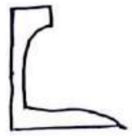
STEEL

IS rolled steel beam section :- (4) series

Section	classification	
Beam	ISJB – Ind. std. Junior Beam	
	ISLB →	light weight beam
	ISMB →	medium weight beam
	ISWB →	wide flange beam
column or Heavy weight beams	ISSC →	column Sections
	ISHB →	Heavy weight beam
channels	ISJC →	Junior channels
	ISLC →	light weight channel
	ISMC →	medium weight channel
	ISMCP →	medium weight parallel-flange channel
Angles	ISA →	equal angles
	ISA →	unequal angles
	ISBA → 	Bulb angles

various types of rolled structural steel section

(I) Rolled steel I section

(II) ——— channel "

(III) ——— Angle "

(IV) ——— T "

(V) ——— Tube "

(VI) ——— bars .

(VII) ——— flats "

(VIII) ——— plates "

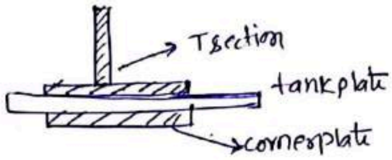
(IX) ——— sheets "

(X) ——— strips "

T-section Application :-

(I) used to transmit bracket loads to column.

(II) used with flat strips to connect plates in the steel ~~tanks~~ rectangular tanks.



Angle section :- uses :-

- designed for resisting axial force (comp & Tension) and transverse forces as purline
- they may be used as connecting elements to connect structural elements like sheets/plates or to form a built up section.

Bulb section :-

- provide Better plate stiffening
- used in ship building
- when the stress is under extreme stress and starts to buckle, this shape is highly resistant and increases the longevity of the structure.

main advantage of steel member :-

(I) High strength to weight ratio

(II) Gas & water tightness

(III) Longer Life

(IV) No sudden failure

(V) Economy in transportation & Handling

(VI) Termite proof & rot proof.

(VII) formwork

Steel density 7850 kg/m³

Rolled steel beams :

① mainly used to resist bending stress when they are designed as beam.

② are used as independent section to resist compressive stress when they are designed for column / strut.

3) are used as independent section to resist tensile stress when they are designed as tie.

design wind pressure (IS 875 part-3) $\boxed{P_z = 0.6 V_z^2}$ P_z (N/m²), V_z (m/s)

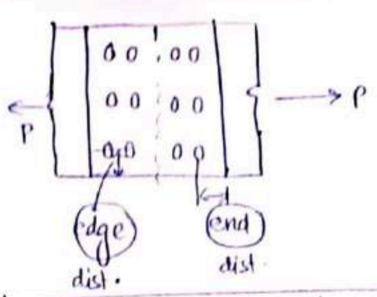
$\boxed{V_z \text{ design wind velocity} = V_b \times \{k_1 \times k_2 \times k_3\}}$

k_1 = Risk cofficient or Probability factor
k_2 = Height, terrain, str. size factor
k_3 = Topography factor.

method of Design of steel framework :-

Simple design	• Based on elastic theory • assume pin joint (no moment transfer) • all connection of beams, girder, trusses are virtually flexible. • most uneconomical method
Semi-rigid Design	• ensure partial fixity is available at support • This method permits a reduction in max. BM in beam suitably connected to support due to partial transfer of moment to another connected members. • economical Than simple design
Rigid Design	• assumption → end connections are fully rigid and capable of transmitting moment & shears • frames sufficient rigid to hold virtually unchanged original angles b/w such members and member they connect. • It is used in convenient cases and gives economy in the weight of steel and saves construction cost

axial compression/tension bending compression/tension	→ $\frac{f_y}{1.1}$	f_y
structural steel	→ $\frac{f_y}{\sqrt{3}\times 1.1}$	
bolt	→ $\frac{f_u}{\sqrt{3}\times 1.25}$	f_u



design strength of bolted connection $\Rightarrow$ min
- strength of bolt in shear (V_{dsb})
- strength of bolt in bearing (V_{dpb})
- strength of bolt in tension (T_{db}) (if exist)

min edge/end dist
- handflamed cut edges & sheared $= 1.7\phi'$
- machine flamecut sawn, plane edges $= 1.5\phi'$

max. edge distance $= 12t\epsilon$ (unstiffened part) $\quad \epsilon = \sqrt{\frac{250}{f_y}}$ $\quad t \rightarrow$ thinner outer plate

→ note :- This will not apply to fasteners interconnecting the components of back to back tension member.

where member exposed to corrosion → max edge dist $\Rightarrow$ $40 + 4t$

design shear strength of bolt or shear capacity of bolt V_{dsb} $= \frac{f_{ub}}{\sqrt{3}\times 1.25}\left[n_n A_{nb} + n_s A_{sb}\right]$

$n_n \rightarrow$ no. of shear plane with thread intercepting plane

$A_{nb} \Rightarrow$ net shear area of bolt at thread $= 0.78\,A_{sb}$
$= 0.78 \times \frac{\pi}{4} d^2$

$A_{sb} \Rightarrow$ nominal shank area of bolt

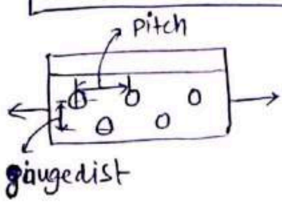
min end/edge dist. required 'or' Bolt holes should not placed too near to the edge Reasons →

(i) The failure of Plate in tension may take place

(ii) The steel of the plate opposite to the hole may bulge out and may crack

note:- Reduction factors for shear capacity of Bolt :-

long joint (β_{lj})	Bolt ($L_j > 15d$) $\quad$ 000000 (L_j) $\beta = 1.075 - (0.5\%)\frac{L_j}{d}$ $\quad 0.75 \le \beta \le 1$ weld ($L_j > 150\,t_t$) → Throat thickness $\beta = 1.2 - 0.2\frac{L_j}{150\,t_t}$	
griplength	$\beta = \frac{8d}{3d + L_g}$	($L_g > 5d$) $\quad L_g$
Packing Plates	($t_{pk} > 6$mm)	$\beta = 1 - 125\times 10^{-4}\,t_{pk}$



Staggered pitch :- c/c dist of staggered bolt measure in direction of load

Bearing capacity of bolt $= \frac{2.5 k_b\, d\, t\, f_u}{1.25}$

$k_b = \min\left\{\frac{e}{3d_0},\ \frac{p}{3d_0} - 0.25,\ \frac{f_u}{f_{ub}},\ 1.0\right\}$

$e \rightarrow$ end, $p \rightarrow$ pitch, $d_0 \rightarrow$ bolt hole, $f_u \rightarrow$ plate, $f_{ub} \rightarrow$ bolt

min pitch required ?

