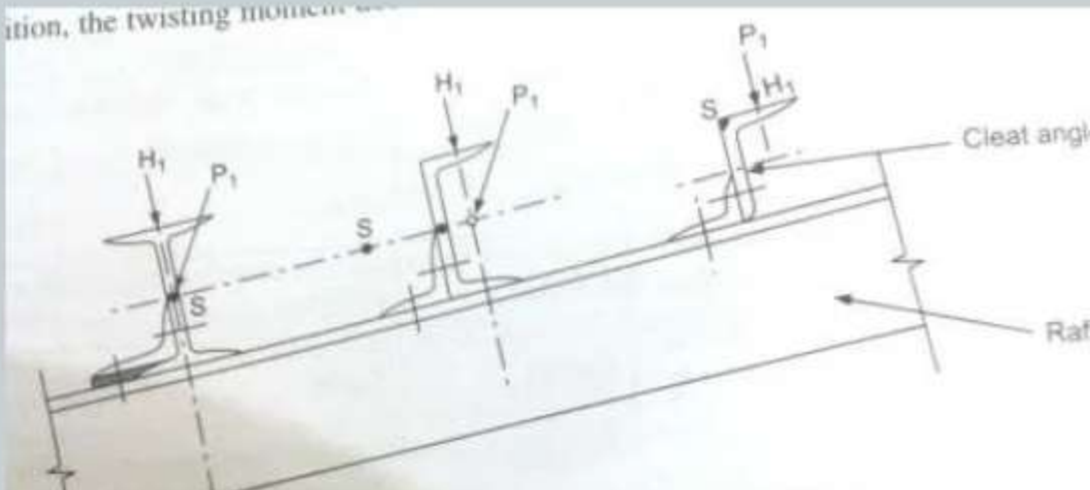
$$Mdz = Zpz.\frac{fy}{\gamma mo} \leq 1.2Zez.\frac{fy}{\gamma mo}$$

And,

$$Mdy = Zpy.\frac{fy}{\gamma mo} \leq \gamma f.Zey.\frac{fy}{\gamma mo}$$

6. The load capacity of the section is checked using the following interaction equation:-

$$\frac{Mz}{Mdz}+\frac{My}{Mdy}\leq 1$$



7. Check whether the shear capacity of the section for both the z and y axes, (for purlins shear capacity will always be high and may not govern the design)

$$Vdz = \frac{fy}{\sqrt{3}.\gamma mo} Avz$$

$$Vdy = \frac{fy}{\sqrt{3}.\gamma mo} Avy$$

Where Avz = h.tw and Avy = 2.bf. tf

The deflection of the purlin calculated should be less than l/180

Design of angle purlins

- An angle is unsymmetrical @ both axes.
- May be used when slope of the roof is < than 30
- Vertical loads and horizontal loads acting on the purlins are determined and the max BM is calculated as PL/10 and HL/10, where P and H are the vertical and horizontal loads respectively.
- The section modulus is calculated by

$$Z = \frac{M}{1.33 \times 066 \times fy}$$

- The trial section is the selected assuming the depth of angle section

ROOF AND TRUSSES

ROOF

ROOF TYPES

CHARACTERISTICS

ROOF TRUSSES/RAFTERS

ROOF ELEMENTS

EXAMPLES

TRUSSES

ROOFS:

- Roof is defined as the upper most part of a building , which is constructed in the form of frame work to give protection to the buildings against rain, heat, snow, wind etc…

ROOF

ROOF TYPES

CHARACTERISTICS

ROOF TRUSSES/RAFTERS

ROOF ELEMENTS

EXAMPLES

TRUSSES

ROOF SHAPES

- Roofs protect buildings and occupants from wind, rain, cold, sun, heat, dust, etc.

- Roofs come in many shapes.

ROOF

ROOF TYPES

CHARACTERISTICS

ROOF TRUSSES/RAFTERS

ROOF ELEMENTS

EXAMPLES

TRUSSES

ROOF TYPES

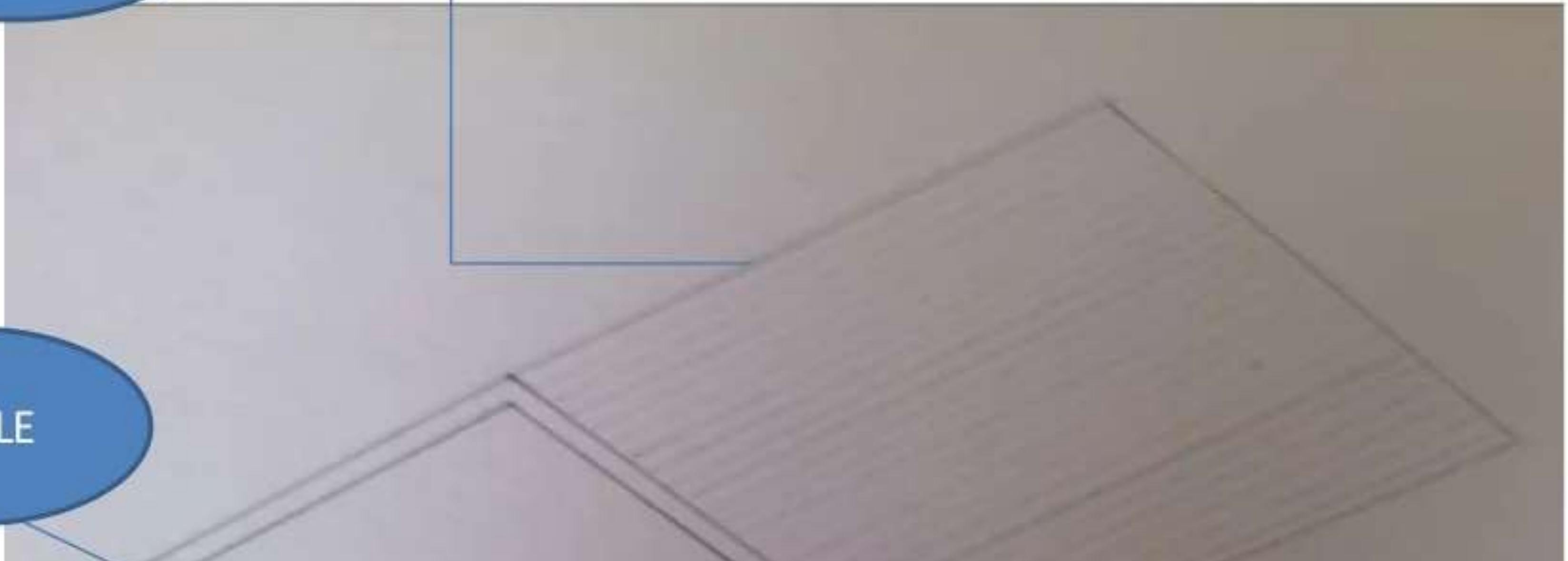
Shed

Gable

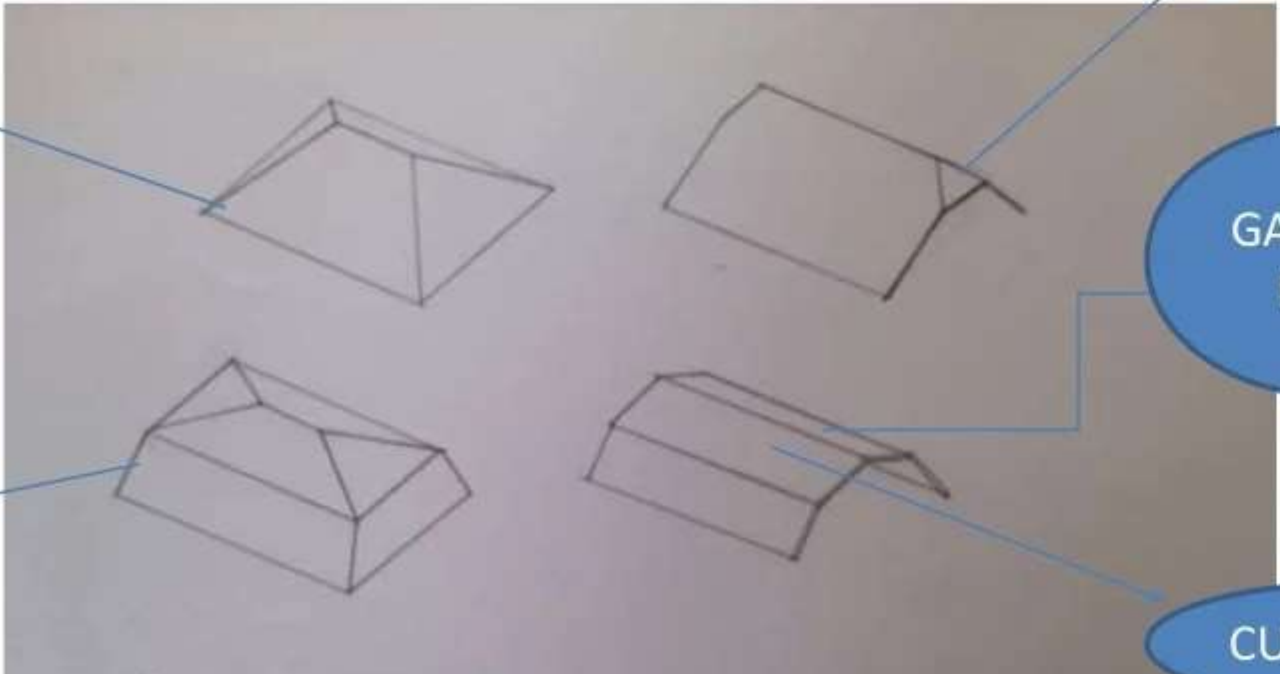
Hip

Gable with
Dormer

ROOF



ROOF TYPES



COMMON ROOFING MATERIAL

Clay/Cement Tiles

Asphalt Shingles

ROOF

ROOF TYPES

CHARACTERISTI
CS

ROOF
TRUSSES/RAFTE
RS

ROOF
ELEMENTS

EXAMPLES

TRUSSES

ROOFS AS SUSTAINABLE SPAC

- Natural light creates a pleasant space for occupants and requires less energy for lighting.

ROOF

ROOF TYPES

CHARACTERISTICS

ROOF TRUSSES/RAFTERS

ROOF ELEMENTS

EXAMPLES

TRUSSES

CHARACTERISTICS

- The characteristics of a roof are dependent upon the purpose of the building that it covers,
- the available roofing materials and the local traditions of construction and wider concepts of architectural design.
- In most countries a roof protects primarily against rain.
- A verandah may be roofed with material that protects against sunlight but admits the other elements.

