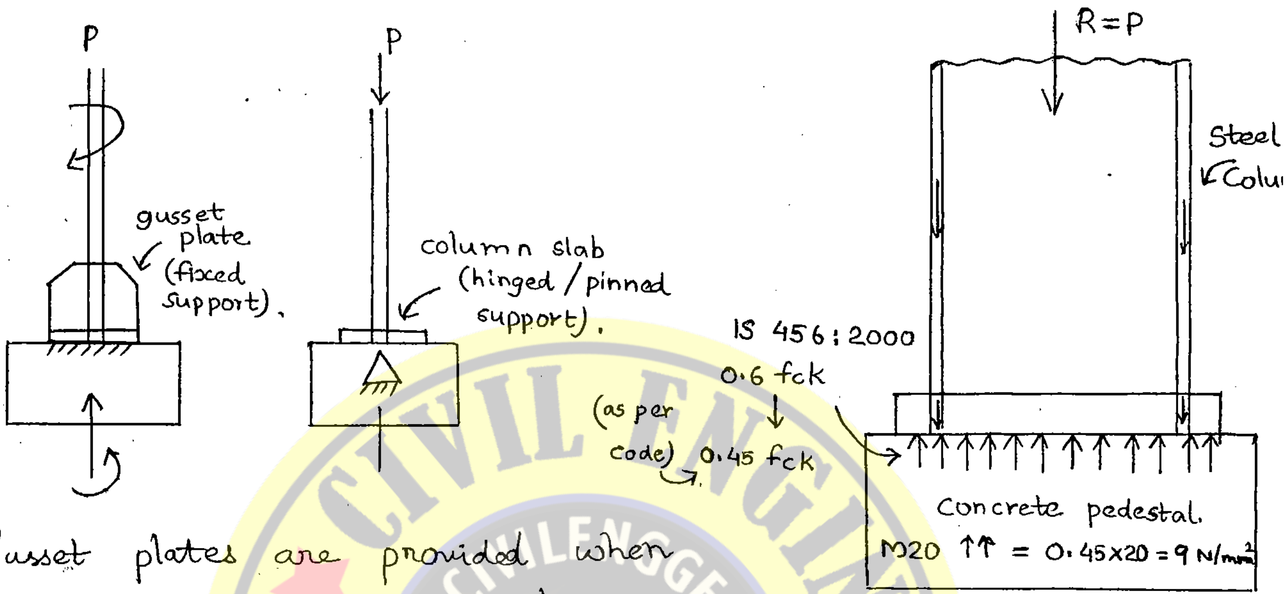
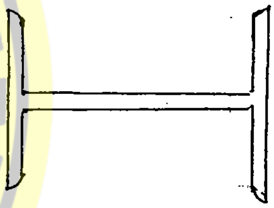
7. COLUMN BASES & COLUMN SPLICES



- Gusset plates are provided when moment is acting on the column.



- Column pl slabs are used when only axial loads act on the column.

→ Types of Column Bases:

– Slab bases :-

To be provided when column is subjected to direct axial loads only.

– Gusseted base :-

To be provided, when column is subjected to heavy axial loads and subjected to axial loads with moments

⊙ Bearing strength of concrete (as per code) = 0.45 fck

$$\frac{P}{A} \leq 0.45\ fck.$$

$$\therefore A = \frac{P}{0.45\ fck}.$$

Min. base area required to safely transmit the axial load $= \frac{P}{0.45}$

3rd Oct,
FRIDAY

→ Design of Slab base:

L → length of slab base

B → width of slab base.

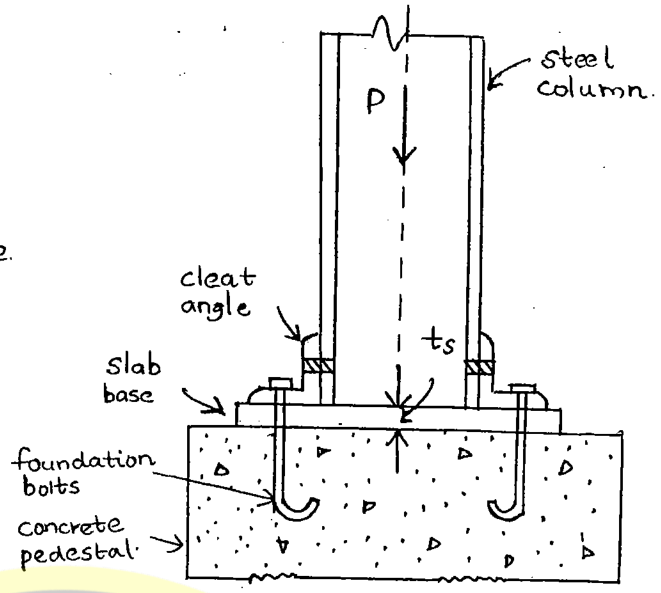
t_s → thickness of slab base.

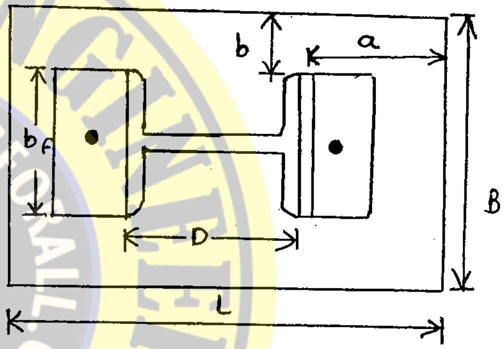
D → depth of steel column section

b_f → width of column flange

a → bigger projection of slab base beyond the steel column.

b → smaller projection of slab base beyond the steel column.





Step 1: Assume suitable grade of concrete. Bearing strength of concrete taken as $0.45 f_{ck}$

Step 2: Area of slab base required = $\dfrac{\text{Factored column load}}{\text{Bearing strength of concrete}}$

$$\boxed{A = \frac{P}{0.45 f_{ck}}}$$

– For square slab base,

Side of slab base, $L = B = \sqrt{A}$

– For rectangular slab base,

$$A = L \times B$$

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

If a & b (projections) are same, the thickness required for slab base will be optimum.

ie, condition for optimum thickness of slab base :

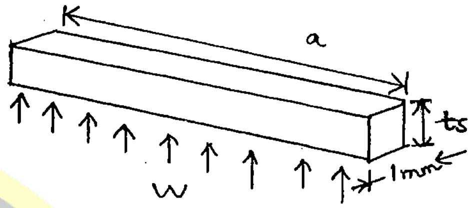
$$a = b$$

– upward pressure from concrete pedestal is w

$$w = \frac{P}{\text{Provided area of slab base}}$$

Consider 1 mm width of slab base,

Net moment due to upward pressure, $M = \frac{wa^2}{2} - \mu \frac{wb^2}{2}$



$$= \frac{w}{2}\left(a^2 - \mu b^2\right)$$

$$\therefore \quad M = \frac{w}{2}\left(a^2 - 0.3\, b^2\right) ; \quad \mu = 0.3 \text{ for steel.}$$

– Design bending strength of slab base,

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

$$M_d = 1.2\, Z_e \frac{f_y}{\gamma_{mo}}$$

Design condition is :-

$$\frac{w}{2}\left(a^2 - \mu b^2\right) = 1.2\, Z_e \frac{f_y}{\gamma_{mo}}$$

$$= 1.2 \frac{t_s^2}{6} \frac{f_y}{\gamma_{mo}}$$

$$Z_e = \frac{I}{y} = \frac{b t_s^3}{12} \Big/ t_s/2 = \frac{t_s^2}{6}.$$

$\Rightarrow$ Thickness of slab base, $t_s = \sqrt{\dfrac{2.5\, w\left(a^2 - 0.3\, b^2\right)\gamma_{mo}}{f_y}}$

$t_s > t_f$; $t_f \rightarrow$ thickness of column flange.

→ Design of Gusseted Base.

$L \rightarrow$ length of base plate

$B \rightarrow$ width of base plate

$P \rightarrow$ Factored column load.

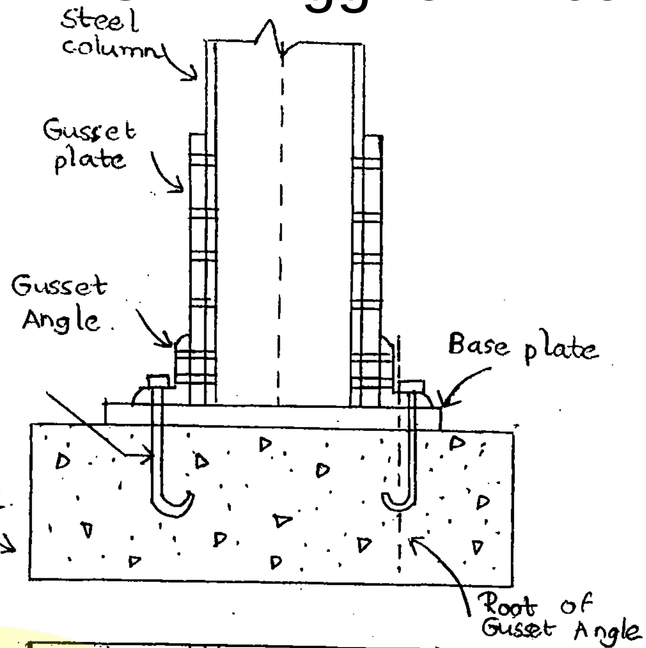
Step 1: Assume suitable grade of concrete.

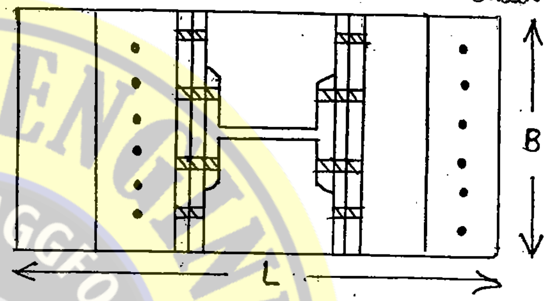
Bearing strength of concrete $= 0.45\, f_{ck}$.

