

# Design of compression members

As per IS 800 : 2007

Dr.S.KAVITHA,Dept of CV,ACSCE

# Compression Members



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# 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

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

where,  $f$  is assumed to be uniform over the entire cross-section

# 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



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# Compression Members

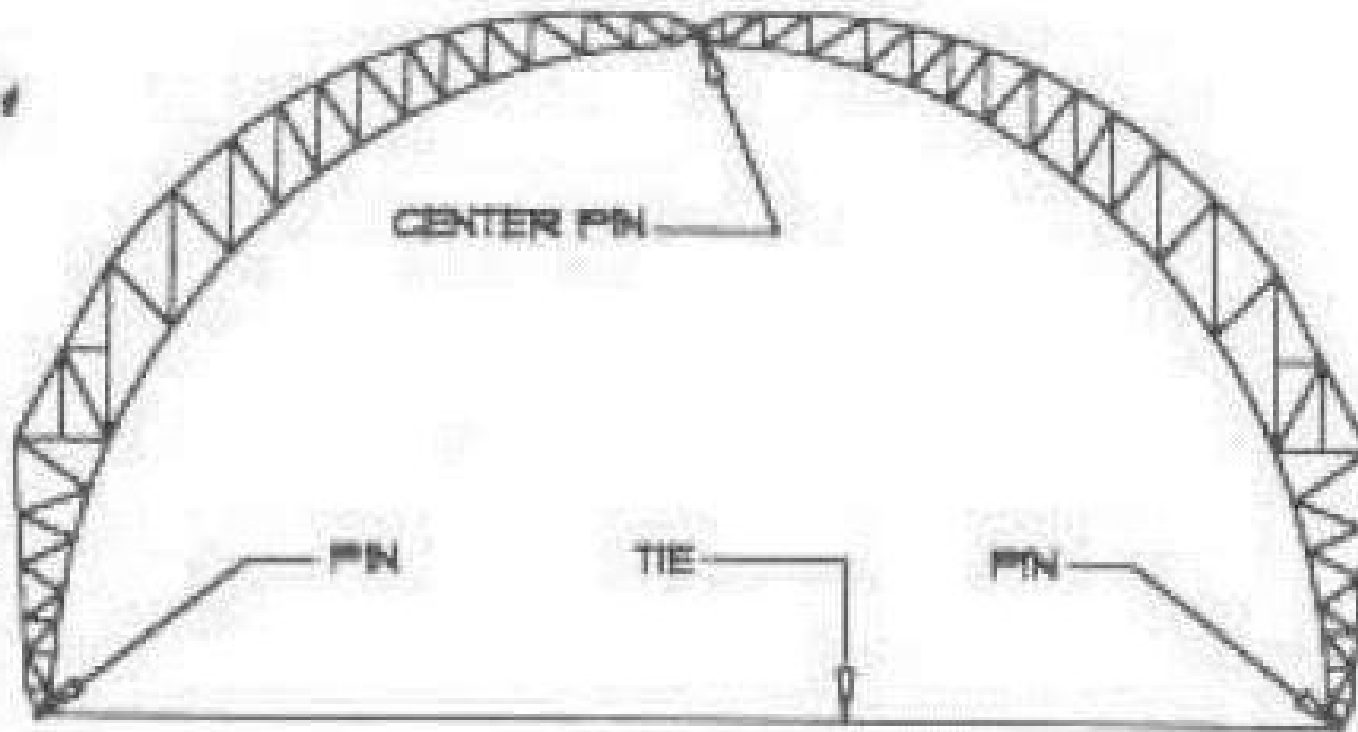


FIG. 2.8 THREE-HINGED TRUSS



# Compression Members

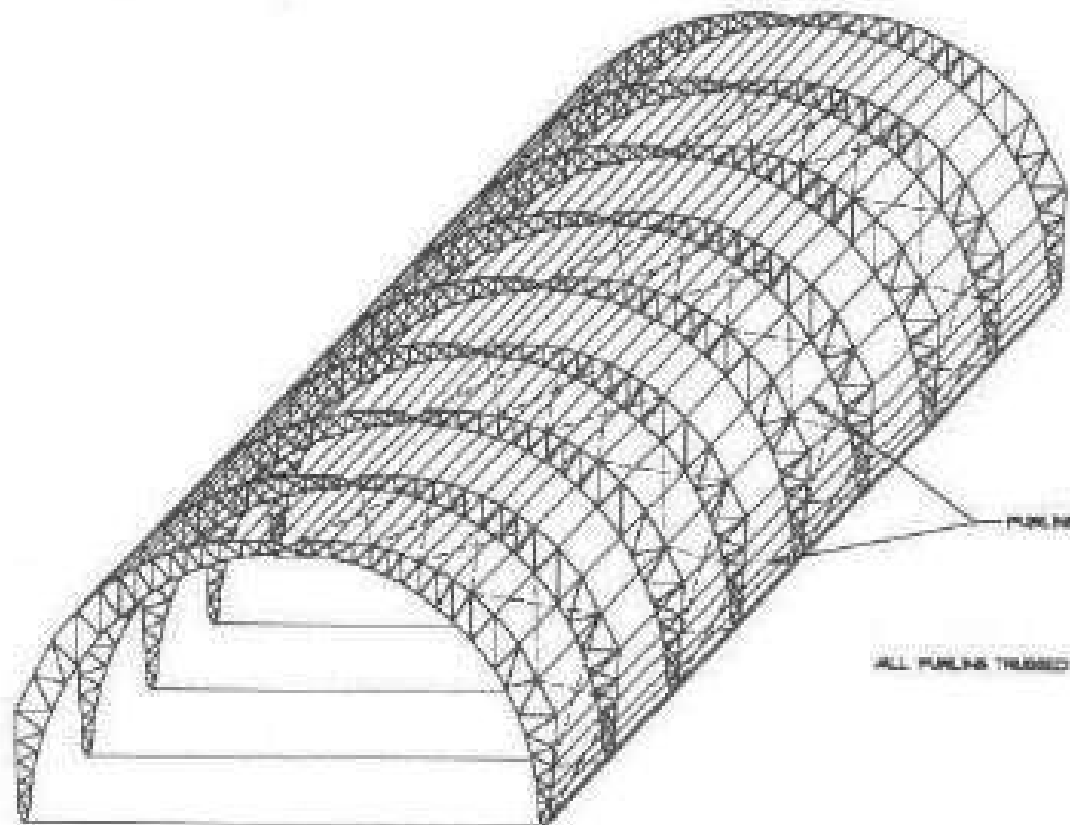


FIG. 2.9 STEEL FRAME WITH  
THREE-HINGED ARCHES

# Compression Members

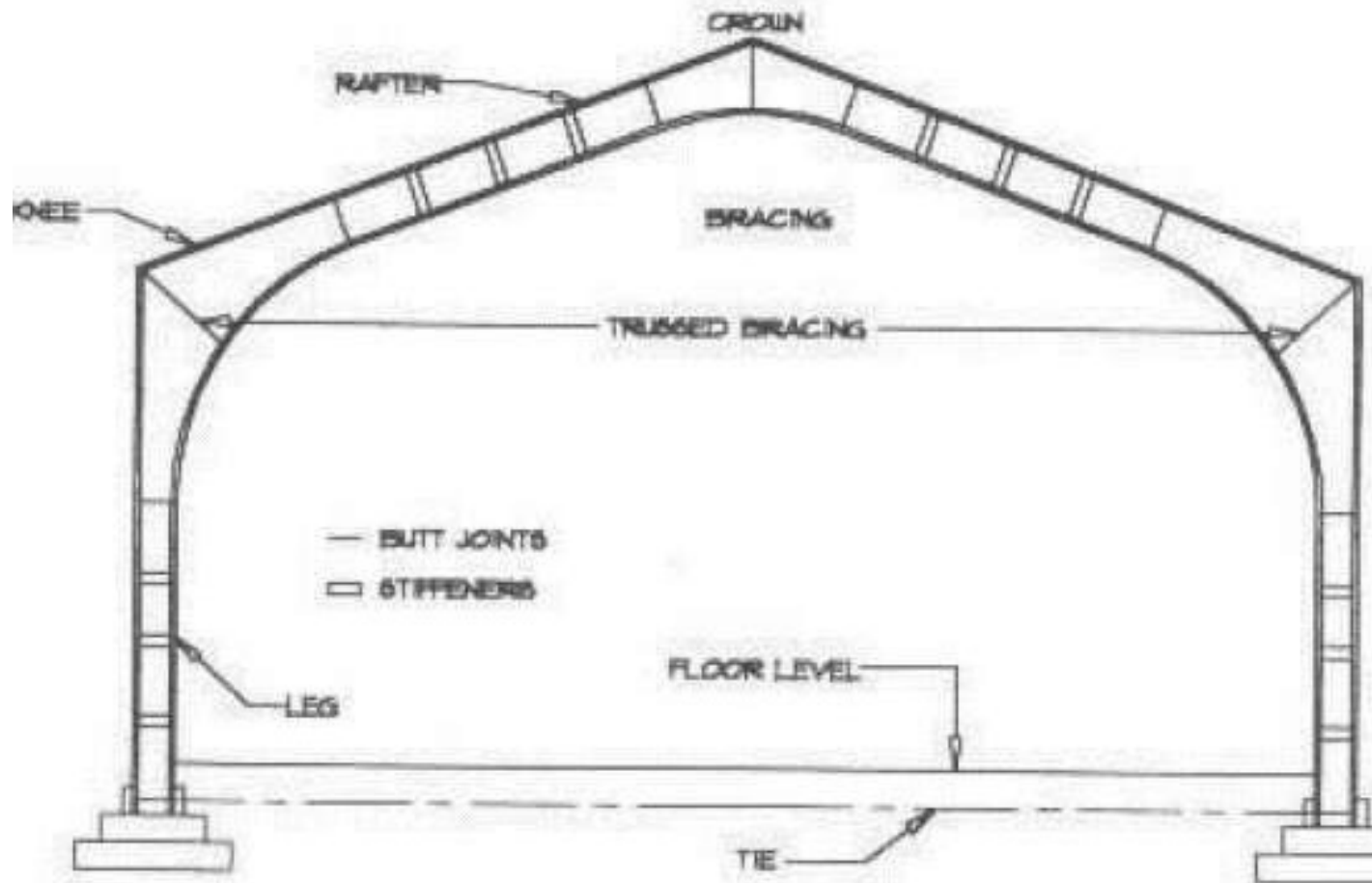


FIG. 2.1 STEEL RIGID FRAME

# Compression Members

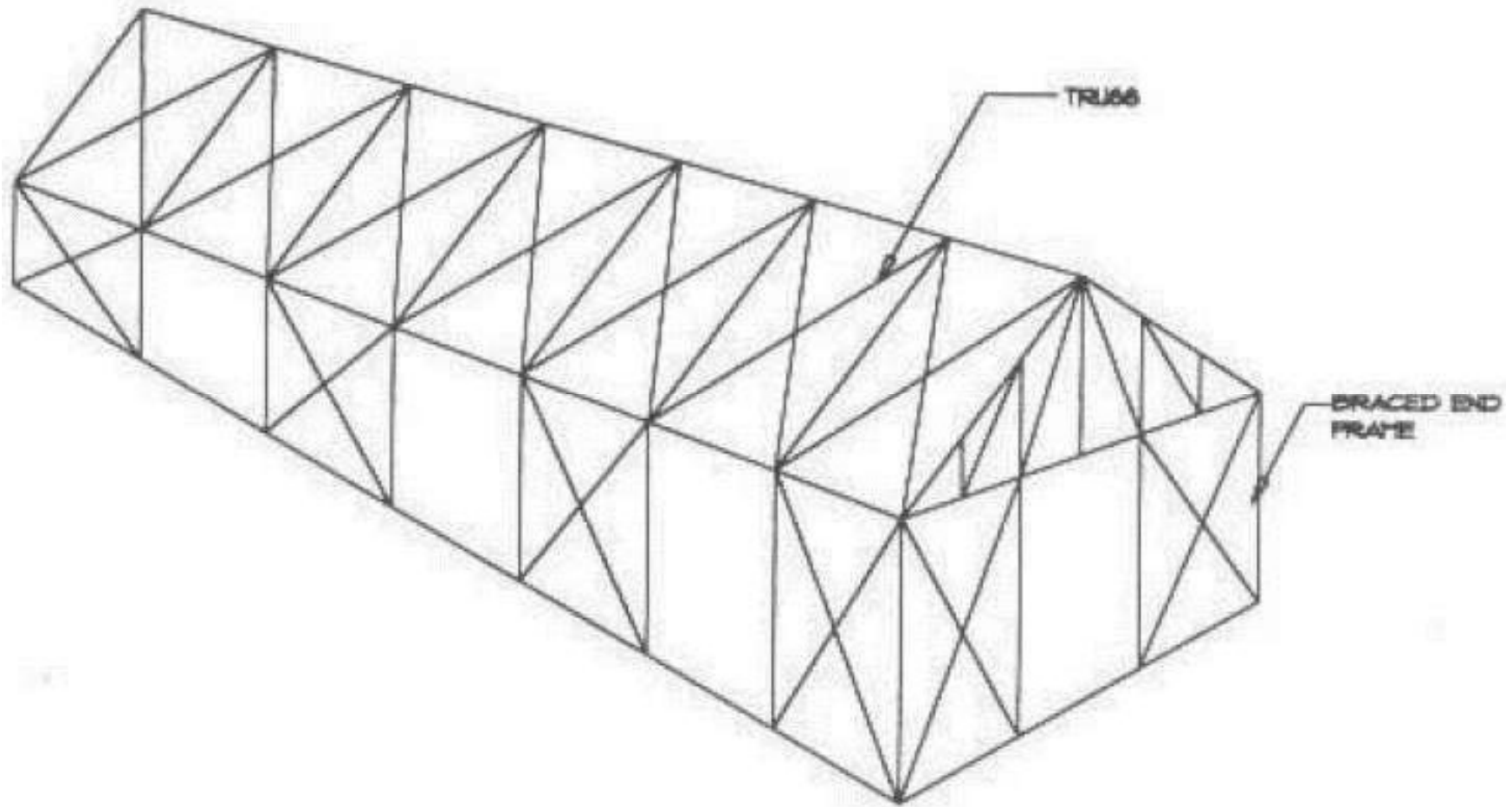


FIG. 2.6 BRACED FRAME

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# Compression Members



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# 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.

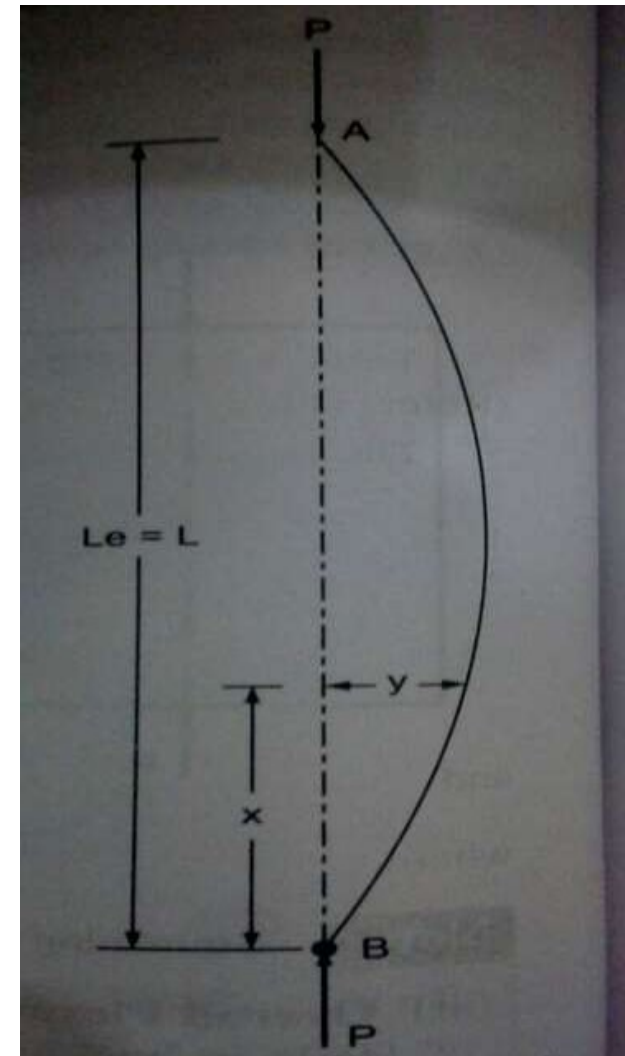
section.

