

Steel structures

Iron

Pure Iron (non Alloy)

(silvery white in colour)

It is the natural metal available directly in the earth ores.

- It is very soft solid (ie can be cut even with knife) having high ductility.
- It is not used in any structural element, (since, it is directly reacts with oxygen → form rust & reacts with moist air → leady corrosion)
- Available in two types (i) Fe^{2+} (Ferrous) (ii) Fe^{3+} (Ferric)

Pig Iron

Basic raw iron is called 'Pig Iron' (in the form of bricks) get transported.

- It is also not used in any structural element, (since it is composed of high/more Carbon content, about 5%.)
- It is highly brittle & low ductility
- The pig iron can be converted into structural iron by removing excess Carbon by adding oxygen and chemicals in molten stage.

Cast Iron

It is the structural element (in specified shape from molten pig iron) having almost same property of pig iron

Wrought Iron

The lowest Carbon Content (0.0-0.1%) of structural iron.

- It has high ductility, easily converted in to specified shape.
- largely used to make thin wires.

Steel

It is an alloy composed of (iron + carbon + chromium + copper + magnesium + nickel + silicon ... etc)

- The structural element used to resist any type of loads

Stainless Steel

Contain a high proportion of Chromium and nickel (Although $< 1\%$) are very resistant to corrosion & other chemical reactions, & are easy to clean, polish & sterilize.

- largely used in household cutlery, scissors, & medical instruments.

Percentage of Carbon

⇒ pig iron > Cast iron > Cast steel > Carbon steel
(4-5%) (2-4.5%) (> 2%) (< 2%)

> High Carbon steel > low Carbon steel
(1-2%) (0.2-0.5%)

> Wrought Iron > Pure Iron
(< 0.1%) (0%)

- While Carbon content increases, its brittleness increases.

Structural Steel

Compare with aluminium alloy

- Aluminium alloy's have greater strength/unit wt.
 → Because of less (E), is liable to buckle easily.
 → It is Corrosion resistance & then it is easy to maintenance.

$$\left[\begin{aligned} \rho_{\text{aluminum}} &= 2 \times \rho_{\text{steel}} \\ E_{\text{aluminum}} &= \frac{E_{\text{steel}}}{3} \end{aligned} \right.$$

Structural Steel

Mild steel is generally used structural steel

It is a low carbon steel, having a Carbon Content (0.2 - 0.35%)

(i) Physical properties (common for all grades)

S.No	Property	Value
1	Unit mass of steel (P)	7850 kg/m ³
2	Modulus of Elasticity (E)	2×10^5 N/mm ²
3	Modulus of Rigidity (G)	0.769×10^5 N/mm ²
4	Coeff of Thermal expansion (α)	$12 \times 10^{-6}/^\circ\text{C}$
5	Poisson's ratio (μ)	
	(i) Elastic range	0.3
	(ii) Plastic range	0.5

(ii) Mechanical properties

S.No	Grade	Yield stress (f_y)			Ultimate Tensile Stress (f_u)	Percentage Elongation (%)
		$t < 20\text{mm}$	20-40	$> 40\text{mm}$		
1	E250 (Fe410)	250	240	230	410	23
2	E410 (Fe540)	410	390	380	540	20
3	E450 (Fe570)	450	430	420	570	20

Note:

→ Fe410 classified as Fe410S, Fe(410W)A, Fe(410W)B + Fe(410W)C.

(∵ Fe410S should not be used for welding of thickness over 20mm, i.e. $t > 20\text{mm}$)

→ % Elongation shall be taken over the gauge length = $5.65 \sqrt{A_s}$ area of specimen

If elongation is non-proportional 0.2% proof stress is taken as f_y .

Design

IS 875-1987 → Design loads for Building & Structures.

IS 800-1984 → Working Stress method of design (Stress Criteria)

IS 800-2007 → Limit State method of design (Load Criteria)

(i) Working Stress method (a) Elastic stress method

In this method, considered that the member can carry upto Elastic limit. Hence member will be designed, such that they can resist less loads as compared to the resistance of max capacity by proper factor of safety to whole.