Area of base plate required,

$$A = \frac{\text{Factored column load}}{\text{Bearing strength of concrete}}$$

$$\boxed{A = \frac{P}{0.45\, f_{ck}}}$$





L = length of base plate.
= width of base plate parallel to web.

L = depth of steel column + 2* thickness of GP + 2* leg width of gusset angle + 2* min. overhand (for bolted connection)

L = depth of steel column + 2* thickness of GP + 2* min overhang (for welded connection)

$$\text{Width of base plate, } B = \frac{\text{area of base plate}}{\text{length of base plate}}$$

– Upward pressure from concrete pedestal,

$$w = \frac{\text{Factored column load}}{\text{Provided area of base plate.}}$$

– Moment due to upward pressure for 1 mm width of base plate, $M = w \cdot c \cdot \frac{c}{2} = \frac{wc^2}{2}$

– Design bending strength of base plate,

$$M_d = Z_p \cdot \frac{f_y}{\gamma_{mo}} = 1.2\, Z_e \frac{f_y}{\gamma_{mo}}$$

c → cantilever projection beyond the root of gusset angle (for bolted connection)

$$M \leq M_d$$

$$\Rightarrow \frac{wc^2}{2} \leq 1.2 \times \frac{t_s^2}{6} \times \frac{f_y}{\gamma_{mo}}$$

$$t_s = c\sqrt{\frac{2.75\,w}{f_y}}$$

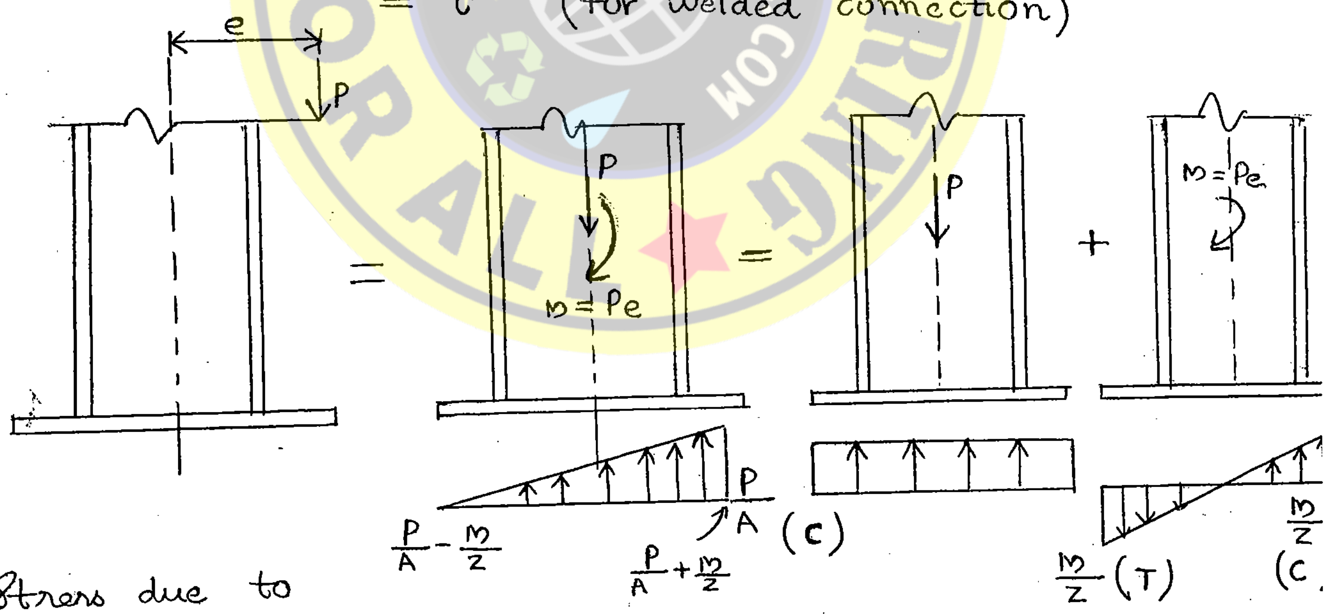
– Thickness of base plate (t_b):

$t_b = t - $ thickness of gusset angle (for bolted connection)

$= t$ (for welded connection)

Q-57

1.



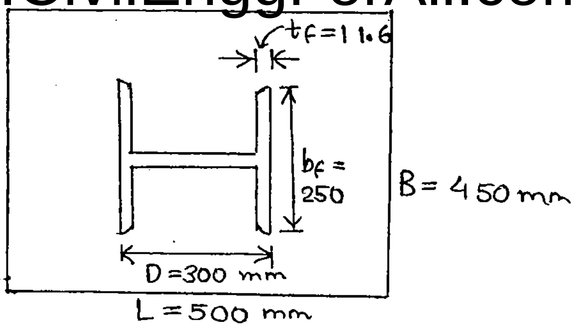
Stress due to direct axial load $= \frac{P}{A} = \frac{P}{L \times B}$ (compression).

Stress due to BM $= \pm \frac{M}{I} y = \frac{6Pe}{BL^2}$

Combined stress due to P & M $= \frac{P}{A} \pm \frac{M}{Z} = \frac{P}{LB}\left(1 \pm \frac{6e}{L}\right)$

3. $a = \frac{L - D}{2} = \frac{500 - 300}{2}$

$= 100$ mm.



$b = \frac{B - b_f}{2} = \frac{450 - 250}{2}$

$= 100$ mm.

Thickness of base plate, $t_s = \sqrt{\frac{2.5 w (a^2 - 0.3 b^2) \gamma_{mo}}{f_y}}$

$= \sqrt{\frac{2.5 \times 9 (100^2 - 0.3 \times 100^2) \times 1.10}{250}}$

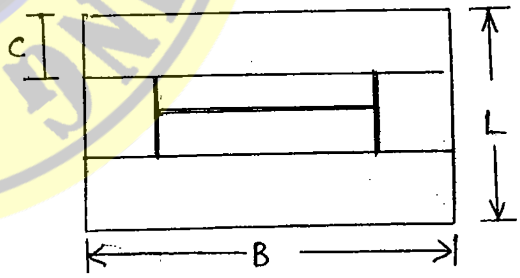
$= \underline{\underline{26.3 \text{ mm}}} \; (\geq t_f = 11.6 \text{mm})$.

4. $t = c \sqrt{\frac{2.75 w}{f_y}}$

For all the 4 options given, area is same.

$\therefore w = \frac{P}{A}$ is also same.

$\therefore t \propto c$.



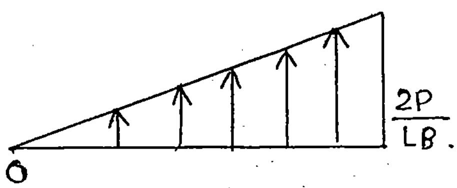
a) $c = \frac{600 - 140}{2} = 230$ mm

b) $c = \frac{600 - 400}{2} = 100$ mm.

c) $c = \frac{500 - 140}{2} = 180$ mm.

d) $c = \frac{720 - 400}{2} = 160$ mm.

05. Combined stress $= \frac{P}{A} \pm \frac{M}{Z}$.



$= \frac{P}{LB} \left(1 \pm \frac{6e}{L}\right)$.

$= \frac{P}{LB} \left(1 \pm \frac{6}{L} \cdot \frac{L}{6}\right) = 0, \; \frac{2P}{LB}$

06. For Fe 410 grade steel,

$f_u = 410$ MPa , $f_y = 250$ MPa.

For M25 grade concrete,

$f_{ck} = 25$ MPa.

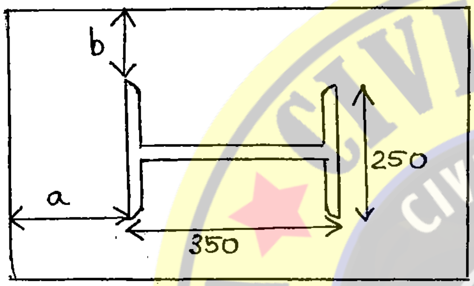
$D = 350$ mm, $b_f = 250$ mm, $t_f = 11.6$ mm.

$P = 2000$ kN

Area of base plate (slab base) $= \dfrac{P}{0.45 f_{ck}}$

$= \dfrac{2000 \times 10^3}{0.45 \times 25}$

$= 177.8 \times 10^3$ mm^2



$A = L \times B = (D+2a)(b_f + 2b)$.

$= (350+2a)(250+2b)$. $(a = b)$.

177777 $= (350+2a)(250+2a)$.

$\Rightarrow a = b = 62.29$ mm $\approx \underline{\underline{65 \text{ mm}}}$

Length of base plate, $L = D + 2a$

$= 350 + 2 \times 65 = \underline{\underline{480 \text{ mm}}}$.

Width of base plate, $B = b_f + 2b$.