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

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

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

(-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

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

---

$$\text{where } r = \sqrt{\frac{I}{A}} = \text{radius of gyration}$$

# 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 in place may have significant bending resulting from the load and initial lateral deflection.



# 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  $A_n = A_g$



# 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 compressive loads will tend to magnify the bending in those members.



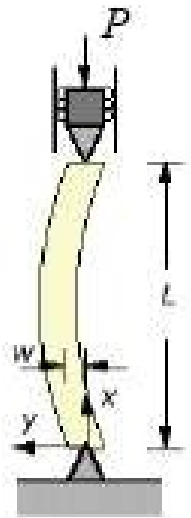
# Compression Member Failure

- 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 in between a short and fat and long and skinny column).

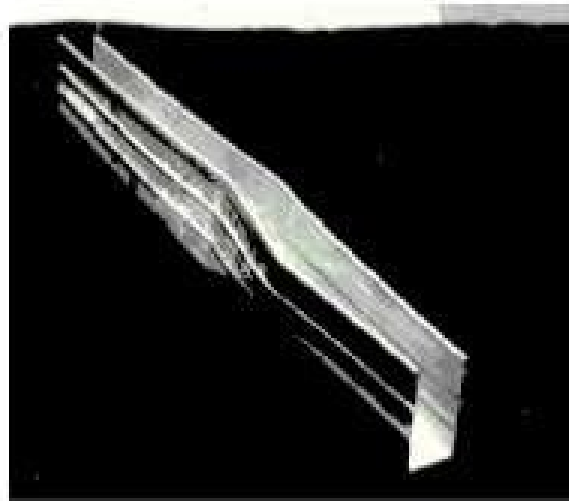


# Compression Member Failure

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

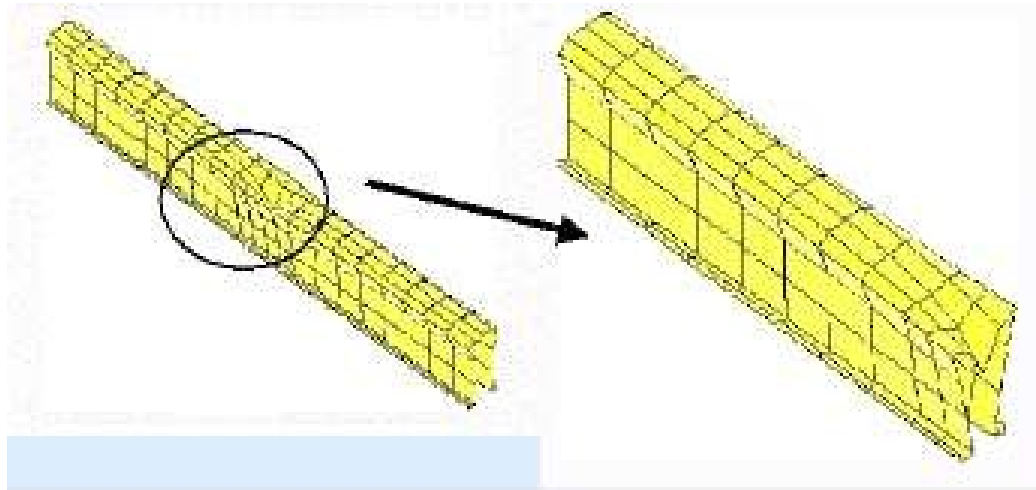


*Simply supported column  
subjected to axial load  $P$*



# 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.



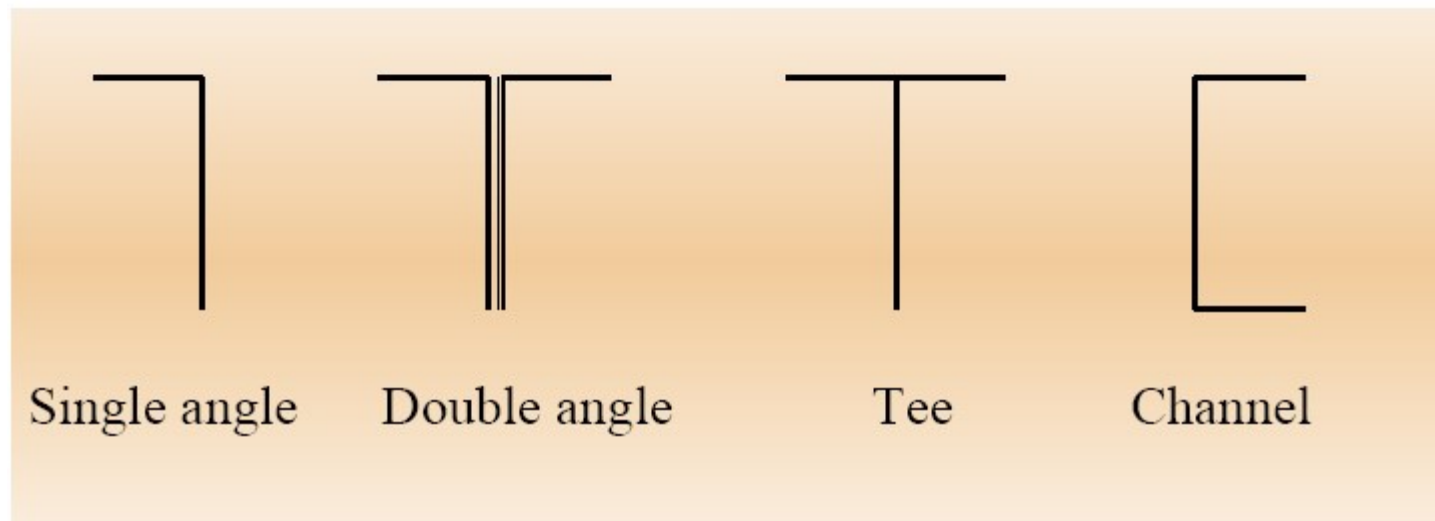
# Sections used for Compression Member

- 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 structure in which the section is to be used.

