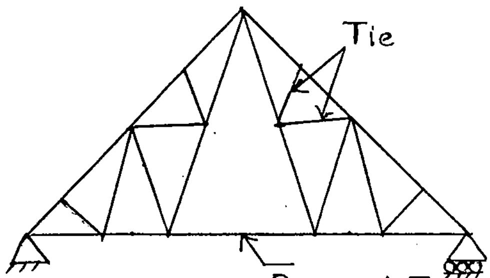
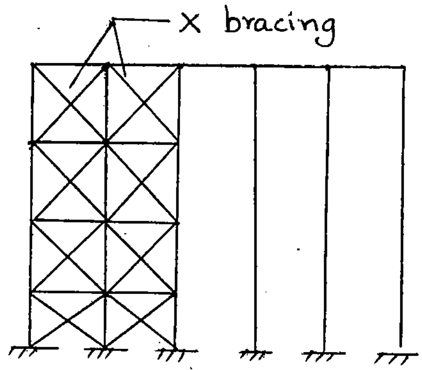
7th Sept,
TUESDAY

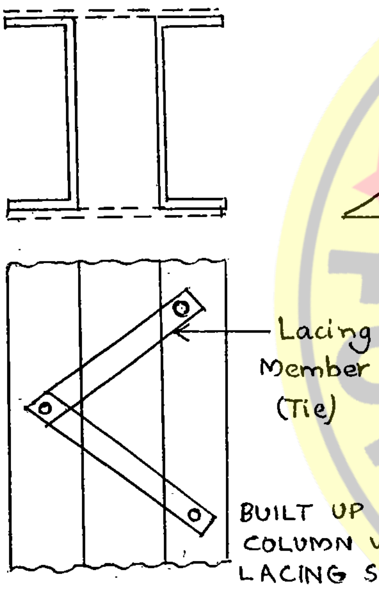
5. TENSION MEMBERS



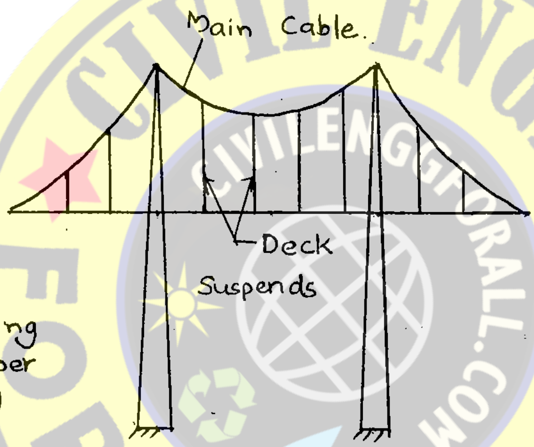
ROOF TRUSS



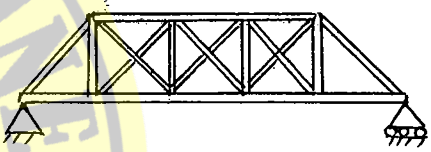
BRACED FRAME



BUILT UP COLUMN WITH LACING SYSTEM



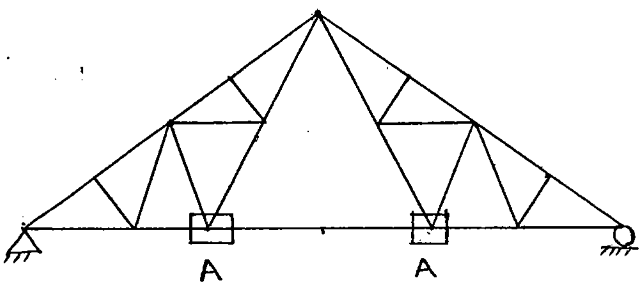
SUSPENSION BRIDGE

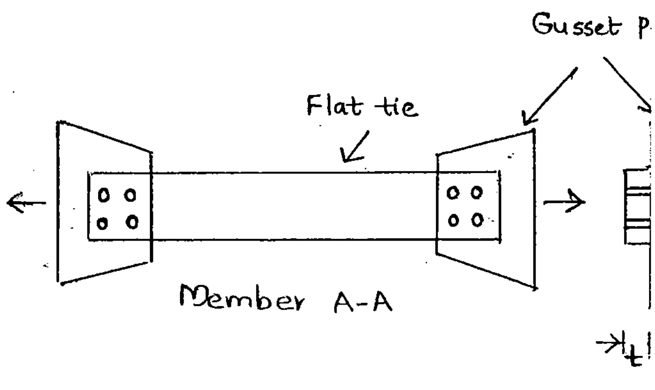


TRUSS BRIDGE

→ Types of Failures in a Tension Member:

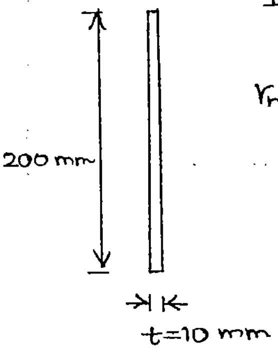
- Gross section yielding failure (T_{dg})
- Net section rupture (or) Fracture failure. (T_{dn})
- Block shear failure. (T_{db})





Member A-A

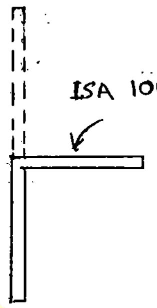
In olden constructions, flat members were used as tension members. Although flat members are strong in tension, they are weak in compression due to small value of radius of gyration. ($P_{cr} = \frac{\pi^2 EA}{(l/r)^2}$). So, angle sections now replace flat members as tension members.



$$I_{min} = \frac{bt^3}{12}$$

$$r_{min} = \sqrt{\frac{I_{min}}{A}} = \sqrt{\frac{bt^3}{12 \cdot bt}} = \frac{t}{\sqrt{12}}$$

$$= \frac{10}{\sqrt{12}} = 2.88 \text{ mm}$$



$r_{min} = r_{vv}$

$= 19.1$ mm

In the above eg, angle section with c/s area as that of a flat member has more radius of gyration.

In flat members, load reversals and bolt holes are the factors of failures. But in angle sections, along with these two factors, eccentricity of loading (causing moment) must also be taken into consideration.

→ Design Tensile Strength of a Tension Member (T_d)

T_d is minimum of T_{dg} or T_{dn} or T_{db}.

* Based on Gross Section Yielding Failure, (T_{dg})

$$T_{dg} = \frac{A_g \cdot f_y}{\gamma_{m0}}$$; A_g → gross sectional area of a member

f_y → yield strength of a material.

γ_{m0} → partial safety factor against yield stress (1.10)

* Based on Net Section Rupture (or) Fracture Failure (T_{dn})

(i) For Plates and Flats.

$$T_{dn} = \frac{0.9 \times A_n f_u}{\gamma_{m1}}$$

$A_n \rightarrow$ net sectional area = gross sectional area − Area of bolt hole.

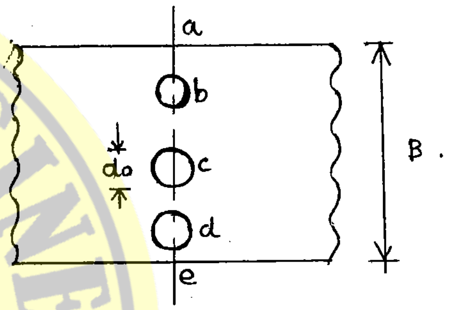
– For chain pattern of bolting:

A_n = A_g − area of bolt holes.