ROOF

ROOF TYPES

CHARACTERISTICS

ROOF TRUSSES/RAFTERS

ROOF ELEMENTS

EXAMPLES

TRUSSES

ROOF TRUSSES

- A roof truss is a simple assembly o members forming a rigid framework of triangular shapes.

ROOF TRUSSS

ROOF

ROOF TYPES

CHARACTERISTICS

ROOF TRUSSES/RAFTERS

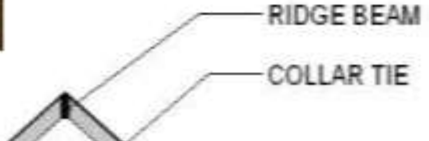
ROOF ELEMENTS

EXAMPLES

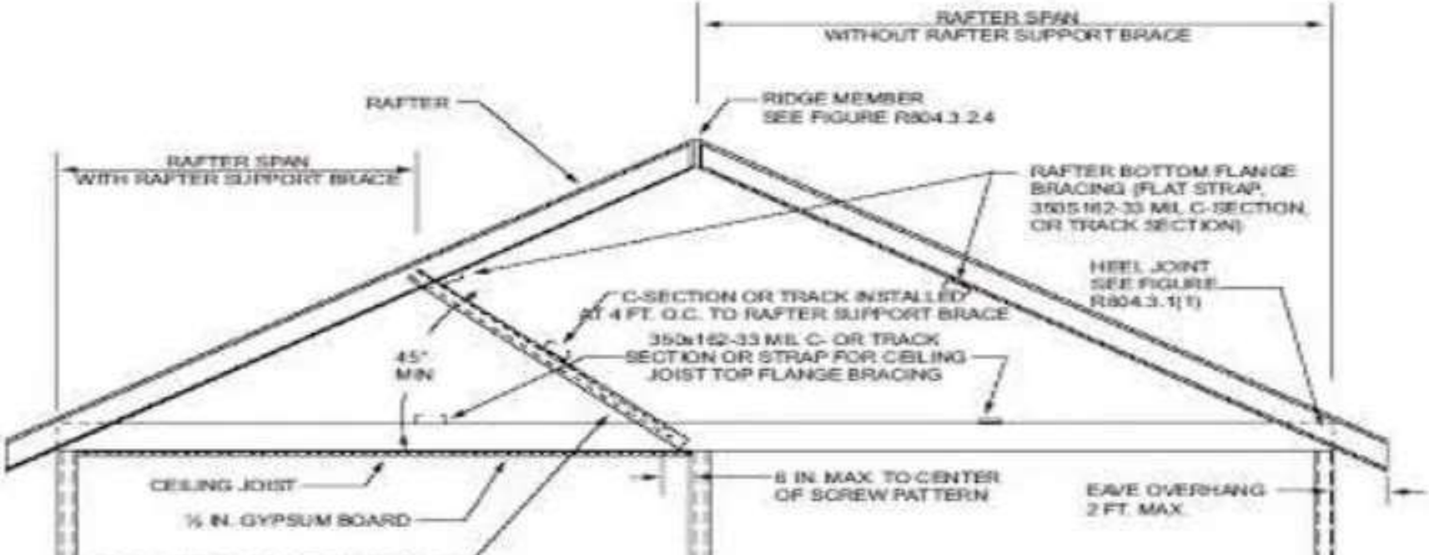
TRUSSES

RAFTE

•Rafters are an alternative to trusses fo
roof support.
•Rafters are roof beams that slope from
the ridge beam to the top of the wall.

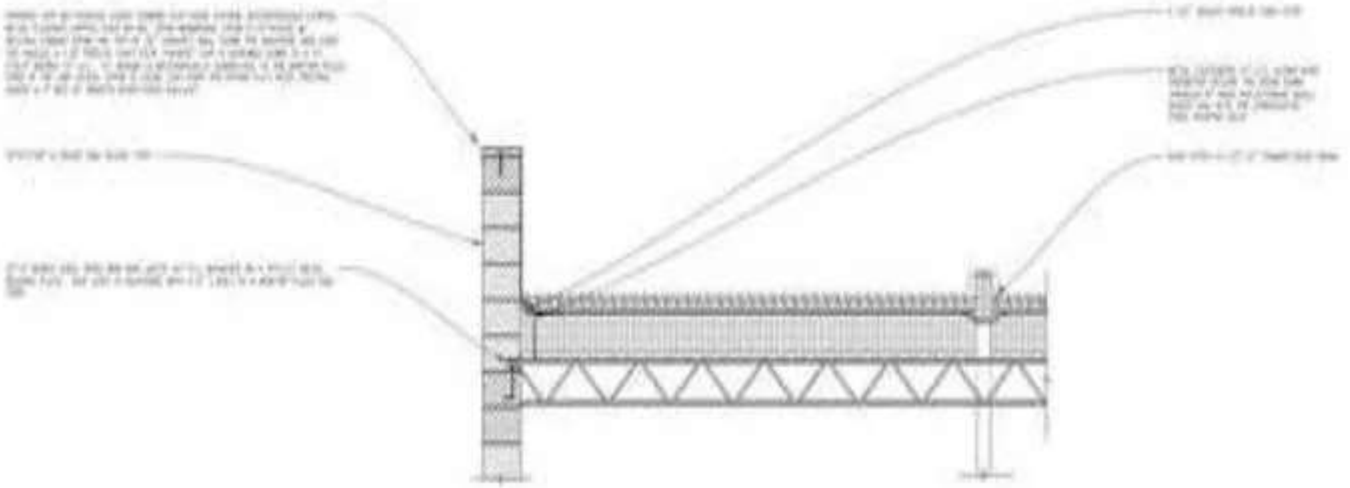


HIP ROOF

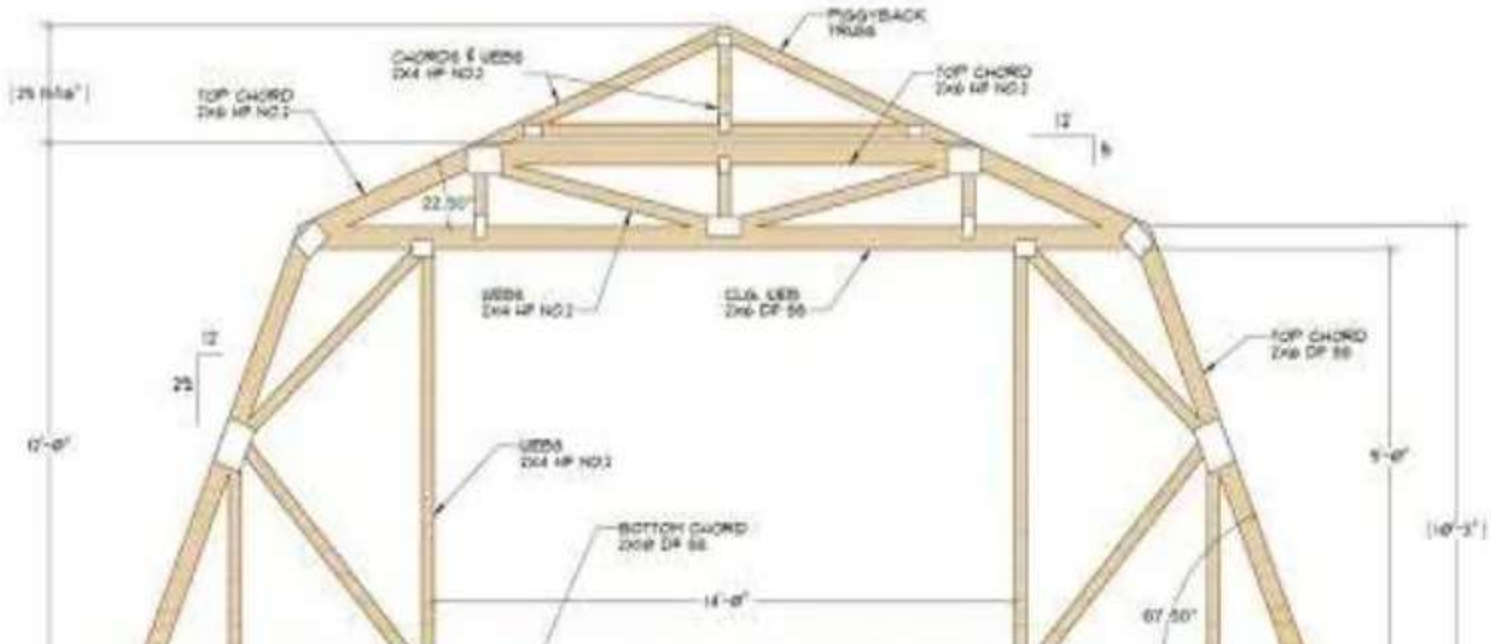


RAFTER SPAN
WITHOUT RAFTER SUPPORT BRACE
RAFTER
RIDGE MEMBER
SEE FIGURE R804.3.2.4
RAFTER SPAN
WITH RAFTER SUPPORT BRACE
RAFTER BOTTOM FLANGE
BRACING (FLAT STRAP,
350S162-33 MIL C-SECTION,
OR TRACK SECTION)
HEEL JOINT
SEE FIGURE
R804.3.1(1)
C-SECTION OR TRACK INSTALLED
AT 4 FT. O.C. TO RAFTER SUPPORT BRACE
350x162-33 MIL C- OR TRACK
SECTION OR STRAP FOR CEILING
JOIST TOP FLANGE BRACING
45°
MIN
CEILING JOIST
½ IN. GYPSUM BOARD
6 IN. MAX. TO CENTER
OF SCREW PATTERN
EAVE OVERHANG
2 FT. MAX