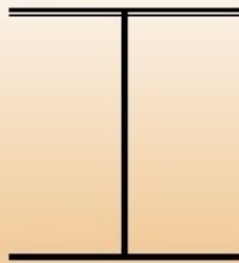


# Sections used for Compression Member

Figure 1. Types of Compression Members



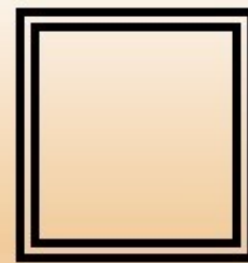
# Sections used for Compression Member



W Column



Pipe or round  
HSS tubing



Square HSS  
tubing



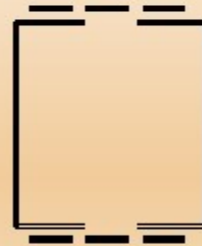
# Sections used for Compression Member



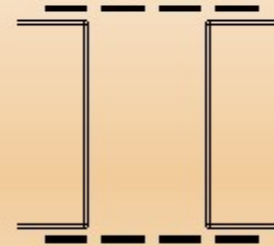
Rectangular  
HSS tubing



Four angle  
box section



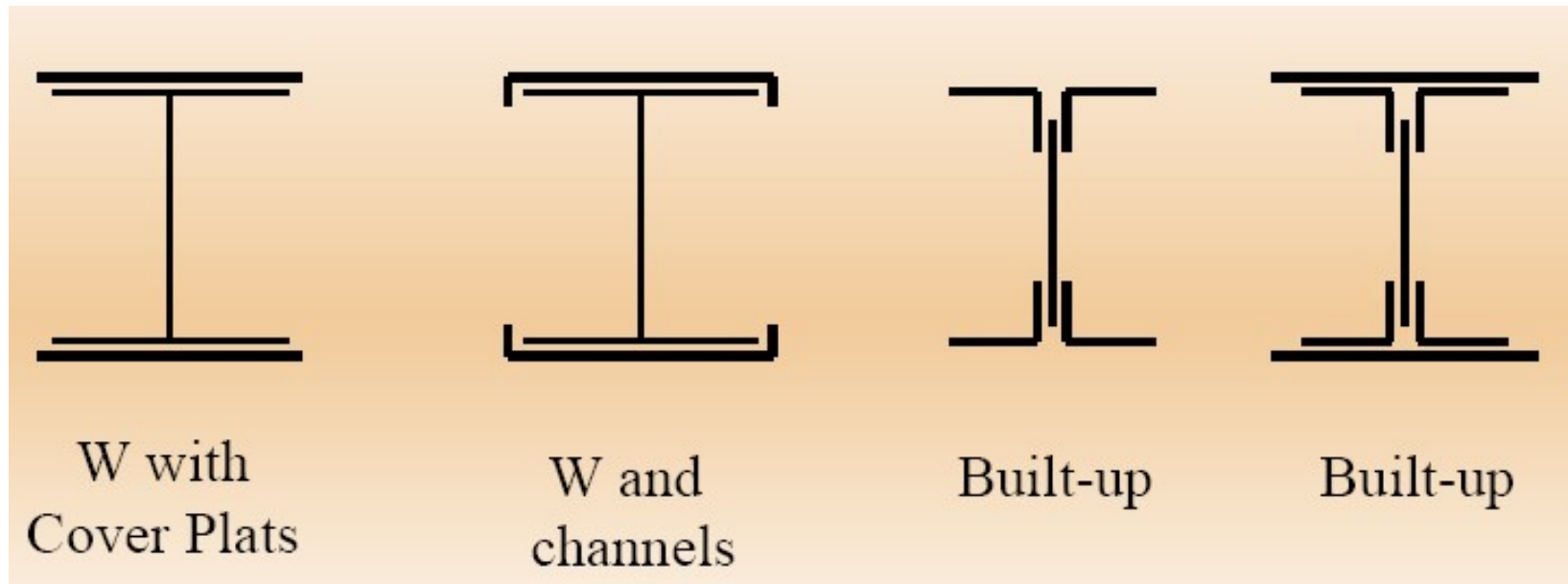
Box section



Box section



# Sections used for Compression Member



# Column Buckling

- Buckling
- Elastic Buckling
- Inelastic Buckling



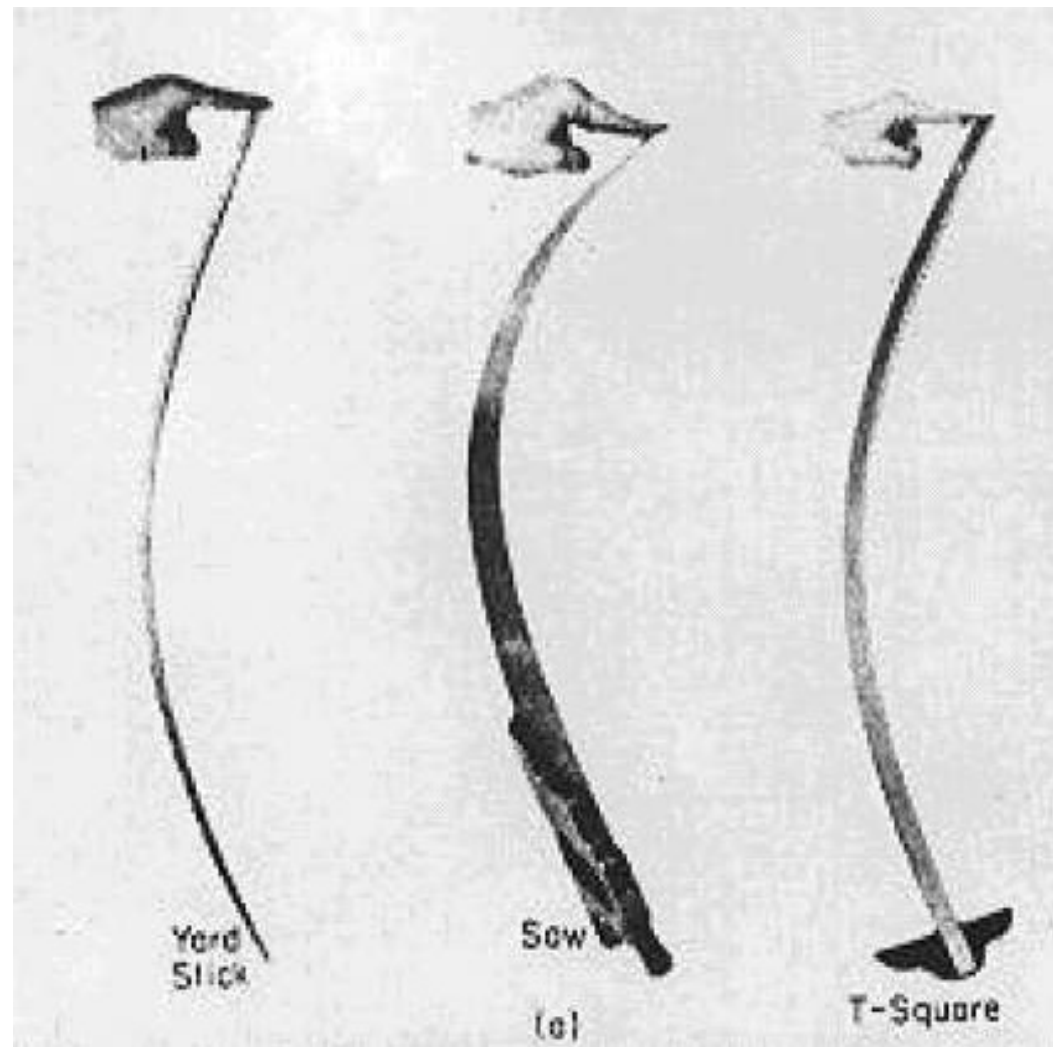
# Column Buckling

- 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

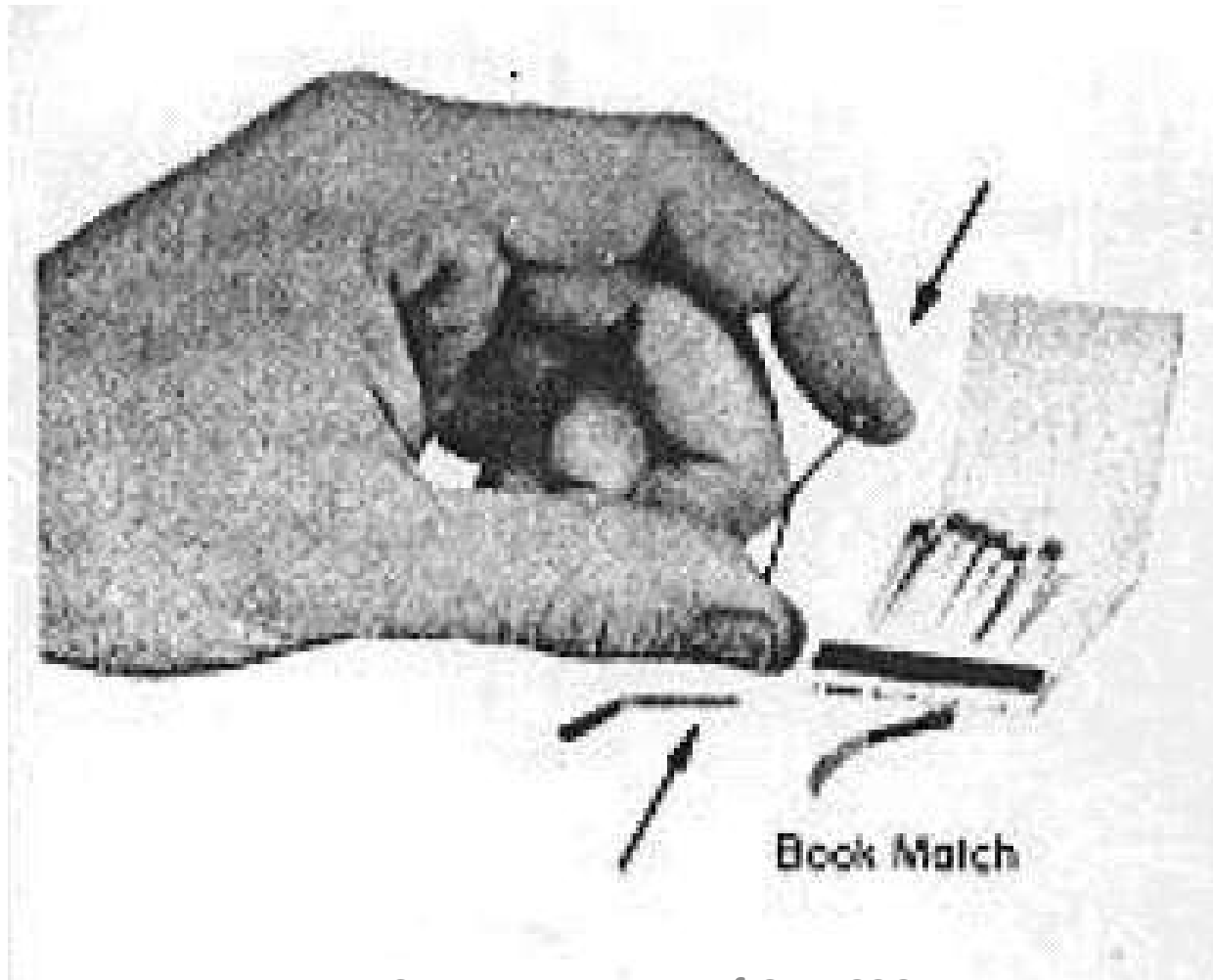
## Example



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# Buckling

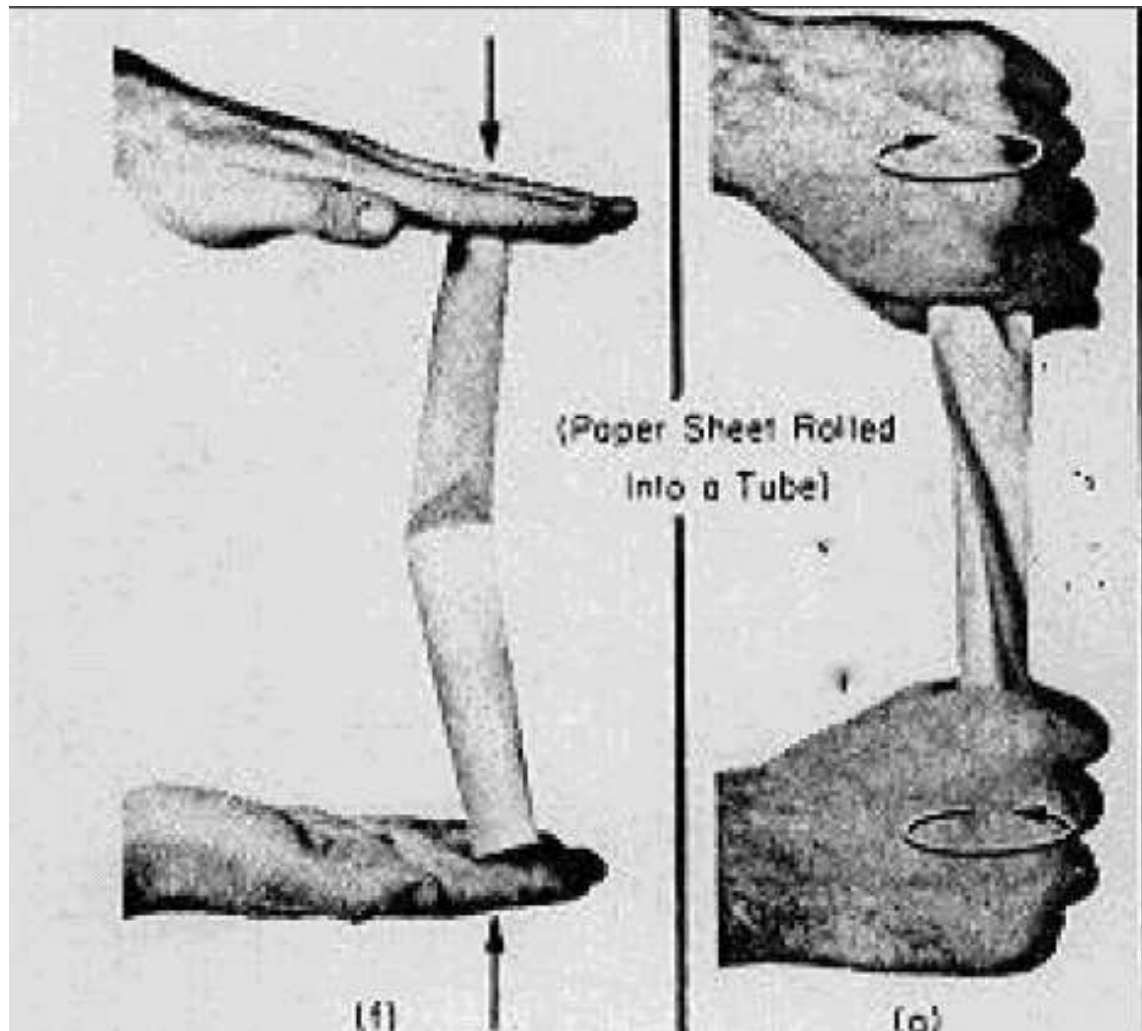
## Example



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# Buckling

## Example



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# Buckling

## Example



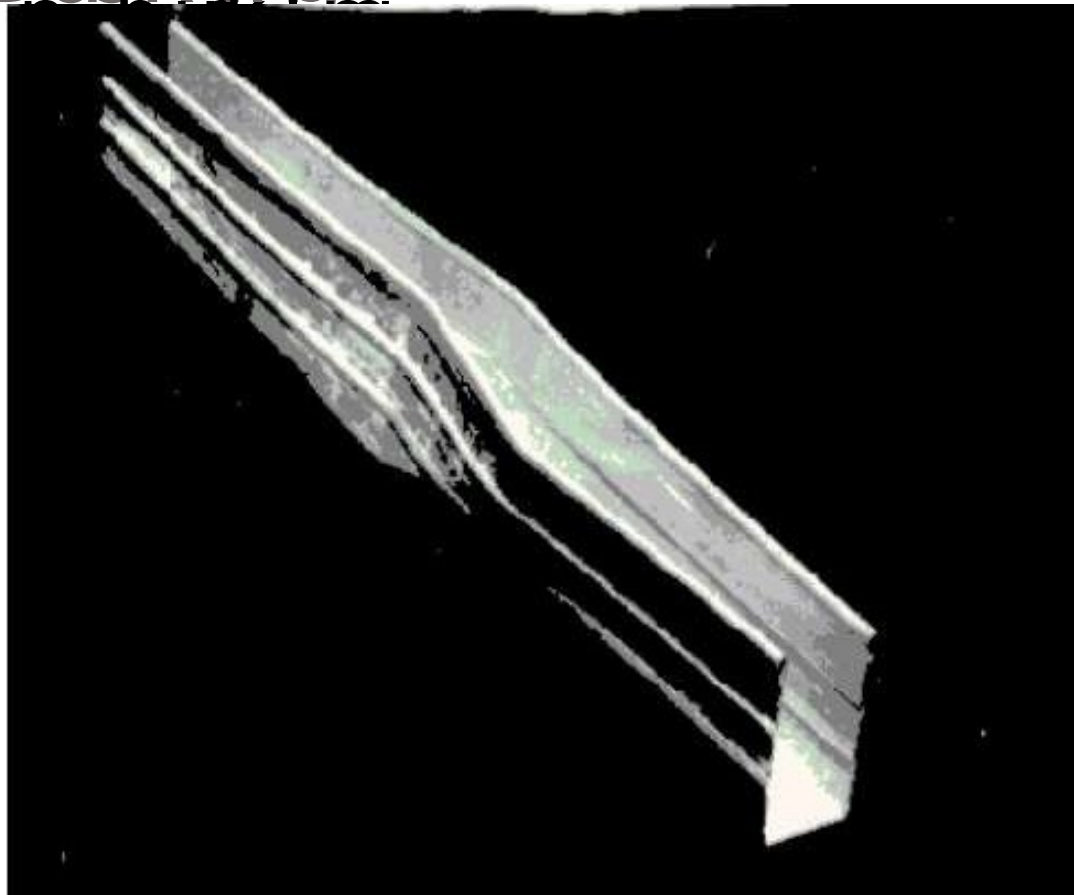
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# Buckling

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



- Column Buckling



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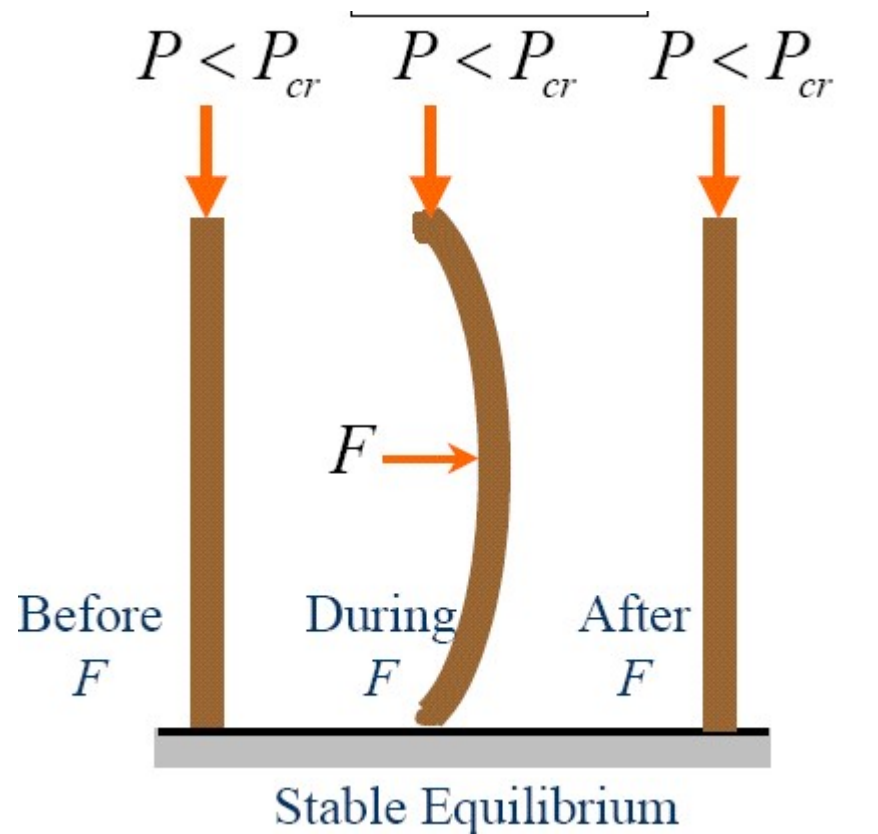
# Mechanism of Buckling

- 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 original straight position.



# 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.
- This action constitutes stable equilibrium.



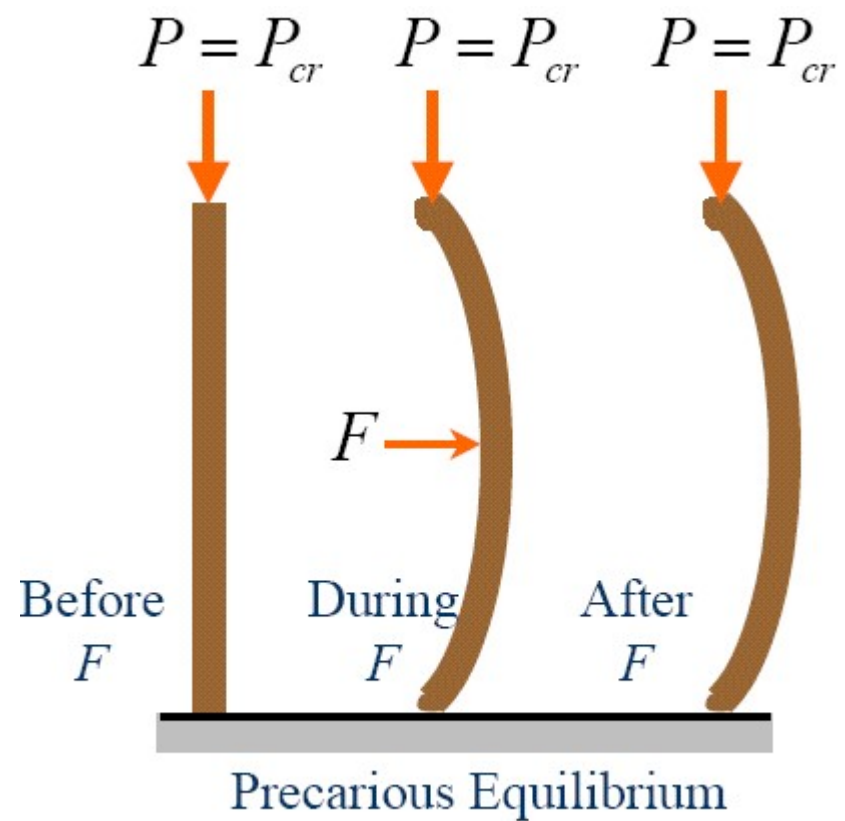
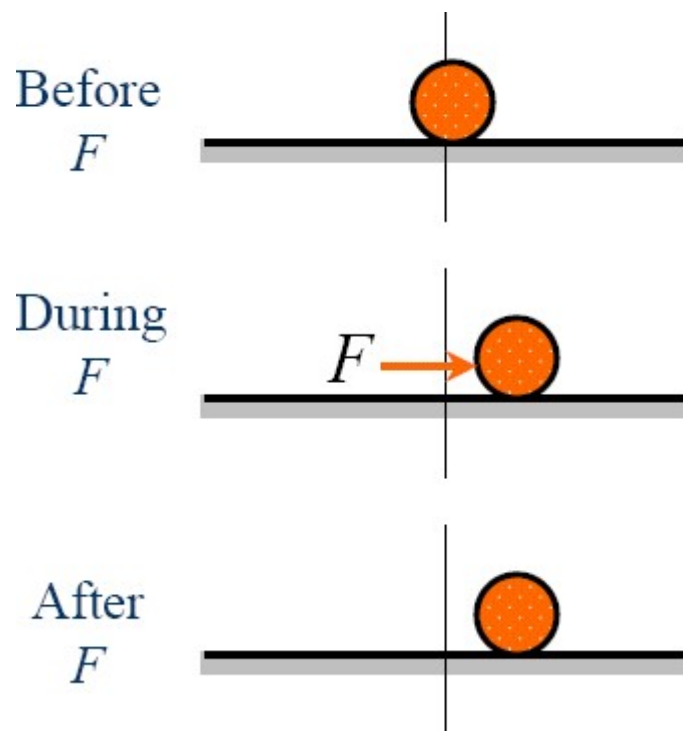
# Mechanism of Buckling

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



# Mechanism of Buckling

**Fig 2**



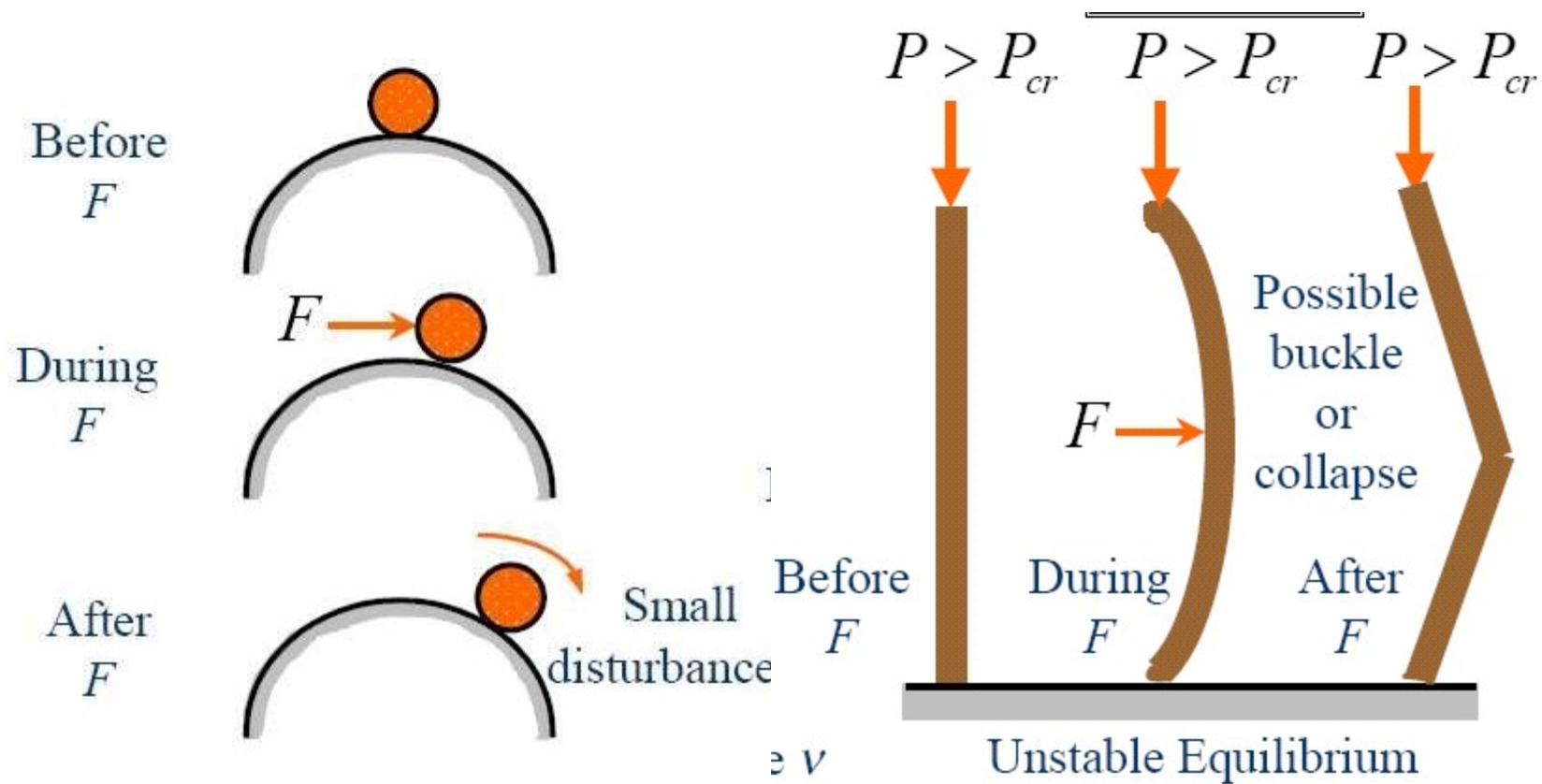
# Mechanism of Buckling

- 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  $P$ , column either remain in bent position, or will completely collapse or fracture.