Along section, a−b−c−d−e:

$= Bt - n d_0 t$

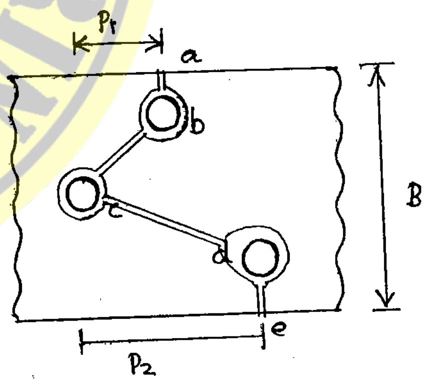
$A_n = (B - n d_0) t$



– For staggered (or) zig-zig pattern of bolting

Along section a−b−c−d−e,

$$A_n = (B - n d_0) t + \underbrace{\frac{P_1^2 t}{4 g_1} + \frac{P_2^2 t}{4 g_2}}_{\text{area connections}}$$



P_1 & P_2 → staggered pitches

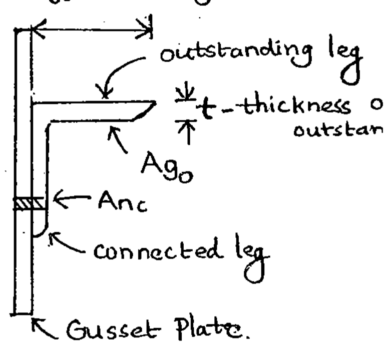
g_1 & g_2 → gauge distances.

■ For n inclination, n number of area connections.

(ii) For Angle, Channel & Other types of Rolled Steel sections

A_{nc} = Net sectional area of connected leg

A_{go} = gross sectional area of outstanding leg.



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

unimportant for GATE

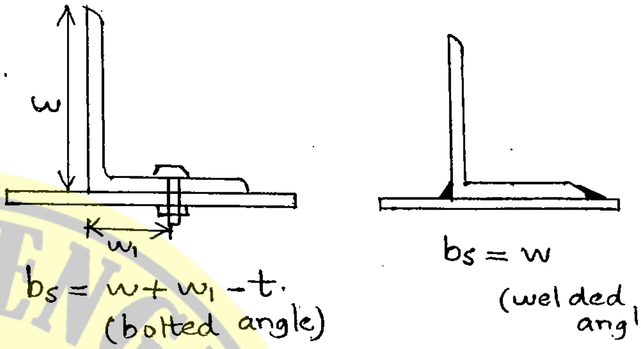
$$\beta = \left\{ 1.4 - 0.076 \left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L_c}\right) \right\} \geq 0.7, \leq \left(\frac{f_u}{f_y} \cdot \frac{\gamma_{m0}}{\gamma_{m1}}\right)$$

b_s → shear leg distance.

L_c → length of end connection

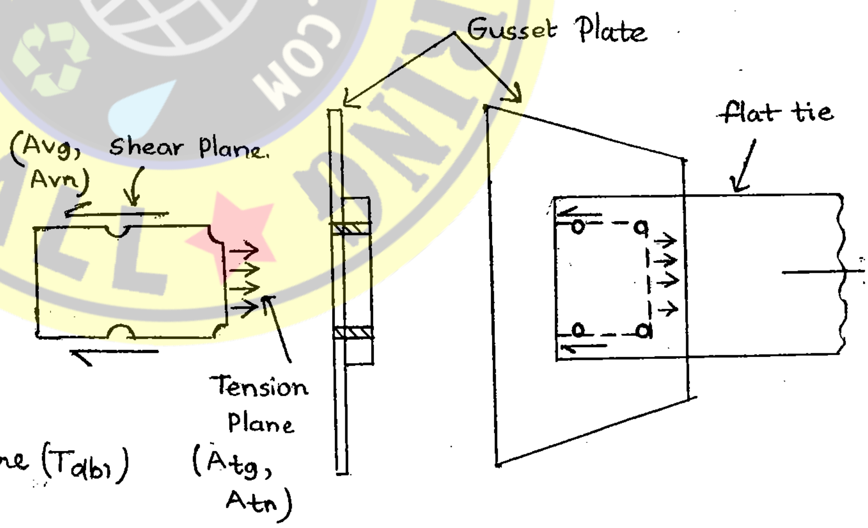
w → outstanding leg distance.

t → thickness of outstand.



- It is better to use unequal angle section for better tensile strength. Longer leg is connected to gusset plate (higher A_{nc}) and correction factor (β) is reduced.

* Based on Block Shear Failure (T_{db})



– Shear yielding & Tension Plane rupture (T_{db1})

$$T_{db1} = \frac{A_{vg} f_y}{\sqrt{3} \cdot \gamma_{m0}} + 0.9\, A_{tn} \frac{f_u}{\gamma_{m1}}$$

– Shear rupture & Tension Yield (T_{db2})

T_{db} : lesser of T_{db1} & T_{db2}

$$T_{db2} = 0.9\, A_{vn} \frac{f_u}{\sqrt{3}\, \gamma_{m1}} + A_{tg} \cdot \frac{f_y}{\gamma_{m0}}$$

A_{vg} & A_{vn} → min gross and net area of shear plane resptly.

A_{tg} & A_{tn} → min gross and net area of tension plane resptly.

* Design requirement for Safety of Tension member

- $T \leq T_d$:- min of $\begin{cases} T_{dg} \\ T_{dn} \\ T_{db} \end{cases}$

- λ of member $\leq \lambda_{limit}$

λ_{limit} → limiting slenderness ratio of tension member as per IS 800 : 2007

→ Design of Axially Loaded Tension Member

* Slenderness ratio of a Tension Member

$$= \frac{\text{unsupported length}}{\text{Min. radius of gyration}} = \frac{l}{r_{min.}}$$

* Limiting Slenderness Ratio (λ_{limit}).

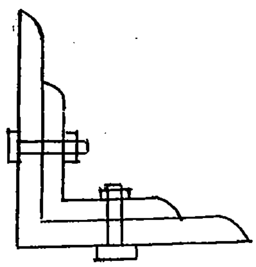
(i) A tension member is subjected to load (or) stress reversals due to loads other than wind (or) earthquake load — 180

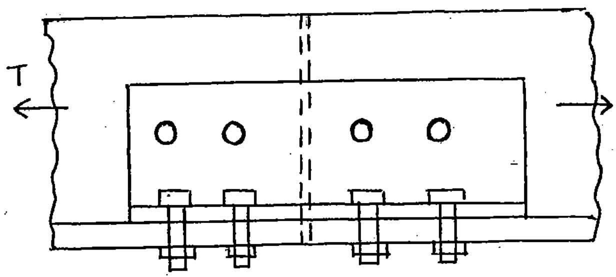
(ii) A member used as tie in roof truss (or) in a bracing system subjected to load reversals due to loads resulting from wind (or) earthquake loads — 350

(iii) For any other tension member (other than pretensioned members) — 400

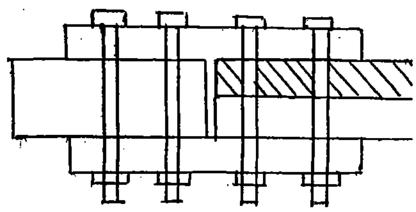
* Tension Splice

Tension splice is a joint for tension member normally used for extending length of a tension member (where the size available from Indian Rolling Mills are limited) and also used for joining two different sizes of tension members





