5th sept,
SUNDAY

09. PLATE GIRDERS

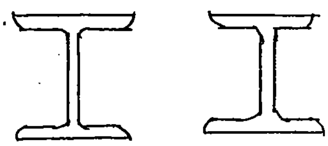
Beams are designed for bending moments (M) and sometimes shear force (V).

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

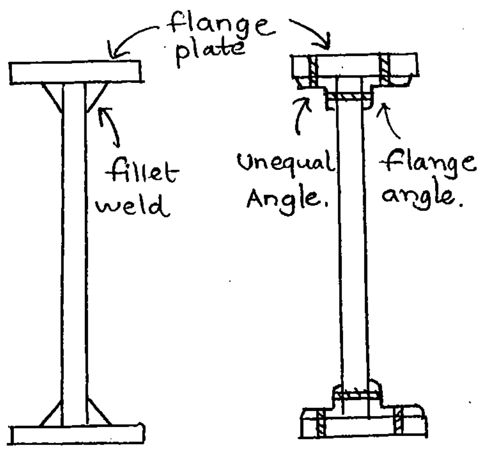
So to meet the design requirement of $M \leq M_d$, higher value of section modulus ($\propto I$), Z, is required. But the maximum depth for an I section given by Indian Rolling mills is 600 mm. So the following sections are considered:

$M_d = f(Z \rightarrow I)$.

(i) Two I-sections placed side-by-side.
(ii) Plate Girder (spans 20m–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

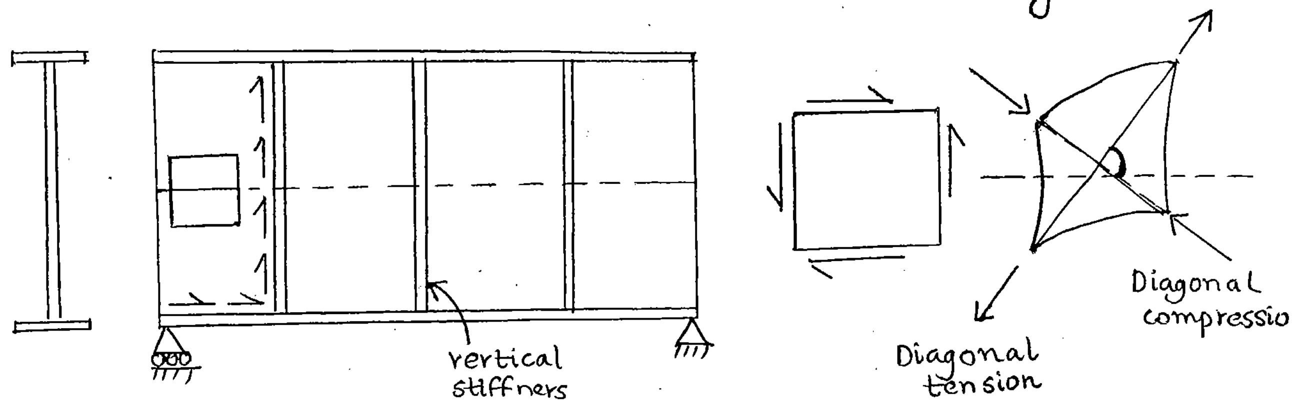
⊙ Weight of welded plate girder $= \frac{W}{400}$ kN/m

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

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

⊙ Shear buckling can also be minimised by providing vertical stiffners so that one component of diagonal comp~~onent~~ression 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 change 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, $f_y = 250$

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

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

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

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

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

$$= 2000 \times 10^3$$

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

oct, [illegible]ESDAY

→ 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.