# 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.
- Therefore, critical load is also called the **buckling load**.



# 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  $\chi$  , and the design compressive stress  $f_{cd}$ , 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,  $f_{cd}$  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  $f_{cd}$  ( design compressive stress) corresponding to non- dimensional effective slenderness ratio  $\lambda$  ( page 35)

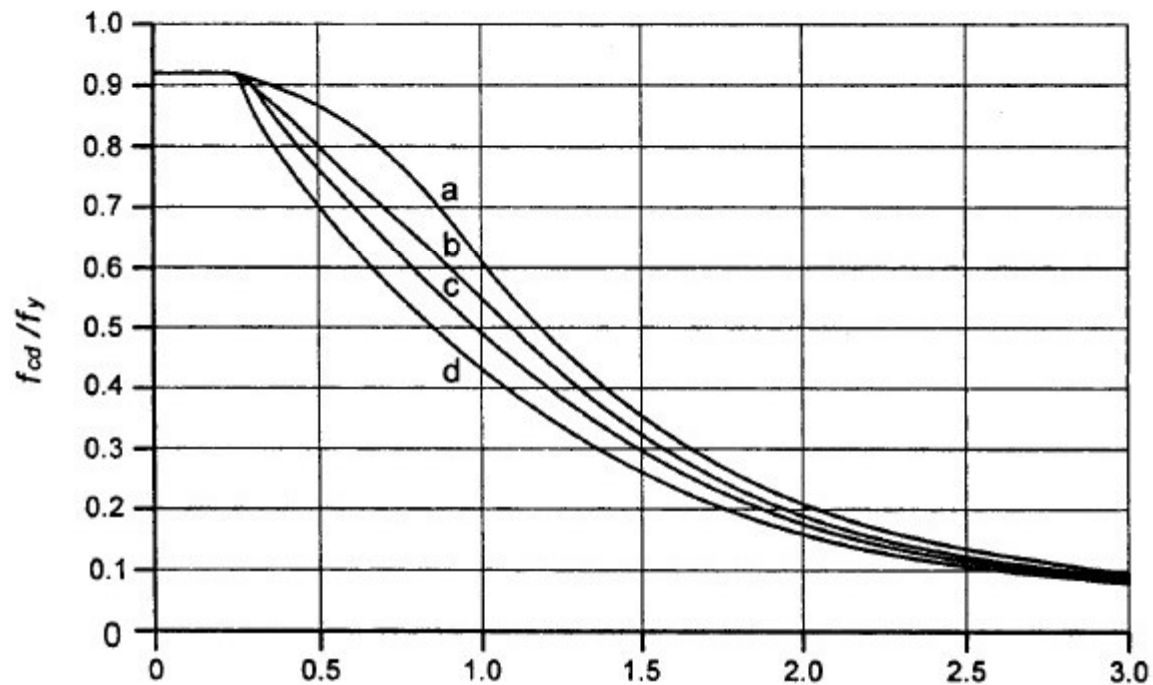
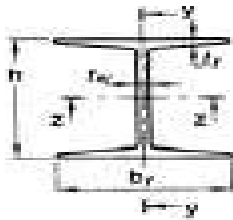
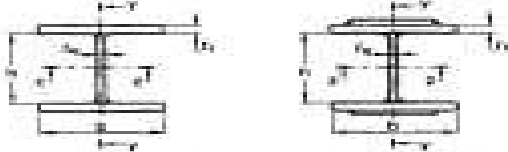
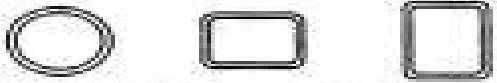
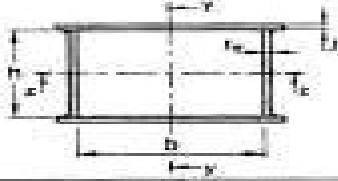
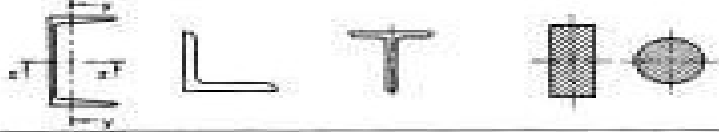



FIG. 8 COLUMN BUCKLING CURVES

**Table 10 Buckling Class of Cross-Sections**  
(Clause 7.1.2.2)

Cross-Section (1)	Limits (2)	Buckling About Axis (3)	Buckling Class (4)
<b>Rolled I-Sections</b> 	$h/b_f > 1.2 :$ $t_f \leq 40 \text{ mm}$  $40 \leq \text{mm} < t_f \leq 100 \text{ mm}$	$x-x$ $y-y$  $x-x$ $y-y$	a b  b c  d d
<b>Welded I-Section</b> 	$t_f \leq 40 \text{ mm}$  $t_f > 40 \text{ mm}$	$x-x$ $y-y$  $x-x$ $y-y$	b c  c d
<b>Hollow Section</b> 	Hot rolled  Cold formed	Any  Any	a  b
<b>Welded Box Section</b> 	Generally (except as below)  Thick welds and $h/t_f < 30$ $b/t_w < 30$	Any  $x-x$ $y-y$	b  c c
<b>Channel, Angle, T and Solid Sections</b> 		Any	c
<b>Built-up Member</b> 		Any	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

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

## 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 calculate effective sectional area.

# 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:

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

where

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

$\lambda$  = non-dimensional effective slenderness ratio

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

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

where

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

$\alpha$  = imperfection factor given in Table 7;

$\chi$  = stress reduction factor (see Table 8) for different buckling class, slenderness ratio and yield stress

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

$\gamma_{m0}$  = partial safety factor for material strength.

**Table 7 Imperfection Factor,  $\alpha$**   
(Clauses 7.1.1 and 7.1.2.1)

Buckling Class	a	b	c	d
$\alpha$	0.21	0.34	0.49	0.76

# 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  $\lambda$  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
Double I - section	30-60

- 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  $f_{cd}$  and the design strength  $P_d = A \cdot f_{cd}$
- For the estimated value of slenderness ratio, calculate the design compressive stress ( $f_{cd}$ ), 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
- $P_d = f_{cd}$  effective cross-sectional area
- The value  $P_d$  should be more than the factored load  $P_u$  for safe design

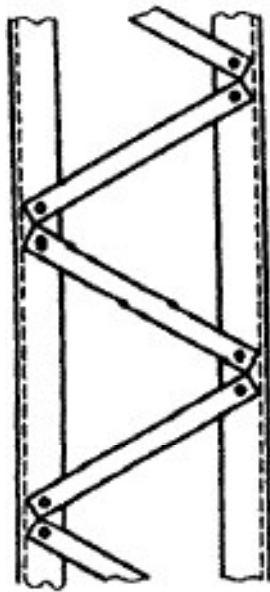
Sl No.	Member	Maximum Effective Slenderness Ratio ( $KL/r$ )
(1)	(2)	(3)
i)	A member carrying compressive loads resulting from dead loads and imposed loads	180
ii)	A tension member in which a reversal of direct stress occurs due to loads other than wind or seismic forces	180
iii)	A member subjected to 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 compression resulting from the action of wind or earthquake forces <sup>1)</sup>	350
vi)	Members always under tension <sup>1)</sup> (other than pre-tensioned members)	400

<sup>1)</sup> Tension members, such as bracing's, pre-tensioned to avoid sag, need not satisfy the maximum slenderness ratio limits.

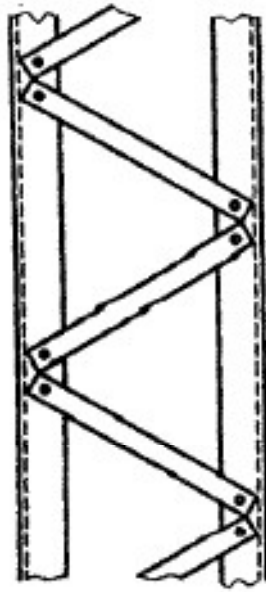
# Built-up Column members

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

# Built- up compression member



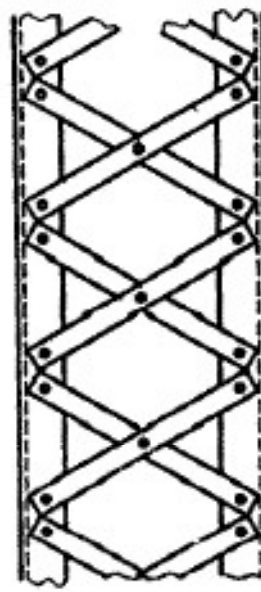
LACING ON  
FACE A



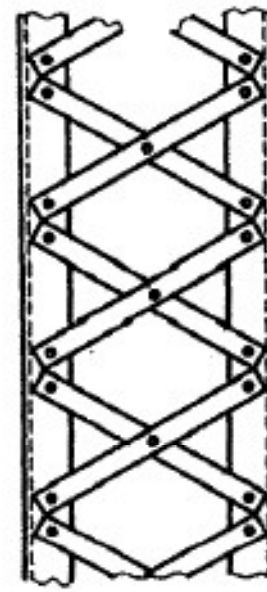
LACING ON  
FACE B

**PREFERRED LACING  
ARRANGEMENT**

10A Single Laced System



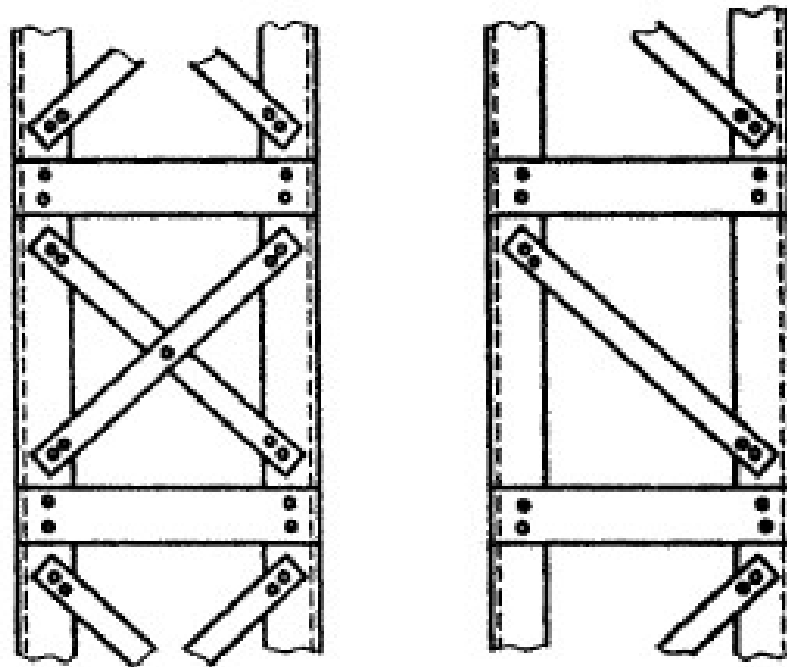
LACING ON  
FACE A



LACING ON  
FACE B

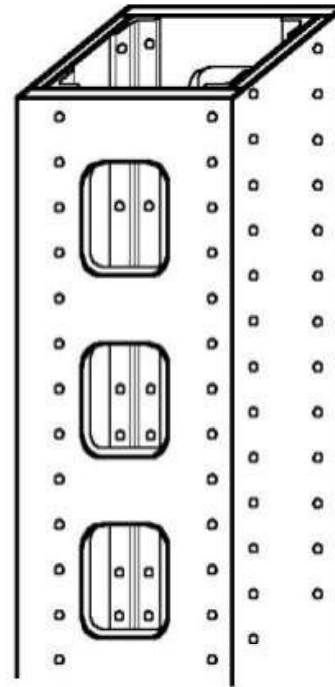
**PREFERRED LACING  
ARRANGEMENT**

10B Double Laced System



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

FIG. 10 LACED COLUMNS



Perforated plate column

Compression members - Dr. Seshu Adiluri



Column with single lacing



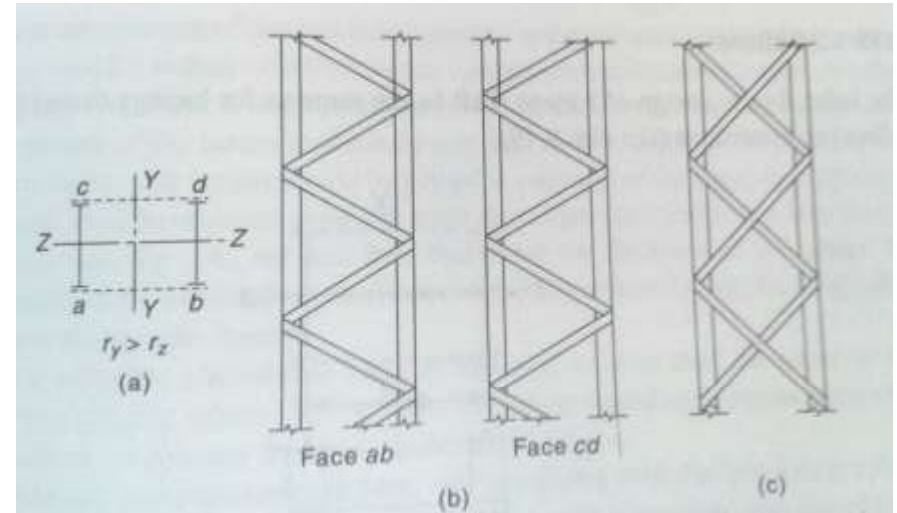
Column with battens

Note that lacings and batten plates are not designed to carry any load. Their primary function is to hold the main components of the built up column in their relative position and equalize the stress distribution, but they may have to resist shear at any point in member or shear due to bending moment or lateral load.

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# 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.  $r_y > r_z$ ) figure (a)
- Lacing system should be uniform throughout the length of column
- Single and double laced systems should not be provided on opposite sides of the same member. ( fig. b and c)



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

- Lacing shall be designed to resist a total transverse shear  $V_t$  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 connecting rivet /bolt; the thickness shall not  $<$  than  $1/40^{\text{th}}$  of effect length for single and  $1/60^{\text{th}}$  for double lacing
- Spacing of lacing bars shall be such that the max slenderness ratio of the components of main member between two consecutive lacing connections is not  $>$  than 50 or 0.7 times the most unfavourable slenderness ratio of the combine column.

- 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 as 1.05 times  $(KL/r)_0$  where  $(KL/r)_0$  is the max actual slenderness ratio of the column, to account for shear deformations effects.
- The required sections of lacing bars as compression/tension members may be determined using the appropriate design stresses  $f_{cd}$  as given before.