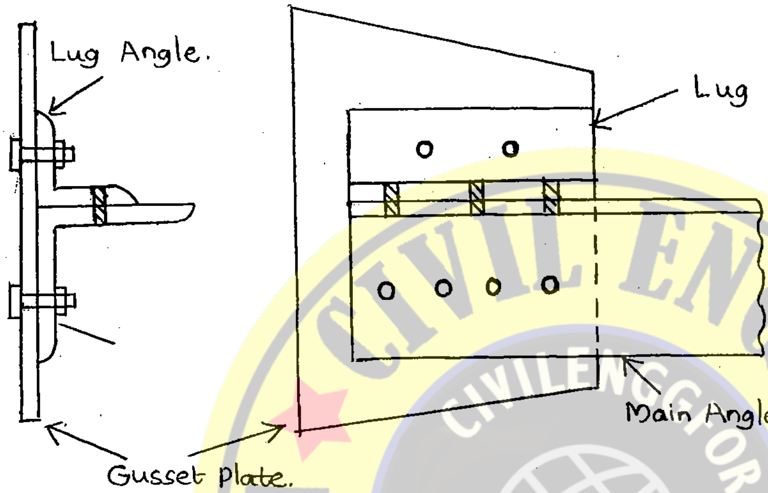
Tension Splice b/w two angle members



Tension splice b/w two diff. sizes of tension members

* Lug Angle.

Lug angle is a short length of an angle used at a joint location to join outstanding leg of an angle to the gusset plate (and also to join outstanding flange of a channel to the gusset plate) so that length of connection or joint can be reduced.



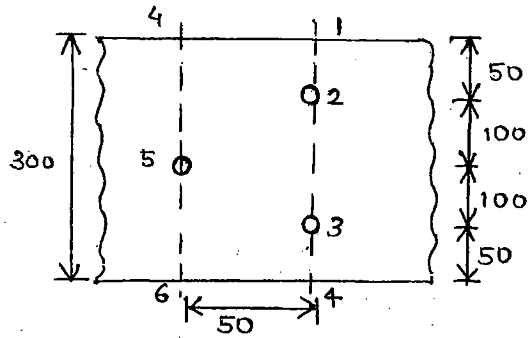
P- 41

1. $B = 300\ mm$, $t = 10\ mm$, $d = 18\ mm$.

Net sectional area of plate with one bolt hole,

$$A_n = (B - nd_o)t = (300 - 1 \times 20) \times 10 = \underline{\underline{2800}}\ mm^2$$

2. Hole diameter, $d_o = 25\ mm$.



Failure sections are:

a) 1 – 2 – 3 – 4

b) 4 – 5 – 6

c) 1 – 2 – 5 – 6

d) 1 – 2 – 5 – 3 – 4

'An' along 1–2–3–4 : (chain pattern).

$$A_n = (300 - 2 \times 25)10$$

$$= \underline{\underline{2500}}\ mm^2$$

Along 1-2-5-6:

$$A_n = (B - n d_o)t + \frac{p^2 t}{4g}$$

$$= (300 - 2\times25)\,10 + \frac{50^2 \times 10}{4\times100} = \underline{\underline{2562.5}}\ mm^2$$

Along 1-2-5-3-4:

$$A_n = (300 - 3\times25)\,10 + \frac{50^2\times10}{4\times100} + \frac{50^2\times10}{4\times100}$$

$$= \underline{\underline{2375}}\ mm^2$$

Effective sectional area $= \underline{2375}\ mm^2$

04. $$T_{dn} = 0.9 A_n \frac{f_u}{\gamma_{m1}}$$

$$A_n = (200 - 3\times18)\times12 = 1752\ mm^2$$

$$T_{dn} = 0.9\times1752\times\frac{410}{1.25} = \underline{\underline{517.19}}\ kN$$

05. $A = 1379\ mm^2$, $f_y = 250$

$$T_{dg} = \frac{A_g f_y}{\gamma_{mo}} = \frac{1379\times250}{1.1} = \underline{\underline{313.4}}\ kN$$

06. $f_u = 410$ MPa ; $f_y = 250$ MPa ; $d_o = 18$ mm

$T_{d} \rightarrow$ min of T_{dg} (or) T_{dn} (or) T_{db}

$d_o = 18$ mm

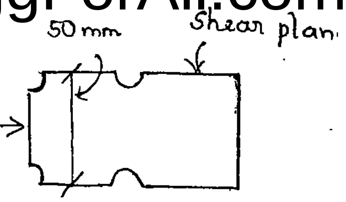
Min end distance, $e_{min} = 1.5\ d_o = 27$ mm.

$e_{provided} > e_{min}$. (no block shear failure).

$$T_{dg} = \frac{A_g f_y}{\gamma_{mo}} = \frac{120\times10\times250}{1.1} = \underline{\underline{272.7}}\ kN$$

$$T_{dn} = 0.9\frac{A_n f_u}{\gamma_{m1}} = 0.9\,(120 - 2\times18)\,10\times\frac{410}{1.25} = \underline{\underline{247.9}}\ kN$$

$A_{tg} = 50 \times 10 = 500 \text{ mm}^2$



$A_{tn} = \left(50 - \frac{d_o}{2} - \frac{d_o}{2}\right) = (50 - 18)10 = 320 \text{ mm}^2$

$A_{vg} = (50 + 35) \times 10 \times 2 = 1700 \text{ mm}^2$

$A_{vn} = \left((50+35) - 18 - \frac{18}{2}\right) \times 10 \times 2 = 1160 \text{ mm}^2$

Shear yield & Tension rupture (T_{db1}) :

$$T_{db1} = A_{vg} \frac{f_y}{\sqrt{3}\,\gamma_{mo}} + 0.9 A_{tn} \frac{f_u}{\gamma_{m1}}$$

$$= 1700 \times \frac{250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 320 \times 410}{1.25} = \underline{317.53} \text{ kN}$$

Shear rupture & Tension yield (T_{db2}) :

$$T_{db2} = 0.9 A_{vn} \frac{f_u}{\sqrt{3}\,\gamma_{m1}} + A_{tg} \frac{f_y}{\gamma_{mo}}$$

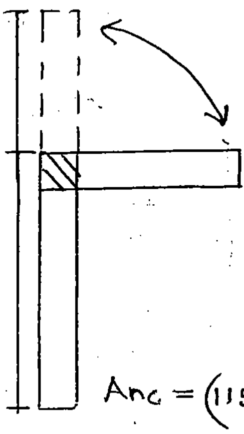
$$= \frac{0.9 \times 1160 \times 410}{\sqrt{3} \times 1.25} + \frac{500 \times 250}{1.1}$$

$$= 311 \text{ kN}$$

T_{db} = min of T_{db1} & T_{db2} = $\underline{311}$ kN

$T_d = \underline{247}$ kN

07 $f_u = 410$ MPa, $f_y = 250$ MPa .