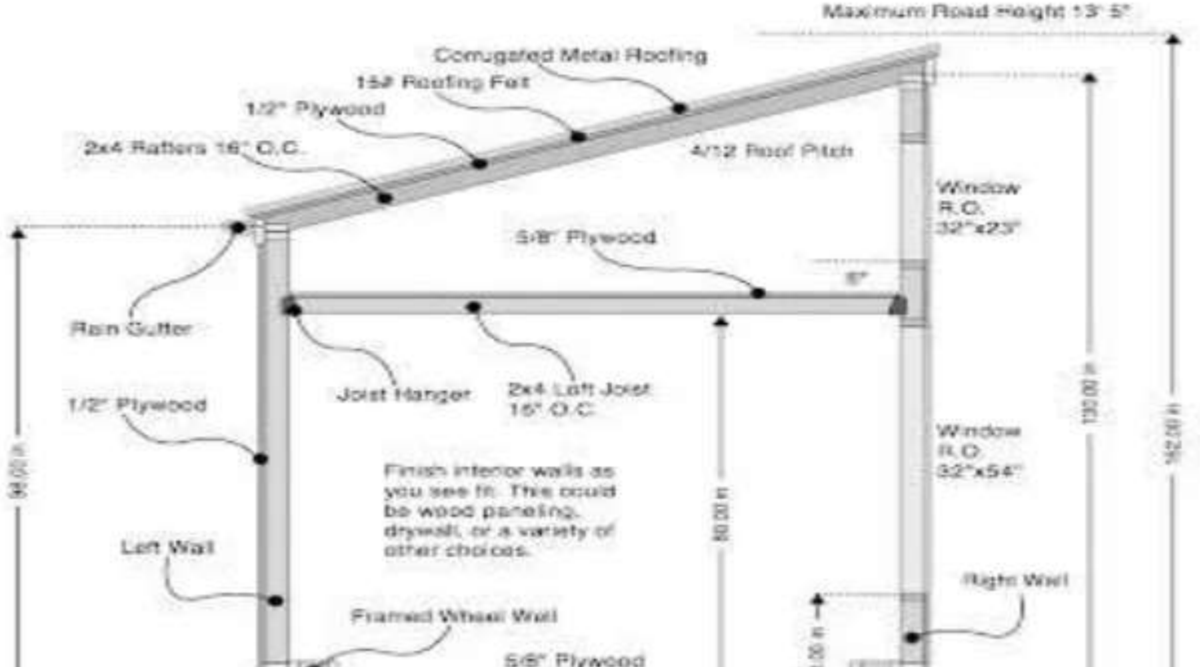
FLAT ROOF



GAMBREL ROOF



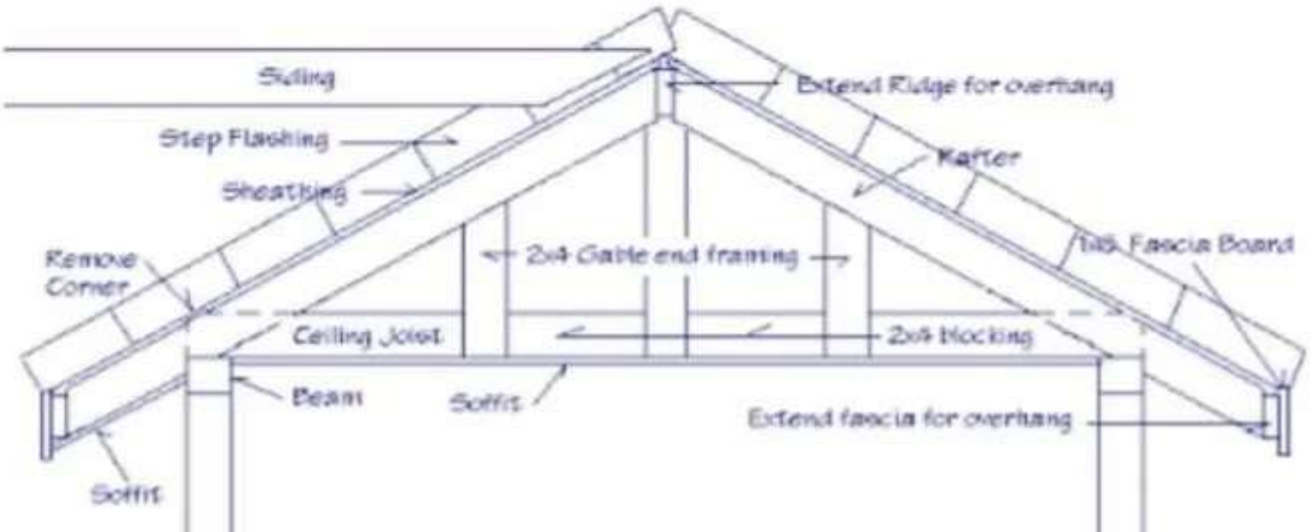
SHED ROOF



Maximum Road Height 13' 5"
Corrugated Metal Roofing
15# Roofing Felt
1/2" Plywood
2x4 Rafters 16" O.C.
4/12 Roof Pitch
Window
R.O.
32"x23"
5/8" Plywood
6"
Rain Gutter
Joist Hanger
2x4 Loft Joist
16" O.C.
1/2" Plywood
Window
R.O.
32"x54"
Finish interior walls as
you see fit. This could
be wood paneling,
drywall, or a variety of
other choices.
Left Wall
Right Wall
Framed Wheel Well
5/8" Plywood

GABLE ROOF

Front View



- ROOF
- ROOF TYPES
- CHARACTERISTICS
- ROOF TRUSSES/RAFTERS
- ROOF ELEMENTS
- EXAMPLES
- TRUSSES

SUFFICIENT ROOF QUALIT

- A roof must be weather resistant to rain, snow, wind and sun.
- The durability of a roof should be equal to or in excess of those materials used in the remainder of the building.
- A roof should have good thermal insulation properties.
- A roof should require a minimum of maintenance
- A roof should be constructed in such a way as to

ROOF ELEM

When designing a roof the following points should be considered in relation to its final appearance.

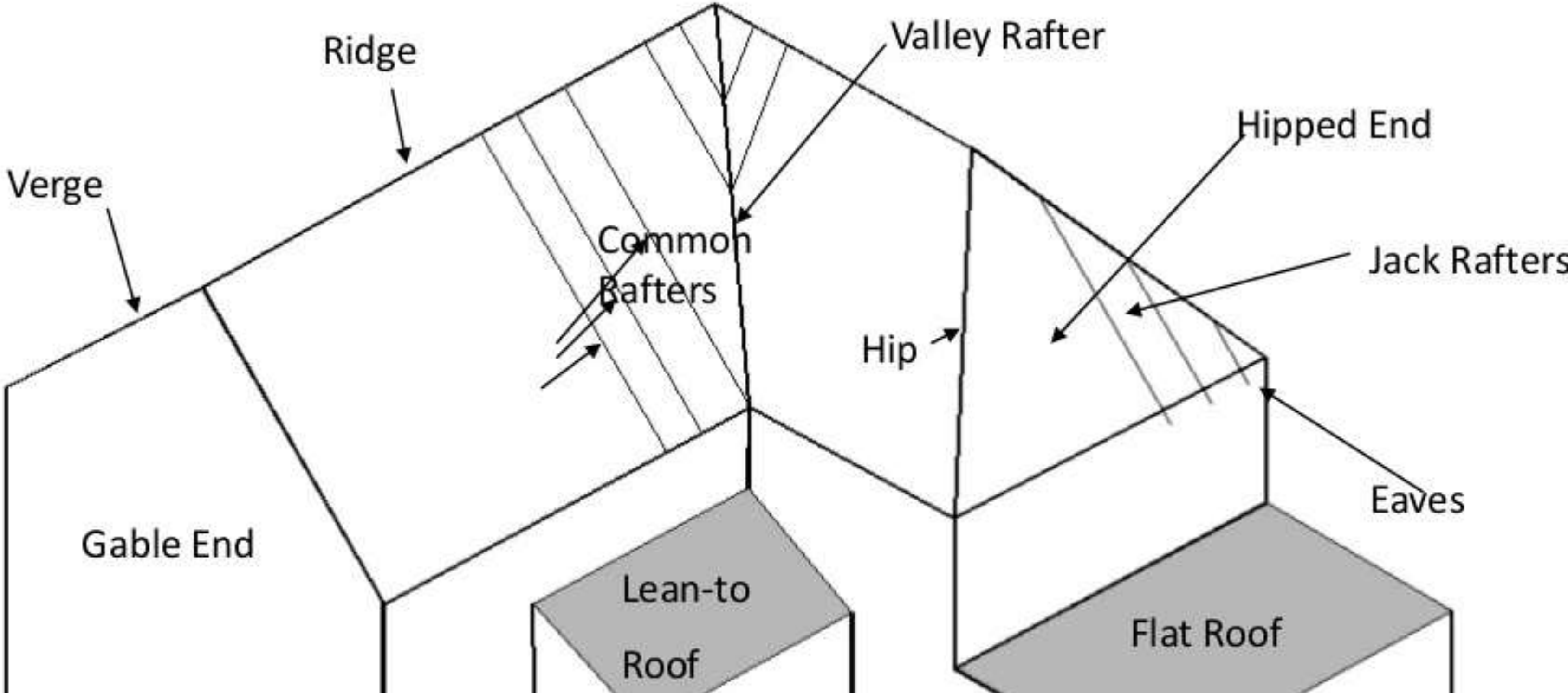
Pitch:

The steeper the pitch the greater the roof area visible. will result in a larger roof space, you can also use small cladding units such as plain tiles and slates.