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

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

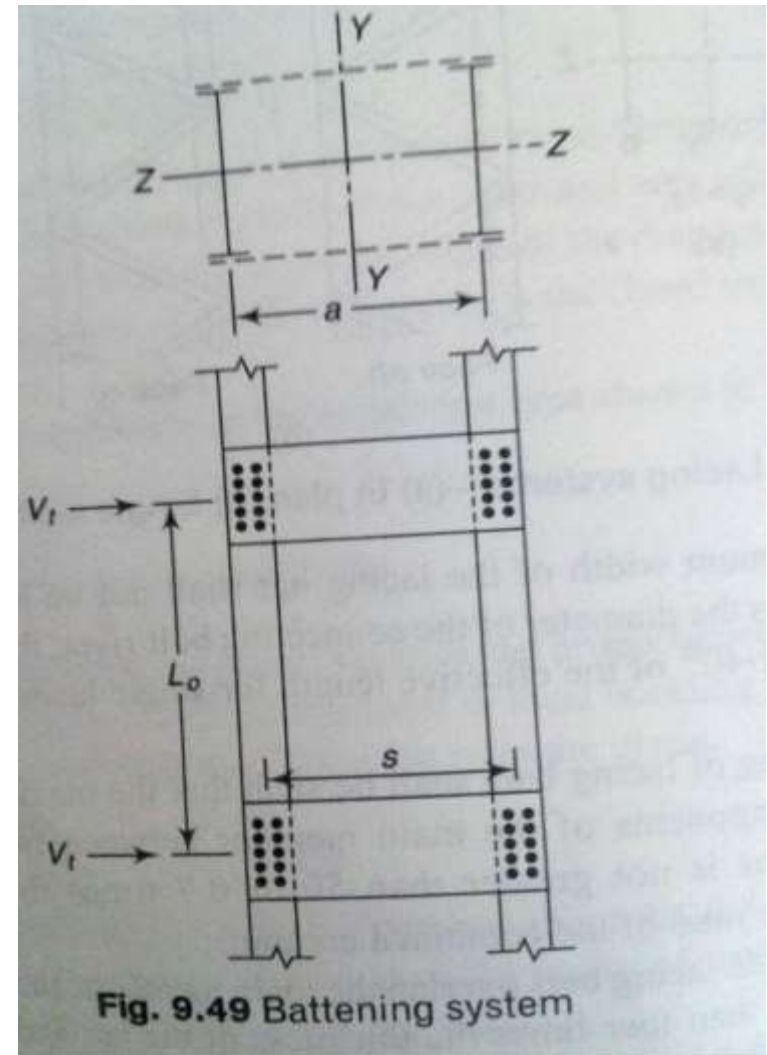
Where

$L_o$  = distance bet c/c of battens, longitudinally

$n$  = no of parallel planes of battens

$s$  = min transverse distance bet centroids of the bolt/rivet group/welding connecting the batten to the main member.

Battens shall be designed to carry BMs and SFs arising from transverse SF,  $V_t$  equal to 2.5% of the total axial force on the whole compression member



# 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 ( $r_y \geq r_z$ ).
- 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  $3/4^{\text{th}}$  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

$$d' = S' + 2c_{yy}$$

Overall depth of end batten

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

Effective depth of intermediate batten

$$d_1' = 3/4^{\text{th}} d'$$

Overall depth of intermediate batten

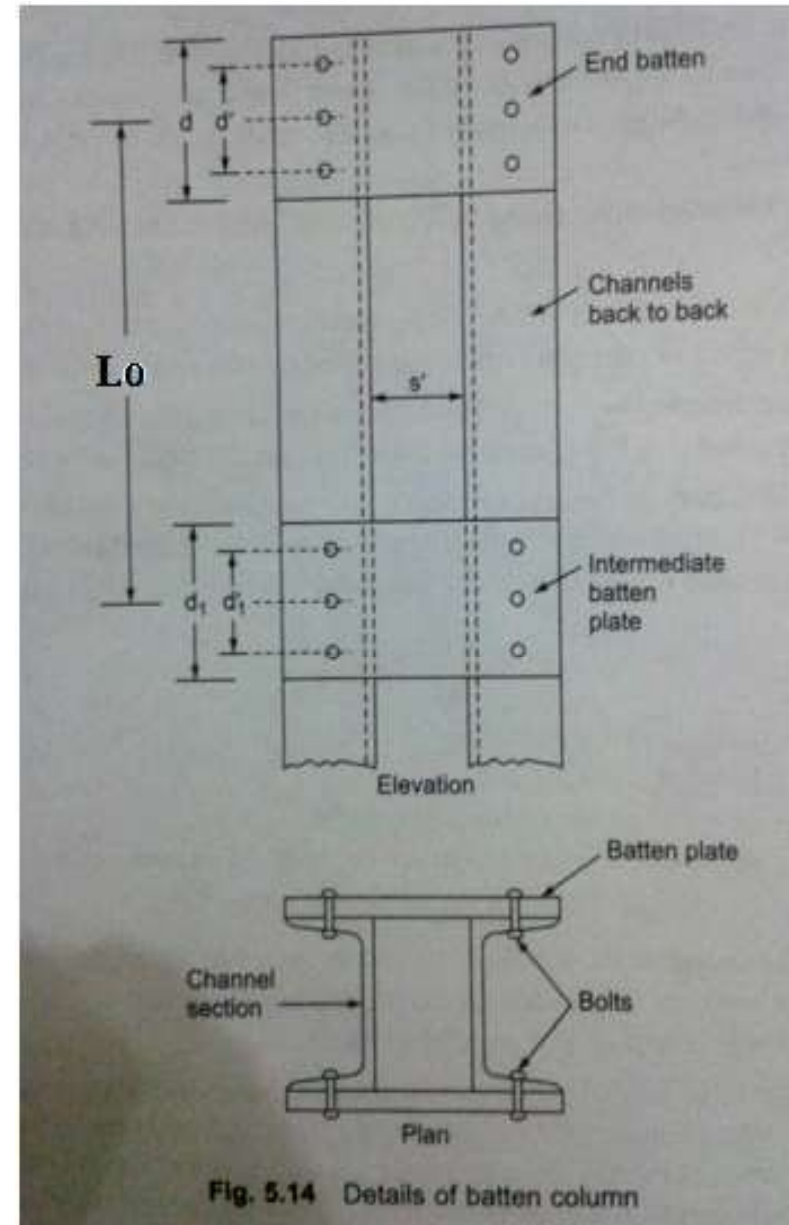
$$d_1 = d_1' + 2 \times \text{edge distance}$$

Where  $c_{yy}$  = the distance taken from steel table for the section selected.

- Thickness shall not be  $< 1/50^{\text{th}}$  of distance between the innermost connecting transverse rivets/bolts or welds.

$$T < 1/50(S' + 2g)$$

- .where  $g$  = gauge distance referred from steel table for the section selected.



- Shear stress calculated in the battens  

$$= (Vb / A1)$$

This should be less than

$$\frac{Vb}{A1} < \frac{fy}{\sqrt{3} \cdot \gamma_{m0}}$$

Where  $A1$  = cross sectional area of batten =  $t \cdot d$

$\gamma_{m0}$  = 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  $< fy / \gamma_{m0}$  as

$$\sigma_{bc,cal} = M/Z = M / (td^2/6) = 6M/td^2 < fy / \gamma_{m0}$$

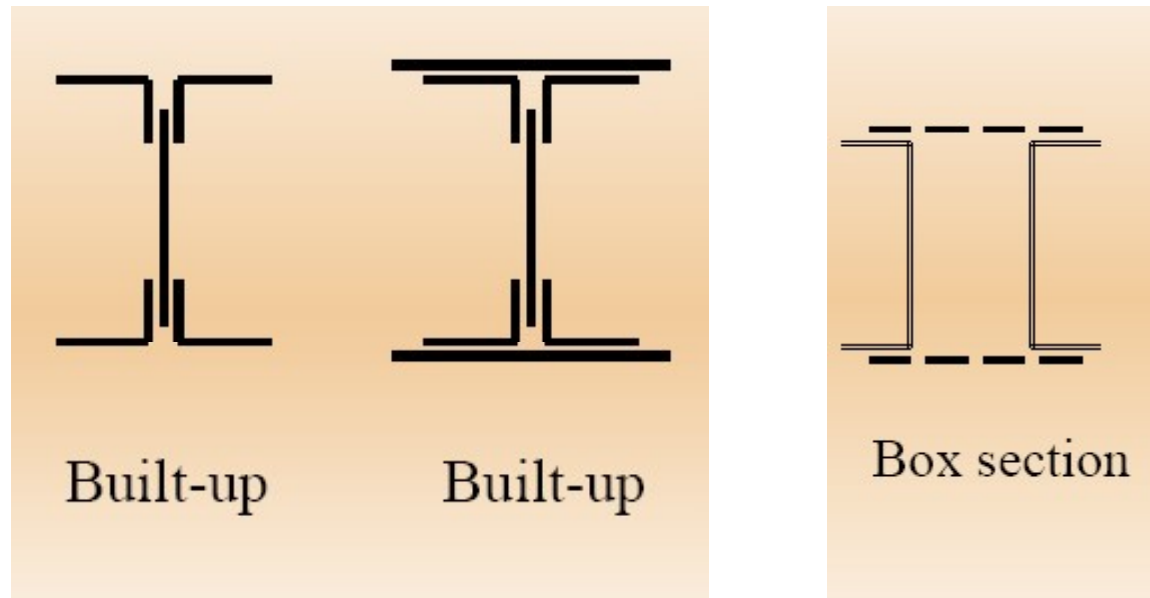
- Requirements of size not required when other rolled sections are used for battens with their legs or flanges perp. to the main member.
- When connected to main members by welds, the length of weld connecting each end of batten shall not  $< \frac{1}{2}$  the depth of the batten plate; atleast  $\frac{1}{3}$ <sup>rd</sup> of its length should be placed at each end of the edge; in addition the weld shall be returned along the other two edges for a length not  $<$  the min lap (i.e not  $< 4$  times thickness 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 times  $(KL/r)_o$ , where  $(KL/r)_o$  is the max actual slenderness

ratio of the column to account for the shear deformation effects

---

Complete section member is composed of two members back to back



- Compression members may be composed of two angles, channels or T's back-to-back in contact or separated by a small distance and connected together by bolting, rivetting or welding. In such case the rules as per IS 800:2007 are as follows

- 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 should be used in each connection.
- Spacing of tack bolts or welds should be less than 600 mm. If bolts are used they should be spaced longitudinally at  $<$  than 4 times the bolt dia and the connection should extend at least 1.5 times the width of the member.

- The bolts/rivets should be 16 mm or more in dia for a member  $\leq 10$  mm thick and 20 mm in dia for a member  $\leq 16$  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  $16t$ , 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 :  $f_y = 250 \text{ N/mm}^2$ ,  $f_u = 410 \text{ N/mm}^2$  and  $E = 2 \times 10^5 \text{ N/mm}^2$ .

## Solution:

### Method 1

Properties of ISHB 400 from handbook/steel table:

$A = 10466 \text{ mm}^2$   $b_f = 250 \text{ mm}$   $t_w = 10.6 \text{ mm}$   $h = 400 \text{ mm}$

$t_f = 12.7 \text{ mm}$   $r_z = 166.1 \text{ mm}$   $r_y = 51.6 \text{ mm}$   $L = 3.5 \text{ m} = 3500 \text{ mm}$

Refer table 10 page 44

$h/b_f =$  if  $> 1.2$  and  $t_f =$   $< 40 \text{ mm}$

From above condition the buckling curve to be used along z-z axis is curve a and that about y-y axis is curve b.

Since  $r_y < r_z$  the design compressive strength is governed by the effective slenderness ratio  $\lambda_y$  (column will buckle about y-y axis)

$\lambda_y=40$  and  $f_y = 250$   $f_{cd} = 206$  Mpa and

$y = KL / r_y$

$$= (0.65 \times 3500) / 51.6$$

$$= 44.09 \quad \text{for both ends fixed } k = 0.65$$

For buckling class b and  $f_y = 250$  MPa, design compressive stress = 200 Mpa =  $f_{cd}$  (refer table 9(b) of IS code) This is obtained by interpolating between the two values of  $\lambda_y$

$\lambda_y=40$  and  $f_y = 250$   $f_{cd} = 206$  Mpa and

so  $(50-40) \rightarrow (194-206)$

$\lambda_y=50$  and  $f_y = 250$   $f_{cd} = 194$  Mpa

$(50 - 44.09) \rightarrow (194 - x)$

Cross multiplying  $10 \times (194 - x) = (-12) \times (5.91)$

$$x = 200 \text{ MPa}$$

## **Method 2**

we can use the formulae

Non-dimensional slenderness ratio

$\lambda$  = non-dimensional effective slenderness ratio

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

$$= \sqrt{[(250 \times (44.09)^2) / (\pi^2 \times 2 \times 10^5)]}$$

$$= 0.5$$

**For  $\lambda = 0.5$** , for buckling curve b,  $f_{cd}/f_y = 0.8$

therefore

$$f_{cd} = 0.8 \times 250 = 200 \text{ MPa}$$

Therefore  $P_d = A_e \times f_{cd} = 10466 \times 200 = 2093 \times 10^3 \text{ N}$  or 2093 kN

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- **Method 3**

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

where

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

$\lambda$  = non-dimensional effective slenderness ratio

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

- Imperfection factor

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

$$\begin{aligned} \text{Find } \phi &= 0.5[1 + 0.34(0.5 - 0.2) + 0.5^2] \\ &= 0.676 \end{aligned}$$

$$\begin{aligned} \chi &= 1 / \{ 0.676 + [0.676^2 - 0.5^2]^{1/2} \} \\ &= 0.884 \end{aligned}$$

$$\begin{aligned} F_{cd} &= (250 \times 0.884) / 1.1 \\ &= 200.9 \text{ N/mm}^2 \end{aligned}$$

$$\begin{aligned} P_d &= A_e \times f_{cd} = 10466 \times 200.9 \\ &= 2102.71 \text{ kN} \end{aligned}$$

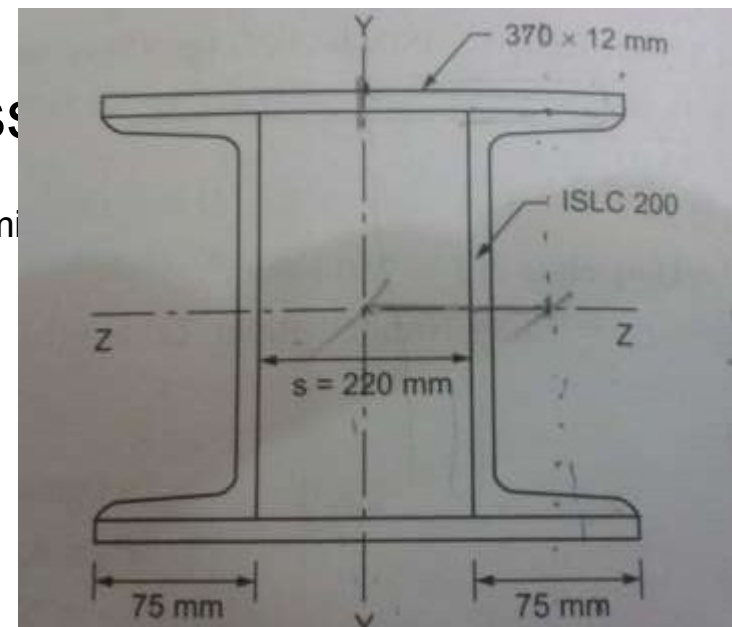
$$f_{cc} = \text{Euler buckling stress} = \frac{\pi^2 E}{\left( \frac{KL}{r} \right)^2}$$

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  $f_y = 250 \text{ N/mm}^2$   $E = 2 \times 10^5 \text{ N/mm}^2$

### Solution

For  $f_{cd}$  what is required

- Need to know buckling class
- Slenderness ratio  $\lambda = L_{\text{eff}} / r_{\text{min}}$   
 $= KL / r_{\text{min}}$



Refer Steel table to get details of section

- [IS SP.1.1964.pdf](#)

Properties of two ISLC 200( back to back)

A=                      I<sub>zz</sub>=                      I<sub>yy</sub>=                      Z<sub>yy</sub>

r<sub>yy</sub>=

Properties of built up section

Total area = Area of Cs + area of cover plates

$I_{zz} = I_{zz} \text{ of channels (back to back)} + I_z \text{ of cover plates @ Z-Z}$

$I_{yy} = I_{yy} \text{ of channels ( back to back)} + I_y \text{ of cover plates @ Y-Y}$

Check which is minimum among these two

[is.800.2007- code of practice for gener steel.pdf](#)( check which buckling class)

- Buckling class=
- Calculate least radius of gyration =
- Corresponding to slenderness ratio and buckling class and for  $f_y = 250 \text{ N/mm}^2$
- 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)**