① to prevent failure of member b/w 2 consecutive rivets.

② to permit efficient installation of rivet

③ to provide adequate resistance to tearout of bolt/rivet

min pitch → 2.5ϕ

max. pitch (32t, 300)
- compression member $(12t, 200)$ *
- tension member $(16t, 200)$ * — thinner

tacking rivet max pitch
- comp. member 600 mm
- Tension member 1000 mm

+

$(32t, 300) \rightarrow$ not exposed to weather

$(16t, 200) \rightarrow$ exposed to weather

note:- tack rivets are used when a member (consist of 2 elements) section, which is suppose to act as single unit then they are — connected by tack bolts.
- tack rivets used to prevent local buckling in compression member.

compression member :- where force are transferred through Butting face

max. pitch = $4.5\phi'$* for a distance of 1.5 X width of member from Butting face.

The distance b/w the centre of any consecutive fasteners in a line adjacent and parallel to an edge of an outside plate

$\not> (100 + 4t, 200)$* in comp. & tension member.

special note :- if staggered fastener :-> at equal interval & $g \not> 75$mm then spacing* increase by 50%.

LSM — bolt hole (clearance)

ϕ (d) (mm)	ϕ' (d_0)
≤14	+1
>24	+3
(16-24)	+2

WSM

rivet d	d1 (Clearance)
≤25	+1.5
>25	+2

Tensile capacity of bolt = $\min\left\{\frac{A_g \times f_y}{1.1}, \frac{A_{net} \times 0.9 f_u}{1.25}\right\}$

Bolt subjected to Shear + Tension: $\left(\frac{V_u}{V_d}\right)^2 + \left(\frac{T_u}{T_b}\right)^2 \le 1$

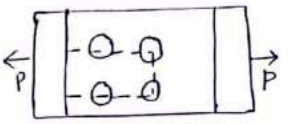
$V_u \rightarrow$ factored SF acting on bolt $\quad V_d \rightarrow$ design shear capacity of bolt

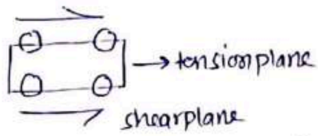
$T_u \rightarrow$ —— tensile force ——

$T_b \rightarrow$ design tension ——

Block shear strength : $\boxed{\text{combination of yielding + rupture (tension)}}$ *

- block shear failure of plate occures along a path involving tension on one plane & shear on perpendicular plane along fasteners.





[in shear: gross area A_{vg}, net area A_{vn}] [in tension: gross area A_{tg}, net area A_{tn}]

① Shear yielding + Tensile rupture

$$\left(A_{vg} \times \frac{f_y}{\sqrt{3} \times 1.1}\right) + \left(A_{tn} \times \frac{0.9 f_u}{1.25}\right)$$

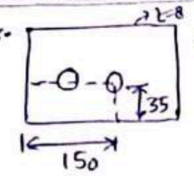
② Shear rupture + tensile yielding

$$\left(A_{vn} \times \frac{0.9 f_u}{\sqrt{3} \times 1.25}\right) + \left(A_{tg} \times \frac{f_y}{1.1}\right)$$

Block shear strength = Min {①, ②}

Imp note:- Block shear failure ⇒ High bearing strength of bolt + High strength of Bolt.

Ex.



$\phi' = 18$

$A_{vg} = 150 \times 8$

$A_{vn} = [150 - (1.5 \times 18)] \times 8$

$A_{tg} = 35 \times 8$

$A_{tn} = (35 - 0.5 \times 18) \times t$ → 8

Prying force :- HSFG bolts → subjected to tensile force then additional forces are considered due to flexibility of connected part

[on basis of Load Transfer]

Bolts type

- (Bearing type)
  [transfer shear by Bearing]
  - Black Bolt or Unfinished Bolt or common bolt or Temporary bolt
    - Imp: ∴ not for Impacts, vibration & fatigue
  - Turned Bolt
- (friction grip type)
  [transfer shear by friction]
  - HSFG Bolt (high strength friction grip bolt)
    - induced initial tension = Proof load = 0.7 × Ultimate tensile strength of bolt.
    - most suitable for bridges where stress reversal may occure.
    - for fatigue load these are ideal

Assumption in design of Rivetted connection :-

1- Rivets are assumed to be stressed equally.
2- The rivet hole is assumed to be completely filled by rivet.
3- friction b/w plate ⟶ Neglected
4- Shear stress is assumed to be uniformly distributed over the gross cross sectional area of rivet.
5- stress in a plate is assumed to be uniform
6- Bending of rivet ⟶ neglected.
7- Bending stress is uniform b/w plate & rivet.

WSM	clearance
$\phi \le 25$	+1.5mm
$\phi > 25$	+2 mm

Unwin's formula

$$\phi = 6.01\sqrt{t_{min}}\ (mm)$$

($t_{min} > 8$)

($t < 8$)

$\sigma_{sc} d t = \frac{\pi}{4} d^2 \sigma_{sc}$

shear   bearing

IS: 800 : 1984 :

permissible stress	value
avg. shear stress	0.40 fy
max. shear stress	0.45 fy
axial comp/tensile stress	0.60 fy
Bending comp/tensile stress	0.66 fy
Bearing stress	0.75 fy
Combined bearing & bending	0.90 fy

(MPa)

Permissible stress in shop rivets :-   value equal

type of rivet	shear	bearing	axial tension
Power driven	100	300	100
Hand driven	80	250	80

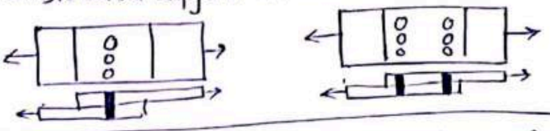
note:- ① field rivets :- (−10%) these values

② if WL or EQ Load is considered then (+25%) above values.

Types of Joints :

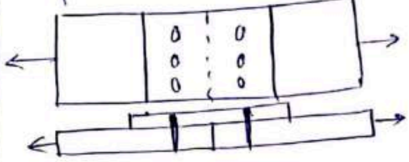
Lap joint — when 2 members to be connected are overlapped & connected together

(i) single bolted Lap joint (ii) double bolted lap joint

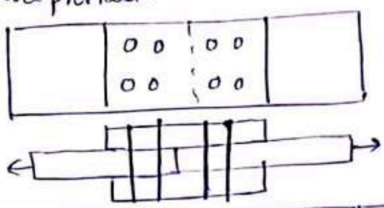


Butt joint — 2 members are placed end to end and are joined by cover plates.

① Single cover butt joint :- when 1 cover plate is provided on one side.



② Double cover butt joint :- when 2 cover plate are provided.