Coverings:

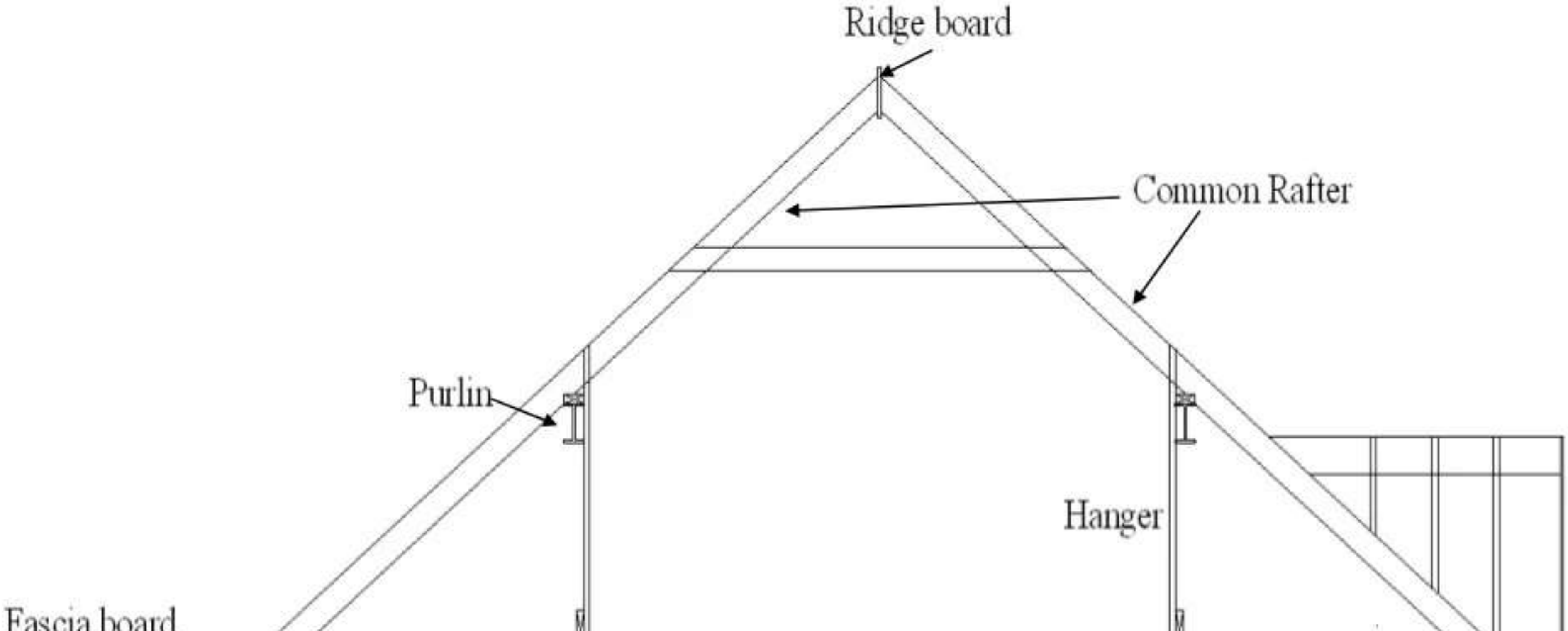
Materials used for roof coverings should harmonise wi the local surroundings.

Roofs



Ridge
Valley Rafter
Hipped End
Verge
Common
Rafters
Jack Rafters
Hip
Eaves
Gable End
Lean-to
Roof
Flat Roof

Traditional cut Roof



ROOF

ROOF TYPES

CHARACTERISTICS

ROOF TRUSSES/RAFTERS

ROOF ELEMENTS

EXAMPLES

TRUSSES

TERMS

Wall plate:

Usually 100 x 50 mm softwood timbers are fixed to the top of load bearing walls to distribute loads and provide fixings for roof timbers.

Ceiling joist:

These are timbers which provide a support for fixing ceiling finishes and act as a collar to prevent rafters spreading.

Common rafters:

These are inclined timbers fixed between wall plate and ridge which transmit live and dead loads to wall plate.

Ridge:

The ridge is a horizontal board set on edge to which

ROOF
ROOF TYPES
CHARACTERISTICS
ROOF TRUSSES/RAFTERS
ROOF ELEMENTS
EXAMPLES
TRUSSES

Hip Rafter:

A hip rafter is a rafter running from the wall plate to the ridge which forms the external angle of the sloping side of a roof.

Purlin:

This is a horizontal roof member supporting the rafters and usually at right angles to these. This enables small section timbers to be used for the rafters.

Hangers :

These are timbers hanging from the purlins to the

ROOF

ROOF TYPES

CHARACTERISTICS

ROOF TRUSSES/RAFTERS

ROOF ELEMENTS

EXAMPLES

TRUSSES

Soffit:

A horizontal board fixed to the underside of rafter outside the building.

Bargeboard:

Verge or gable board.

Eaves:

The lower part of the roof, which usually includes the end of

the rafter, ceiling joist, soffit, fascia and gutter.

Dormer:

ROOF

ROOF TYPES

CHARACTERISTI
CS

ROOF
TRUSSES/RAFTE
RS

ROOF
ELEMENTS

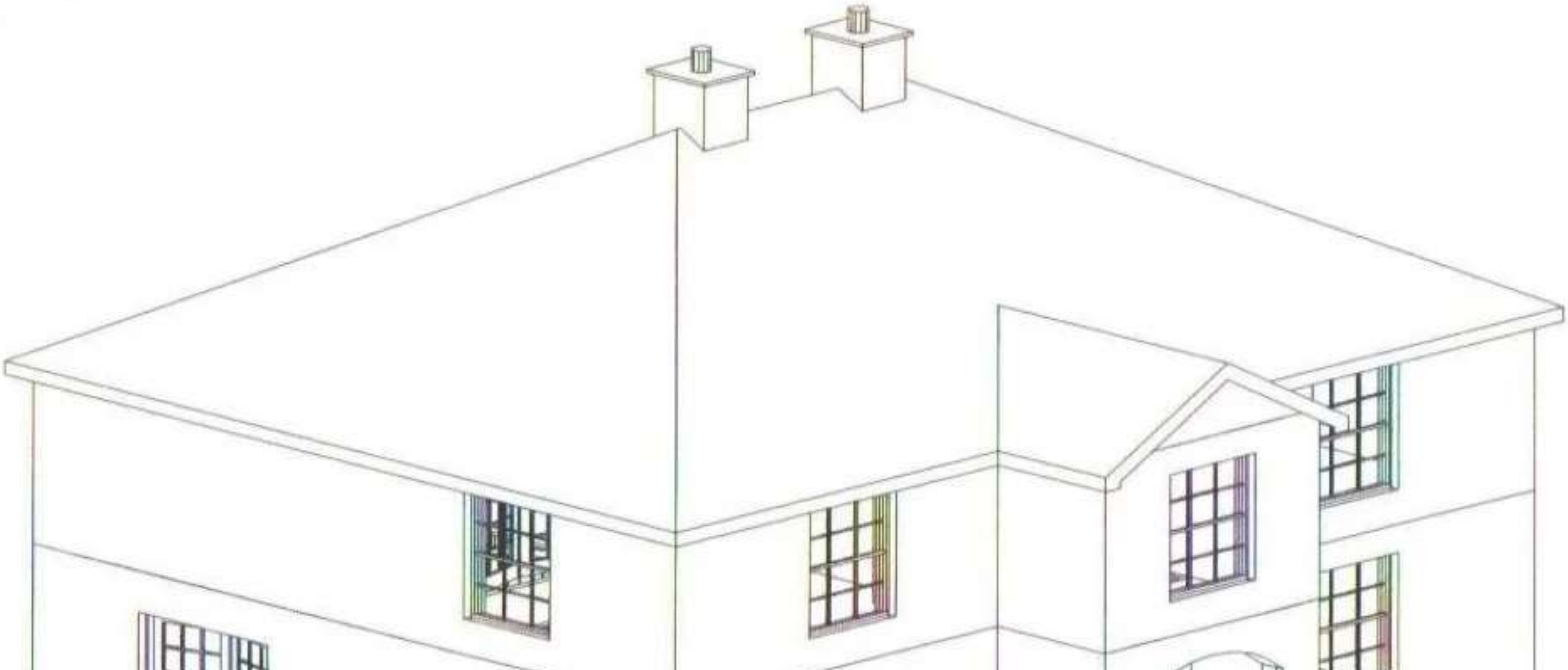
EXAMPLES

TRUSSES

PITCHED ROOF

- If the roof space is or may be intended to be used in the future, the structure should be designed accordingly.

HIPPED ROOF



Pitched roof with dormer windows

•When a steep pitched roof is used in

ROOF
ROOF TYPES
CHARACTERISTI CS
ROOF TRUSSES/RAFTE RS
ROOF ELEMENTS
EXAMPLES
TRUSSES

TRADITIONAL CUT ROOF STRUCTURE

- A traditional cut roof was the first development to create pitched roofs as we know them today.

This type of roof structure is still widely used for individual dwellings or for roofs of a complicated shape.

Steel purlin

Pad stone built into the

ROOF
ROOF TYPES
CHARACTERISTICS
ROOF TRUSSES/RAFTERS
ROOF ELEMENTS
EXAMPLES

TRUSSED RAFTERS

- A high percentage of roofs designed now are constructed using prefabricated trussed rafters. They have been developed since the mid 1960's.
- The principal disadvantage is that the roof space is occupied by a large number of timber sections.

- The following are the main reasons for developing truss rafters:

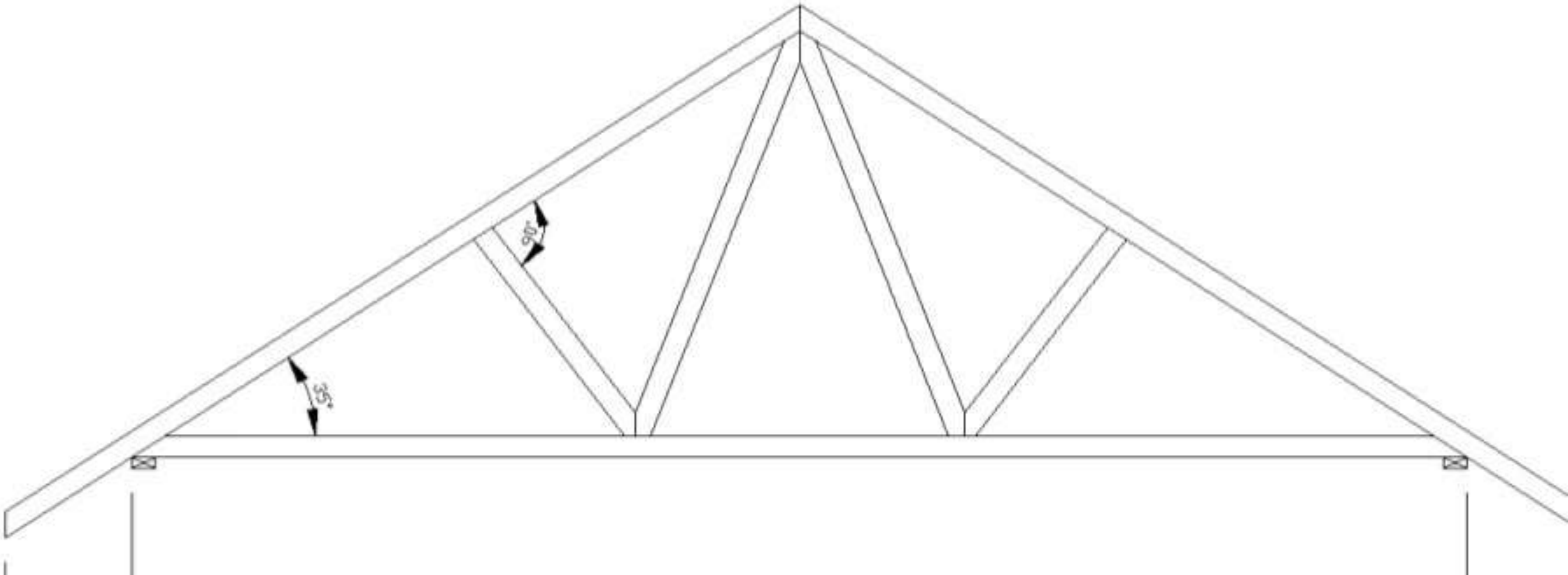
1.Smaller and light sections of timber can be used.