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

$$= \left[\left(150 + 115 - 8\right) \times 8\right] \times \frac{250}{1.1} = \underline{467.7} \text{ kN}$$

$A_{nc} = \left(115 - \frac{8}{2}\right.$

$= 1168$

08. $T_{dn} = 0.9 A_{nc} \frac{f_u}{\gamma_{m1}} + \beta A_{go} \frac{f_y}{\gamma_{mo}}$

$$\beta = 1.4 - 0.076 \left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L_c}\right) \geq 0.7 \quad \& \quad \leq \frac{f_u}{f_y} \cdot \frac{\gamma_{mo}}{\gamma_{m1}}$$

$$= 1.4 - 0.076 \times \frac{115}{8} \times \frac{250}{410} \times \frac{115}{140} \geq 0.7 = \underline{0.85}$$

$A_{nc} = \left(150 - \frac{8}{2}\right) 8 = 1168 \ mm^2$

$A_{go} = \left(115 - \frac{8}{2}\right) 8 = 888 \ mm^2$

$T_{dn} = \underline{\underline{516.3 \ kN}}$

op. $\lambda \leq \lambda_{limit}$

$\frac{l_{max}}{r_{min}} \leq \lambda_{limit}$

$\frac{l_{max}}{20} \leq 350 \quad (r_{min} = 20 \ mm).$

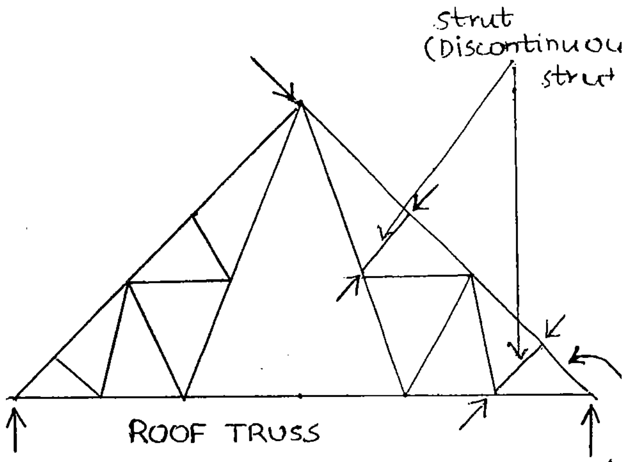
$l_{max} \leq 350 \times 20$

$\leq 7000 \ mm$

$\leq 7.0 \ m.$

12th Sept,
FRIDAY

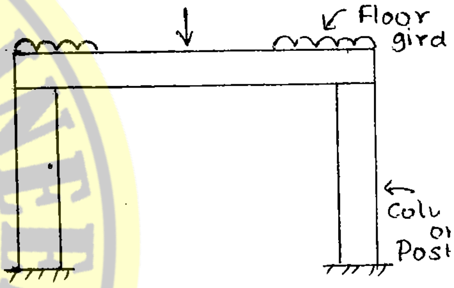
6. COMPRESSION MEMBERS



⊙ Principale Rafter – normally used in roof truss as a main strut.

⊙ Strut – generally used in roof truss (or) in a bracing system.

⊙ Column (or) Post (or) Stanction – (steel column) used in industrial building to support floor (or) floor girders.



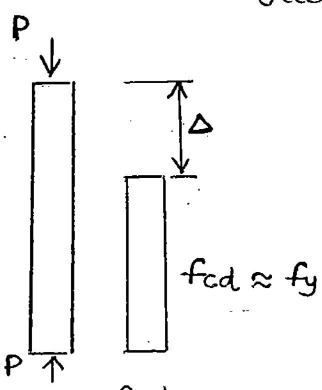
⊙ Boom – Principle compression member in a crane system.

→ Types of Failures in a Compression Member

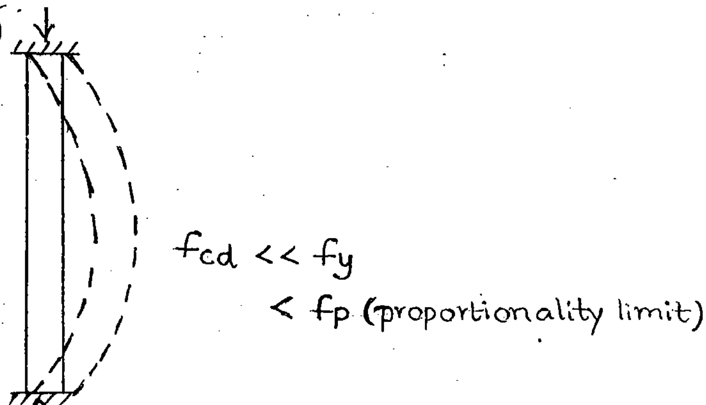
(i) Short Column or short strut – Normally fail by yielding or crushing of a material.

(ii) Intermediate Column or Intermediate strut – generally fail by inelastic buckling.

(iii) Long Column or Long strut – generally fail by elastic buckling.



(Yielding failure in Short Column)



(Elastic buckling failure in long column)

— Compressive strength of a column or strut is influenced by:

(i) Initial imperfection in the member (due to handling, transportation & erection).

(ii) Residual stress in the c/s (developed due to cooling from high temp while moulding to ambient temp.)

(iii) Eccentricity of loading (creating additional moments)

→ Design Compression Strength of a Member (P_d)

$$P_d = f_{cd} \cdot A_e$$

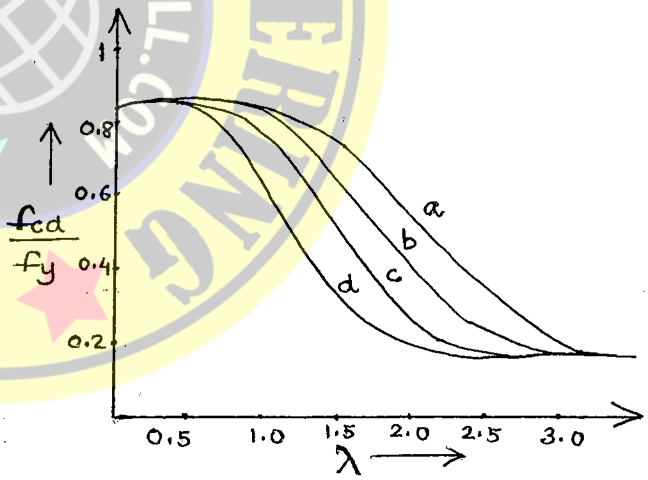
f_{cd} → design stress in axial compression

A_e → effective sectional area.

⊙ IS 800:2007 proposes multiple curves a, b, c & d based on PERRY ROBERTSON's Approach.

λ = non-dimensional effective slenderness ratio.

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



f_{cc} = elastic buckling stress

$$= \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}$$

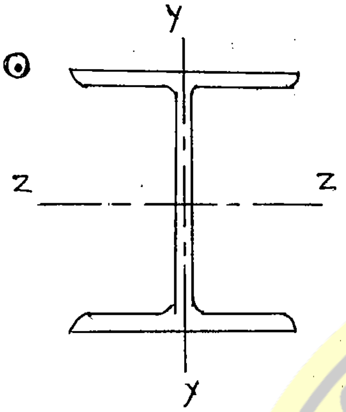
⊙ As per IS 800:1984,

$$P_d = \sigma_{ac} \cdot A_e$$

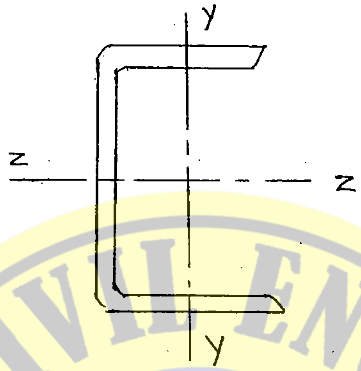
$$\sigma_{ac} = \frac{0.6 f_y f_{cc}}{\left((f_y)^n + (f_{cc})^n\right)^{1/n}} \leq 0.6 f_y$$ (MERCHENT RANKINE FORMULA)

⊙ Class a (tubular sections) members have more compressive strength compared to class c (solid circular sections). Both class a & c members are initially straight and free from eccentric loading. But residual stresses will be more in class c compared to class A as uniform cooling takes place in the interior and exterior of tubular sections.