$= 250 + 2 \times 65 = \underline{\underline{380}}$ mm

Upward pressure, $w = \dfrac{P}{\text{provided area of base plate}} = \dfrac{2000 \times 10^3}{480 \times 380}$

$= 10.96$ N/mm^2

Thickness of base plate, $t_s = \sqrt{\dfrac{2.5\, w\, (a^2 - 0.3 b^2)\, \gamma_{mo}}{f_y}}$

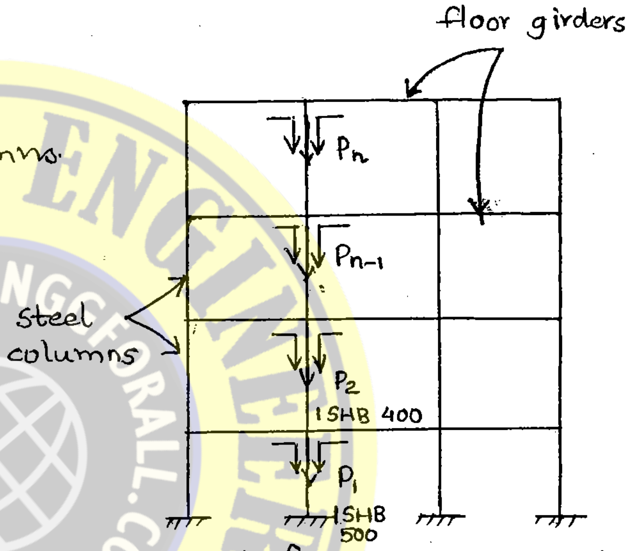
$= \sqrt{\dfrac{2.5 \times 10.96\,(65^2 - 0.3 \times 65^2) \times 1.1}{250}}$

$= 18.88 \approx \underline{\underline{20\text{ mm}}}$ $(> t_f = 11.6\text{mm})$

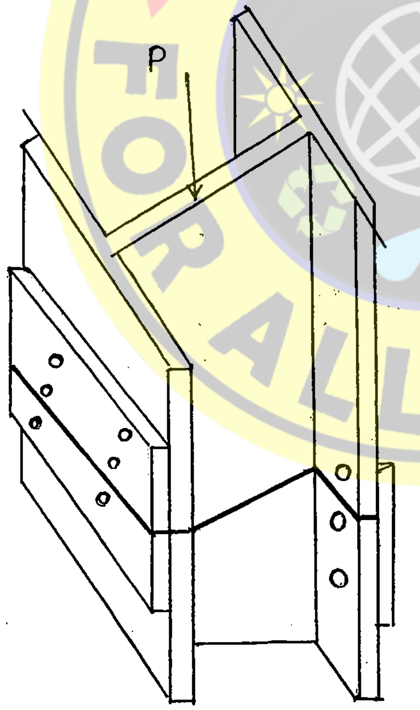
→ COLUMN SPLICE

$P_1, P_2, \cdots P_{n-1}, P_n$ factored column loads in various columns.

$P_1 > P_2 > P_3 \cdots > P_{n-1} > P_n$.

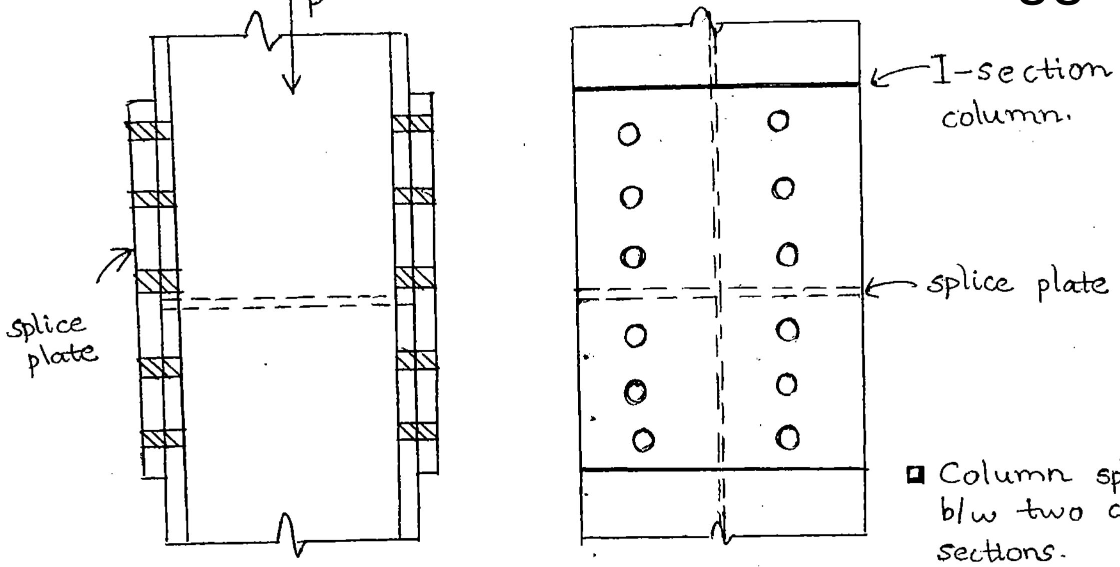


Steel frame for multi-storeyed building.



Column splice is a joint for steel column to be provided for extending length of column section. (length available from Indian Rolling mill is less) and also provided when two different sizes of columns are to be joined.

Column splice is should be designed as a short column and recommended to locate just above floor level.

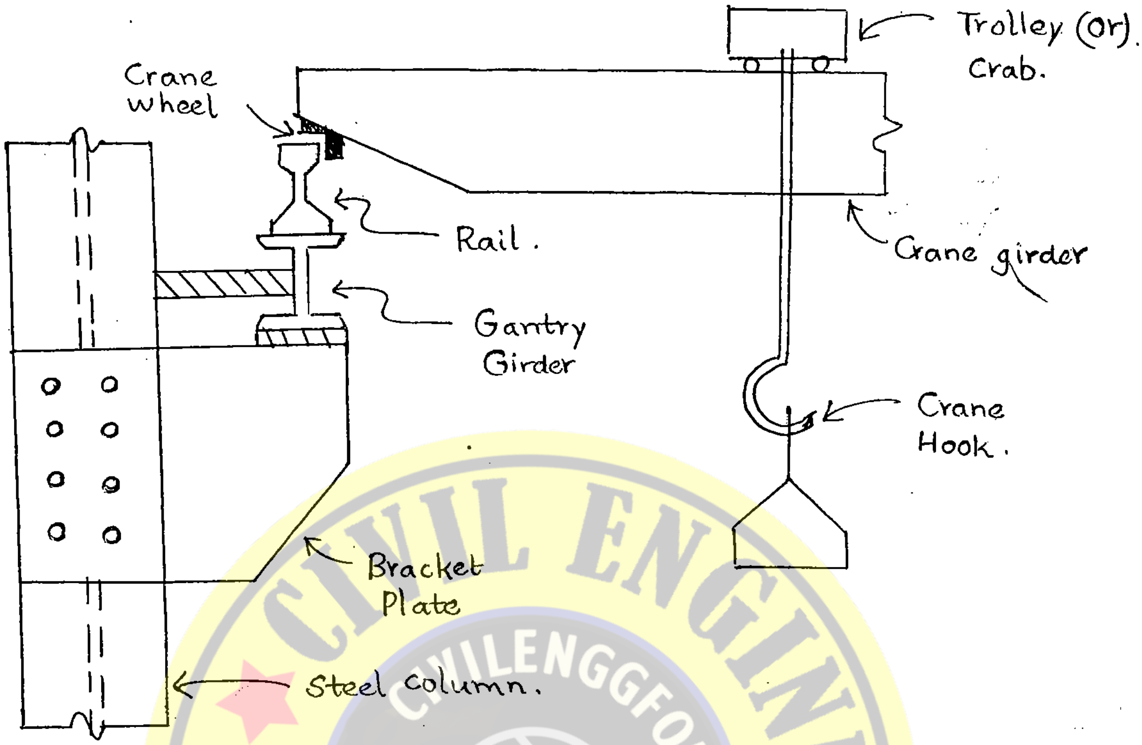


■ Column splice b/w two column sections.

If end of a column is machined or milled, theoretically no connecting system is required. But in practise, splice must be designed for 50% factored column load.

3rd Oct,
FRIDAY

10. GANTRY GIRDERS



EOT Cranes :- Electrically Operated Overhead Travelling Cranes.

MOT or HOT Cranes :- Manually (or) Hand Operated Overhead Travelling Cranes.

→ Design loads on Gantry Girder:
- Vertical loads (Gravity Loads)
- Lateral loads (Surge Loads)
- Longitudinal loads (Drag Loads)
- Impact loads (Sudden Actions)

→ Additional loads on Girder due to Impact effect

(i) Vertical Loads	Additional loads
- EOT cranes	25% max. static wheel load.
- MOT cranes.	10% max. static wheel loads.

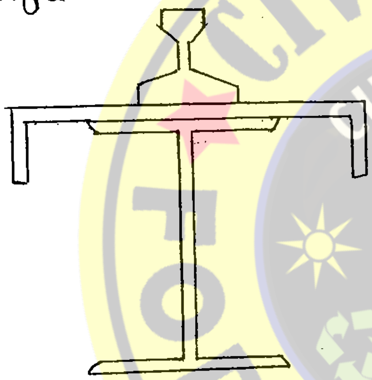
	Additional Loads
(ii) Lateral Loads	
– EOT cranes	10% (wt. of crab + wt. lifted crane)
– MOT cranes	5% (wt. of crab + wt. lifted crane)
(iii) Longitudinal Loads	5% max. static wheel load.

→ Types of Sections

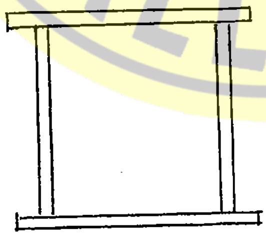
– ISWB : to resist more lateral loads.

$$(I_{yy})_{ISWB} > (I_{yy})_{ISLB/ISJB/ISMB/ISHB}$$

– Reinforced with channel section.



– Box Girder : for more torsional resistance.



→ Limiting Deflections (IS 800: 2007)

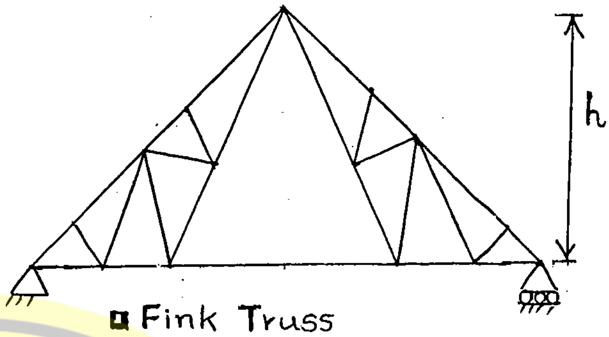
	Δlimit	
– For MOT (or) HOT cranes	L/500	L → span of gantry girder
– For EOT cranes with crane capacity upto 50 t or 500 kN	L/750	
– EOT cranes with crane capacity more than 50 t (or) 500 kN	L/1000	
– For other moving equipments like charging cars etc	L/600	

4th Sept,
SATURDAY

11. ROOF TRUSSES

→ Selection criteria for Roof truss :

- Span of the truss.
- Pitch of the truss.