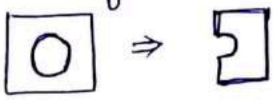


Joint	η %
Lap single rivetted	50-60
double "	60-70
Tripple "	72-80
Butt single rivetted	55-60
double rivetted	76-84
Tripple rivetted	80-88

...ious mode of failure of riveted connection :-

(A) failure of Plate

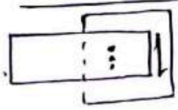
① **By failure of Plate b/w rivet hole & edge**



- due to less/insufficient edge dist.

⇒ to avoid this failure provide min edge dist.

② **Tearing of plate b/w rivet holes :-**

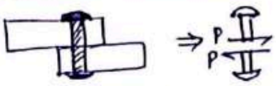


Tearing strength of plate $= (b - nd')\,t\,\sigma_{at}$

n → no. of rivets

(B) failure of rivet

① **failure of rivet in shear :-**



strength of rivet in shear $= \sigma_{shear} \times \frac{\pi}{4} d'^2$

② **Failure of rivet in bearing :-**

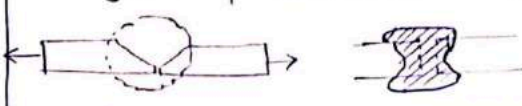
strength of rivet in bearing $= \sigma_{br} \times d' \times t$

t → thickness of thinner plate

Buttweld (Grooveweld)

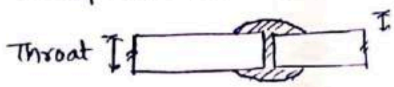
- Provided when member to be jointed are in one plane (i.e Butt joint)

- Butt weld refers to the beads that are deposited in a groove b/w 2 members to be joined



single V buttweld   square groove weld

- size of Butt weld = throat dimension (effective throat thickness)



effective Throat thickness (t_t)

- double U, V → in complete penetration → (thickness of thinner member)
- (single V groove weld) → in incomplete penetration → ($5/8$ of thinner thickness)

note:- reinforcement → extra weld metal which makes the throat thickness at least 10% greater than thickness of welded material

fn → to increase η of joint.

- reinforcement $\ngtr$ 3 mm
- In calculation neglect reinforcement.

axial Strength of Buttweld :- govern by yielding (in comp. or tension)

$$\Rightarrow \frac{(l_{eff} \times t_t) \times f_y'}{1.25} \rightarrow \text{for shop weld}$$

$$\Rightarrow \frac{(l_{eff} \times t_t) \times f_y'}{1.50} \rightarrow \text{for Field weld}$$

basically $f_y' \rightarrow \min \left\{ \begin{array}{l} \text{axial strength of weld} \\ f_{yw} \end{array}, \begin{array}{l} \text{yield strength of parent metal} \\ f_y \end{array} \right\}$

Shear strength of buttweld :- govern by yielding

$$\Rightarrow \frac{(l_{eff} \times t_t) \times f_y'}{\sqrt{3} \times 1.25} \rightarrow \text{shop weld}$$

$1.50 \rightarrow$ Field weld

- effective length of Buttweld = length of full size weld
- min length of Buttweld = 4 × size of weld = 4S

intermittent Buttwelding :-

min effective length = 4 × S

& spacing $\ngtr 16 t_{min}$ (thinner)

→ note :- The intermittent weld shall not be used in position subjected to dynamic, repetitive and alternating stresses.

Buttweld shall be treated as parent metal with a thickness equal to throat thickness, stress $\ngtr$ permitted in parent metal.

Check for combination of stress in Buttweld need not to be carried out if →

(I) Buttwelds are axial loaded.

(II) in single & double bevel weld :-

[sum of normal + shear stress $\ngtr$ design normal stress]

[shear stress $\ngtr$ 50% of design shear stress]

Buttweld :-

Combined bearing, bending, shear :-

$$f_{eq} = \sqrt{(f_{br})^2 + (f_b)^2 + (f_{br} f_b) + 3q^2}$$

fillet weld :- (check shear criteria)

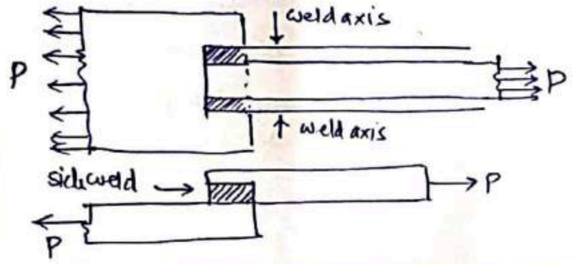
- Provided when 2 members to be jointed are in different plane and Lap joint

(Single fillet) (Double fillet)

- a fillet weld joins 2 surfaces at approximate right angle to each other.

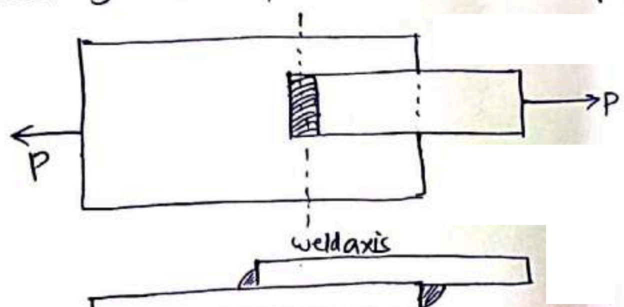
Side fillet weld :- when member with side weld is loaded, the load axis is parallel to the weld axis.

- weld is subjected to shear
- weld shear strength is limited to just about half the weld metal tensile strength
- ductility is high in side weld

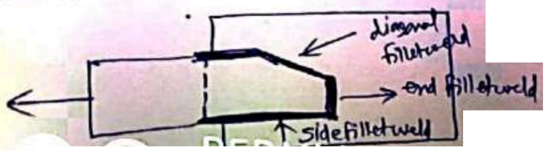


end fillet weld :- when a member with end weld is loaded, the load axis is perpendicular to weld axis.