2. Speed of erection.

3. Semi-skilled labour can be used.

4. Eliminates the need for purlins and ridge boards.

Standard trussed rafter



ROOF

ROOF TYPES

CHARACTERISTI
CS

ROOF
TRUSSES/RAFTE
RS

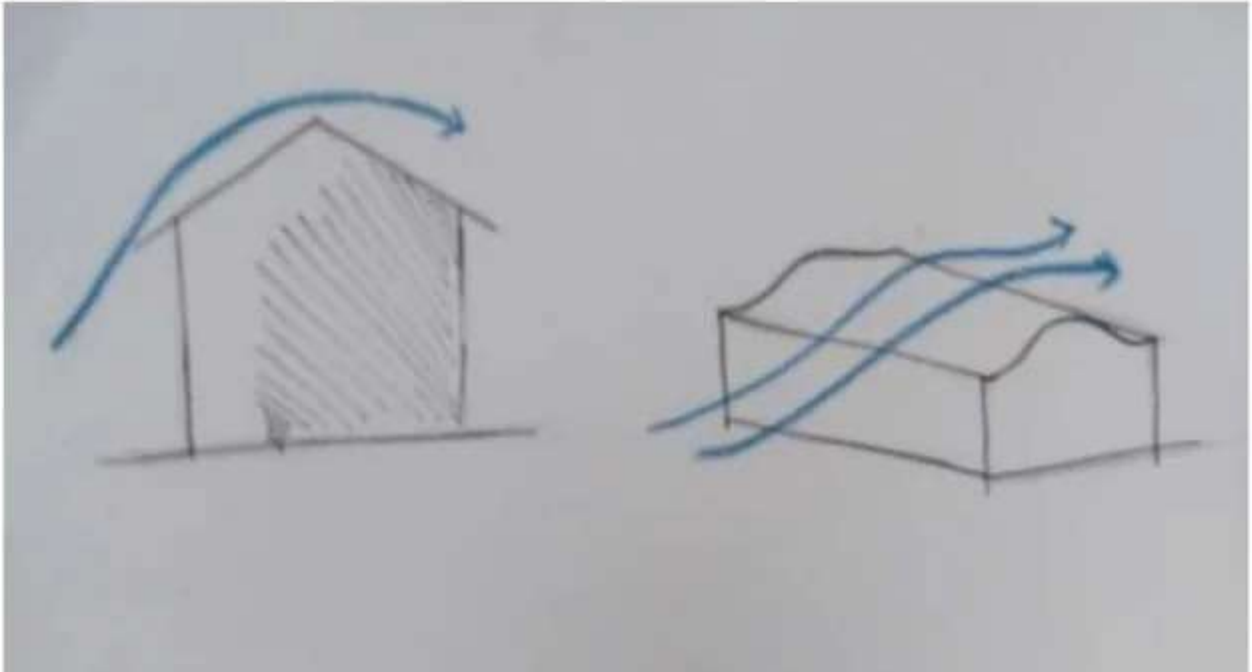
ROOF
ELEMENTS

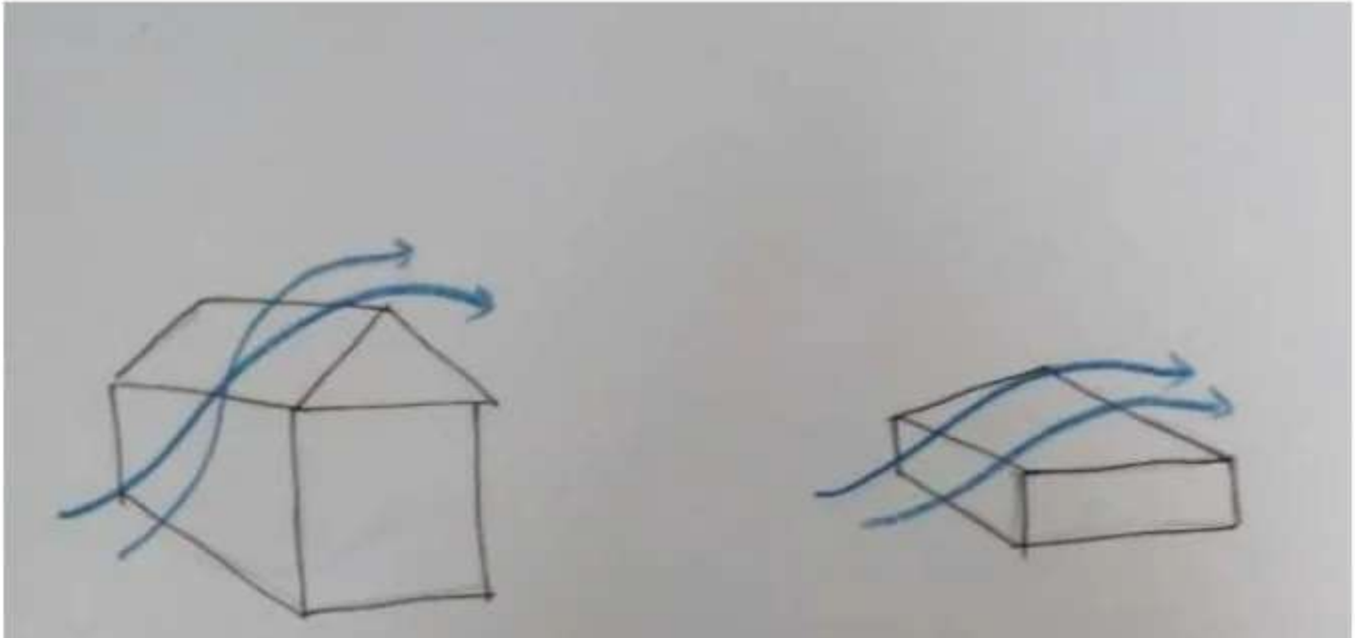
EXAMPLES

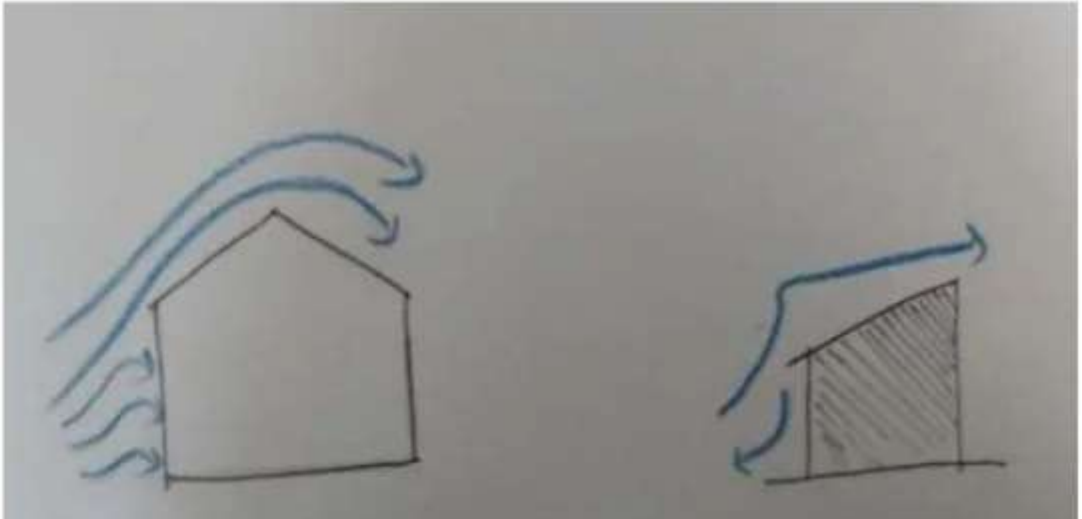
TRUSSES

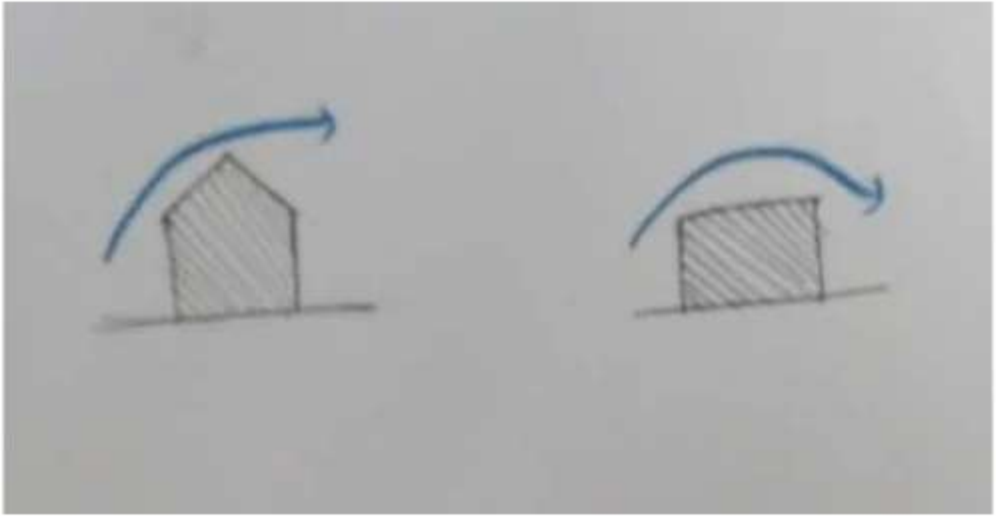
ROOF COVERINGS

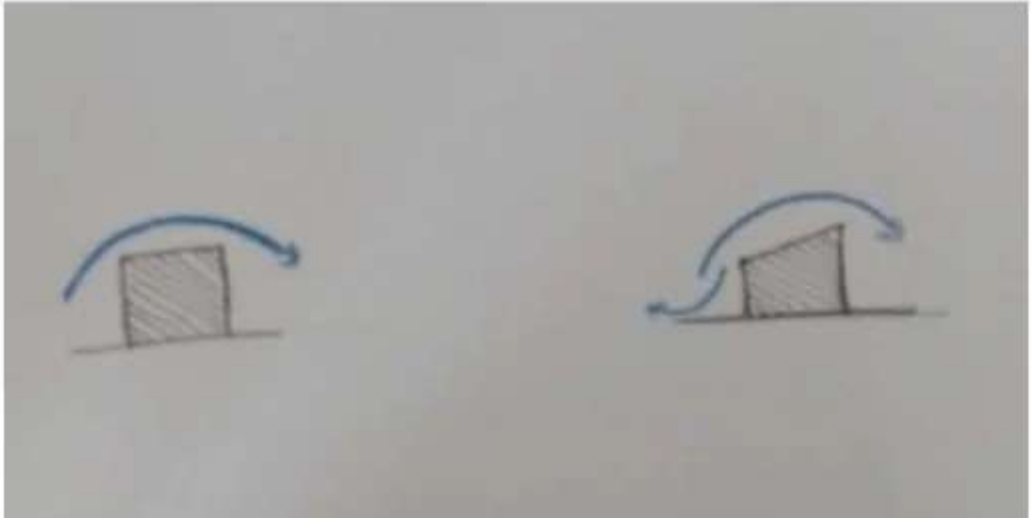
- A wide variety of different types of roof coverings are available including:
- natural slates
- man made slates
- concrete tiles
- Profiled metal sheet