■ Fink Truss

Pitch of the truss depends on :

(i) Type of roof covering material to be used for truss (like AC sheets, GI sheets, plastic sheets)

(ii) Lightening & ventilation requirement.

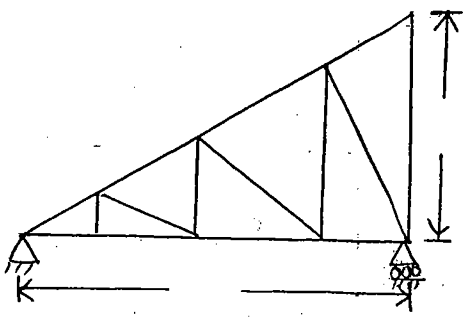
$$\text{Pitch} = \frac{\text{Rise}}{\text{Span}} = \frac{h}{L}$$

$$\text{Slope} = \tan\theta = \frac{\text{Rise}}{\text{Half span}} = \frac{h}{L/2} = \frac{2h}{L}$$

$\boxed{\text{Slope} = 2 * \text{pitch}}$ (for symmetric truss)

* North Light Roof Truss

- Spans upto 10 m
- Day light is main criteria for selection of North light roof truss.

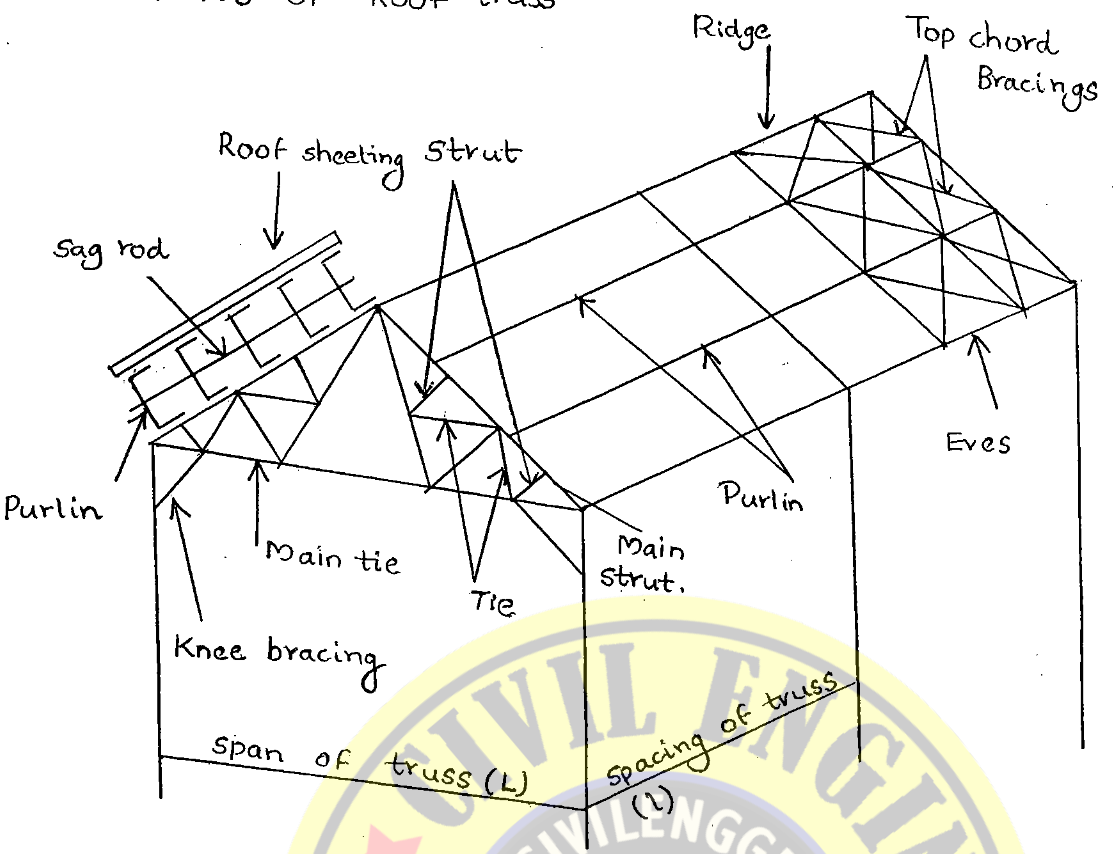


■ North light Roof Truss

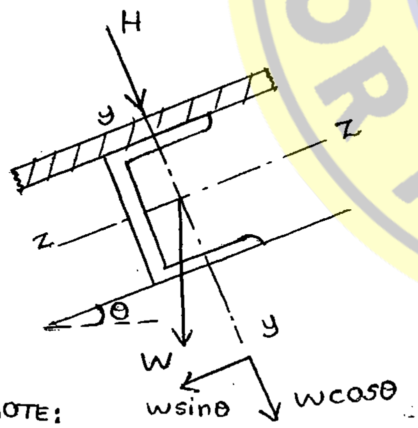
$$\text{Pitch} = \frac{h}{L}$$

$$\text{Slope} = \tan\theta = \frac{h}{L} \Rightarrow \boxed{\text{Pitch} = \text{Slope}}$$

→ Elements of Roof truss



⊙ Top Chord bracings are not required when masonry walls are provided.



$$M_{zz} = \frac{\gamma_L (H + w\cos\theta)\, l^2}{10}$$

$$M_{yy} = \frac{\gamma_L (w\sin\theta)\, l^2}{10}$$

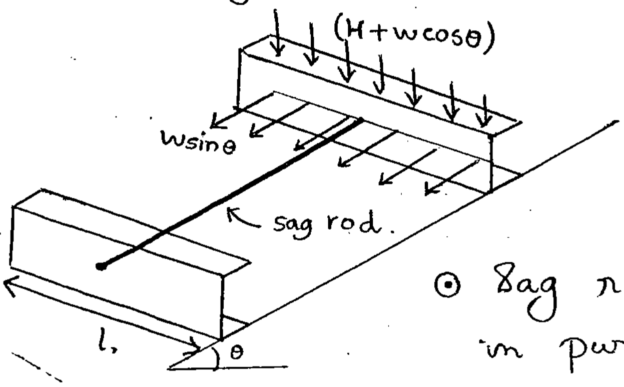
w → gravity load due to DL & LL (N/m or kN/m)

H → load due to wind pressure (N/m or kN/m)

γ_L → load factor.

NOTE:

⊙ Purlin member may be designed as simply supported beam or cantilever beam or continuous beam. IS 800 : 2007 recommends to design as continuous beam subj to biaxial bending moment

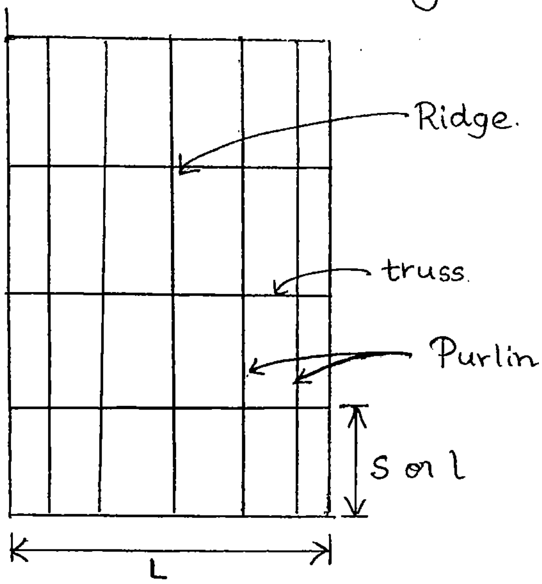


$$M_{yy} = \frac{\gamma_L\, w\sin\theta\, (l/2)^2}{10}$$

$$= \frac{1}{4}\,\gamma_L \frac{w\sin\theta\, l^2}{10}$$

⊙ Sag rod will minimise ~~slopes~~ deflection and BM in purlin member about minor principal axis

→ Economical spacing of truss (S or l)



t → cost of truss per unit area

p → cost of purlin per unit area

r → cost of roof sheeting per unit area.

x → total (or) overall cost of roof building per unit area.

$$x = t + p + r$$

⊙ $t \propto \frac{1}{s} \Rightarrow t = \frac{C_1}{s} \quad \therefore C_1 = ts$

$$M = \frac{wl^2}{10} \Rightarrow p \propto s^2$$

⊙ $p = C_2 s^2 \Rightarrow C_2 = \frac{P}{s^2}$

⊙ As spacing increases, no: of joints b/w roofing sheets increase and as a result cost of erection increases.

$$r = C_3 S \Rightarrow C_3 = \frac{r}{s}$$

$$x = t + p + r$$

$$x = \frac{C_1}{s} + C_2 s^2 + C_3 S$$

To have minimum cost of roof building,

$$\frac{dx}{ds} = 0$$

$$\frac{d}{ds}\left(\frac{C_1}{s} + C_2 s^2 + C_3 s\right) = 0.$$

$$-\frac{C_1}{s^2} + 2C_2 s + C_3 = 0$$

$$\frac{C_1}{s^2} = 2C_2 s + C_3.$$

$$\frac{ts}{s^2} = \frac{2P}{s^2} \times s + \frac{r}{s} \Rightarrow \boxed{t = 2P + r}$$

Cost of truss per unit area = 2 * cost of purlin per unit area + cost of roof covering per unit area.

$$\boxed{l \text{ or } s = \frac{L}{3} \text{ to } \frac{L}{5}}$$

→ Design Loads on Roof truss

- design dead loads.
- design live (or) imposed loads.
- design wind loads.
- design snow loads.

* Design Dead Loads (IS 875 : 1987 Part I)

(i) Self weight of purlins.
(ii) Self weight of bracings.
(iii) Self weight of roof sheeting
(iv) Self weight of truss.