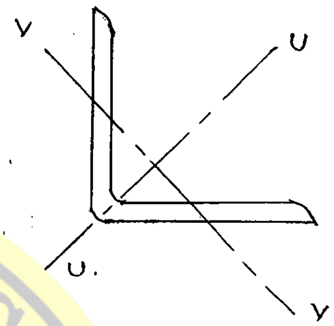
⊙



$r_{zz} > r_{yy}$



$r_{zz} > r_{yy}$



$r_{uu} > r_{vv}$.

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

∴ λ will be more along y-y axis for bending. Higher λ means lower $\frac{f_{cd}}{f_y}$ value from the curve.

$$\therefore (P_d)_{z-z} > (P_d)_{y-y}.$$

Economic sections or best sections are those in which

$(P_d)_{zz} = (P_d)_{yy}$. (tubular circular sections).

* Perry Robertson's Equation

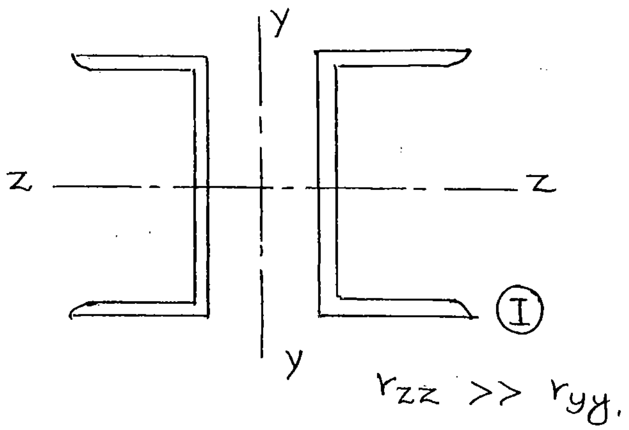
$$f_{cd} = \frac{f_y/\gamma_{m0}}{\phi + (\phi^2 - \lambda^2)^{0.5}} \leq \frac{f_y}{\gamma_{m0}}$$

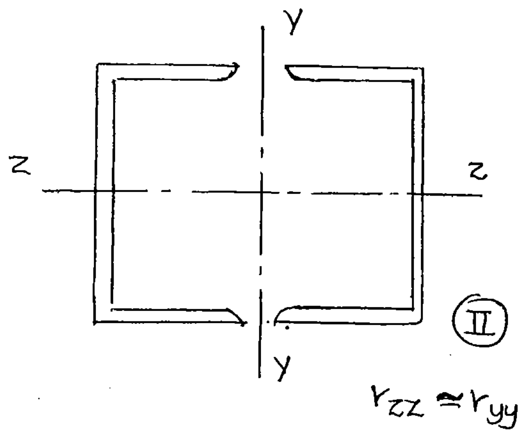
λ → non dimensional effective slenderness ratio $= \sqrt{\frac{f_y}{f_{cc}}}$

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

II^{nd} section will be subjected to eccentric loading wrt y axis

However for I^{st} section, loading will be CG of section.





For same area of section, II^{nd} section will be more economical and effecient than I^{st} section.

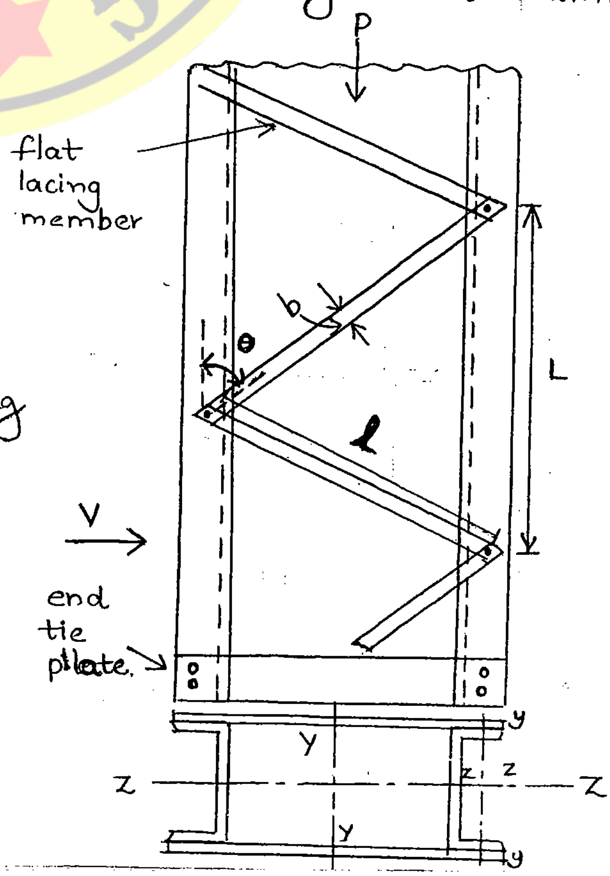
- For struts, tacking bolts, or tacking welds or tacking rivets are used to connect different sections.
- For columns, bracings or lacing systems are used.

* Connecting Systems for Built up Columns are:

- Lacing System (preferred for eccentric loaded column)
- Batten system (generally used for axially loaded columns)

* Lacing System

- Flats, angles, channels or tube sections are mostly used as section for lacing member.



$\theta \rightarrow$ angle of inclination of lacing with longitudinal axis.

$L \rightarrow$ spacing of lacing member

$b \rightarrow$ width of flat lacing

$l \rightarrow$ length of lacing member.

Step 3 : Select a trial section with approximate area equal to area required with higher minimum radius of gyration.

Step 4: Based on end condition, KL and effective slenderness ratio (KL/r) and fcd of a trial section may be calculated.

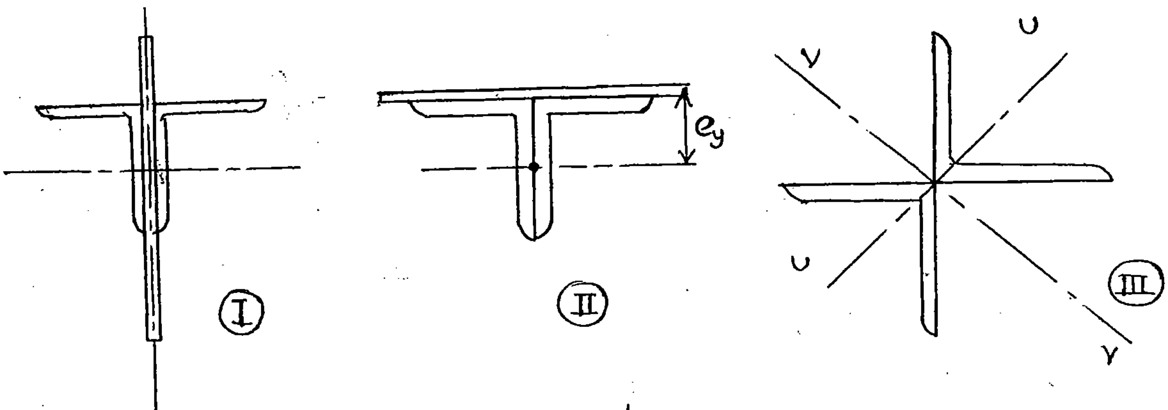
Step 5: $(P_d)_{trial} = (f_{cd})_{trial} \times A_e$.