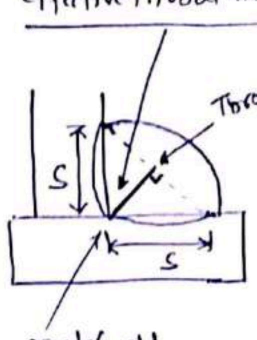
- when a connection with end fillet is loaded in tension, The weld high strength & the strength developed in weld = value of weld metal.



imp note :- end fillet weld stronger than side weld. But for calculation & analysis both are taken same.



effective throat thickness (t) $= ks$ ← size of weld

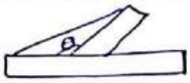


- $t_{min} = 3$ mm
- $t_{max} \not> 0.7\,t_{\text{thinner thickness}}$
- $t_{max} \not> 1 \times$ thickness of thinner member (in special case)

$S \rightarrow$ size of weld (length of smallest side of triangle length)

$t \rightarrow$ throat thickness (shortest perpendicular distance from root to hypotenuse)

$k \rightarrow$ depends on fusion faces angle



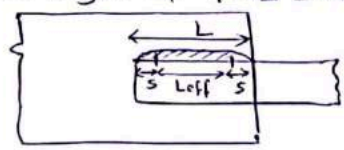
θ degree	k
60-90	0.70
91-100	0.65
101-106	0.60
107-113	0.55
114-120	0.50

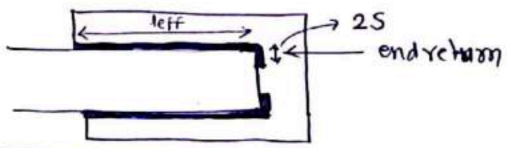
* fillet weld **not** recommended if $\theta < 60$ & $\theta > 120$

effective length of fillet weld :-

$l_{eff} \not< 4S$ ← $l_{eff} = L - 2S$ (L = total length) or $L = L_{eff} + 2S$

- for which specified size & throat thickness of weld exist.
- in practice actual length of weld is made of the effective length + plus 2 times weld size





Intermittent fillet weld :- min effective length = max(40, 4S)

max. clear spacing :-

- compression $\not>(12t, 200)$
- tension $\not>(16t, 200)$

min overlap in Lap Joint $\Rightarrow$ max. $\{40, 4t\}$ → thinner min. size of weld $(S)_{min}$:- (Based on thickness of thicker plate)

thickness of thicker plate (mm)	size of weld mm (S)
0-10	3
10-20	5
20-32	6
32-50	8mm (1st run) 10 mm (final size)

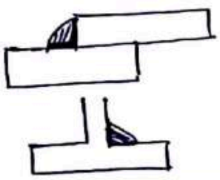
note: (I) min size ?? to avoid risk of cracking in absence of Preheating.

(II) for thicker plate > 50mm special precaution like Preheating of plate will be taken.

max. size of weld :- → based on thinner plate thickness

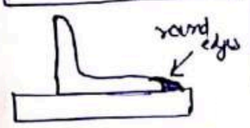
for square edges

⇒ thickness of thinner plate − 1.5



for round edges

⇒ $\frac{3}{4}$ × nominal thickness of round edge



Design shear strength of fillet weld :- → Based on throat area.

$$\Rightarrow (l_{eff} \times t) \times \frac{f_u}{\sqrt{3} \times 1.25 \rightarrow \text{shop} \quad 1.50 \rightarrow \text{field}}$$

f_u → smaller of ultimate stress of weld & Parent metal

t → (KS)

Reduction factor for long joint in weld (side fillet) :-

if $(L_j > 150 t_t)$ $\boxed{\beta = 1.2 - \frac{0.2 L_j}{150 t_t} \leq 1}$

L_j → length of joint or length of side fillet weld in direction of force.

• combined stress in fillet weld — normal stress (f_a), shear stress (q)

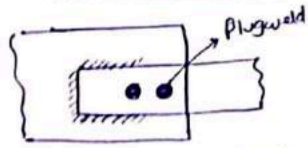
$$\boxed{f_{eq} = \sqrt{f_a^2 + 3q^2} \leq \frac{f_u}{\sqrt{3} \times 1.25 \rightarrow \text{shop} \quad 1.50 \rightarrow \text{field}}}$$

Imp: no need to check combination of stress if

(I) side fillet weld joining cover plates & flange plates.

(II) fillet weld where normal stress + shear stress ≯ f_{wd} = $\frac{f_u}{\sqrt{3} \times 1.25 / 1.50}$

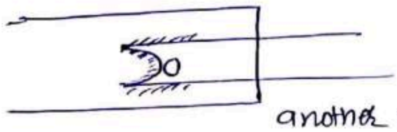
Plug weld :-



→ in which small holes are made in one plate and kept over another plate to be connected & then entire hole is filled with filler material

Slot weld :- used when overlapping length of weld is smaller than required weld Length so to increase weld length are made

• in which a Plate with circular hole is kept with another plate to be joined & then fillet weld is made along the periphery of hole.



Ties → steel members designed to carry axial Tensile load (tension member)

↳ tie beam in a truss is a horizontal beam connecting 2 rafters.

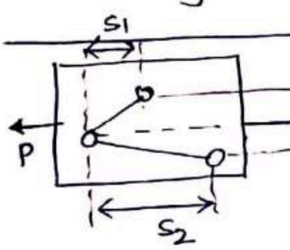
<u>various forms of tension member</u> :-

① <u>wires</u> → wire ropes are exclusively used for hoisting purpose

② <u>cables</u> → Used in suspension bridge.
- generally long, flexural stiffness → negligible
- initial sag & other geometrical effect must be accounted in the design.

③ <u>Rods & bars</u> :- used for small tension members
- such members in general welded to gusset plate or may be threaded and bolted

④ <u>plates & flats</u> :- used as tension member in - transmission tower, foot bridge.
They are also used in column to keep the component member in their correct position like lacing flat, batten plates, end tie plate



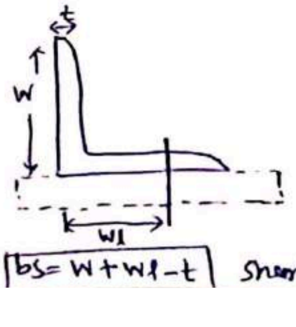
Anet ⇒

$$\left[B - n \times d' + \frac{s_1^2}{4g_1} + \frac{s_2^2}{4g_2}\right] \times t$$

S → pitch g → gauge n → no of holes

<u>Rupture strength of Angle</u> :- connected Through 1 leg is affected by shear lag.

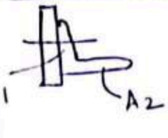
$$\Rightarrow \boxed{\frac{(0.9 f_u)\, A_{net\,connected}}{1.25} + \beta\, \frac{A_{gross\,unconnected} \times f_y}{1.1}}$$



$\boxed{b_s = W + W_1 - t}$ shear lag width

$\boxed{b_s = W}$

- single angle connected only 1 leg :-

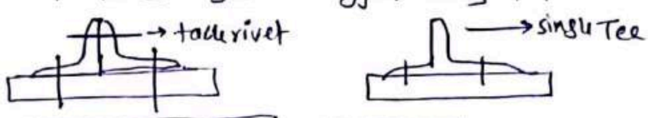


$\boxed{A_{net} = A_1 + kA_2}$ $\boxed{k = \frac{3A_1}{3A_1 + A_2}}$

$\boxed{A_1 = (l_1 - d' - t/2)t}$ $\boxed{A_2 = (l_2 - t/2)t}$

- pair of angle placed back to back (or a single Tee) connected by only one leg of each angle (or by the flange of tee) to the same side of gusset plate

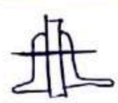
'or' if the 2 angles are tagged along a-a



$\boxed{A_{net} = A_1 + kA_2}$ $\boxed{k = \frac{5A_1}{5A_1 + A_2}}$

A_1 → area of connected leg
A_2 → Area of outstand

- if 2 angles are placed back to back and connected to both side of gusset plate

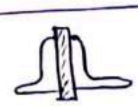


$A_{net} = A_1 + A_2$ (k=1) when tack riveted

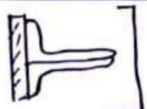
if not tack riveted then both will be considered separately and use $k = 3A_1/(3A_1 + A_2)$

WSM :- strength or Anet↑ : opposite tack riveted > same side tack riveted > not tack riveted (whether same side or opposite side)

LSM :-



&



design strength = 2× one angle strength.