Permissible Stress

It is the maximum load carried by the member without deformation.

$$\text{Permissible stress} = \frac{\text{Yield stress } (f_y)}{\text{Factor of safety}}$$

$$\begin{aligned} F.O.S &= 1.67 \text{ for members subjected to Axial tension/compression} \\ &= 1.50 \text{ for members subjected to Bending.} \end{aligned}$$

(∵ In axial loading all fibers reach max stresses, but in bending only extreme ^{fibers} stress will reach max stresses. Hence F.O.S will be less for bending)

Working Stress

Considering, in the field there are always worst combination of loads will present (i.e. DL+LL+WL+EL+SL+...). Hence member will be design such that they can resist more & more loads as actually we needed. ultimately size & c/s area of the member increase ⇒ working/failure stress decrease.

$$\text{Working Stress} = \frac{\text{Load Supplied to the members}}{\text{Cross-Sectional area}}$$

(a)

$$\text{Working Stress} = \frac{\text{Yield stress } (f_y)}{\text{Factor of safety}}$$

Note:

From the above discussion it states that Working Stress < Permissible Stress. Since we are considered more F.O.S, i.e. $(F.O.S)_{ws} > (F.O.S)_{ps}$

Merits of W.S.M

- The members cannot be failed in future, having large life span
- The design is very simple (∵ we cannot view in economical)

Demerits

- weight of the structure to increase, hence it is uneconomical.

(ii) Plastic (a) Limit State method

The design of the member ^{may be} touch plastic range, i.e. the F.O.S will be decided for each loads by considering load combinations & (Strength & Serviceability requirements). Hence it is called partial safety factor

$$\therefore \text{Design load } (P_d) = \frac{\text{Corresponding Characteristic load } (P)}{\text{Partial safety factor } (\gamma_m)}$$

$$\text{eg: For DL+LL} \rightarrow \gamma_m = 1.5 \text{ (Strength)} = 1.0 \text{ (Serviceability)}$$

$$\text{For DL+EL} \rightarrow \gamma_m = 1.2 \text{ (Strength)}$$

→ Self wt will be less, hence it is Economical.

→ Design is not simple. Overloading will not happened

$$\therefore (F.O.S)_{Lsm} \leq (F.O.S)_{Lsm}$$

Rolled steel sections

IS — (Indian Standard, 3rd term — Property, 4th term — Shape)

1) I-Sections (IS-B, Indian Standard — Beam)

- a) ISLB (Light) b) ISJB (Junior) c) ISMB (medium wt)
d) ISWB (wide flange) e) SC (Indian Standard Column Section)

2) Channel Sections (IS-C Indian Standard — Channel)

- a) ISPG (Gate channel) b) ISLC (Light) c) ISJC (Junior)
d) MC (Indian Standard medium wt channel with sloping flange)
e) MCP (Indian Standard medium wt channel with parallel flange)

3) Tee-Sections (IS-T Indian Standard — Tee bars)

- a) ISNT (Rolled normal) b) ISDT (Rolled deep legged)
c) ISLT (Slit light weight) d) ISMT (Slit medium weight)
e) ISHT (Indian Standard slit Tee bars from H-sections)

Riveted Connections

A rivet is made of a round ductile steel bar piece (Mild or High tensile) called 'shank', with a head at one end.

a) Size of rivet = Dia of Shank (ϕ)

b) Grip of rivet = Shank length inside the connection ($\geq 4d$)