If $P \leq (P_d)_{trial}$; section is safe. Otherwise same procedure is repeated with higher c/s area.

* Limiting Slenderness Ratio.

	Limiting λ
⊙ A compression member subjected to compressive loads resulting from DL + LL combination.	≤ 180
⊙ A member subjected to compressive loads resulting from wind load or earthquake loads, WL / EQL	≤ 250
⊙ For compression flange of beam.	≤ 300

→ Built up Sections (Built up Coloumns)



Out of three sections, III[rd] one is most effecient as more area thrown away from buckling axis and hence more r

$\frac{KL}{r}$ → effective slenderness ratio.

KL → effective length of a compression member.

K → effective length constant ; L → unsupported length.

$$\phi = 0.5\left(1 + \alpha(\lambda - 0.2) + \lambda^2\right)$$

α → imperfection factor.

Buckling Class	a	b	c	d
α	0.21	0.36	0.49	0.76

It considers —

a) Eccentricity of a load.

b) Initial straightness.

c) Residual stress present in c/s

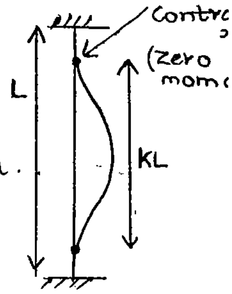
→ Effective Length of a Compression Member : (KL)

KL depends upon :—

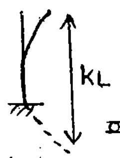
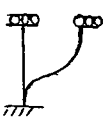
(i) type of end condition

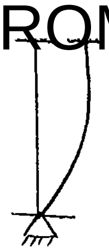
(ii) No: of members meeting at a joint location.

(iii) No: of rivets or bolts used at a joint.



* Effective Length of a Column (KL)

Type of End Condition		Effective Length (KL)	Effective length constant (K)
Fixed — Fixed.		0.65L	0.65
Fixed — Pinned (or hinged)		0.80L	0.80
Pinned — Pinned (Hinged — Hinged)		1.0L	1.0
Fixed — Free		2.0L	2.0
Fixed — Moment Roller.		1.2 L	1.2

Pinned-moment roller		2.0L	2.0

18th Sept,
THURSDAY

→ Design of Axially Loaded Compression Members.

* Design Requirement for Safety of Compression Members:

- Design compressive load (P) ≤ Design compression strength of a member.

- $\left(\frac{KL}{r}\right)_0$ of section ≤ Limiting Slenderness Ratio. (to take care of erection loads)

⊙ Design of compression member is an indirect method of design as the failure stress, fcd, depends on lot of factors.

$$f_{cd} = f\left(\frac{KL}{r}, \alpha, \phi = \sqrt{\frac{f_y}{f_{cc}}}\right)$$

⊙ For a tension member, section is dependent on A_g.
For a compression member, section is dependent on A_e & r
For a beam, section is dependent on moment of inertia.

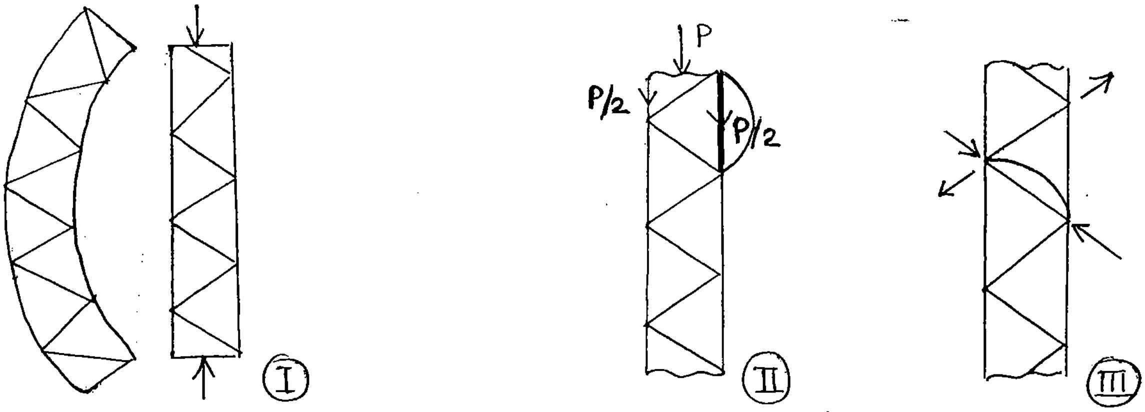
Step 1: Assume design stress in axial compression (f_{cd})

f_{cd} = 90 MPa (for angle struts)
= 135 MPa (for I-section columns)
= 200 MPa (for heavily loaded columns)

Step 2: Effective sectional area required for factored compressive load, P is :

$$(A_e)_{req} = P/(f_{cd})_{assumed}$$

– Failure of lacing system



(I) → buckling of whole component.

(II) → local buckling of column component.

(III) → local buckling of lacing member.

– General Specifications:

⊙ The radius of gyration normal to the plane of lacing not less than parallel to the plane of lacing

ie, $r_{yy} \nless r_{zz}$

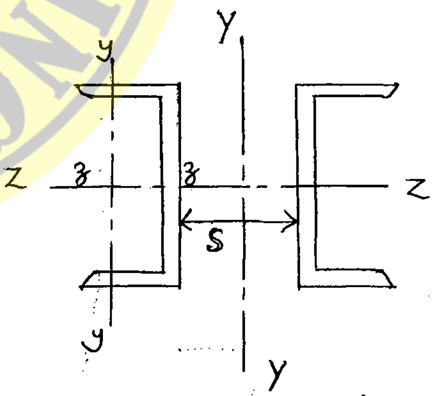
To have economy in design, condition must be

$$r_{yy} = r_{zz} \Rightarrow I_{yy} = I_{zz}$$

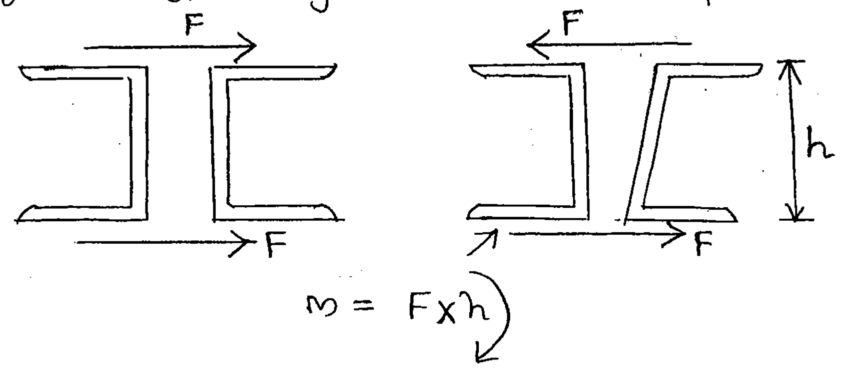
$$I_{zz} = I_{yy}$$

$$2I_{zz} = 2\left(I_{yy} + Az^2\right)$$

$$= 2\left(I_{yy} + A\left(\frac{S}{2} + C_{yy}\right)^2\right)$$

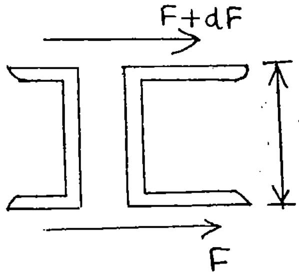


⊙ For single lacing system, lacing on one plane should reflect mirror image to the other plane.