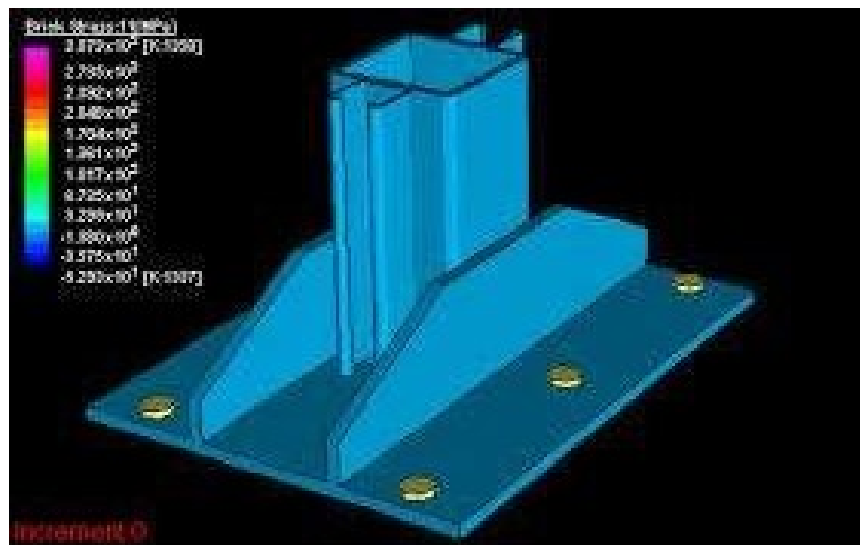
- Column bases like base plates are used to transmit load from columns to foundations
- It reduces intensity of loading and distributes it over the foundations.
- Area of base plate chosen is so chosen such that the intensity of load distributed is less than the bearing capacity of concrete on which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

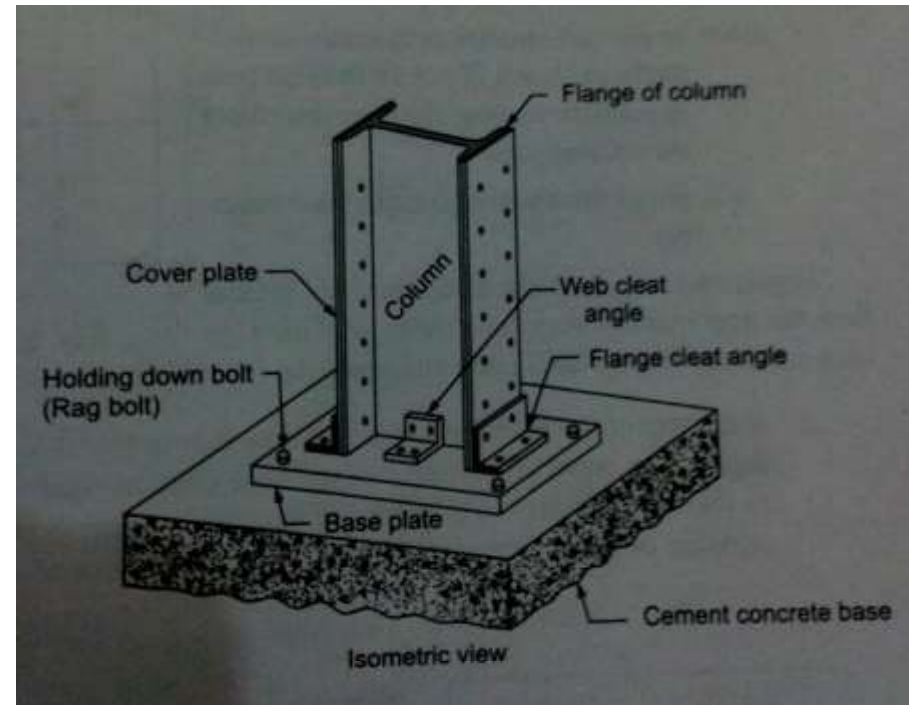
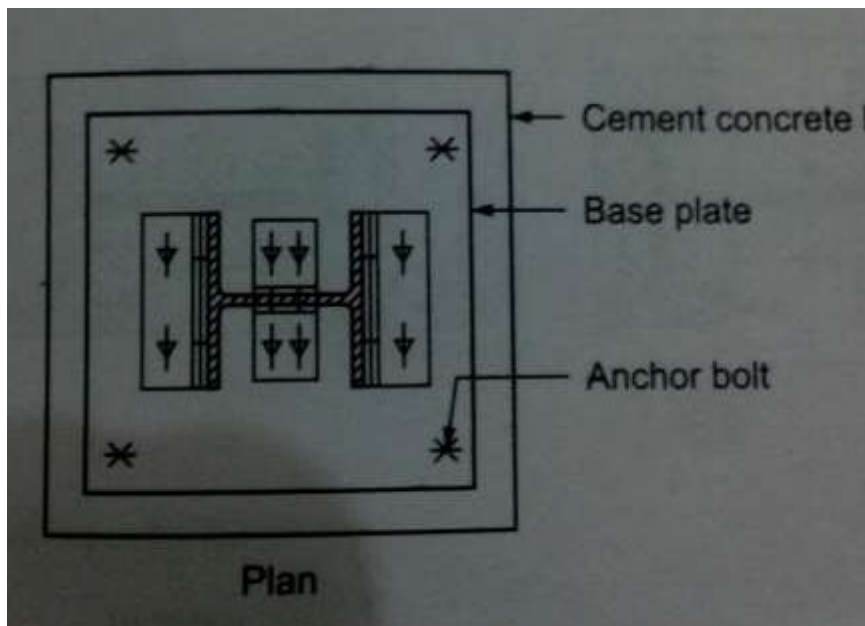
- ✓ Slab base
- ✓ Gussetted base
- Pocket base



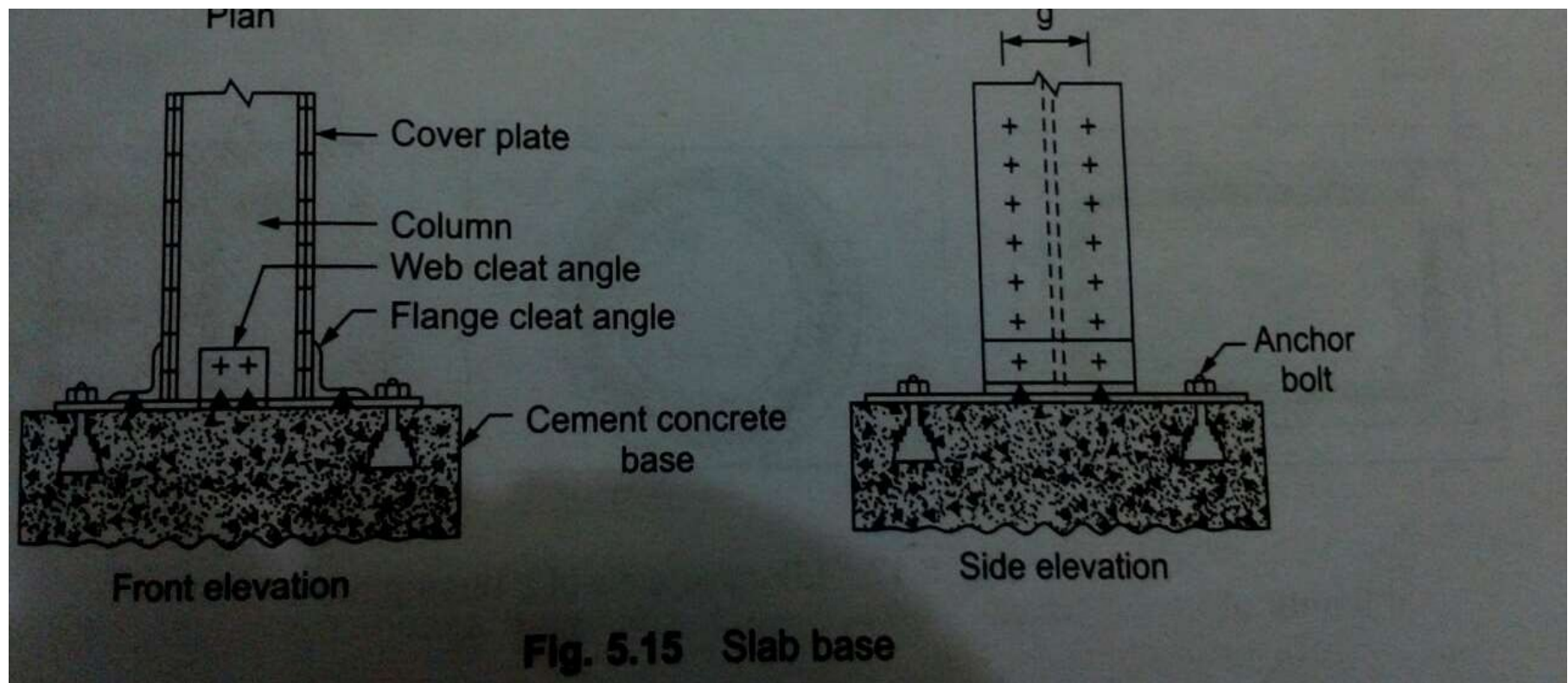
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# Slab base

- 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
- Column base is connected by welding or bolted angle iron cleats.



- Base plate is subjected to
- bending in two principal directions under action of load exerted by the column on base plate and
  - upward pressure by concrete foundation

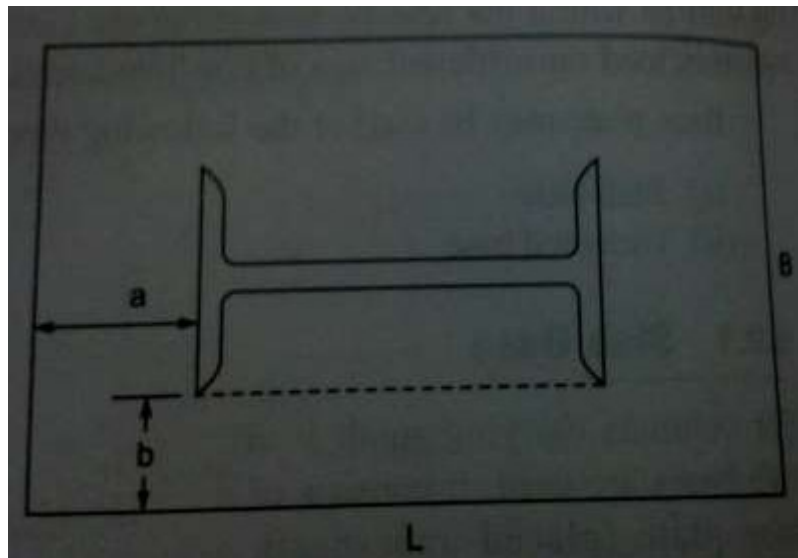
# Specifications by IS Code

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

$$< t_f$$

$$t_s = \sqrt{\frac{W_u}{5w(a - 0.3b)}} \cdot \gamma / f_y$$

Where  $w$  = uniform pressure from below slab base  
 $a, b$  = larger and smaller projection, respectively

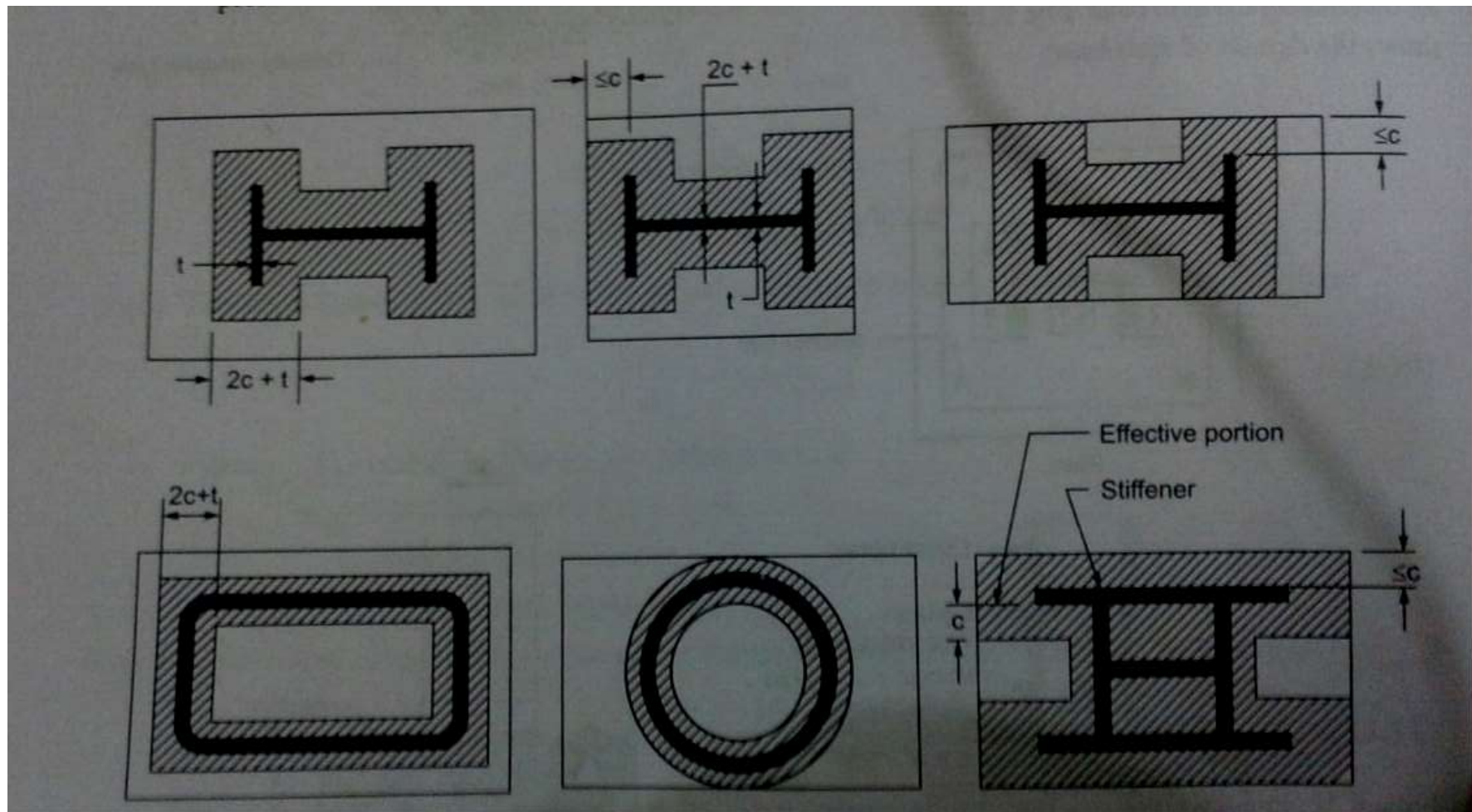


in figure beyond the rectangular describing the column

$t_s$  = 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;

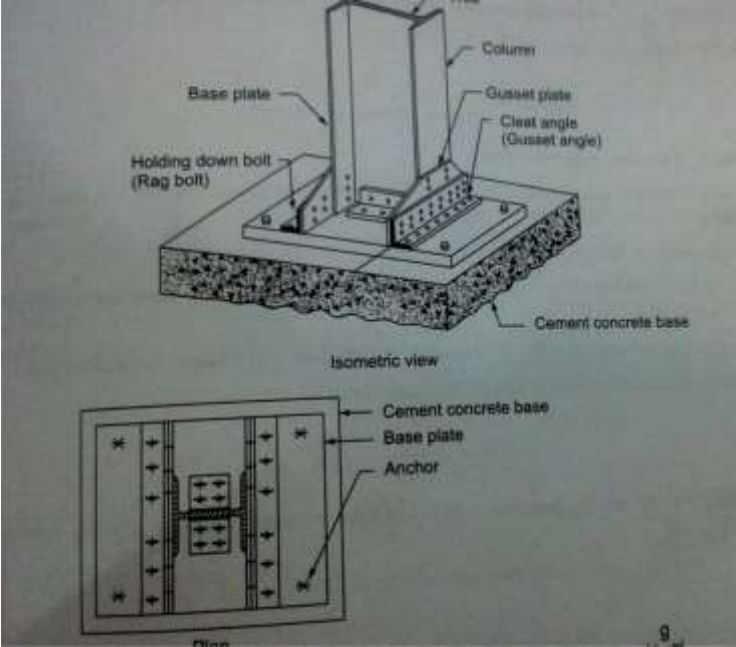
- If size of the base plate is larger than required to limit the bearing pressure on the base support, an equal projection  $c$  of the base plate beyond the face of the column and gusset may be taken as effective in transferring the column load as given in figure such that the bearing pressure of the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