Self weight of truss $= 100\ N/m^2$ to $150\ N/m^2$ Plan area
$= \left(\frac{L}{3}+5\right) \times 10\ N/m^2$ Plan area.
(L = span of truss)

* Design Live (or) Imposed loads (IS 875 : 1987 Part II)

- Slope of roof, $\theta \leq 10$

$LL = 1500\ N/m^2$ (If access is provided for repair & maintenance)

$= 750\ N/m^2$ (if access is not provided for repair & maintenance)

- Slope of roof, $\theta > 10°$

$$LL = (750 - 20(\theta - 10))\ N/m^2 \not< 400\ N/m^2$$

* Design Snow Load. (IS 875 : 1987 Part IV)

$SL = 25\ N/m^2$ per every 'cm' depth of snow
$= 2.5\ N/m^2$ per every 'mm' depth of snow.

$\theta > 50°$; Snow load need not to be considered.

5th sept,
SUNDAY

* Design Wind Load. (IS 875: 1987 Part III)

V_b = Basic wind speed in m/s at a height of 10m from MSL

V_z = Design wind speed in m/s at a height z.

$$\boxed{V_z = k_1 k_2 k_3 V_b}$$

where $k_1 \rightarrow$ probability (or) risk factor.
$k_2 \rightarrow$ size, shape and structure factor.
$k_3 \rightarrow$ topography factor.

P_z = Design wind pressure at a height z of a structure.

$$P_z = k \cdot V_z^2$$

$$\boxed{P_z = 0.6\ V_z^2} \qquad (k = 0.6)$$

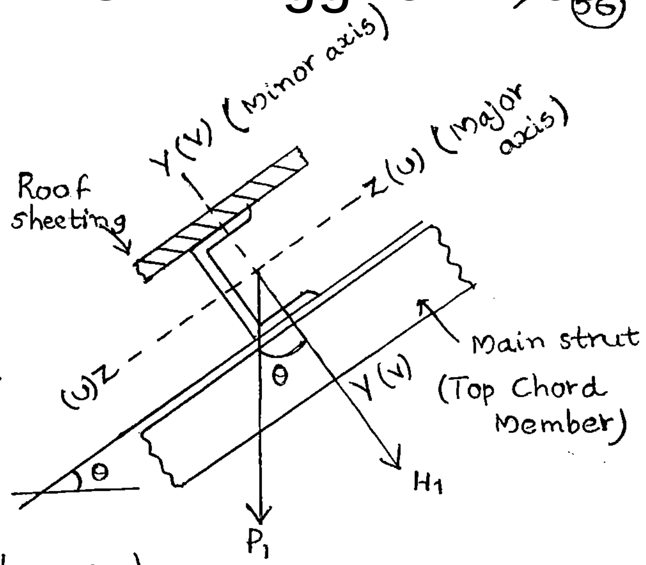
Design wind load $= (C_{pe} - C_{pi}) P_z \cdot A_e$.

where $A_e \rightarrow$ exposed area.

$C_{pe} \rightarrow$ external wind pressure coeffecient (depends on slope of roof)

$C_{pi} \rightarrow$ internal wind pressure coeffecient (depends on degree of permeability (or) no. of openings in structure)

→ Design of Purlin



P_1 = Gravity load due to sheeting and live load. (in kN/m)

H_1 = load due to wind pressure in kN/m.

l = span of purlin (spacing b/w two adjacent trusses).

θ = slope of roof.

IS 800:2007 recommends to design a purlin as a continuous beam subjected to biaxial (unsymmetrical) BMs.

Load along minor axis (YY axis)

$$P = \gamma_L (H_1 + P_1 \cos\theta) \rightarrow M_{zz}$$

Load along major axis (ZZ axis)

$$H = \gamma_L (P_1 \sin\theta). \rightarrow M_{yy}$$

Bending moment about major axis, $M_{zz} = \frac{Pl^2}{10}$.

BM about minor axis, $M_{yy} = \frac{Hl^2}{10}$

* Deflection (or) Serviceability limits: (IS 800:2007)

(i) Brittle cladding (or sheet) $= \frac{L}{180}$

Eg: AC roofing sheet

(ii) Elastic cladding $= \frac{L}{150}$

Eg: GI, Plastic roofing sheet

– Design bending strength about major axis (M_{dz}).

$$M_{dz} = Z_{pz} \cdot \frac{f_y}{\gamma_{mo}}.$$

– Design bending strength about minor axis (M_{dy}).

$$M_{dy} = Z_{py} \cdot \frac{f_y}{\gamma_{mo}}$$

Z_{pz} & Z_{py} : Plastic section modulus of purlin about major & minor axis resptly.

– For safety of purlin, Interaction Equation as per IS 800 : 2007 must be satisfied:

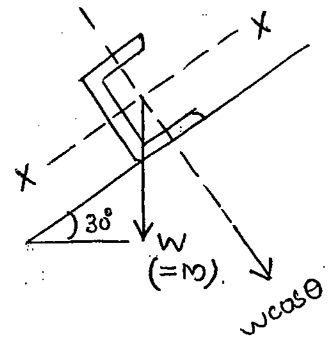
⊙ $\frac{M_{zz}}{M_{dz}} + \frac{M_{yy}}{M_{dy}} \leq 1.0$ $M_{dz} > M_{zz}$, $M_{dy} > M_{yy}$.

⊙ $\Delta_{cal} \leqslant \Delta_{limit}$; Δ_{limit} = limiting deflection.

2 For w, $M_{xx} = M$

For $w\cos 30$, $M_{xx} = M\cos 30$

$$= \frac{\sqrt{3}}{2} M$$



3. $M_{dz} = Z_{pz} \frac{f_y}{\gamma_{mo}}$

$M_{dy} = Z_{py} \frac{f_y}{\gamma_{mo}}.$

Section modulus (Z_{pz} & Z_{py}) is independent of orientation of sections.

04. As per IS 800 : 1984, angle iron purlins were used

* Assumptions :

(i) Slope, $\theta \leq 30$ (exposed area for wind force gets minimised).

(ii) Bending moment about minor axis neglected.

(iii) LL = 750 N/mm^2.

08 Slope = 1

$\tan\theta = 1 \Rightarrow \underline{\underline{\theta = 45^\circ}}$

5th sept,
SUNDAY

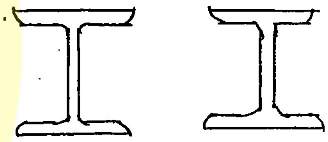
09. PLATE GIRDERS

Beams are designed for bending moments (M) and sometimes shear force (V).

Plate girders are the major beams in a structure. For heavy loads and large spans, M will be very large.

So to meet the design requirement of $M \leq M_d$, higher value of section modulus ($\propto I$), Z, is required. But the maximum depth for an I section given by Indian Rolling mills is 600 mm. So the following sections are considered: $M_d = f(Z \rightarrow I)$.

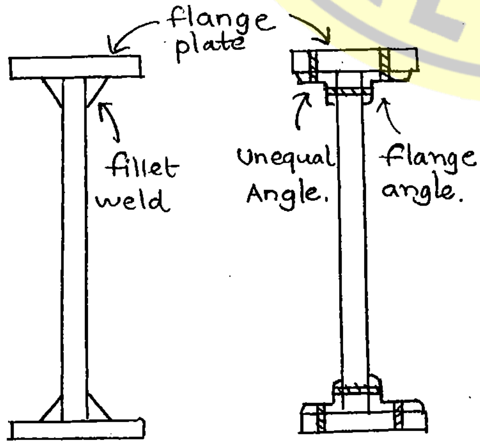
(i) Two I - sections placed side-by-side.



(ii) Plate Girder (spans 20 m - 100 m)

(iii) Truss Girder (spans > 100 m).

→ Plate Girders



▪ welded PG ▪ Bolted PG

Unequal angles are provided to:

(i) increase moment of inertia.

⊙ Weight of bolted/riveted plate girder $= \frac{W}{300}$ kN/m

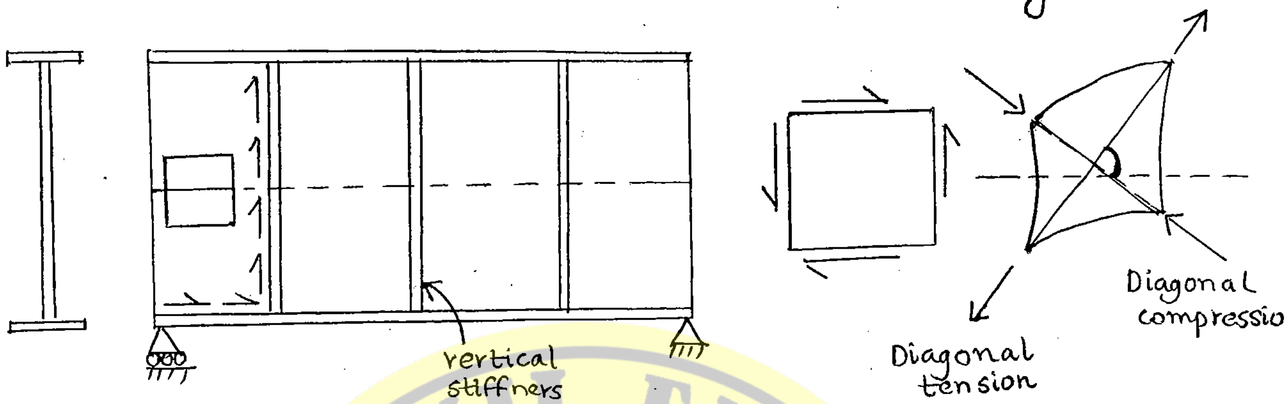
⊙ Weight of welded plate girder $= \frac{W}{400}$ kN/m

w → superimposed load in kN

- For plate girders, $\frac{d}{t_w}$ ratio will be very high, leading to local buckling failures before yielding failures

(i) Shear buckling Failure.