ROOF

ROOF TYPES

CHARACTERISTI
CS

ROOF
TRUSSES/RAFTE
RS

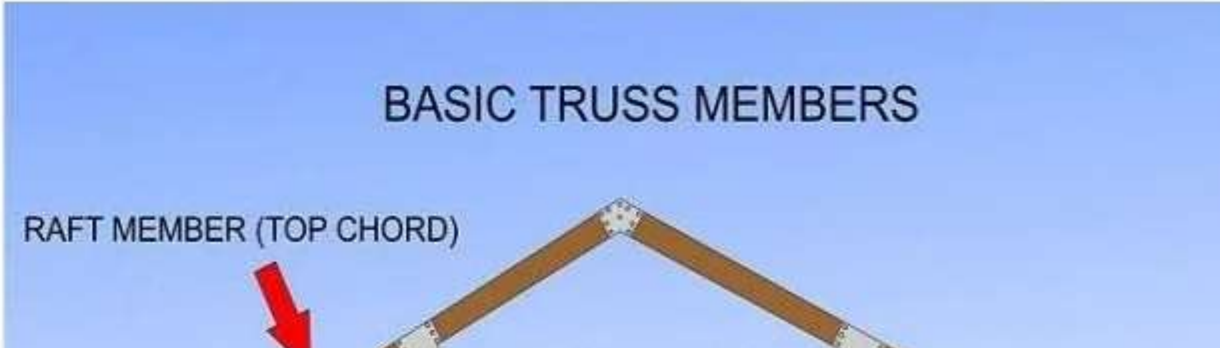
ROOF
ELEMENTS

EXAMPLES

TRUSSES

TRUSSES

- A "two-force member" is a structural component where force is applied to only two points.
- Truss- The framework , usually of triangles and designed to support the roof covering or ceilin over rooms , is known as a roof truss.



ROOF
ROOF TYPES
CHARACTERISTICS
ROOF TRUSSES/RAFTERS
ROOF ELEMENTS
EXAMPLES
TRUSSES

CHARACTERISTICS

- A truss consists of typically straight members connected at joints, traditionally termed panel points.
- A triangle is the simplest geometric figure that will not change shape when the lengths of the sides are fixed.
- In comparison, both the angles and the lengths of a four-sided figure must be fixed for it to retain its shape.

ROOF

ROOF TYPES

CHARACTERISTICS

ROOF TRUSSES/RAFTERS

ROOF ELEMENTS

EXAMPLES

TRUSSES

TYPES OF TRUSS

- Some of the usual form of roof trusses are as follows:

i. PLANER TRUSS
ii. SPACEFRAME
iii. PRATT TRUSS
iv. BOWSTRING TRUSS
v. KING POST
vi. QUEEN-POST TRUSS
vii. LENTICULAR TRUSS
viii. COMBINATION OF KING-POST AND QUEEN-POST TRUSS
ix. TOWN'S LATTICE TRUSS
x. MANSARD ROOF TRUSS

ROOF

ROOF TYPES

CHARACTERISTICS

ROOF TRUSSES/RAFTERS

ROOF ELEMENTS

EXAMPLES

TRUSSES

PLANER TRUSS :

- The simplest form of a truss is one single triangle.
- This type of truss is seen in a framed roof consisting of rafters and a ceiling joists.
- A planar truss lies in a single plane. Planar trusses are typically used in parallel to form roofs and bridges.

ROOF

ROOF TYPES

CHARACTERISTI
CS

ROOF
TRUSSES/RAFTE
RS

ROOF
ELEMENTS

EXAMPLES

TRUSSES

SPACEFRAME

- A space frame trus
a three-dimension
framework of
members pinned a
their ends.
- Space frames can
used to span large
areas with few
interior supports.
the truss, a space
frame is strong

ROOF

ROOF TYPES

CHARACTERISTI
CS

ROOF
TRUSSES/RAFTE
RS

ROOF
ELEMENTS

EXAMPLES

TRUSSES

PRATT TRUSS

- The **Pratt truss** was patented in 1844 by two Boston railway engineers,Caleb Pratt and his son Thomas Willis Pratt.
- The design uses vertical members for compression and horizontal members to respond to tension.

ROOF

ROOF TYPES

CHARACTERISTICS

ROOF TRUSSES/RAFTERS

ROOF ELEMENTS

EXAMPLES

TRUSSES

BOWSTRING TRUSS

- Named for their shape, bowstring trusses were first used for arched truss bridges often confused with tied-arch bridges.
- A structural truss consisting of a curved top chord meeting a bottom chord at each end.

ROOF

ROOF TYPES

CHARACTERISTI
CS

ROOF
TRUSSES/RAFTE
RS

ROOF
ELEMENTS

EXAMPLES

TRUSSES

KING POST

- One of the simplest truss styles to implement, the **king post** consists of two angled supports leaning into a common vertical support.

ROOF

ROOF TYPES

CHARACTERISTI CS

ROOF TRUSSES/RAFTE RS

ROOF ELEMENTS

EXAMPLES

TRUSSES

QUEEN-POST TRUSS

- A queen-post truss differs From a king-post truss in having two vertical posts, rather than one.

ROOF

ROOF TYPES

CHARACTERISTI CS

ROOF TRUSSES/RAFTE RS

ROOF ELEMENTS

EXAMPLES

TRUSSES

LENTICULAR TRUSS

- Lenticular trusses, patented in 1878 by William Douglas, h the top and bottom chords of the truss arched, forming a having fish belly shape.

TOWN'S LATTICE TRUSS

- American architect Ithiel Town designed **Town's Lattice Truss** as an alternative to heavy-timber bridges.
- Uses easy-to-handle planks arranged diagonally with short spaces in between them.

ROOF

ROOF TYPES

CHARACTERISTICS

ROOF TRUSSES/RAFTERS

ROOF ELEMENTS

EXAMPLES

TRUSSES

VIERENDEEL TRUSS

- The **Vierendeel truss** is a structure where the members are not triangulated but form rectangular openings, and is a frame with fixed joints that are capable of transferring and resisting bending moments.

ROOF

ROOF TYPES

CHARACTERISTICS

ROOF TRUSSES/RAFTERS

ROOF ELEMENTS

EXAMPLES

TRUSSES

COMBINATION OF KING-POST AN QUEEN-POST TRUSS

- Queen-post trusses are suitable for spans upto 12m.
- For greater spans, the queen-post truss can strengthen by one member, called ***princess-post*** to each side.
- the resulting combination of king-post and queen-post trusses, which are suitable upto 18m span.

ROOF
ROOF TYPES
CHARACTERISTICS
ROOF TRUSSES/RAFTERS
ROOF ELEMENTS
EXAMPLES
TRUSSES

MANSARD ROOF TRUSS

This roof truss is a combination of king-post and queen-post trusses.

It is a two storey truss, with upper portion consisting of king-p truss and lower portion consisting of queen-post truss.

The use of this truss results in economy in space, since a room may be provided between the two queen-posts.

However, it has become obsolete because of odd shape.

ROOF

ROOF TYPES

CHARACTERISTICS

ROOF TRUSSES/RAFTERS

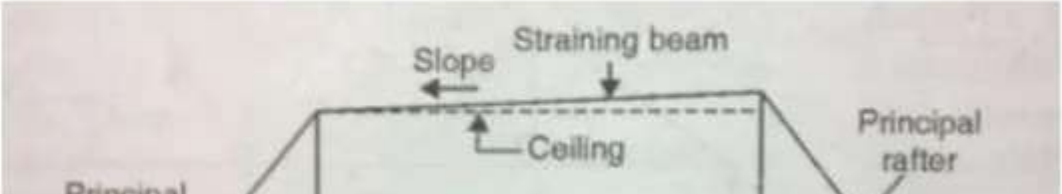
ROOF ELEMENTS

EXAMPLES

TRUSSES

TRUNCATED TRUS

- A truncated truss is similar to Mansard truss, except that its top is formed flat, with a gentle slope to one side.
- This type of truss is used when it is required to provide a room in the roof,