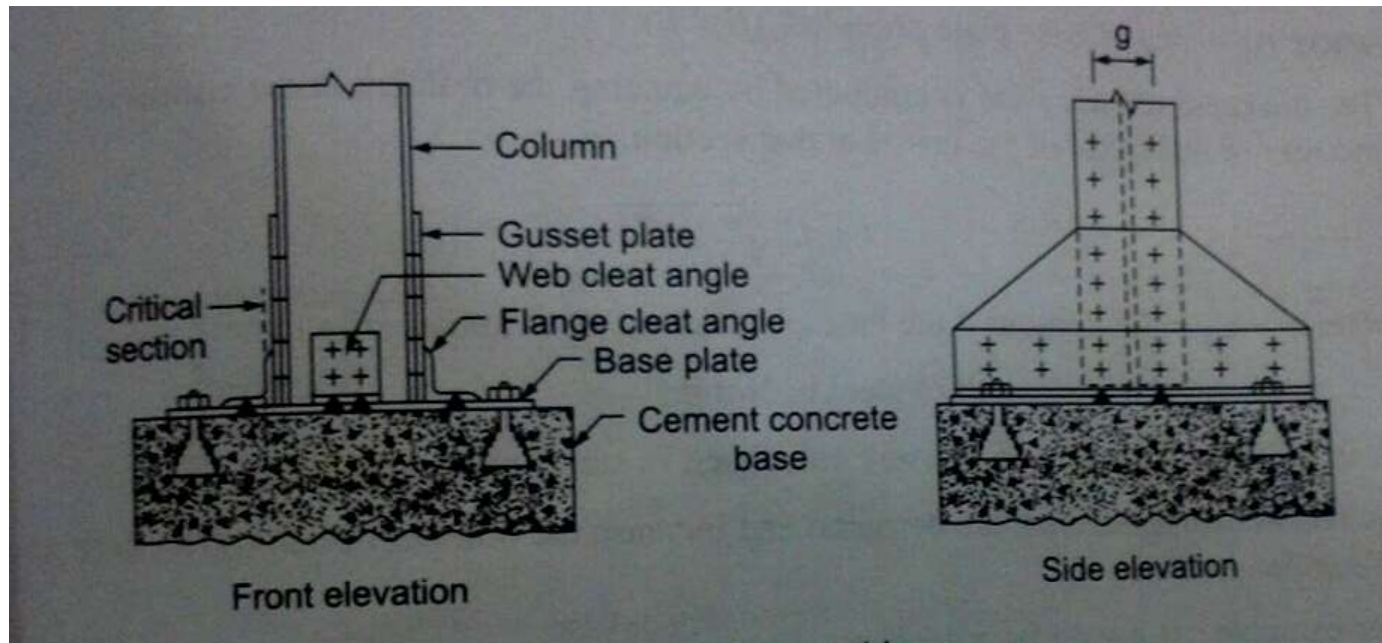


## Effective area of base plate

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# 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)
  - Cleat angles are used to connect column base plate.
- 
- The diagram illustrates the components of a gusseted base. The isometric view shows a column with a base plate, gusset plates, and cleat angles (also called gusset angles) connecting them. A holding down bolt (lag bolt) is shown securing the base plate to the cement concrete base. The plan view shows the layout of the base plate, gusset plates, and anchors within the cement concrete base.
- Here the thickness of the base plate will be less than slab base for the same axial load as the bearing area of the column on the base plate increases by the gusset plate



- 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 ( $f_{ck}$ ) the bearing strength of concrete can be determined by  $0.45f_{ck}$
2. The area required of base plate is computed by

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

**Bearing strength of concrete**

where  $P_u$  = factored load on column

3. The size of plate is calculated from  $A(\text{plate})$ . 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

**$L = \text{depth of section}(d) + 2 (\text{thickness of gusset plate} + \text{leg length of angle} + \text{overhang})$**  (for bolted plate)

**$L = \text{depth of section } (d) + 2(\text{thickness of gusset plate}) + \text{overhang}$**  ( for welded plate)

the other dimension B can now be calculated as

$$\mathbf{B = A(plate) / L}$$

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

$$\mathbf{\text{Bearing pressure , } w = P / A_1}$$

where  $A_1$  = area of base plate provided,  $(B \times L)$

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

$$t = C1 \sqrt{2.7 w / f_y}$$

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

$f_y$  = yield strength of the steel in N/mm<sup>2</sup>

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 for eg. If  $n = 13.04$ ; provide 16 nos.

# Elastic Buckling of Columns

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

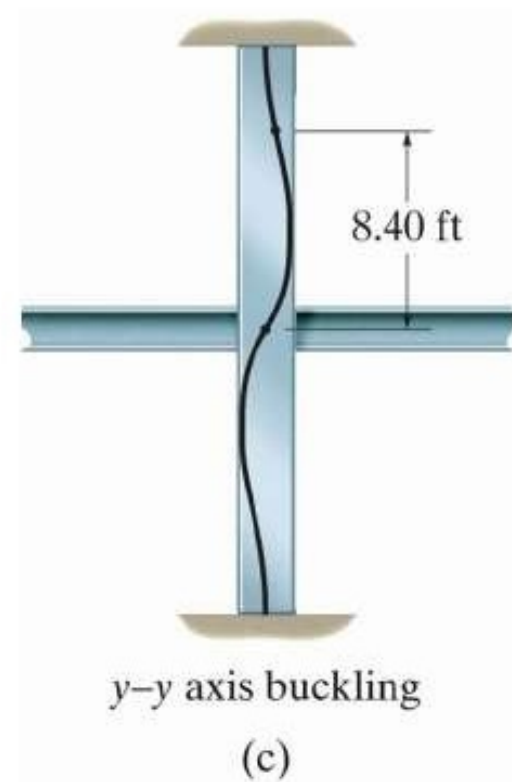
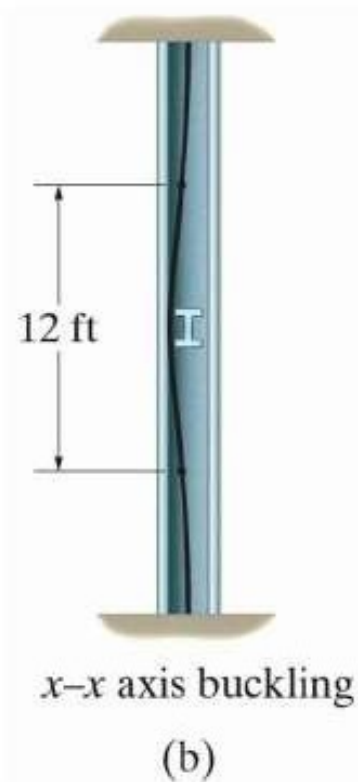
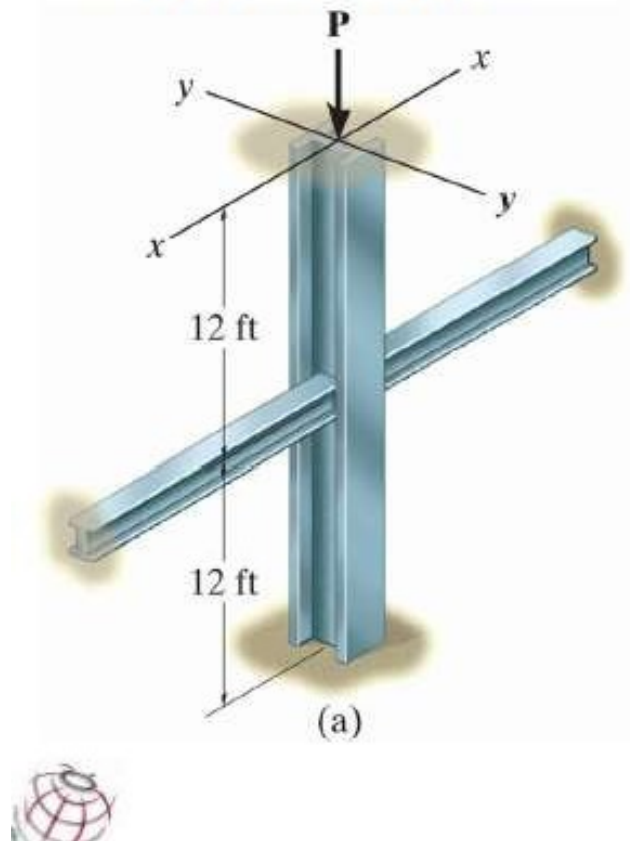
$$P_E = \frac{\pi^2 EI}{L^2}$$

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

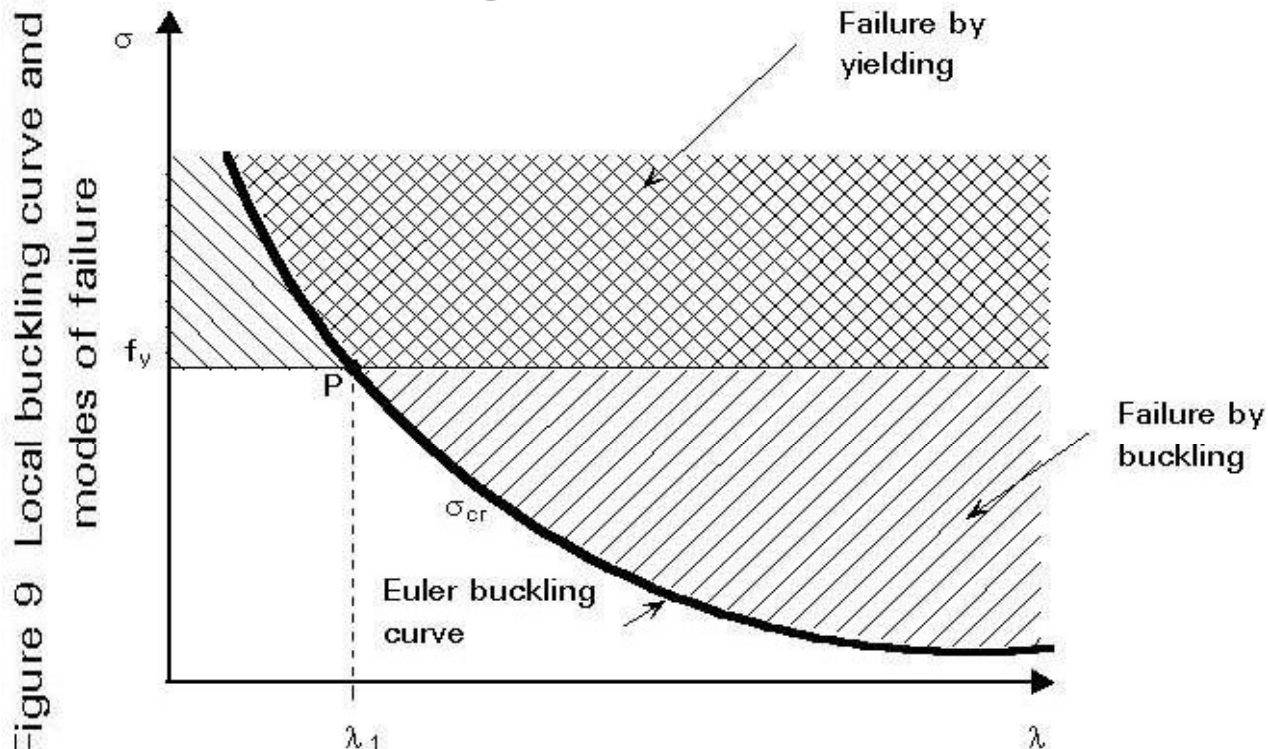
$$F_E = \frac{\pi^2 E}{(L/r)^2}$$

- Tendency of compression members to buckling is governed by  $L/r$

# Effective lengths in different directions



# Elastic Buckling of Columns



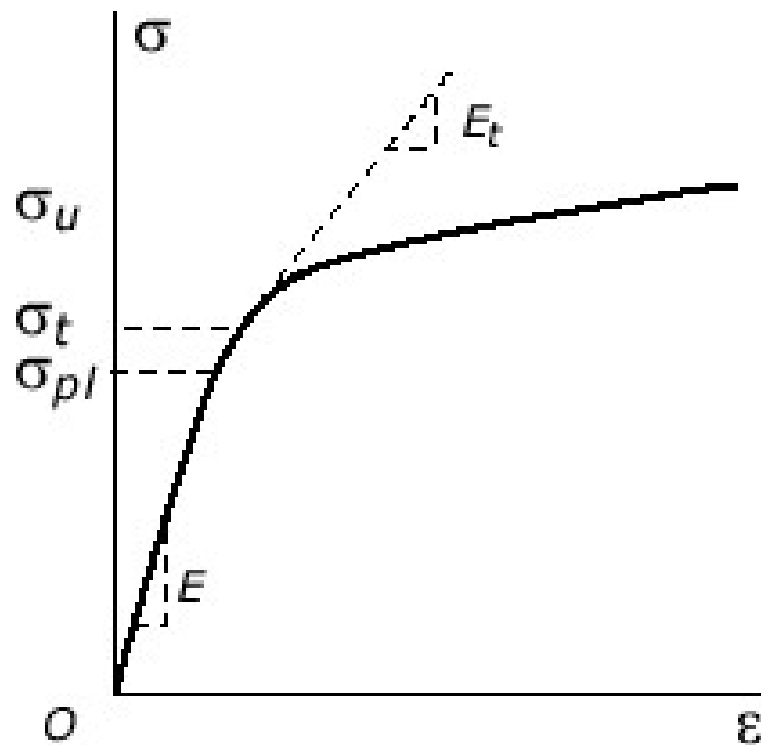
The intersection point P, of the two curves represents the maximum theoretical value of slenderness of a column compressed to the yield strength. This maximum slenderness (sometimes called Euler slenderness)

# 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 ultimate strength. This kind of situation is called *inelastic buckling*.

# Inelastic Buckling of Columns

## Tangent-Modulus Theory



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

# 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 modulus depends on the stress, which is a function of the bending moment that varies with the displacement  $w$ .

# 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 side is still below the elastic limit.

# 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 tangent-modulus theory.

# Inelastic Buckling of Columns

## Reduced Modulus Theory

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

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

The corresponding critical stress is,

$$\sigma_r = \frac{E_r \pi^2}{(L_{\text{eff}}/r)^2}$$

# 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 function of the bending moment that varies with the displacement  $w$ .

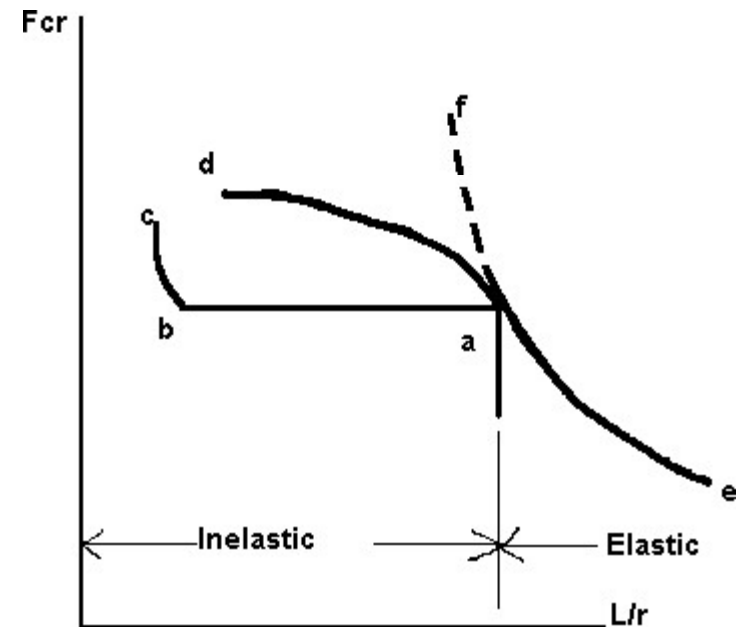
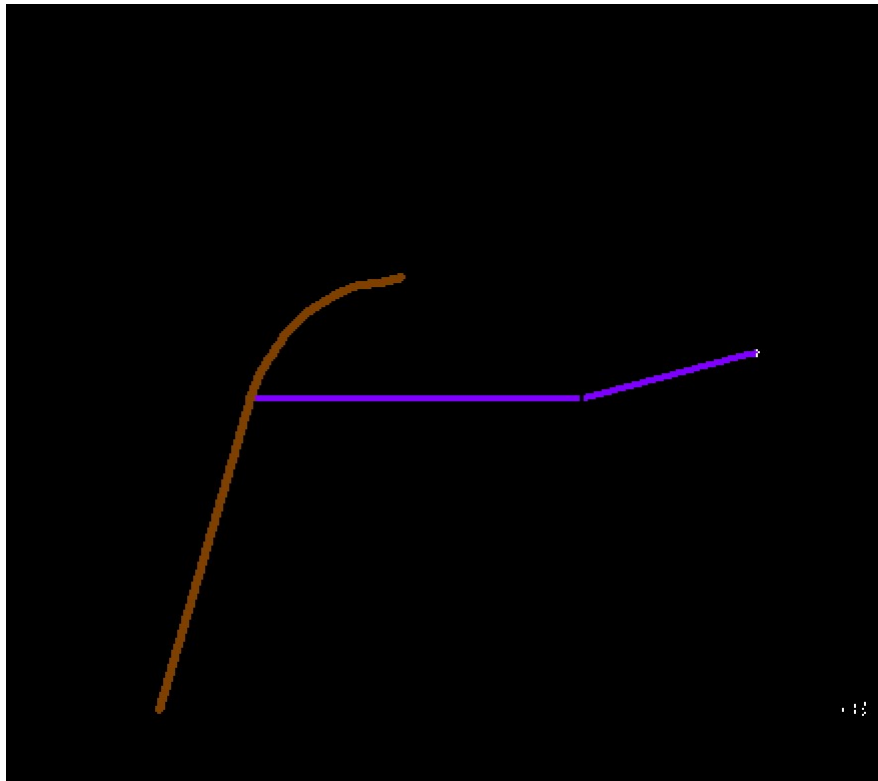
# Inelastic Buckling of Columns

## 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.
- This is the reason why many design formulas are based on the overly-conservative tangent-modulus theory.