$$M = F \times h$$

⊙ There should not be any variation in lacing system



$M = dF \times h$

– Design Specifications: (important)

⊙ $40^\circ \leq \theta \leq 70^\circ$

Optimum angle for lacing member, $\theta = 45^\circ$ to 50°

If $\theta < 40^\circ$, lacing member may chance to carry some of the the column load and same load may transfer from one column component to another column component like truss member.

If $\theta > 70^\circ$, the tieing force carrying capacity of lacing member is decreased.

⊙ Effective slenderness ratio of lacing member should be less than or equal to 145.

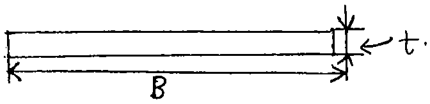
$$\left(\frac{KL}{r}\right)_{lacing} \leq 145$$

Above condition is required to avoid local buckling of lacing member b/w individual column components.

	Kl
(i) For single lacing system with one bolt or one rivet.	1.0 l
(ii) For single lacing with welds.	0.7 l
(iii) For double lacing system	0.7 l

where Kl → effective length of lacing member (Kl)

For flat lacing (B x t)



$$I_{min} = \frac{Bt^3}{12}$$

$$r_{min} = \sqrt{\frac{I_{min}}{A}} = \sqrt{\frac{Bt^3}{12 \cdot Bt}} = \frac{t}{\sqrt{12}}$$

$$\frac{\sqrt{12}\, Kl}{t} \leq 145$$

Min width of flat lacing member, $B_{min} \simeq 3 *$ shank diameter of bolt.

Shank diameter of bolt	M16	M18	M20	M22
B (min) (mm)	50	55	60	65

- Min thickness of lacing member

  $t_{min} \not< \frac{l}{40}$; for single lacing system

  $\not< \frac{l}{60}$; for double lacing member.

- $\frac{L}{r^c_{min}} \leq 50$

  $\leq 0.7 \times$ effective slenderness ratio of whole built up column $\left(\frac{KL}{r}\right)_0$

  whichever is less.

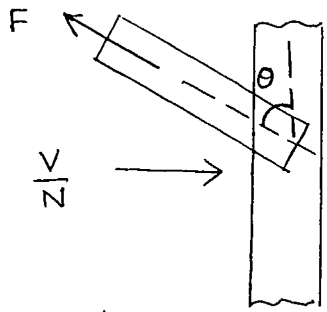
$r^c_{min} \rightarrow$ min. radius of individual column component.

The above condition is required to eliminate local buckling of individual column component b/w lacings.

Lacing members ~~may~~ should be designed for a transverse shear of 2.5 % factored column load.

Transverse shear, V = 2.5 * factored column load

$$V = \frac{2.5\,P_u}{100}$$



$$\frac{V}{N} - F\sin\theta = 0$$

$$F = \frac{V}{N}\sin\theta$$

- Design axial force in lacing member (F)

$$F = \frac{V}{N\sin\theta}$$

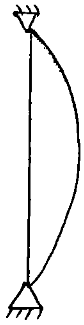
$$F = \frac{V}{2\sin\theta} \quad (\text{for single lacing, } N = 2$$

$$= \frac{V}{4\sin\theta} \quad (\text{for double lacing system } N = 4)$$

- Effective slenderness ratio of lacing column should be increased by 5% in order to take care shear deformations due to unbalanced horizontal forces in lacing members.

LEVEL-2
P-52
9th SEPT, FRIDAY

1. Effective slenderness ratio, $\frac{KL}{r} = 200$

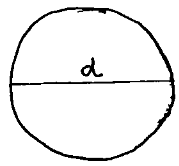


$\frac{l}{d} = ?$

$KL = 1.0\,L$ (Ends are hinged)

$\frac{KL}{r} = 200$

$\therefore \frac{L}{r} = 200$



$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{\pi d^4/64}{\pi/4\ d^2}} = \frac{d}{4}$

$\frac{L}{d/4} = 200 \Rightarrow \frac{l}{d} = 50$

4. Transverse shear, $V = \frac{2.5P}{100}$

P → Factored load ; P_s → service load.

$P = \gamma_L \times P_s$

$= 1.5 \times 1000 = 1500$ kN.

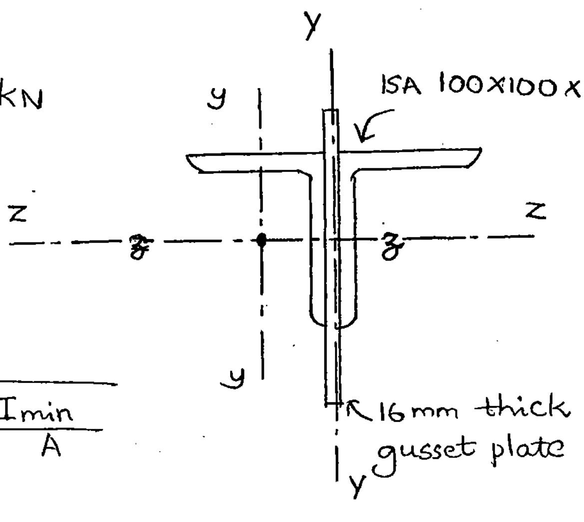
$\therefore V = \frac{2.5 \times 1500}{100} = 37.5$ kN

5. Axial force in lacing, $F = \frac{V}{N\sin\theta}$

$\theta = 45^0$; $N = 2$.

$F = \frac{37.5}{2\sin 45} = 26.52$ kN

6. $A = 1903\ mm^2$



$I_{zz} = I_{yy} = 177 \times 10^4\ mm^4$

Min radius of gyration, $r_{min} = \sqrt{\frac{I_{min}}{A}}$

I_{min} = min of I_{yy} or I_{zz}.

$I_{ZZ} = 2(I_{GG} + Ay^2)$

$= 2(I_{zz} + A0^2) = 2 I_{zz}$.

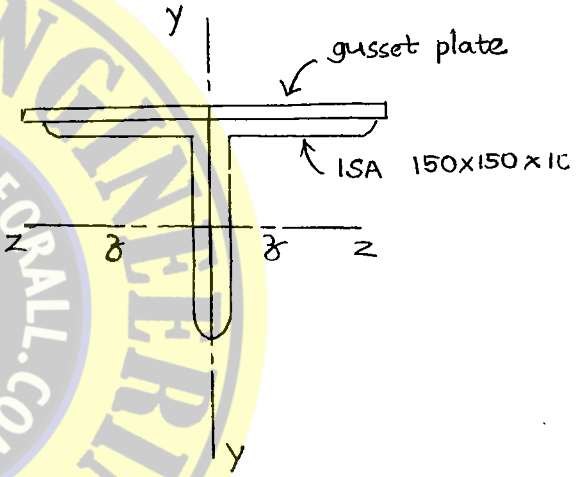
Gusset plates are provided only at points of junctions.
$\therefore$ I & A of gusset plates are not included

$I_{YY} = 2(I_{yy} + A(t/2 + C_{zz})^2)$.