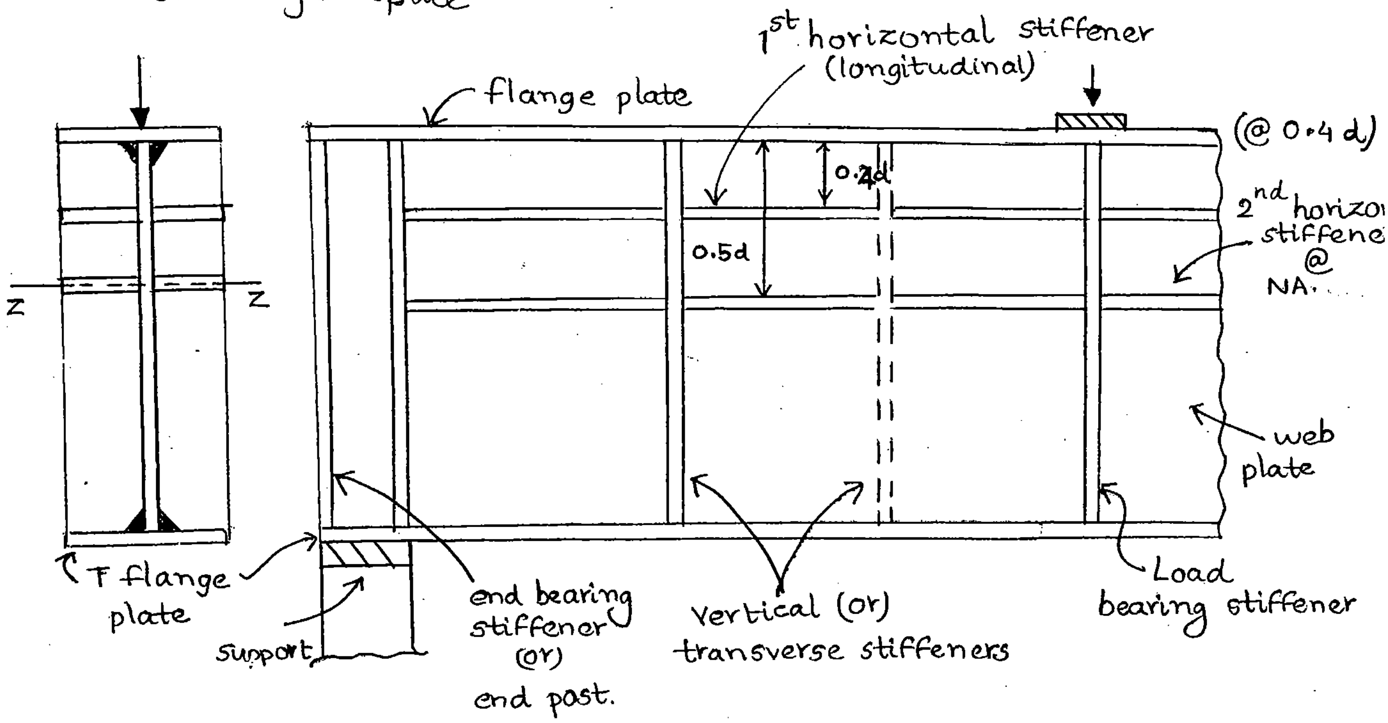
a) Load bearing Stiffeners. (under conc. loads)

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 riverts.

→ 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.

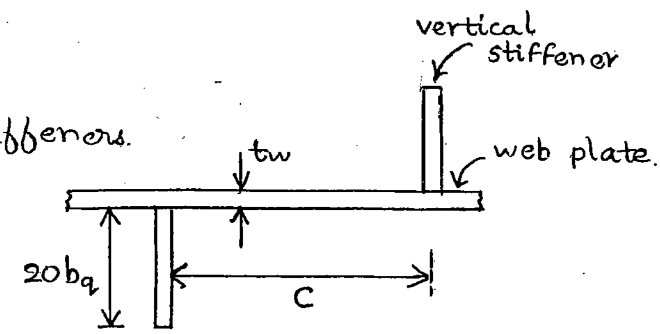
- $\frac{d}{t_w} \leq 400\ \epsilon_w$

→ 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)