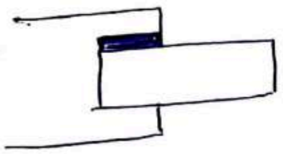
Type of member	λ_{max} (Slenderness ratio)
lacing bar in compression	145
Bracing member incase of Hangers	160
always compressive (DL+LL)	180
compressive (WL+EQ)	250
tension member (load other than EQ)	180
tension member (WL+EQ)	350
compression flange of beam against lateral torsional buckling	300
member always in tension except in pretensioned member	400

note:- Tension member (Bracing) is Pretensioned to avoid sag
• need not to satisfy max. slenderness ratio

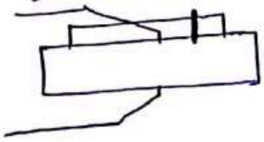
Lug angle :- (I) reduce length of connection (save gusset plate)
(II) reduce shear Lag { ∴ η of Tension member ↑↑↑ }
Thus stress-strain uniform hence no shear Lag.

IS code specifications :- (Lug angle) :-

(I) effective connection of lug angle shall be as far as possible Terminate at end of member



(II) fastening of Lug angle to main member :- shall preferably start in advance of direct connection of the member to the gusset or other supporting member.



(III) lug + gusset plate ⇒ (min 2 bolts)

(IV) main member → angle / channel

(Gm)	angle	channel
Lug angle to gusset plate	$+20\% \times F_{out}$ strength	$+10\% \times F_{out}$
Lug angle to main member	$+40\% \times F_{out}$ strength	$+20\% \times F_{out}$

Shear Lag :- when stress in one part lags behind the other part of section.
• connected Leg will have highest stress at failure than outstanding leg
• Shear Lag reduce the effectiveness of component plates of tension member that are not connected directly to a gusset plate.
Imp (for this reason unequal angle with long leg connected is preferred)

Tension splice :- when 2 tension members are connected use Jt.
• design strength of tension splice → Yield / Rupture / Block shear } min.
• splice connection design force ⇒ max { 0.30 × member design capacity in Tension (gross (yield) / net / Block) ; design action (factored Load) }

IS 800-1984 :-

Merchant Rankine formula :-

$$\boxed{\sigma_{ac} = \frac{0.6 f_{cc} f_y}{\left[(f_{cc})^{1.4} + (f_y)^{1.4}\right]^{\frac{1}{1.4}}}}$$

σ_{ac} → Permissible stress in axial compression

f_{cc} = elastic critical stress in compression

$f_{cc} = \frac{\pi^2 E}{\lambda^2}$ λ slenderness ratio

$\lambda = \frac{l_{eff}}{r_{min}}$

$r_{min} = \sqrt{\frac{I_{min}}{A}}$ A → gross

IS 800-2007 :-

$P_u = f_{cd} \times A_g$ $f_{cd} < \cancel{f_{cc}}$ → $\frac{\pi^2 E}{\lambda^2}$ { based on Perry Robertson approach.

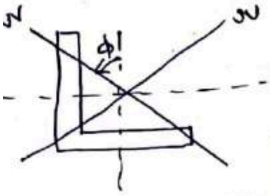
↳ design comp. strength

$$\boxed{f_{cd} = \frac{f_y/1.1}{\phi + \sqrt{\phi^2 - \lambda^2}}}$$ $$\boxed{\phi = 0.5\left[1 + \alpha(\lambda - 0.2) + \lambda^2\right]}$$

$\lambda = \sqrt{\frac{f_y}{f_{cc}}}$

buckling class	a	b	c	d
Imperfection factor α	0.21	0.34	0.49	0.76

Principle axis $I_{xy} = 0$ (product moment of area)



$$\boxed{\begin{matrix} I_{UU} \\ I_{VV} \end{matrix} = \frac{I_{xx}+I_{yy}}{2} \pm \sqrt{\left(\frac{I_{xx}-I_{yy}}{2}\right)^2 + I_{xy}^2}}$$

$$\boxed{\tan 2\phi = -\left(\frac{2 I_{xy}}{I_{xx} - I_{yy}}\right)}$$

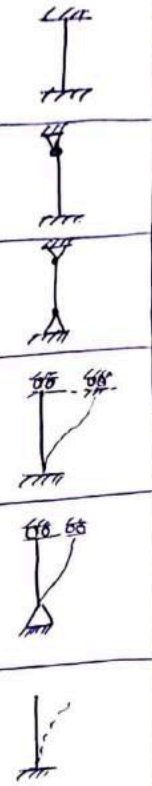
$$\boxed{\begin{matrix} r_{min} \\ (r_{vv}) \end{matrix} = \sqrt{\frac{I_{vv}}{A}}}$$

$\boxed{r_{min} = r_{vv}}$ $\boxed{r_{min} = r_{yy}}$ $\boxed{r_{min} = r_{xx}}$

note :: $\boxed{\text{Product moment of Inertia} \Rightarrow (+), (-), \text{zero canbe}}$

note :: if 2 axis passing through c.g = Principal axis.

effective length of column :



	Theortical Leff	IS code provision
	$0.50 l$	$0.65 l$
	$\frac{l}{\sqrt{2}} = 0.707 l$	$0.80 l$
	l	l
		$1.2 l$
		$2 l$
		$2 l$

note :- (i) effective length :- distance b/w the points of contraflexure

in IS code slight larger value than theortical value to account for the Lack of 100% fixidity at support

(ii) for laced column above value increase by 5%

(iii) for Batten " ——— +10%

for compression member consisting of Angle section :

Section		type	Allowable Permissible stress
single or double angle		continuous	σ_{ac}
single angle	one rivet	Dis—	$0.8\sigma_{ac}$
	>1 rivet or weld	Dis—	σ_{ac}
double Angle	Placed back to back on opposite side of gusset plate	Dis-continuous	σ_{ac}
	Placed back to back on same side of gusset plate	Dis-continuous	$0.8\sigma_{ac}$

1.vimp

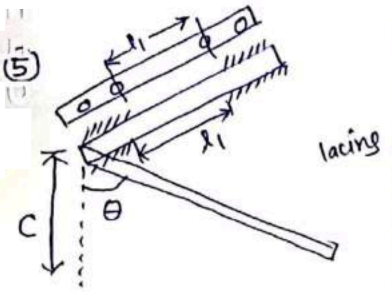
Lacing Built up column :-

① $\lambda_{max} = 14.5 = \frac{l_{eff}}{r_{min}}$ $\left\{ r_{min} = \frac{t}{\sqrt{12}} \right\}$

② $L_{eff} = 1.05 \times L_{eff}$ {increase by 5%}
(to account for shear deformation due to unbalance horizontal SF.