ROOF

ROOF TYPES

CHARACTERISTICS

ROOF TRUSSES/RAFTERS

ROOF ELEMENTS

EXAMPLES

TRUSSES

BEL-FAST ROOF TRUSS

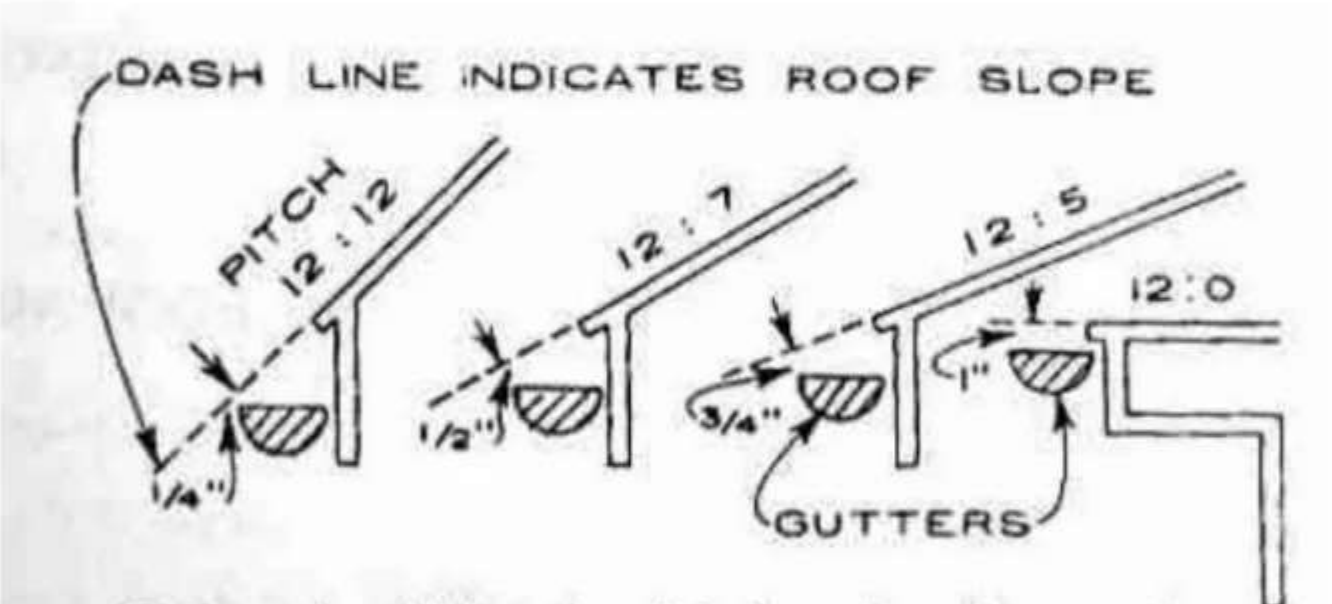
- This truss, in the form of a bow, consists of thin sections of timber, with its top chord curved.
- If the roof covering is light, this roof truss can be used upto 30m span. The roof truss is also known as ***latticed roof truss***.

- ROOF
- ROOF TYPES
- CHARACTERISTI CS
- ROOF TRUSSES/RAFTE RS
- ROOF ELEMENTS
- EXAMPLES
- TRUSSES

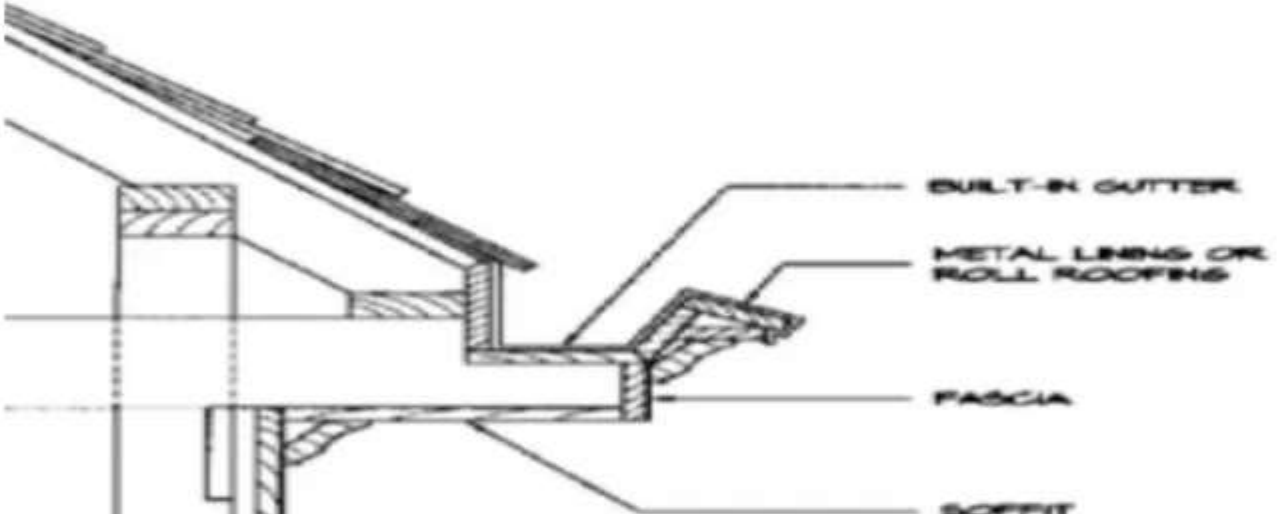
Composite roof truss

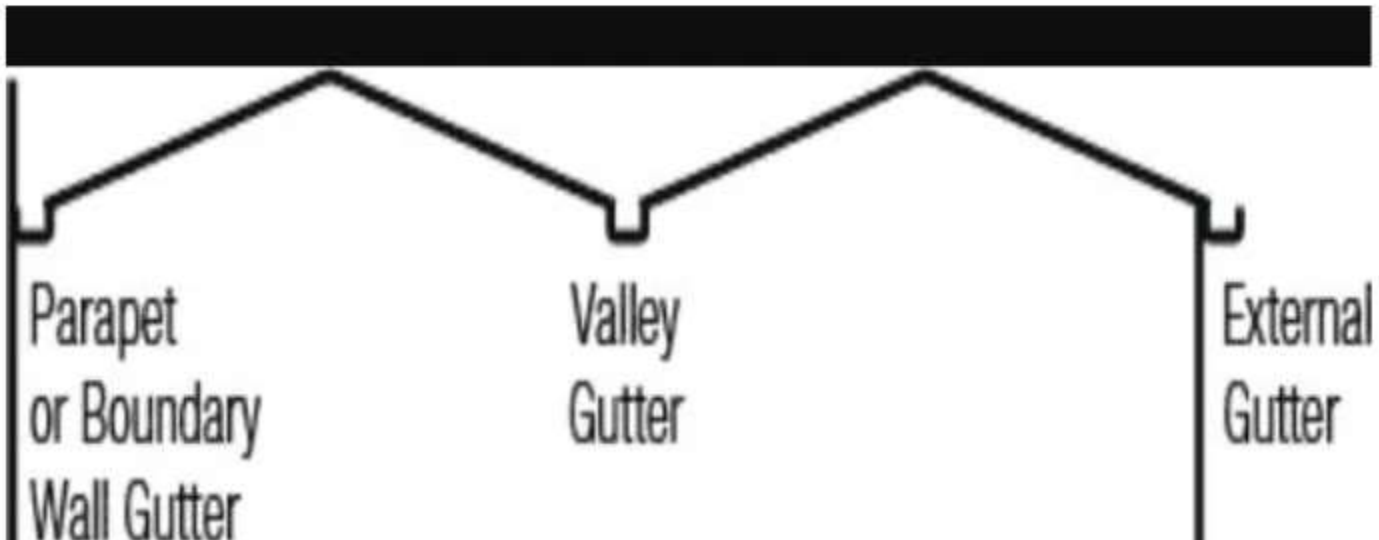
- Roof trusses made of two materials, such as timber and steel, are known as ***composite*** roof trusses.
- In a composite truss, the tension members are made of steel, while compression members are made of timber.
- Special fittings are required at the junction of steel and timber members.