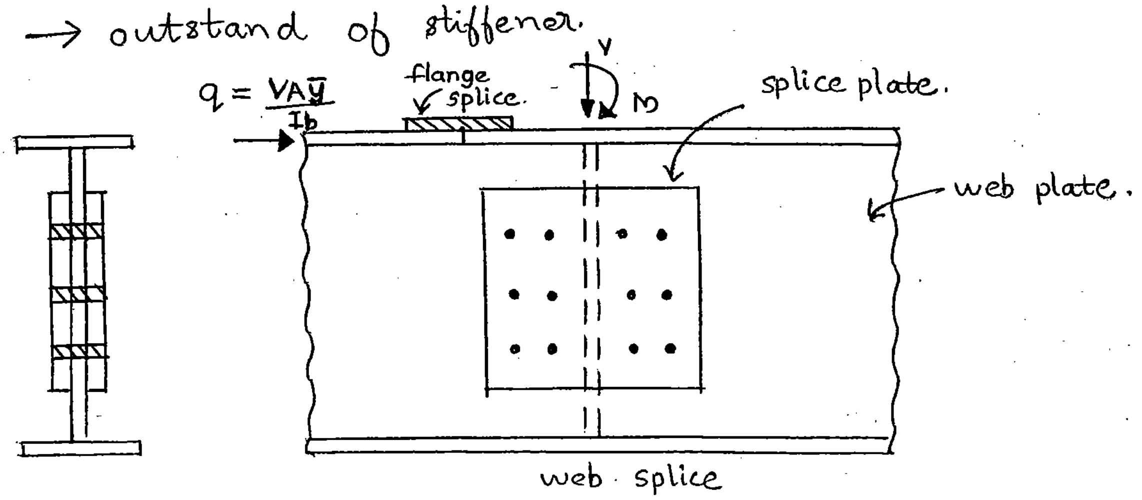
NOTE:

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.

- 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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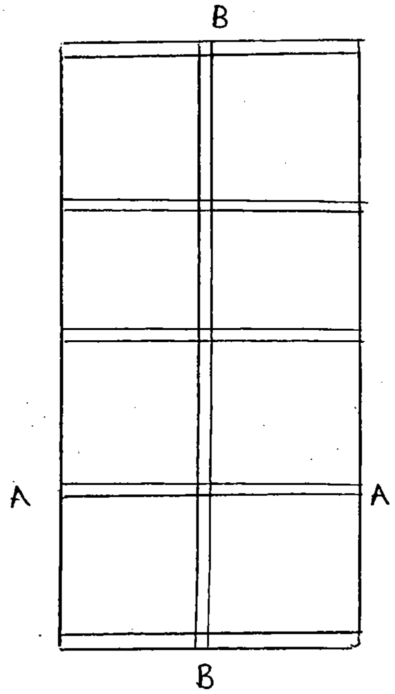
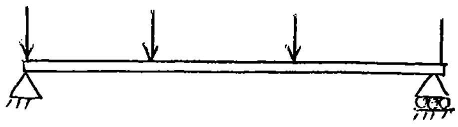
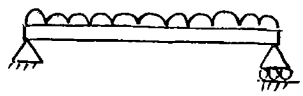
12th Oct,
SUNDAY

BEAMS

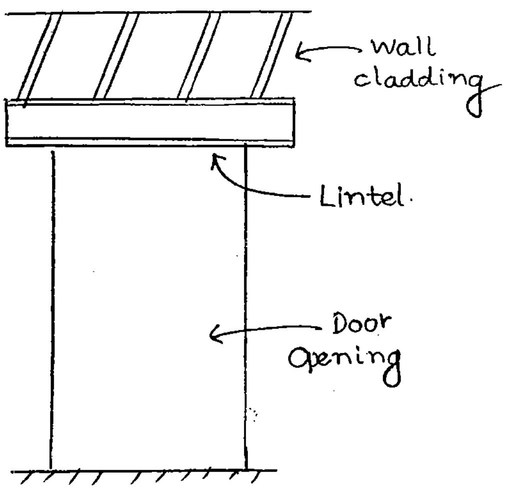




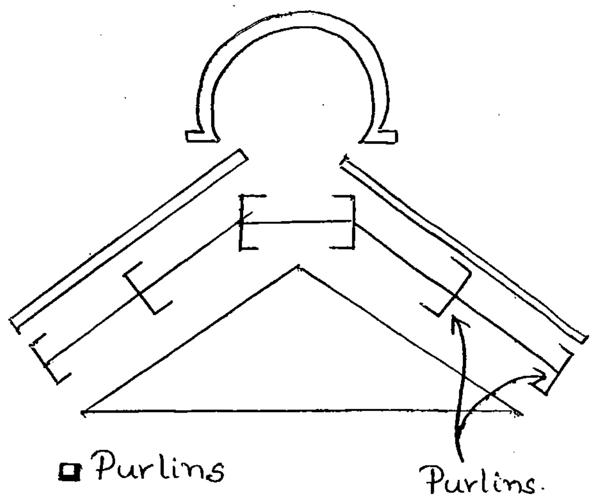
Beam B–B
(FLOOR BEAM).