Also called Diagonal compression buckling failure.



⊙ No shear buckling failure in web when

$$\frac{d}{t_w} \leq 67\epsilon \; ; \; \epsilon = \sqrt{\frac{250}{f_y}}$$

⊙ Shear buckling can also be minimised by providing vertical stiffners so that one component of diagonal compression taken care of by vertical stiffners and another component by flange plates.

(ii) Horizontal (or) Longitudinal buckling failure.

Due to compressive bending stresses, web plate may have a chance to buckle. ($I_{zz} >>> I_y$). about minor axis. So to avoid this horizontal stiffners are provided.

(iii) Vertical (or) bearing buckling failure of web plate.

Stiffners are to be provided at concentrated loads or supports. Such stiffners are called Load bearing stiffners.

19. For unstiffened web plate, no stiffener is required.

For Fe 450 grade steel, $f_y = 250$

$$\epsilon = \sqrt{\frac{250}{f_y}} = 1.$$

$$\frac{d}{t_w} \leqslant 200\,\epsilon.$$

$$\therefore\ t_w \geqslant \frac{d}{200\,\epsilon}$$

$$\Rightarrow\ t_w \geqslant \frac{2000}{200 \times 1} = \underline{\underline{10\ mm}}$$

20. $I_s \geqslant d_2\, t_w^3$

$$= 2000 \times 10^3$$

$$= \underline{\underline{2 \times 10^6}}\ mm^4$$

Oct, Tuesday

→ Elements of a Plate Girder

- Web plate
- Flange plate with flange angles for bolted/riverted plate girder.

Flange plates only for welded plate girder.

* Stiffeners :

(i) Intermediate Stiffeners

a) Vertical or Stability or Transverse stiffeners.

b) Horizontal or longitudinal stiffeners.

(ii) Bearing Stiffeners.

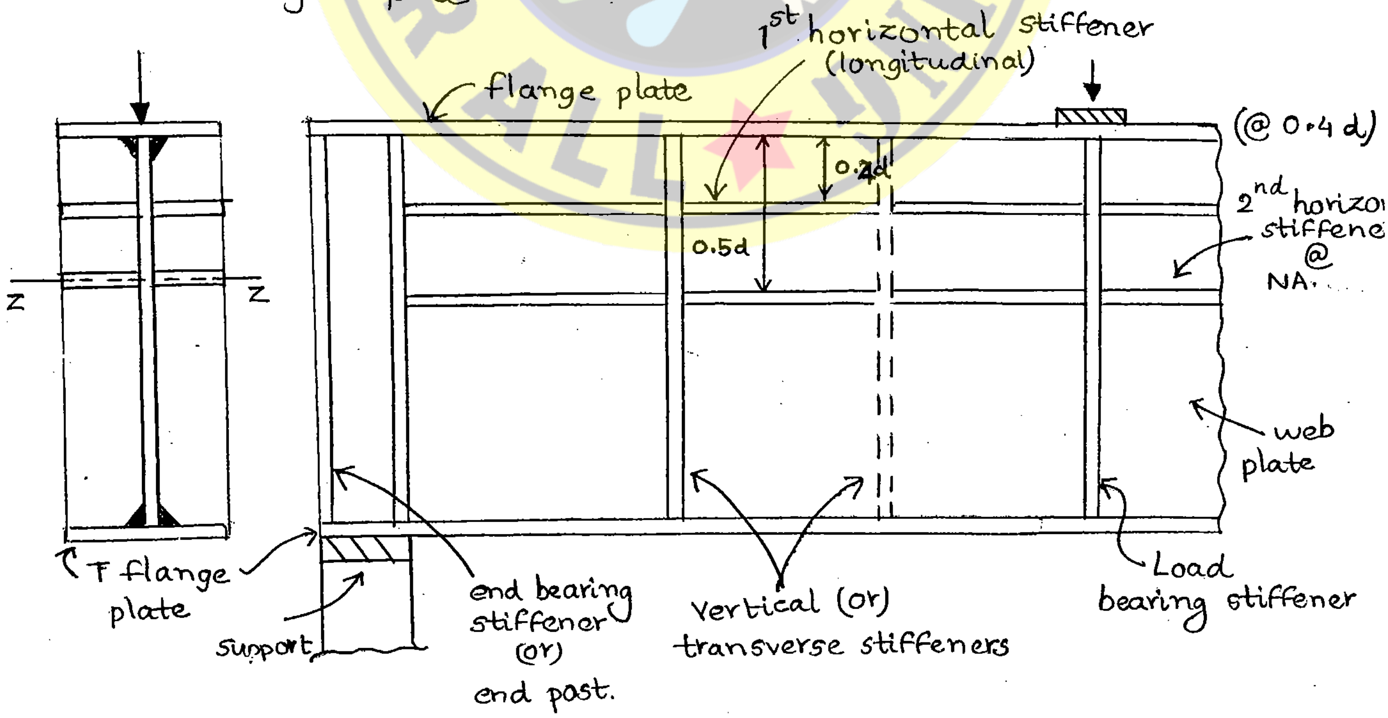
a) Load bearing Stiffeners. (under conc. loads)

b) End bearing Stiffeners. (at supports).

* Splices:

(i) Web Splice.

(ii) Flange Splice



Length of plate available from Indian Rolling Mills is only 7.5 m. But plate girders are built for a length of 20 m. So web splice is used to join web plates and flange splice is used to connect flange plates.

Web plates are provided to support SF. So web splices are not provided at points of max. SF like supports, under conc loads etc. Similarly, flange plates are designed to support the moments. So flange splices are not provided at max. BM locations like under conc. loads. Splices at these critical locations increase the cost and no: of bolts and rivets.

→ Web Plate.

Economical depth of web plate (IS concept based on minimum area of steel (min. wt) to be provided for girder)

$$\boxed{d = \left(\frac{M_z K}{f_y}\right)^{1/3}} \quad ; \quad K = \frac{d}{t_w}$$

M_z = design bending moment

f_y = yield strength of material.

* Min. thickness of Web Plate (should meet serviceability criteria & compression flange buckling requirement)

– min thickness of web plate (based on serviceability criteria)

a) No vertical (Transverse) stiffeners required.

– $\frac{d}{t_w} \leq 200\,\epsilon$ (web connected to flange along both longitudinal edges)

- $\frac{d}{t_w} \leq 90\epsilon$ (web connected flange along one longitudinal edge only)

b) When vertical stiffeners are to be provided.

- $\frac{d}{t_w} \leq 200\, \epsilon_w$ (for $3d \geq c \geq d$)

- $\frac{c}{t_w} \leq 200\, \epsilon_w$ ($0.74\, d \leq c < d$)

- $\frac{d}{t_w} \leq 270\, \epsilon_w$ ($c < 0.74\, d$) ; $\epsilon_w = \sqrt{\frac{250}{f_{yw}}}$

When $c > 3d$, plate girder to be treated as unstiffened.

c = spacing b/w vertical stiffeners.

d) When vertical stiffeners + 1st horizontal stiffener + 2nd horizontal stiffener at NA.

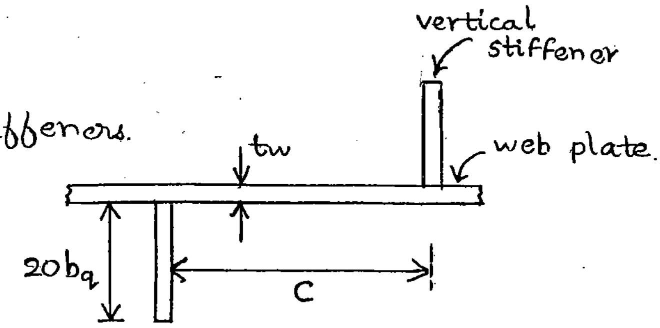
- $\frac{d}{t_w} \leq 400\, \epsilon_w$

→ Stiffeners

- Vertical (or) Transverse (or) Stability Stiffeners.

b_q → outstand of stiffener

c → spacing b/w vertical stiffeners.



NOTE:

Vertical stiffeners are to be provided to eliminate shear buckling failure in web plate. The shape of the stiffener must be angle section for bolted plate girder and flat section.

for welded plate girder.

– minimum MI required (I_s)

$$I_s = 0.75\, d\, t_w^3 \quad \left(\text{when } \frac{c}{d} \geq \sqrt{2}\right)$$

$$I_s = \frac{1.5\, d^3 t_w^3}{c^2} \quad \left(\text{when } \frac{c}{d} < \sqrt{2}\right)$$

– 1st horizontal (longitudinal) stiffener @ $\frac{2}{5}d$ from compression flange to NA.

Minimum MI required : $I_s \geq c\, t_w^3$

– 2nd horizontal stiffener at NA.

$$I_s > d_2 t_w^3$$

where $d_2 = 2 *$ distance from compression flange to NA $(= d)$

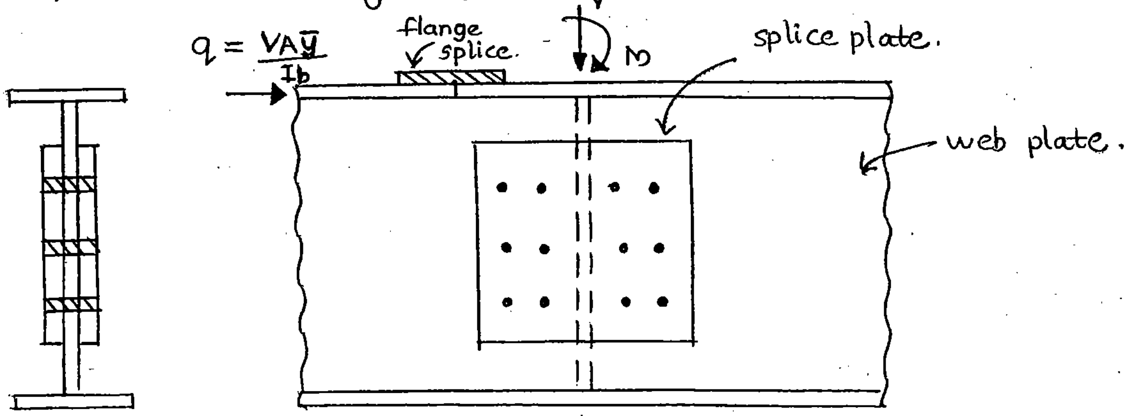
NOTE:

The connection b/w vertical stiffener to the web plate, horizontal stiffener to the web plate should be designed min. shear not less than $\frac{t_w^2}{5 b_s}$

$$\text{Minimum shear} = \frac{t_w^2}{5 b_s} \quad (\text{kN/mm})$$

$t_w \rightarrow$ thickness of web plate

$b_s \rightarrow$ outstand of stiffener.



web splice

→ Web Splice.