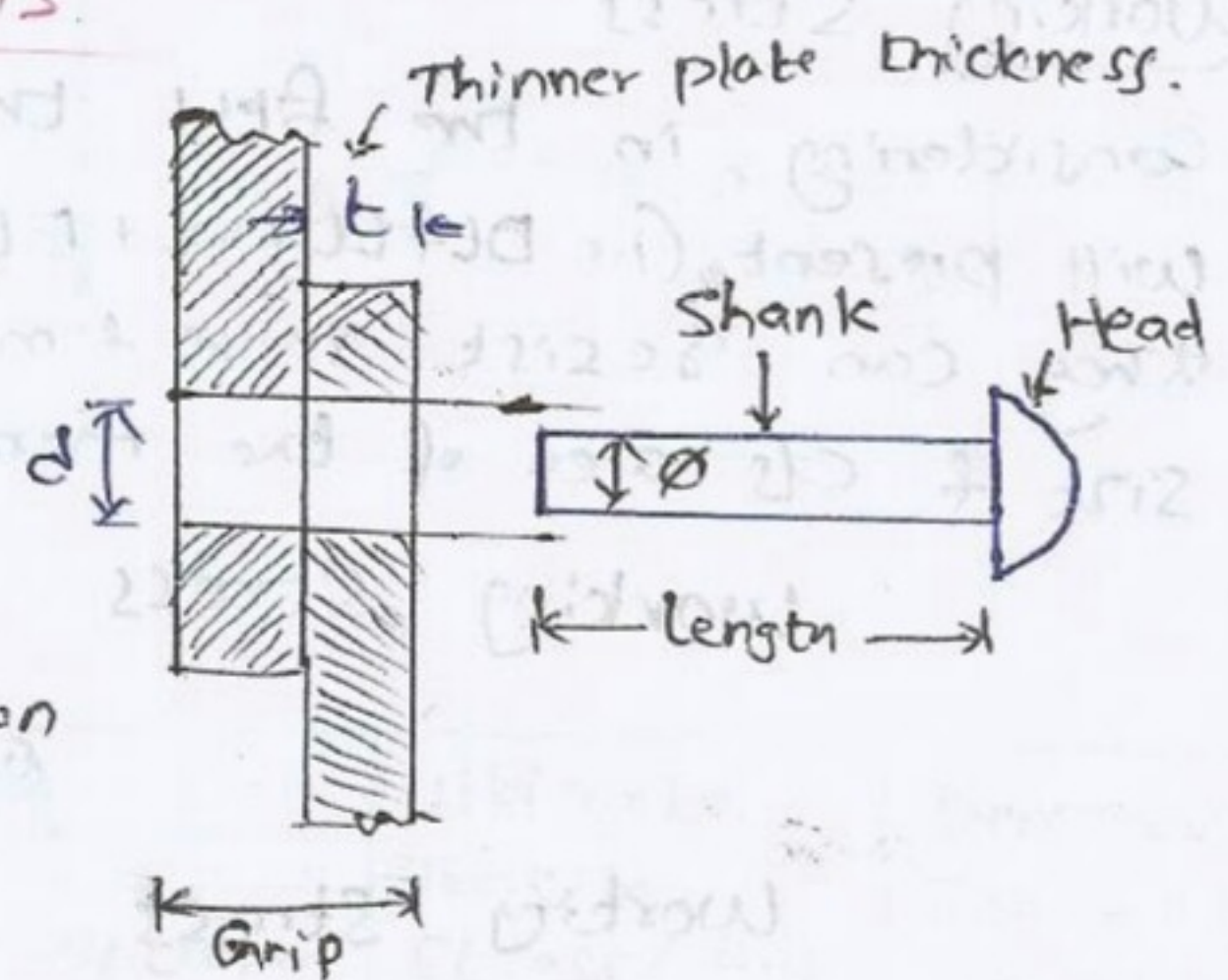
c) Nominal dia of rivet (ϕ) = Dia of Shank

d) Gross dia or Eff Size of rivet (d) = Diameter of rivet hole.

$$d = \phi + 1.5 \text{ mm if } \phi \leq 25 \text{ mm}$$

$$= \phi + 2.0 \text{ mm if } \phi > 25 \text{ mm}$$

$$\phi = 6.05 \sqrt{E} \quad [\text{Unwin's formula}]$$



Rivets

1. Hot driven rivets

They are heated (1000-1950°F) red hot before driving, These are two types

- a) Shop driven rivets b) Field driven rivets

2. Cold driven rivets (Not adopted for dia (ϕ) > 10mm)

They are driven at atmospheric condition (it is difficult to create 2nd head)

Strength of Cold > Hot & Shop > Field driven rivet.