③ design as slender compression member (truss member)

④ lacing member — Rolled section, tubes of equal strength
ISF, ISA, ISLB

⑤ 

$\boxed{40 < \theta < 70}$

lacing

$\frac{C}{r_{min}}$ ≯ 50
≯ 0.7 λ_{whole}

if fails then → θ↑ / Provide double lacing

$l_1 \Rightarrow$ inner to inner of bolt/weld

⑥ effective length

Single lacing	double lacing & welding
$(l_{eff} = l_1)$	$l_{eff} = 0.7\, l_1$

⑦ t_{min} < $\frac{l_1}{40}$ single lacing & welding
$\frac{l_1}{60}$ double lacing

⑧ tie plates at end of lacing system →
• to prevent distortion of Built up c/s due to Unbalance horizontal force.

⑨ lacing should be designed to resist a transverse shear (V) = 2.5% of axial column load

⑩ force in each lacing bar (F) $= \frac{V}{N \sin\theta}$ $\left\{ \begin{array}{l} N = 2 \text{ single} \\ N = 4 \text{ double lacing} \end{array} \right.$

∴ no of rivet req. $= \frac{2F\cos\theta}{R_v}$

⑪ lacing ⟶ Best for eccentric loading

if eccentric load then it is designed to resist additional shear cause due to BM.
↳ due to eccentric load.

⑫ $\boxed{\text{width of lacing bar} = 3 \times \text{dia of bolt}}$

φ	B
16	50
18	55
20	60
22	65

Buckling of strut component (tack Bolted)
Trick T-24

$\frac{l_1}{r_{min}}$ ≯ 40
0.6 λ_{whole}

$l_1 \rightarrow$ dist. b/w tack bolts

Battened Built up column :-
→ only axial load
→ not for eccentric load
→ design as frame

• Batten subjected to long. SF & BM

• flat plate are used for Batten

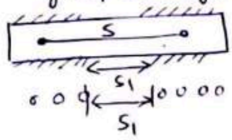
• The no. of Battens ⇒ so that member divided into not less than 3 parts longitudinally
min

{ min. 4 batten plates or min 2 intermediate batten & 2 two end battens }

• effective length of Battened column ⇒ (+10%↑)
increase by 10%.

• V = 2.5% of axial load

• $\boxed{t_{min} \not< \frac{S_1}{50}}$ $S_1 \Rightarrow$ transverse dist b/w centroid of inner end bolt group or rivet group



• $\boxed{f = \frac{VC}{NS}}$

$\boxed{M = f \times S/2 = \frac{VC}{2N}}$

• effective depth of batten (d) ⇒ (D − 2 × edge dist)
D: total Depth of batten

$d > \frac{3a}{4}$	⟶	for intermediate batten ✓
$d > a$	⟶	for end batten ✓
$d > 2b$	⟶	for any batten

Design of slab base (IS800:2007) :

$$t = \sqrt{\frac{2.5w(a^2 - 0.3b^2)}{f_y/1.1}}$$

1) **filler Joist** :- • Steel beam of light section Plain cement concrete.

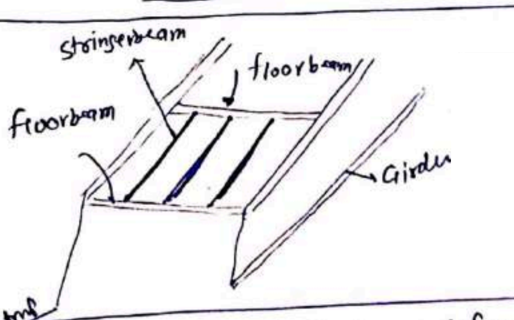
[Joist → a beam supporting floor construction but not a major beam.]

2) **Girder** :- floor beams used in Building
↳ also major beam in any structure.

3) **floor beams** :-
↳ major beam supporting the beams
↳ span b/w girder

4) **Stringer beam** → span b/w floor beam

stringers → members used in bridge parallel to the traffic to carry the deck slab, they will be connected by transverse floor beam



5) **Spandrels** :- exterior beam at floor level of building which carry part of floor load + exterior wall.

6) **Lintel** :- Beam members used to carry wall loads, over wall opening for door, window.

7) **Purlins** :- A roof beam usually supported by roof truss
↳ supported on principal rafter

8) **Rafter** :- A roof beam usually supporting building.

[rafter
purlin
roof truss]

9) **Girt** :- A horizontal members fastened to and spaning b/w periferal column of an industrial building. used to support wall cladding like corrugated metal sheeting.

---

note :- Castellated Beam :- (मतलब बीम में Hole)

↳ wide flange I beam, I beam subjected to longitudinal cut along web.

Design criteria of beam :-

1- Design for BM
2- Design for SF
3- check for Deflection [WSM max permissible deflection in SSB steel ≯ span/325]
4- check for secondary failure
   - local buckling of compression flange or web
   - web crippling under concentrated load

Beam
- laterally supported Beam — whose compression flange restrain against lateral torsional buckling
- laterally unsupported beam → opposite

(i) proving shear connector on compression flange.
(ii) Bracing the compression flange of adjacent beam.