- it is recommended to locate web splice at a point away from max. shear.

- web splice must be designed for shear force & BM at spliced locations.

- web splice is a joint for web plate to be used for extending length of web plate.

→ Flange Splice.

- It is joined for flange plate for extending length of flange plate

- flange splice should not be located at a point of max. BM.

- flange splice should be designed for horizontal shear (axial load to the flange plates) due to transverse loads

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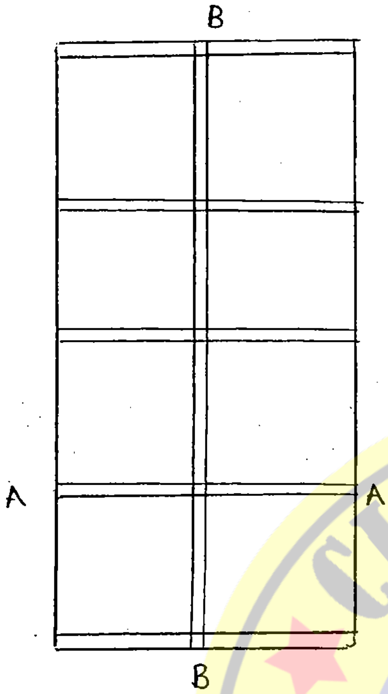
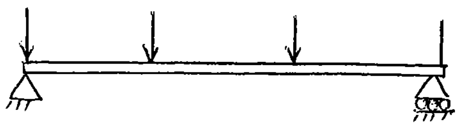
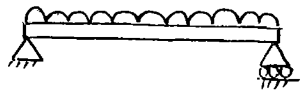
12th Oct,
SUNDAY

BEAMS

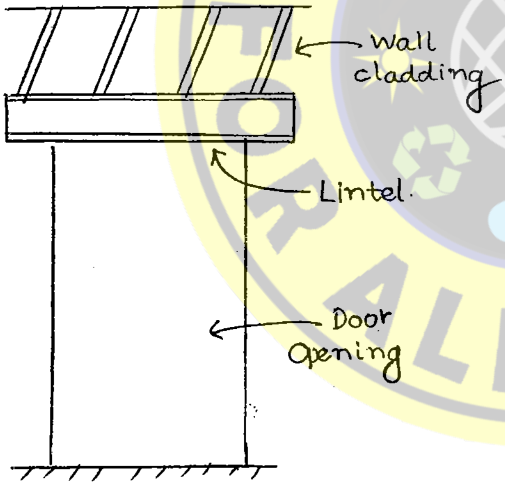




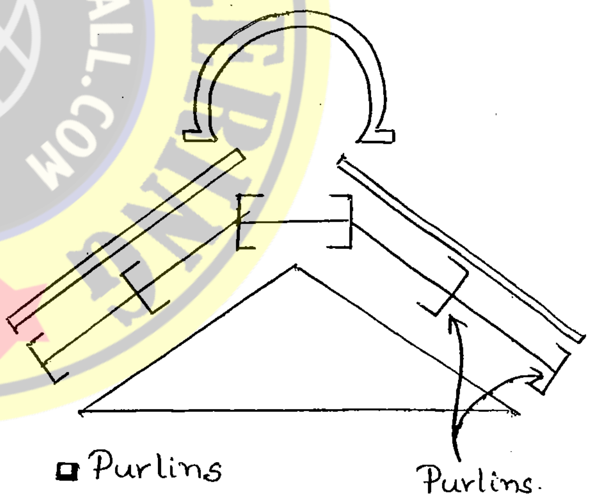
Beam B–B
(FLOOR BEAM).



Beam A–A
(JOIST)



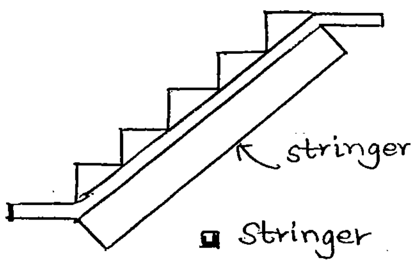
■ Lintel.



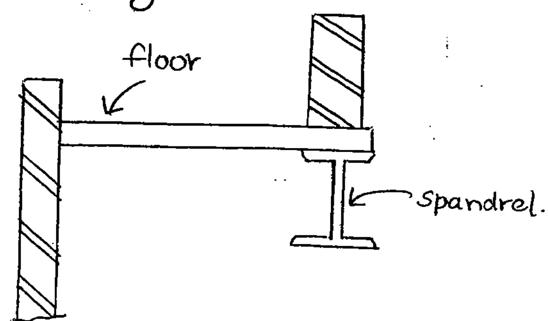
■ Purlins

* Girder – major beam.

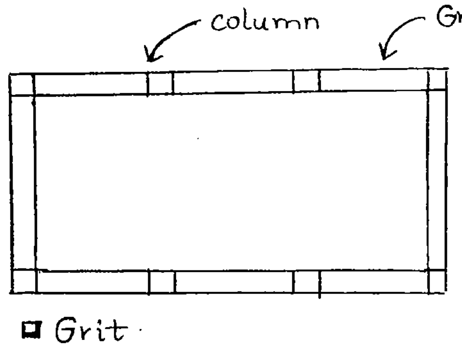
Eg: Floor beam in an industrial building.



■ Stringer



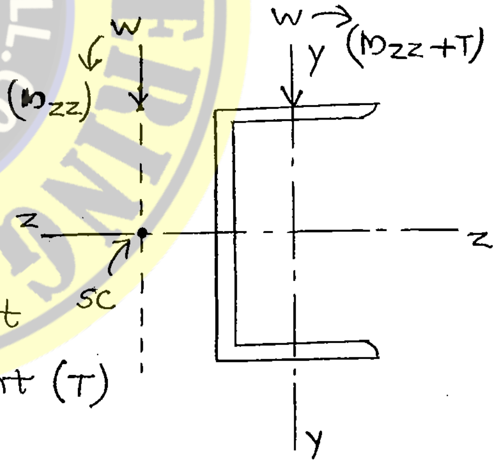
■ Spandrel.



* Header — transverse loaded structure provided in well openings of stairs.

Bending moment is a function of loading and span.
$M = f(w, l)$ or $M = f(w, l^2)$. But tranverse loaded structures like grit, purlin, lintel, etc are secondary beams with shorter spans. So the design BM will be less for them.
∴ they are designed with channel sections although I-section are the best beam sections; as I-sections becomes uneconomical here.

For channel sections, load must pass through the SC to produce simple bending along zz axis. If they act at a different point, it will cause twisting moment (T) and BM about ZZ axis (M_{zz}).

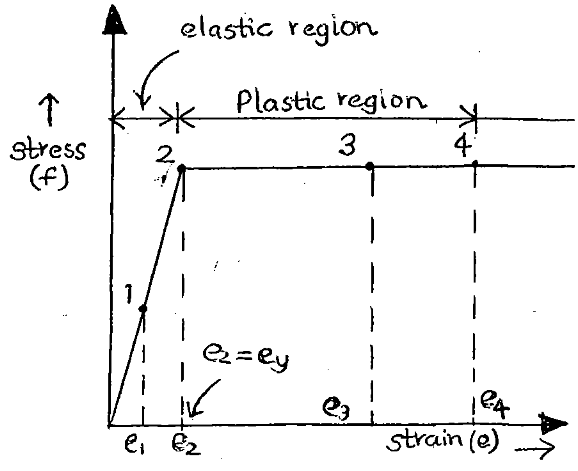


→ Behaviour of Beam in Flexure.

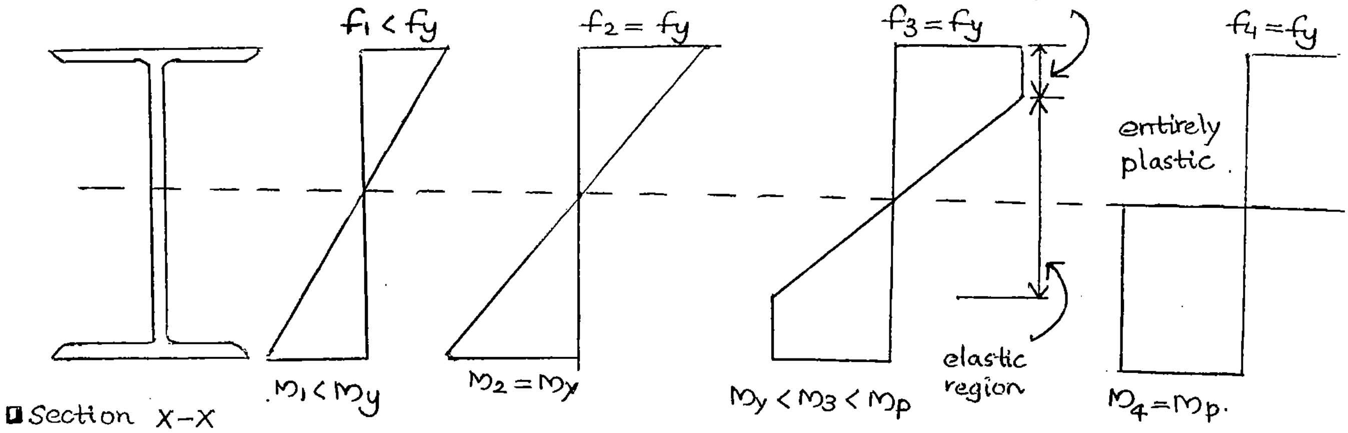
Idealised stress strain curve of mild steel is also known as elasto-plastic strain curve.