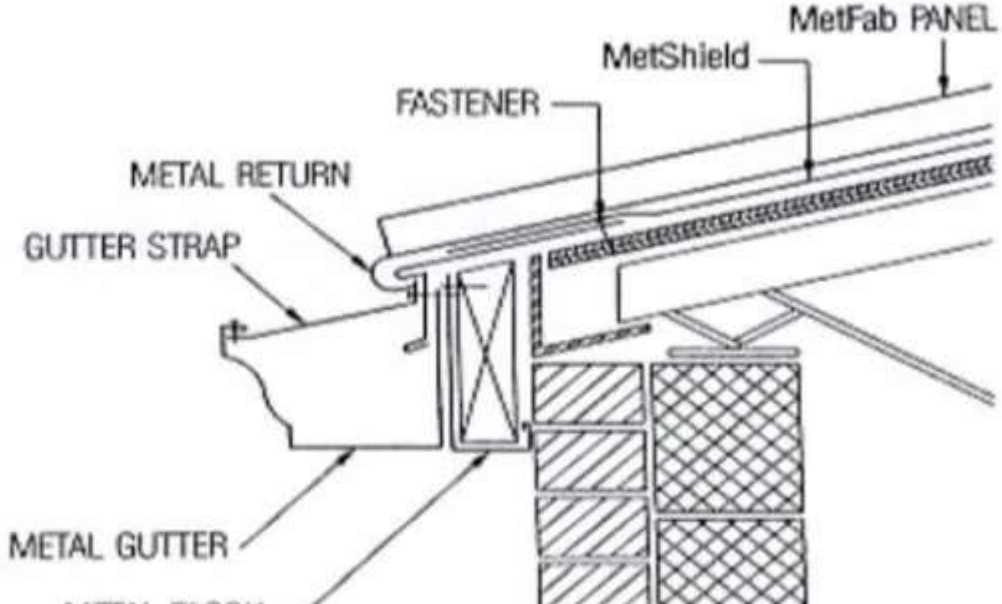
TYPES OF GUTTERS USED IN ROOFS



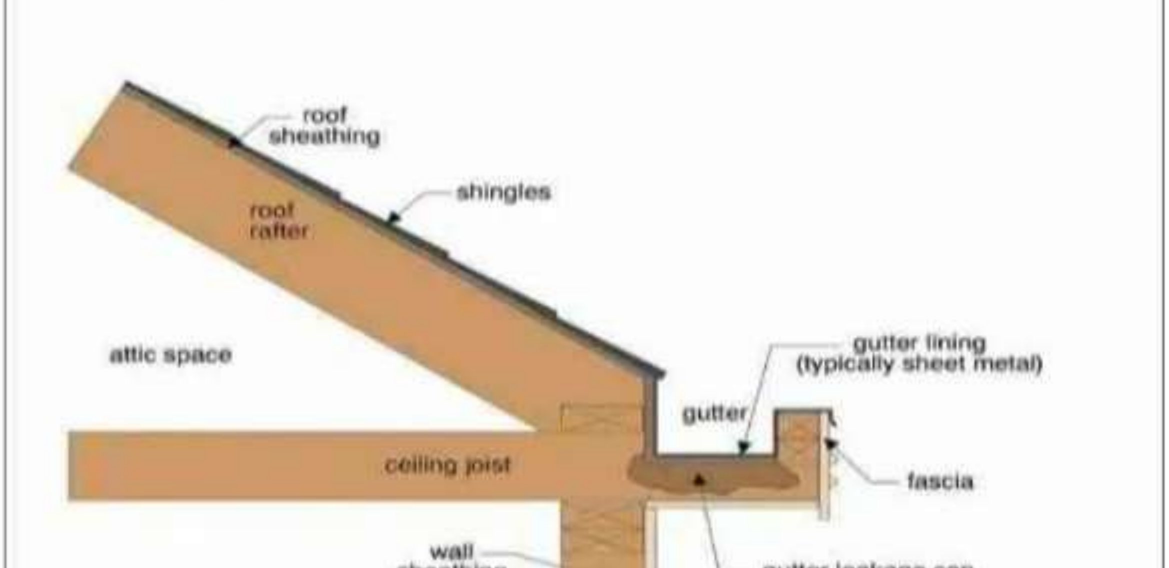
BOX CORNICE WITH BUILT-IN GUTTER
LINED WITH METAL OR SMOOTH ROLL
ROOFING




Parapet
or Boundary
Wall Gutter
Valley
Gutter
External
Gutter


MetFab PANEL
MetShield
FASTENER
METAL RETURN
GUTTER STRAP
METAL GUTTER

Integral gutter



ROOF

ROOF TYPES

CHARACTERISTICS

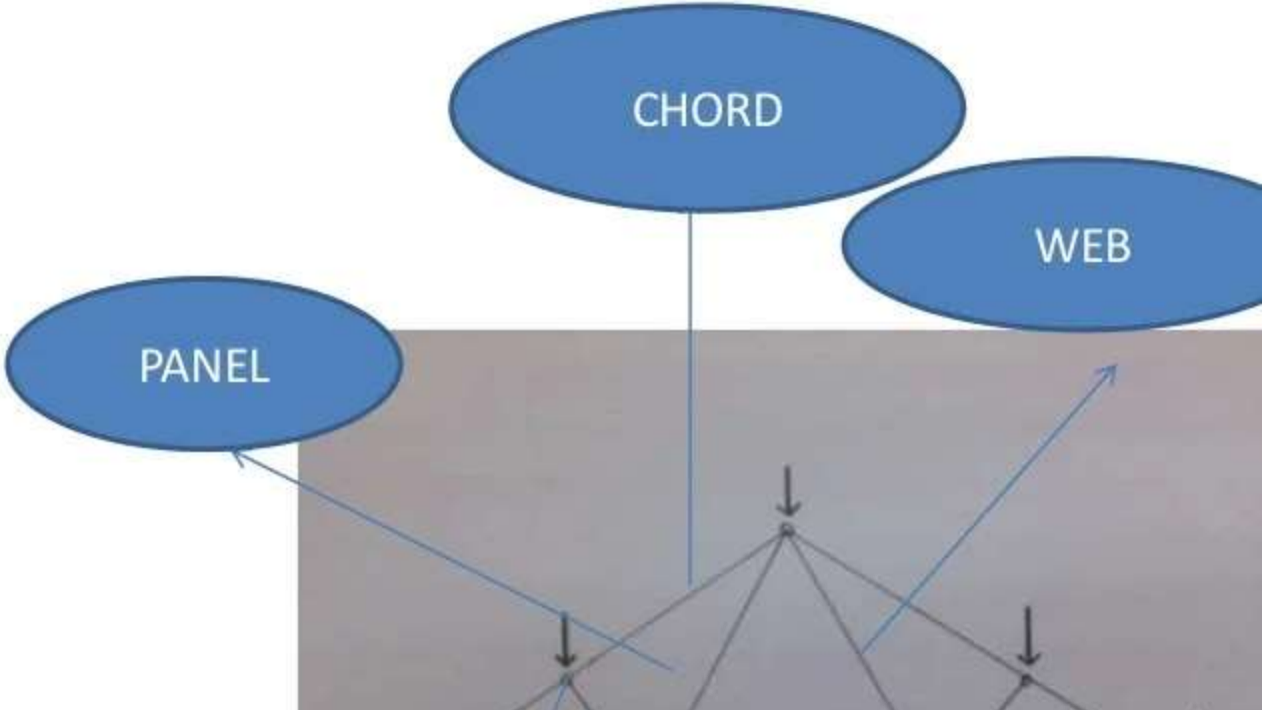
ROOF TRUSSES/RAFTERS

ROOF ELEMENTS

EXAMPLES

TRUSSES

BASIC TRUSS MEMBERS



ROOF

ROOF TYPES

CHARACTERISTICS

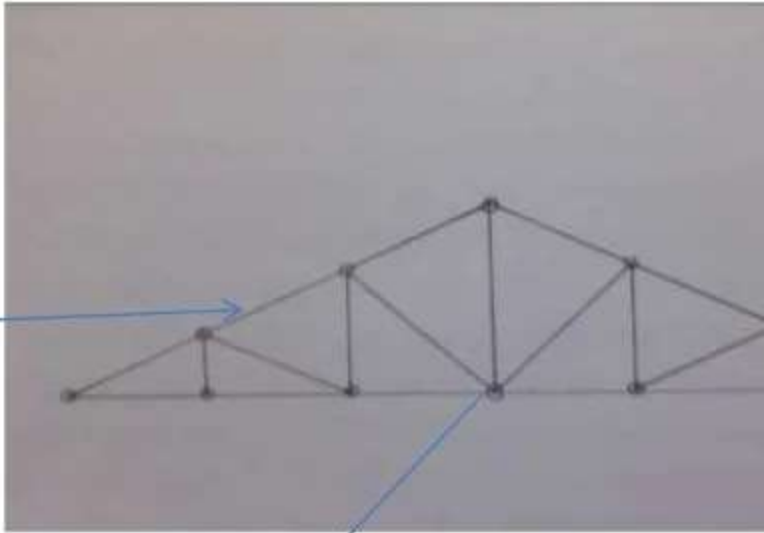
ROOF TRUSSES/RAFTERS

ROOF ELEMENTS

EXAMPLES

TRUSSES

PITCHED TRUSS



PRATT

ROOF

ROOF TYPES

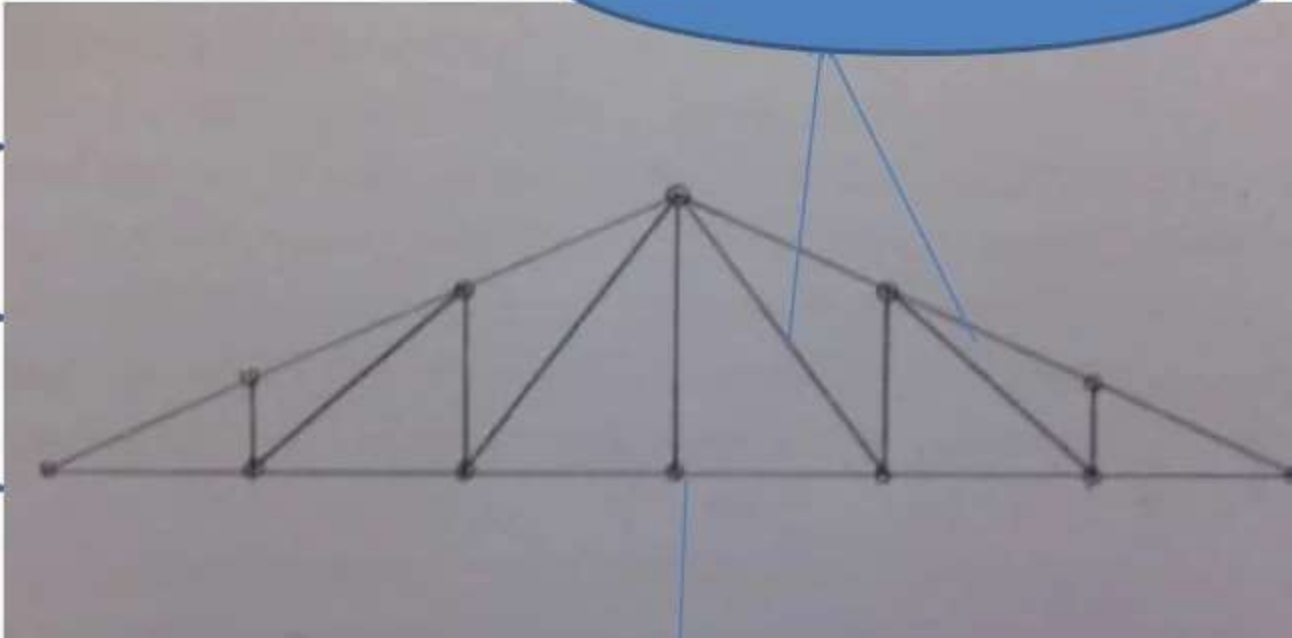
CHARACTERISTICS

ROOF TRUSSES/RAFTERS

ROOF ELEMENTS

EXAMPLES

TRUSSES



ROOF

ROOF TYPES

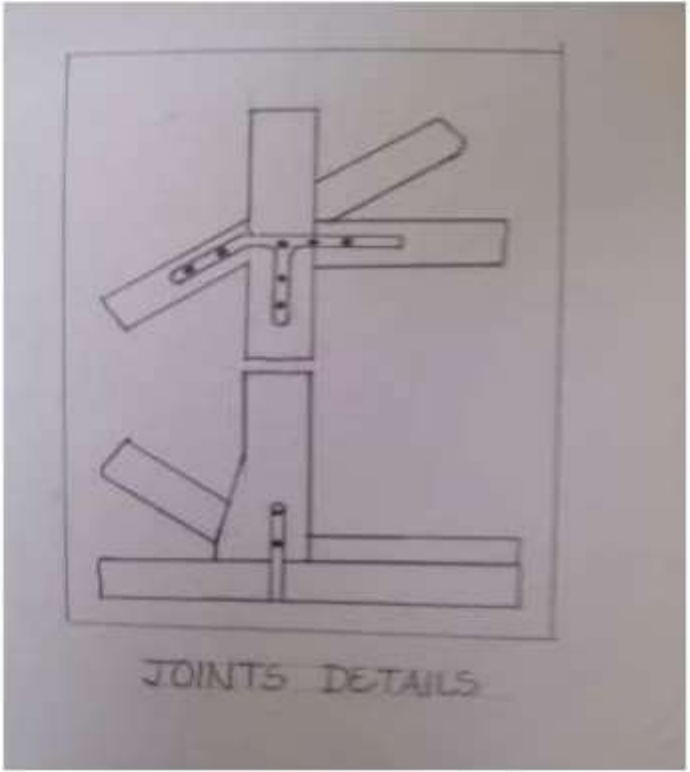
CHARACTERISTI
CS

ROOF
TRUSSES/RAFTE
RS

ROOF
ELEMENTS

EXAMPLES

TRUSSES



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THANK YOU