Bending (flexural strength) :- (LSM)

Design BM (M) $\leq$ Design bending strength (M_d)

(A) Laterally supported beam :-

when $\frac{d}{t_w} < 67\varepsilon$ (no shear buckling in web)

Low shear case

factored SF: $V_u \leq 0.6 V_d$

design shear strength $= \frac{A_v \times f_y}{\sqrt{3} \times 1.1}$

A_v → shear area of web

⇓

web of section not susceptible to buckling under shear before yielding.

design bending strength of beam section (M_d)

Plastic & compact section	Semi compact section
$M_d = \frac{(Z_p) f_y}{1.1}$	$M_d = \frac{Z_e f_y}{1.1}$

note: (v.v.imp)

for SSB $M_d < 1.2 \frac{Z_e f_y}{1.1}$
for cantilever $M_d < 1.5 \frac{Z_e f_y}{1.1}$

for slender section

$M_d = Z_e f_y'$

f_y' → reduced design strength

High shear case

$V_u > 0.6 V_d$

Plastic or compact section —

$M_{dv} = M_d - \beta(M_d - M_{fd})$

$\leq \frac{1.2 Z_e f_y}{1.1}$

$\beta = \left(\frac{2V}{V_d} - 1\right)^2$

Semi compact

$M_{dv} = \frac{Z_e f_y}{1.1}$

(B) laterally unsupported :- (refer IS code)

Deflection limit :- Excessive deflection may lead to crack in plaster and may damage the material attached (or) supported by beam

IS 800 : 2007 :

(imp)

Vertical deflection for		value
cantilever span	elastic cladding	$\frac{span}{120}$
	Brittle cladding	$\frac{span}{150}$
~~Simply~~ ~~span~~ Purlin & grit	elastic cladding	span/150
	Brittle cladding	span/180
simply span	elastic cladding	span/240
	brittle "	span/300

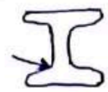
Web crippling :- occures at a point where concentrated load act. (imp)

hence due to reaction at support, high compressive stresses are produced in web close to upper flange or lower flange

- web crippling is a $\boxed{\text{bearing failure}}$
- near the support web of beam may cripple due to lack of bearing capacity

(imp)

- The crippling occures at root of radius.



As per IS code ⇒ to find web crippling of web

$f_w = [(b_1 + n_c) t_w] \frac{f_y}{1.1}$

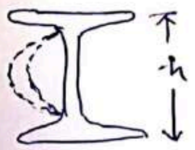
b_1 → $(2.5 t_f)$

n_c → length obtain by dispersion through the flange to web jn at slope 1 : 2.5

note :- How to avoid web ~~buckling~~ crippling

- Provide bearing stiffner
- Provide thicker webs

Web buckling :- • web buckling is the sudden sideway deflection of str. member under Application of compressive load.



• web buckling (vertical buckling occures when Intensity of vertical compressive stress near centre of section becomes greater than the critical bucking stress for the web acting as a column, The buckling of column is much influenced by the restraint provided for the flanges.

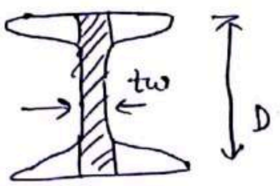
• web buckling is not a problem with rolled beam section, this possibility exist in the thin webs of deep plate girders.

Imp.
• normally if web is safe in crippling it will be safe in buckling also.

V.V Imp conclusion :-

① Local flange Buckling →	due to Bending compression
② Web crippling →	more bearing stress at root of fillet
③ Web buckling →	diagonal compression due to shear

$$\text{Shear capacity of web} = \left[\frac{f_y}{\sqrt{3}\times 1.1}\right](D \times t_w)$$



Built up beam :

① symmetrical built up beam

$$\boxed{\text{Area of each cover plate } A_p = \frac{Z - Z_1}{d}}$$

d → depth of beam

Z_1 → Section modulus of rolled I Section available

② unsymmetrical built up beam

$$\boxed{A_p = \frac{1.2(Z - Z_1)}{d}}$$

Imp
note :- Gross sectional area for flange plate is taken 20% more than the net cross-sectional area to allow for rivet holes and approximation in calculation.

Gantry girder :- (designed as laterally unsupported beam used in Industrial building]

• subjected to unsymmetrical bending due to lateral Thrust.

• type of load acting over this

① Trans-verse load	due to dead load (DL) → Gravity load
② lateral load	due to moving & stopping of crab
③ longitudinal load	movement of truss of rails, starting & stopping of crane

max. Deflection of Gantry girder under DL+LL (IS-800: 1984)

where cranes → manually operated	$\frac{L}{500}$
EOT cranes < 50 t	$\frac{L}{750}$
EOT cranes > 50 t	$\frac{L}{1000}$
other moving loads such as charging car	$\frac{L}{600}$

Seated connection :-

The connection b/w one beam to other beam or column with the angle at top & bottom.

(1) **unstiffened seated connection :-**
if packing stiffner is not provided with angle at top & bottom

(2) **stiffened seated connection :-** if packing stiffner is provided with angle at top & bottom.

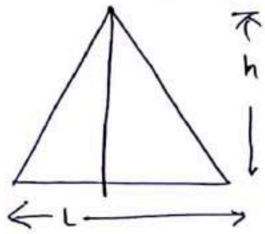
Purlin → flexure member
Girts → unsymmetrical bending

$$\boxed{\text{max. BM in purlin} = \frac{WL^2}{10}}$$

$$\boxed{t = 2p + r}$$

cost of truss, cost of purlin, roof cost } per unit area

• $\boxed{\text{economic spacing of truss} = \frac{L}{3} - \frac{L}{5}}$



$$\boxed{\text{roof truss slope} = 2 \times \text{pitch}\left(\frac{h}{L}\right)}$$

Plate Girder [for Larger & heavy Gravity load]

- riveted or Bolted plate girder
  - Provide Angles (unequal leg with longer leg horizontal)
- welded plate girder
  - provide flatter plate section
    - ∵ (5-15%) less weight than riveted/Bolted plate girder

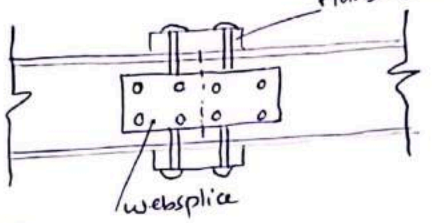
Purpose of stiffner in a plate girder :-

- to prevent buckling of web plate

Vertical stiffner (stability stiffner) (transverse stiffner)	• increase buckling resistance of web against shear • Intermediate vertical stiffners are Joggled.
horizontal / longitudinal stiffner	• increase buckling resistance of web against bending. • provided when the depth of web is more & there is tendency of web buckling. • It is provided when vertical stiffners becomes to close and only thin plates are available for web. • provided in compression zone of web.
Bearing stiffner or end bearing stiffner or load stiffner	• used to transfer concentrated loads on the girder & heavy reactions at support to full depth of web • provide straight • design as column with the length of web 20 times the thickness of web on both side. • It prevents the web from crushing & buckling sideways. • It relieves the rivet connecting the flange angles & web, from vertical shear.
diagonal stiffners	Safe web against shear + bearing
Torsional stiffner	• To transmit Tensile forces applied to web through a flange • provided at support to restrain the girder against Torsional effect.

Flange splice :-

- a joint in the flange element provided to increase the length of flange plate



- flange splice is designed for axial force only.