Beam A–A
(JOIST)



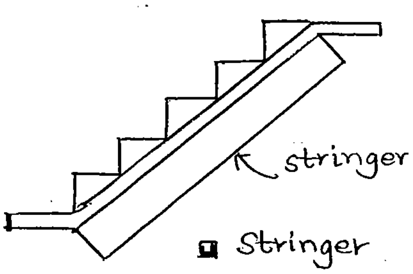
■ Lintel.



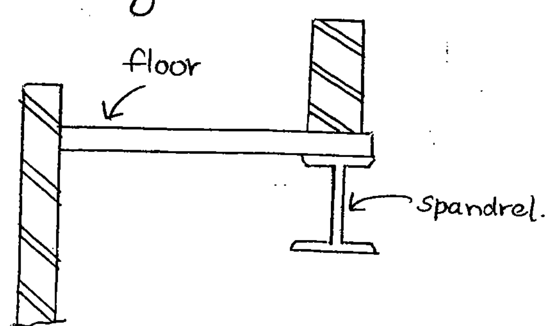
■ Purlins

* Girder – major beam.

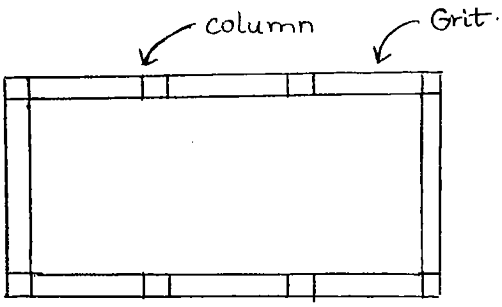
Eg: Floor beam in an industrial building.



■ Stringer



■ Spandrel.

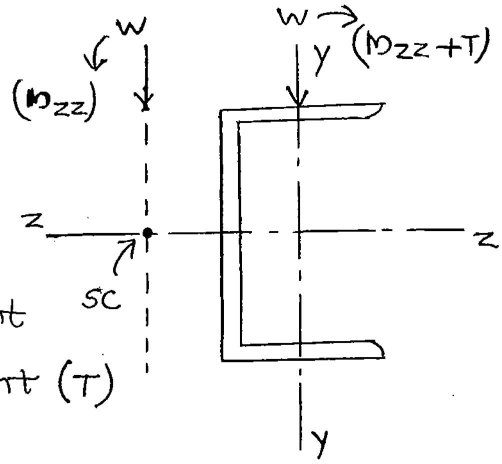


Grit.

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

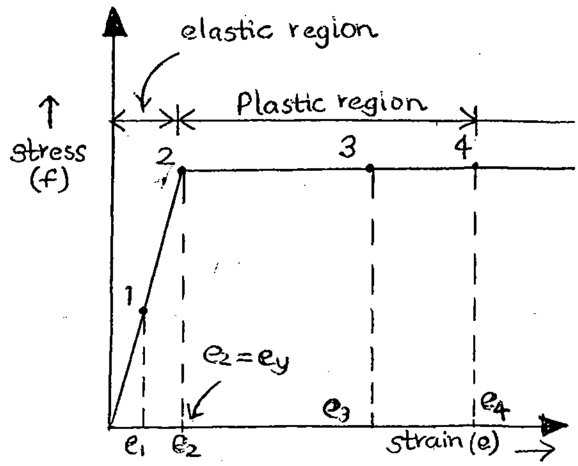
Bending moment is a function of loading and span. $M = f(w, l)$ or $M = f(w, l^2)$. But tranverse loaded structures like grit, purlin, lintel, etc are secondary beams with shorter spans. So the design BM will be less for them. ∴ they are designed with channel sections although I-section are the best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