1) Power driven rivets (Driven by power operated equipment)

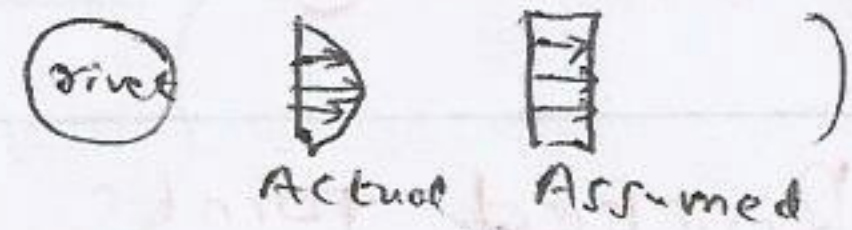
Hand driven rivets (Driven by hand operated hammer)

Permissible Stresses in Rivets

Type of Rivet	Axial Tension (MPa) (σ_{at})	Shear (σ_s) (MPa)	Bearing (σ_b) (MPa)
(i) Power Driven			
a) Shop rivets	100	100	300
b) Field rivets	90	90	270
(ii) Hand Driven	80	80	250

* The above values are increased by 25%, if wind or Earthquake is taken into account.

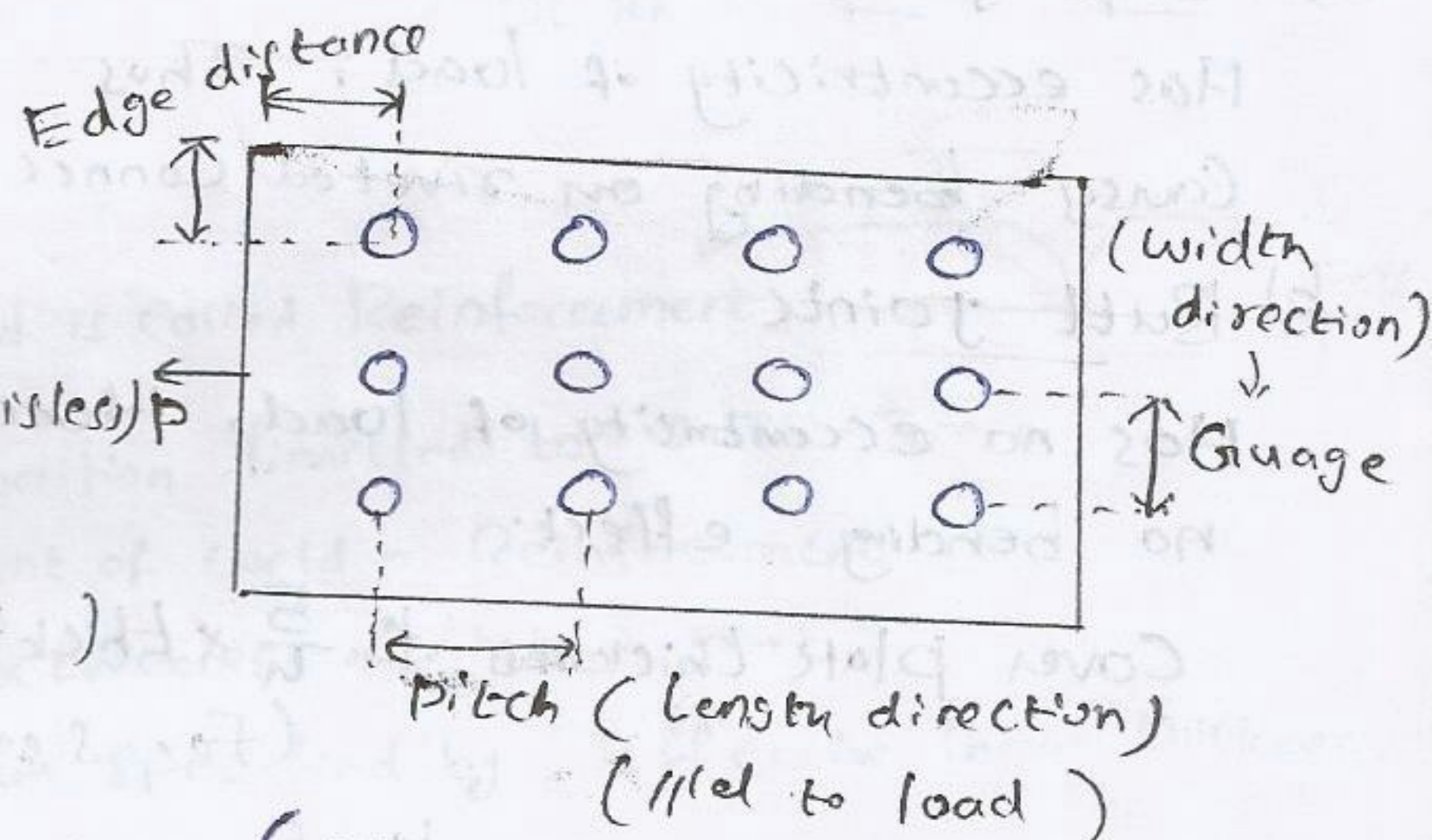
Assumptions in Riveted Connection

1. Friction between plates is neglected
2. Bending stress in the rivet is neglected (if grip $< 8d$)
3. Shear stress is uniform over the c/s of rivet ($\therefore$ )
4. Rivets fill the hole completely ($\therefore d$ is used in strength calculation)