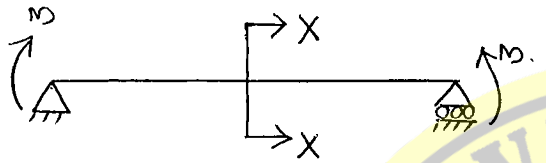
$$\frac{M}{I} = \frac{f}{y}$$

$$M_e = f \frac{I}{y} = f Z_e$$



Bending Stress Distributions corresponding to Various points on stress strain curve:-





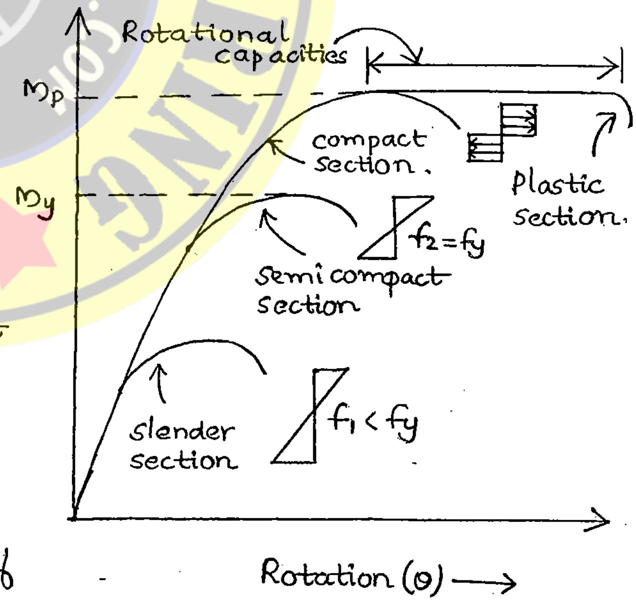
$M_p = f_y Z_p$.

$$\boxed{M_p = (1.10 \text{ to } 1.20) M_y}$$

→ Classifications of Sections :

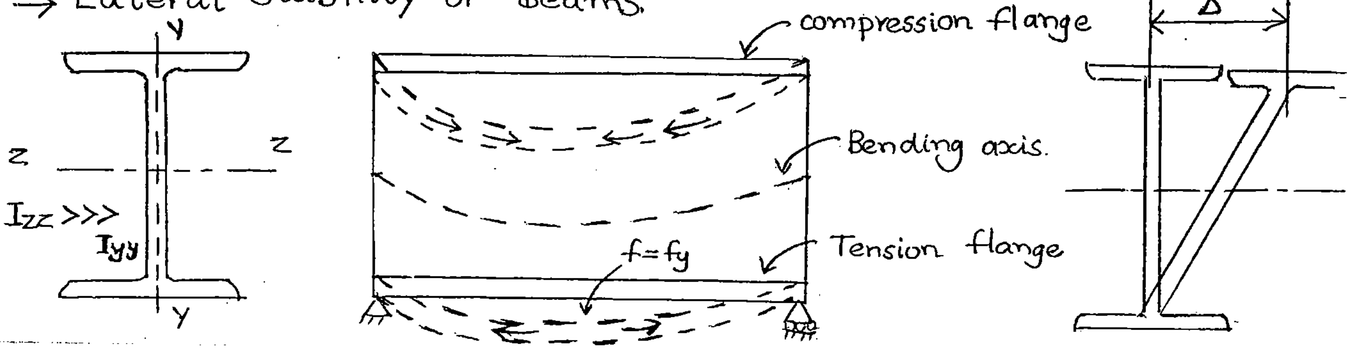
Based on yield moment (M_y), plastic moment (M_p), $\frac{d}{t_w}$ ratio of flange & web, the four classes of sections are:

(i) Plastic section.
(ii) Compact Section.
(iii) Semi compact Section (Non compact).
(iv) Slender Section.



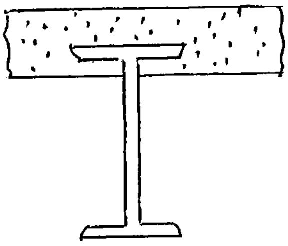
I-sections and channel sections have plastic moment of resistance whereas plate girders do not develop plastic moment of resistance.

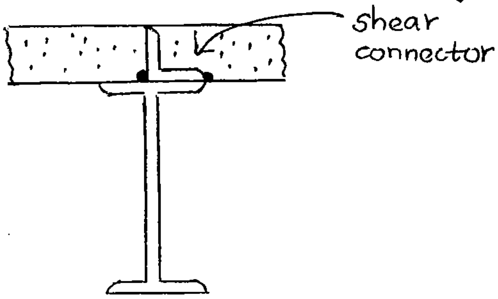
→ Lateral Stability of Beams.



(i) Laterally Restrained Beams (or) Laterally Supported Beams

Here compression flange of beam is not affected by lateral or lateral torsional buckling.

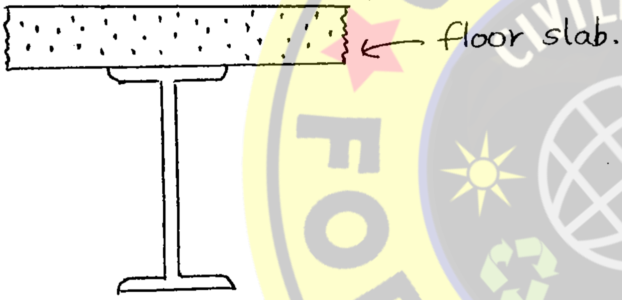




~~(ii) Lateral Stability of Beams.~~

(ii). Laterally unrestrained (or) laterally unsupported beams.

Compression flange of beam section is affected by lateral (or) lateral torsional buckling.



→ Design Criteria of Beam:

- Flexural Strength Criteria (Design for BM (M)).
- Shear Strength Criteria (Design for SF (V)).
- Deflection Criteria. (Serviceability Criteria)
- Check for secondary effects like web buckling & web crippling failures.

* Design Flexural (or) Bending Strength. (M_d).

a) Laterally Restrained Beam. (No lateral or lateral torsional buckling).

b) When $\frac{d}{t_w} \leq 67\,\epsilon$ (No shear buckling of web).

c). Low shear case (when $V \leq 0.6\, V_d$)

$V \rightarrow$ design shear force

$V_d \rightarrow$ design shear strength of a beam.

$$\frac{f_y}{\sqrt{3}} = 0.577 f_y = 0.6 f_y$$

$V > 0.6 V_d$ (High Shear Case)

$$\boxed{M_d = \beta_b Z_p \frac{f_y}{\gamma_{mo}} \leqslant 1.2 Z_e \frac{f_y}{\gamma_{mo}}}$$

(for simply supported beams)

$\beta_b = 1.0$ (for plastic & compact sections)

$= \frac{Z_e}{Z_p}$ (for semi-compact sections).

$$\boxed{M \leqslant M_d}$$

$M_d = Z_e f'$ (f' = reduced bending stress)
(for slender section).

For semi compact sections,

$$M_d = \frac{M_y}{\gamma_{mo}} = \frac{f_y}{\gamma_{mo}} \cdot Z_e \quad \left(\Rightarrow \beta_b = \frac{Z_e}{Z_p}\right).$$

For laterally unrestrained beams,

$$M_d = \beta_b \cdot Z_p \cdot f_{cd}.$$

$f_{cd} \rightarrow$ compressive strength of flange of a beam which is calculated by Perry Robertson equation.

f_{cd} depends on slenderness ratio.

- Slenderness ratio for compression flange of a beam is limited to 300.

* Design Shear Strength of the beam (V_d)

a) Lateral restrained (or). Supported beams.

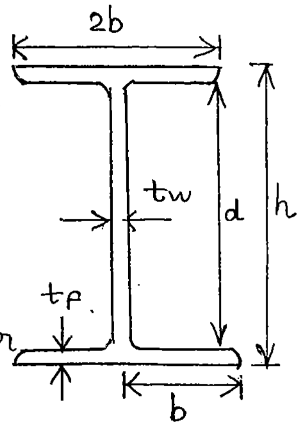
b) when $\frac{d}{t_w} \leqslant 67\epsilon$ (No shear buckling of web).

where $\epsilon = \sqrt{\frac{250}{f_y}}$

$$V_d = \frac{V_n}{\gamma_{mo}} = \frac{V_p}{\gamma_{mo}}$$

V_p = Plastic shear strength of the section

$$= \text{Shear area} * \frac{f_y}{\sqrt{3}}$$



$$\therefore \quad V_d = \text{Shear area} * \frac{f_y}{\sqrt{3} \cdot \gamma_{mo}}$$

For rolled I-section :

Shear area = $h \cdot t_w$ (about major axis)

For welded I-section :

Shear area = $d \cdot t_w$ (about major axis).

* Limiting Deflections : (As per IS 800 : 2007).

– For simply supported beam

$$\Delta_{limit} = \frac{Span}{240} \quad \text{(Elastic cladding)}$$

$$= \frac{Span}{300} \quad \text{(Brittle cladding)}$$

– For cantilever beam = 2 * simply supported beam

$$\Delta_{limit} = \frac{Span}{120} \quad \text{(Elastic cladding)}$$

$$= \frac{Span}{150} \quad \text{(Brittle cladding)}$$

$$\boxed{\Delta_{cal} \leq \Delta_{limit}}$$

3-64.

30. $V_d = \text{Shear area} \times \dfrac{f_y}{\sqrt{3} \cdot \gamma_{mo}} = 500 \times 10.2 \times \dfrac{250}{\sqrt{3} \times 1.1} = \underline{\underline{669.20 \text{ kN}}}$