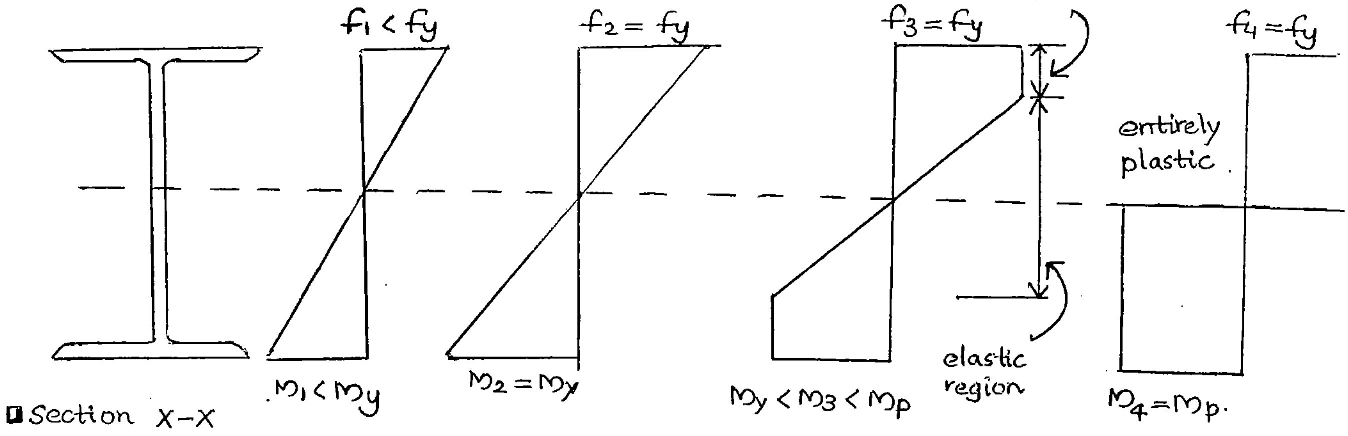
Idealised stress strain curve of mild steel is also known as elasto-plastic strain curve.

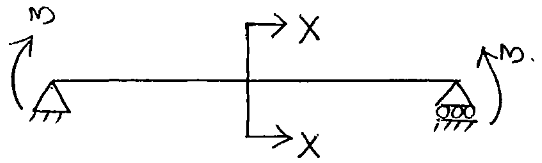


$$\frac{M}{I} = \frac{f}{y}$$

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

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:

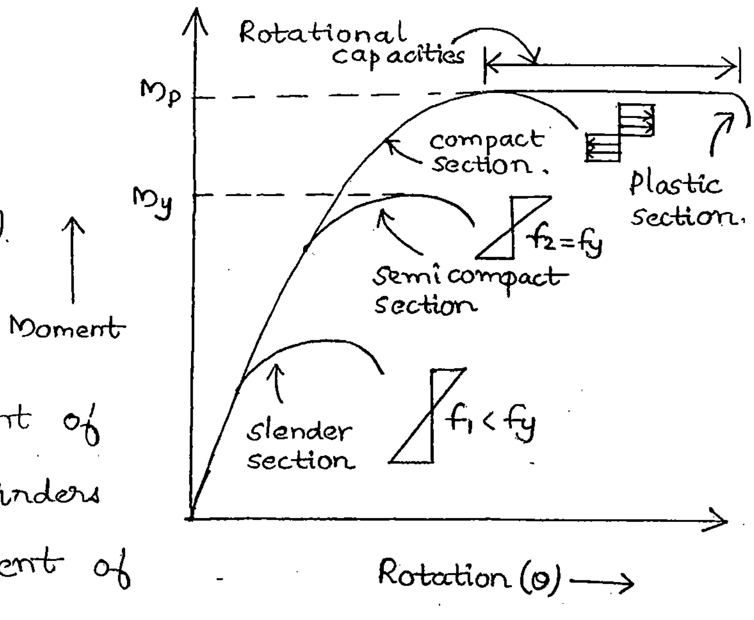
Based on yield moment (M_y), plastic moment (M_P), $\frac{d}{t_w}$ ratio of flange & web, the four classes of sections are:

(i) Plastic section.