$\Rightarrow I_{ZZ} < I_{YY} \quad \therefore I_{min} = I_{ZZ}$.

$r_{min} = \sqrt{\frac{2 I_{zz}}{2A}} = \underline{30.6 \text{ mm}}$

8. $I_{min} = \min\ I_{ZZ}$ or I_{YY}



$I_{ZZ} = 2(I_{zz} + \cancel{Ay^2}^{\,0})$

$= 2 I_{zz}$

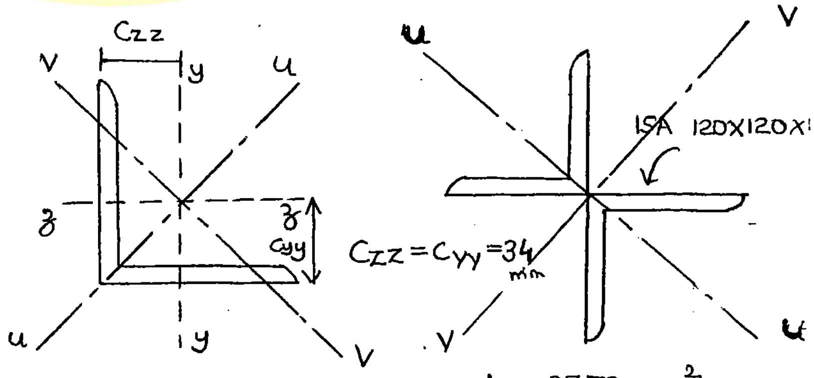
$I_{YY} = 2(I_{yy} + Az^2)$

$= 2 I_{yy} + 2A(40.8)^2$

$I_{min} = I_{ZZ}$

$r_{min} = \sqrt{\frac{I_{min}}{A}} = \sqrt{\frac{2 I_{zz}}{2A}} = \sqrt{\frac{6.335 \times 10^6}{2921}} = \underline{46.57 \text{ mm}}$

9. $r_{min} = \sqrt{\frac{I_{min}}{A}}$



$I_{min} = \min$ of I_{VV} or I_{UU}

$I_{UU} = 2(I_{uu} + Av^2)$

$= 2(584 \times 10^4 + 2750 \times 0^2)$

$= 1168 \times 10^4 \text{ mm}^4$

$I_{VV} = 2(I_{vv} + Au^2)$.

$= 2(151 \times 10^4 + 2750 \times 2312)$.

$= 1573.6 \times 10^4$

$A = 2750 \text{ mm}^2$

$I_{zz} = I_{yy} = 368 \times 10^4$

$I_{uu} = 584 \times 10^4$

$I_{vv} = 151 \times 10^4$.

$u^2 = C_{zz}^2 + C_{yy}^2 = 34^2 + 34^2 = 2312$.

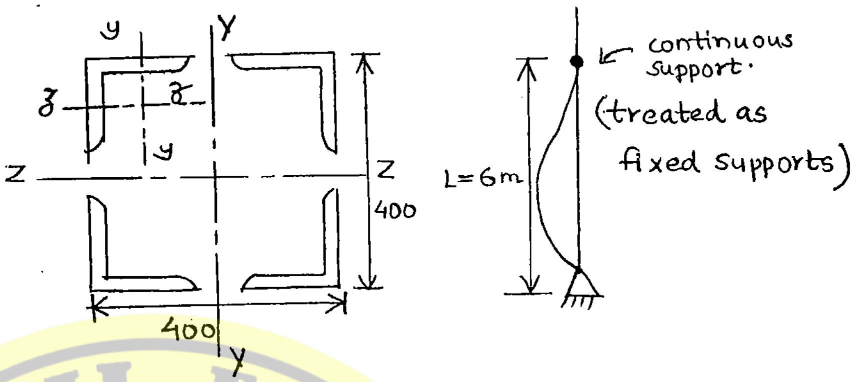
$I_{min} = I_{uu} = 1168 \times 10^4 \ mm^4$.

$$r_{min} = \sqrt{\frac{I_{uu}}{A}} = \sqrt{\frac{1168 \times 10^4}{2 \times 2750}} = \underline{\underline{46.08 \ mm}}$$

10. $A = 1903$, $I_{zz} = I_{yy} = 177 \times 10^4 \ mm^4$.

$C_{yy} = C_{zz} = 28.4$ mm.

$r_{yy} = r_{zz} = 30.5$ mm.



P-47 : 6th Point.

Buckling about any axis of built-up member → class C

Design axial compressive load, $P = f_{cd} \times A_e$.

f_{cd} = Design axial compressive stress

A_e = Effective sectional area $= 4 \times 1903 \ mm^2$

$$f_{cd} = \frac{f_y/\gamma_{mo}}{\phi + (\phi^2 - \lambda^2)^{0.5}} \leq \frac{f_y}{\gamma_{mo}}$$

Assume grade of steel, Fe 410

$\Rightarrow f_y = 250$ MPa, $\gamma_{mo} = 1.10$

Built up members — class C $\Rightarrow$ Imperfection factor, $\alpha = 0.49$

λ = non dimensional effective slenderness ratio

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

$KL = 0.8L$ (one end hinged & other end fixed).

$= 0.8 \times 6000 = 4800$ mm.

MI of built up section : $I_{zz} = I_{yy} = (I_{GG} + Ay^2)\, 4$

$$= 4\left(177 \times 10^4 + 1903 \times \left(\frac{400}{2} - 28.4\right)^2\right)$$

$$= 231.227 \times 10^6 \ mm^4.$$

$$r_{min} = r_{yy} = r_{zz} = \sqrt{\frac{I_{zz} \text{ or } I_{yy}}{4A}}$$

$$= \sqrt{\frac{231.227 \times 10^6}{4 \times 1903}} = 174.28 \ mm.$$

Effective slenderness ratio $= 1.05 \dfrac{KL}{r}$ (5% ↑ for built up column).

$$\lambda = \sqrt{\frac{f_y \left(\frac{KL}{r}\right)^2}{\pi^2 E}} = \sqrt{\frac{250 \times \left(\frac{1.05 \times 4800}{174}\right)^2}{\pi^2 \times 2 \times 10^5}} = \underline{\underline{0.325}}$$

$$\phi = 0.5\left(1 + \alpha(\lambda - 0.2) + \lambda^2\right)$$

$$= 0.5\left(1 + 0.49(0.325 - 0.2) + 0.325^2\right)$$

$$= 0.5834$$

$$f_{cd} = \frac{f_y / \gamma_{m0}}{\phi + (\phi^2 - \lambda^2)^{0.5}} = \frac{250/1.10}{0.58 + (0.58^2 - 0.325^2)}$$

$$f_{cd} = 214.33 < \frac{250}{1.1} \ N/mm^2$$

$$P_d = f_{cd} \times A = 214 (4 \times 1903)$$

$$= 1628 \times 10^3 \ N = \underline{\underline{1628 \ kN}}$$