# Inelastic Buckling of Columns

## Shanley's Theory



# Factors effecting Buckling

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

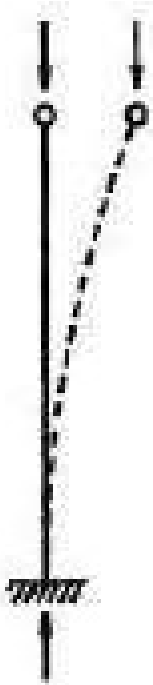
# Factors effecting Buckling

## 1. End Connections

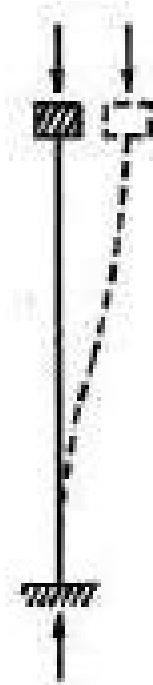
- Rotation of ends of columns in building frames is usually limited by beams connecting to them.

# Factors effecting Buckling

## 1. End Connections: Effective length



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



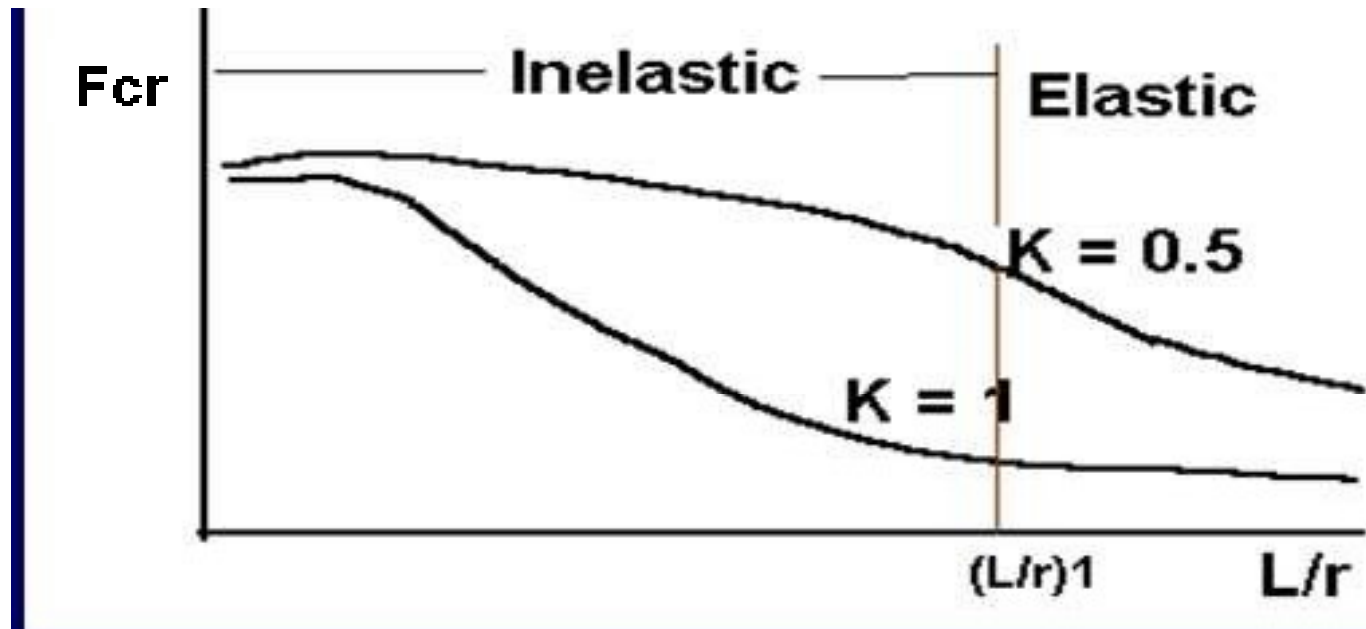
$$P_E = \frac{\pi^2 EI}{L^2}$$

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

- $KL$  is called **effective length** of column and  $K$  **effective length factor**.

# Factors effecting Buckling

## 1. End Connections: Effective length



- A column with fixed ends can support four times as much load as a column with pinned ends
- This benefit decrease with decreasing  $L/r$  until  $F_{cr}$  finally becomes virtually independent of  $K$

# Factors effecting Buckling

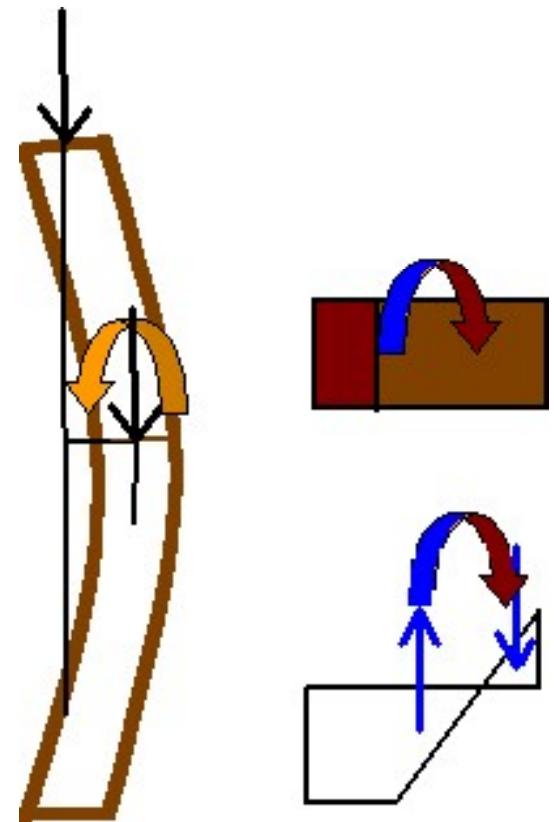
## 2. Effect of initial crookedness

- The initial out-of-straightness is also termed "initial 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.



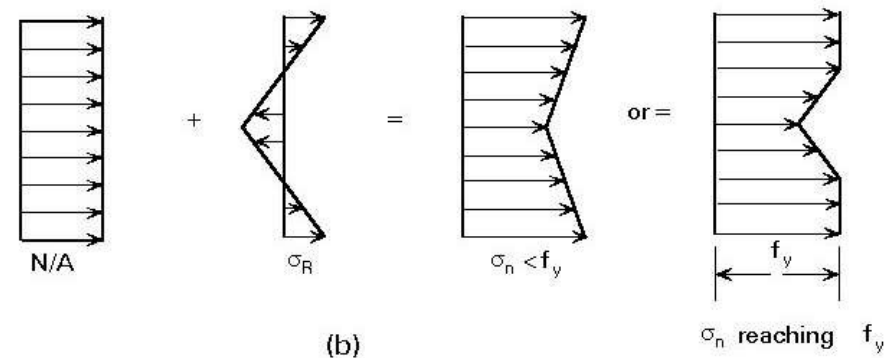
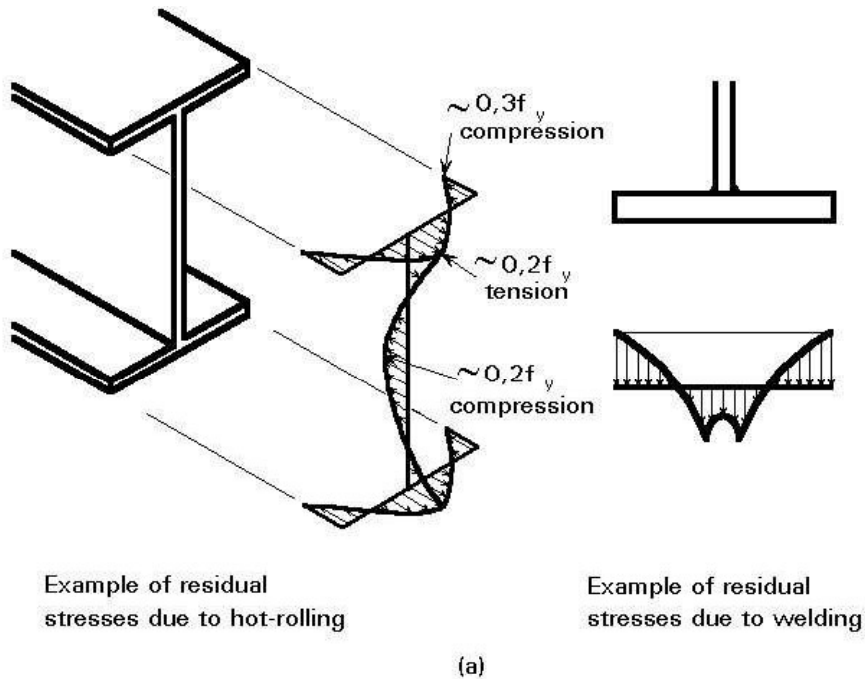
# Factors effecting Buckling

## 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

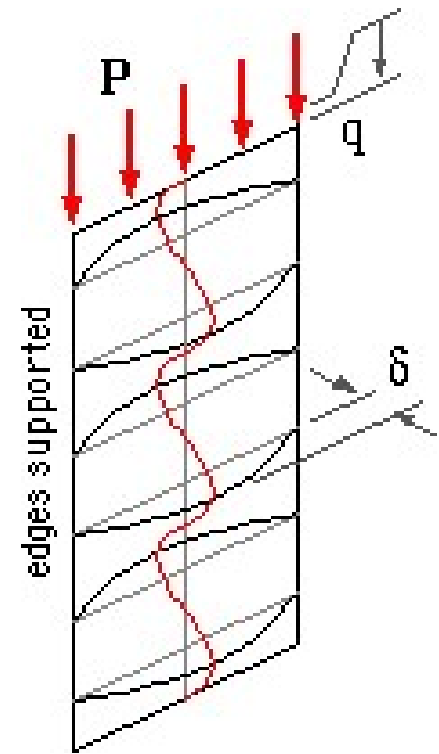


Combination with axial stresses

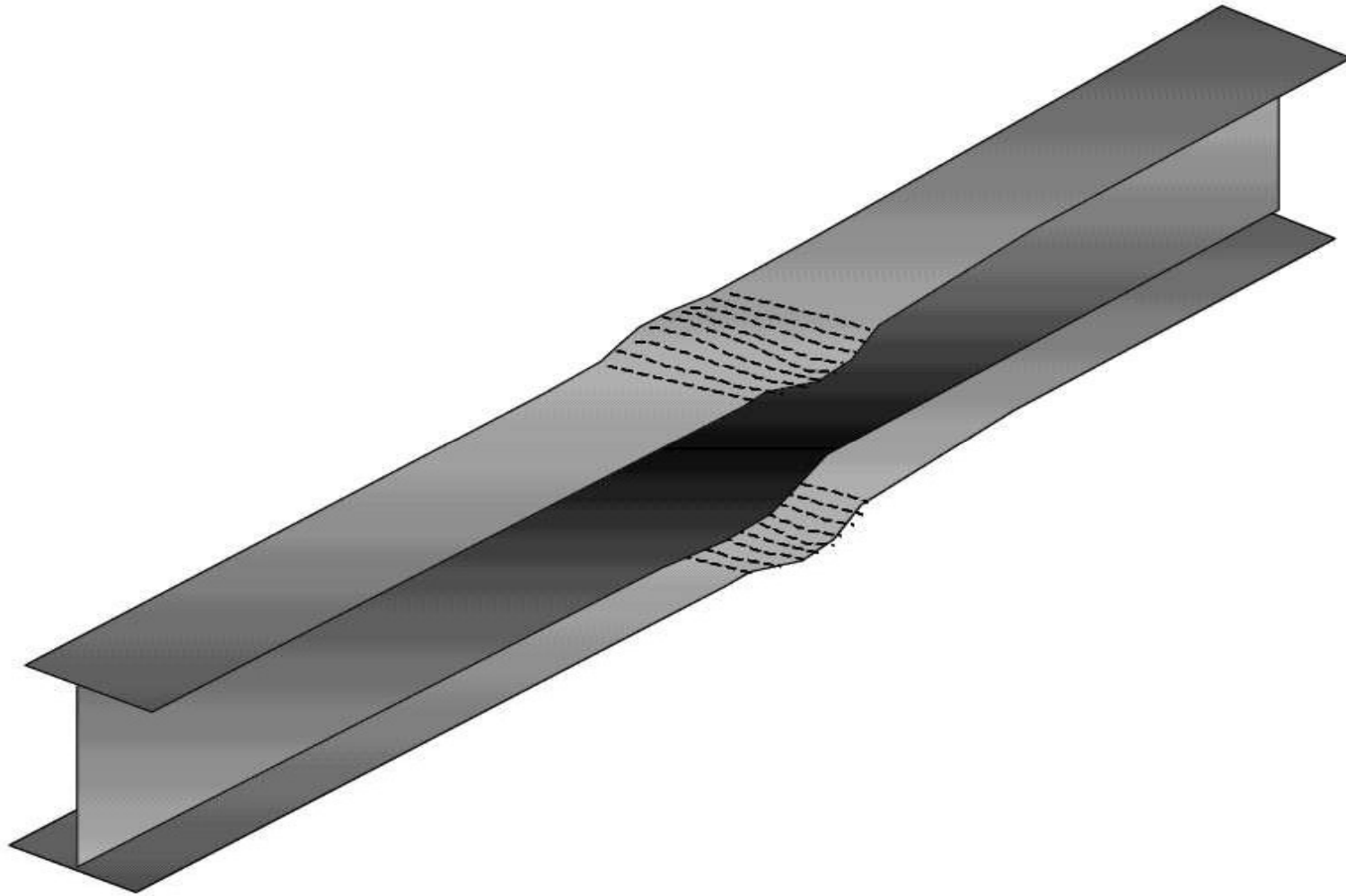
Figure 12 Example and effect of residual stresses

# 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.



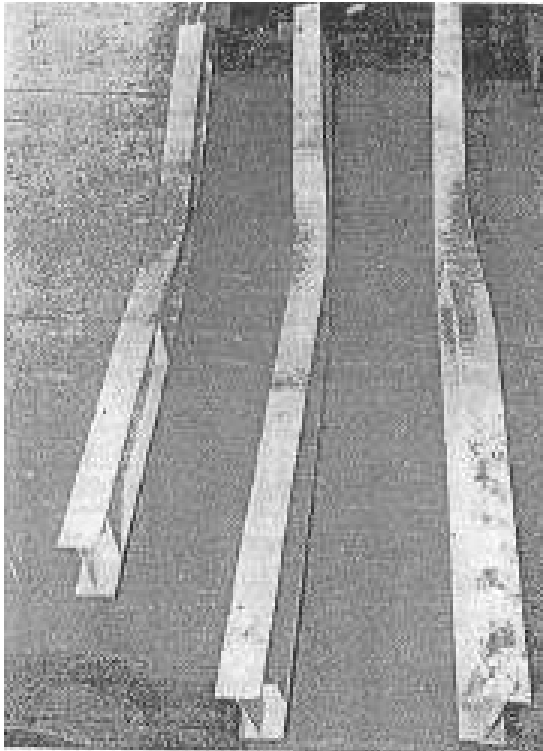
# Local Buckling



**Figure 4.** Local buckling of columns

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# Local Buckling



Laterally buckled beams



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.