(ii) Compact Section.

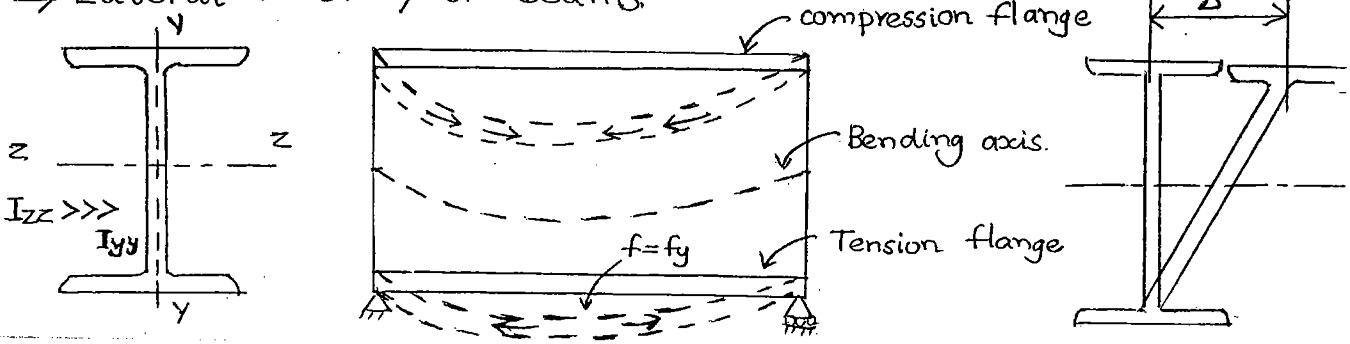
(iii) Semi compact Section (Non compact).

(iv) Slender Section.



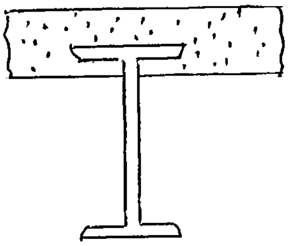
I-sections and channel sections have plastic moment of resistance whereas plate girders do not develop plastic moment of resistance.

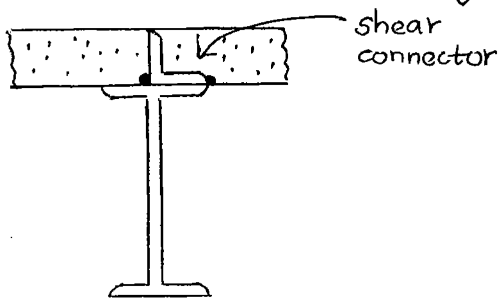
→ Lateral Stability of Beams.



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

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

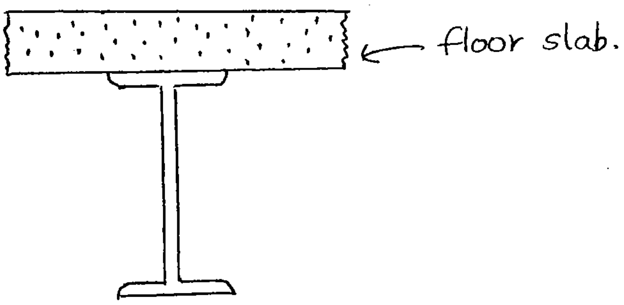




~~(ii) Lateral Stability of Beams.~~

(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

$V_d \longrightarrow$ 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,

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

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.

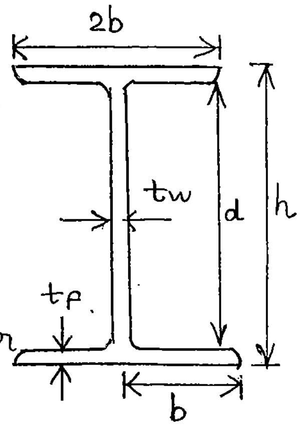
b) when $\frac{d}{t_w} \leqslant 67 \epsilon$ (No shear buckling of web).

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

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



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

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

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

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

– For cantilever beam

= 2* simply supported beam

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

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

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

-64.

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