Web splice :- A joint in web plate to increase its Length

- web splice designed for shear & moment at the spliced section
- the splice plates are provided on each side of web

$\frac{d}{t_w} \le 85$	unstiffened → does not require any stiffner
$\frac{d}{t_w} > 85$	Provide transver stiffner
$85 < \frac{d}{t_w} < 200$	only Intermediate vertical stiffner required.
$200 < \frac{d}{t_w} < 250$	vertical stiffner + 1 horizontal stiffner i.e. at a distance from compression flange equal to 2/5 th of the distance from the compression flange to the neutral axis are provided
$250 < \frac{d}{t_w} < 400$	vertical stiffner + 1 HS + 2nd HS at N.A.

- min distance b/w vertical stiffner ⇒ $0.33d$
- max ⇒ $1.5d$

- min unsupported Length of stiffner = $180 t_w$
- max = $270 t_w$

effective flange area in Compression = $\boxed{A_f + \frac{A_w}{6}}$

Tension = $\boxed{A_f + \frac{3}{4}\left(\frac{A_w}{6}\right)}$

Bernauli → straindiagram linear

$Z_p = A_c Y_c + A_t Y_t$ — first choose equal area axis

↓

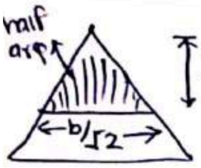
Plastic section modulus

load factor = fos × shape factor

$\frac{mp}{m}$ = $\frac{\text{Collapse load}}{\text{working load}}$; $\frac{my}{m}$; $\left(\frac{Z_p}{Z_e}\right) = \frac{mp}{my}$

margin of safety = fos − 1

half area



$\frac{h}{\sqrt{2}}$; $b/\sqrt{2}$

$$\frac{d^2y}{dx^2} = \frac{1}{R} = \frac{M}{EI} = \frac{\sigma}{y} = \infty$$

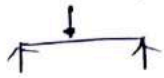
$\because y \to 0$

∴ Rate of change of slope = ∞ at plastic limit or curvature

Section	$\frac{b}{d_f}$	value Ex.	Condition
class-1 (plastic)	<9.5	$\frac{16MP}{l^2}$	Hinge develop, sufficient hinge to fail structure.
class-2 (~~...~~ compact)	9.5-10.5	$\frac{12MP}{l^2}$	Hinge develop, inadequate plastic hinge rotation capacity
class-3 (semi compact)	10.5-15.7	my	extreme fibre at fy, due to local buckling can not develop MP
class-4 (slender)	>15.7	<My	Before reaching My, element buckle locally

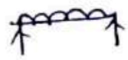
Plastic length Length (l_p) :- (length of elastoplastic zone)
↳ zone of yielding (mp to my)

① SSB (conc. load mid)



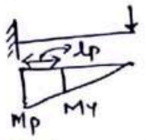
$L_p = L\left(1 - \frac{1}{S}\right)$

② SSB (udl)



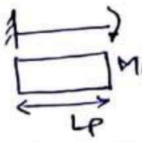
$L_p = L\sqrt{1 - 1/S}$

③ cantilever (conc. load free end)



$L_p = L(1 - 1/S)$

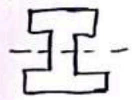
④ cantilever (conc. moment at end)



$L_p = L$

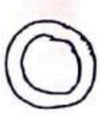
Lp depend → loading (conc. load or udl) ; → Geometry (shape factor (S)) ; → length of beam

Shape factor :- (S)



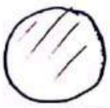
strong axis $\boxed{(1.12 - 1.14)}$ & (1.1-1.2)

weak $\boxed{(1.50)}$



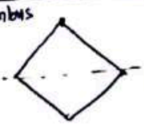
tubular $\boxed{1.27}$ (4/π)

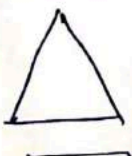
(rectangle) $\boxed{1.50}$; wide flange (weak axis)



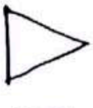
$\boxed{1.70}$ or 1.69 &

Rhombus





$\boxed{2.34}$

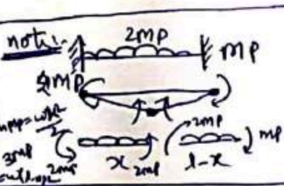


$\boxed{2}$

D/d = k

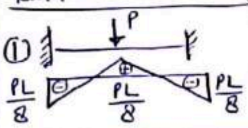
$1.69\left(\frac{1-k^3}{1-k^4}\right)$

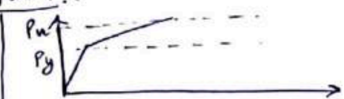
note:-



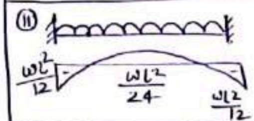
load Deflection diagram :-

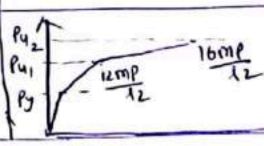
①

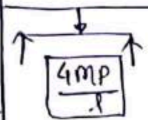


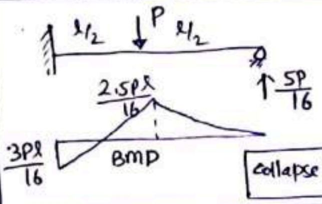


(ii)





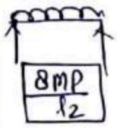


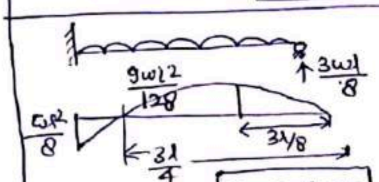


$W_{u_1} = \frac{16}{3}\frac{MP}{l} = 5.33\frac{MP}{l}$

$W_{u_2} = \frac{6MP}{l}$

collapse load $= \frac{6MP}{l}$





$W_{u_1} = 8mp/l^2$

$W_{u_2} = 11.656\frac{MP}{l^2}$

$x = 0.414l$ → Plastic hinge for propped

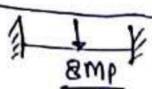
collapse load $= 11.656\frac{MP}{l^2}$

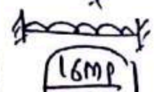
Design

- elastic
  - eqb condition
  - compatability condition
  - moment curvature relation
- Plastic
  - equilibrium
  - mechanism
  - yield condition

(Ub) Kinematic Theorem (upper bound Theorem) (mechanism) $P \geq P_u$

static Theorem (lower bound Theorem) $P \leq P_u$





note:-

① no. of plastic hinge = degree of indeterminacy + 1

② no. of possible independent mechanism = n − r → degree of redundancy.

n → no. of possible plastic hinge