Pitch & Gauge

1) Pitch (P)

- a) $P_{min} \geq 2.5 \phi$
- b) $P_{max} \geq 16t$ or 200mm (whichever is less) (in Tension)
- $\geq 12t$ or 200mm (in Compression)



2) Gauge (g)

(Chain Riveting)

c/c Distance b/w two bolts in L^r direction of applied load.

* 'Diamond or Zig-zag riveting' is more efficient than 'Chain riveting'.

1) Strength of rivet

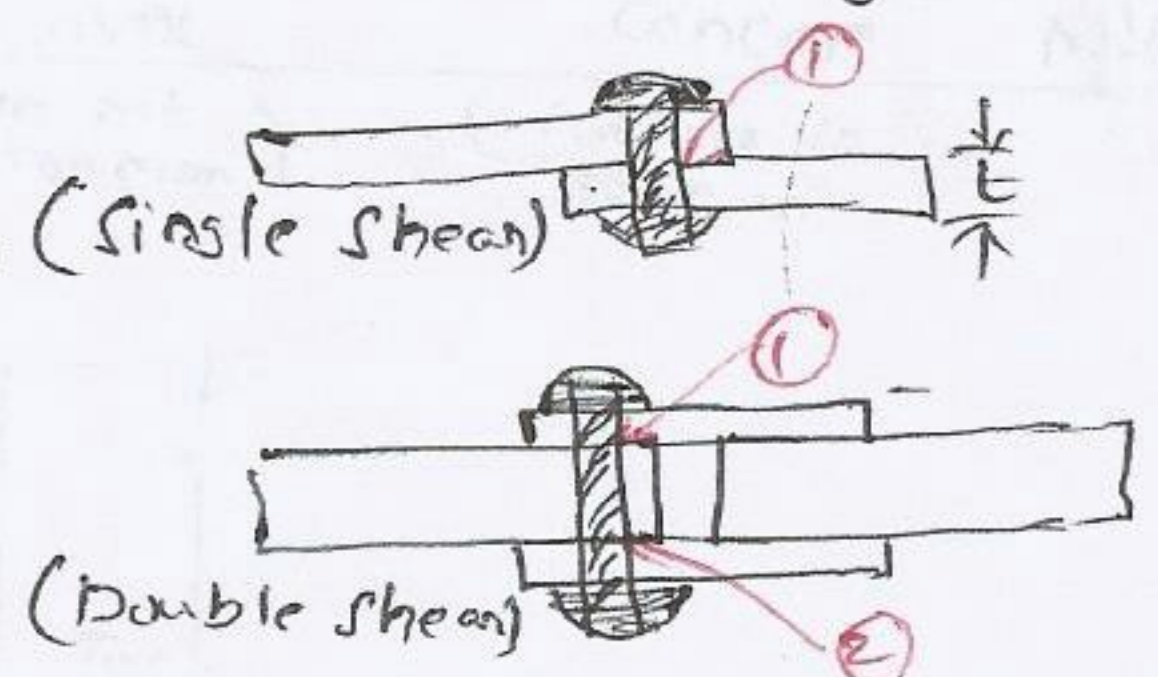
Rivets resisting the load by shearing & bearing only. (Single Shear)

a) Strength of Rivet in Shear (P_s)

$$P_s = \text{Area of hole} \times \text{permissible stress in shear}$$

$$= (\pi d^2) \times \sigma_s \rightarrow \text{Single Shear}$$

$$= 2 \times (\pi d^2) \times \sigma_s \rightarrow \text{Double Shear}$$



b) Strength of Rivet in Bearing (P_b)

$$P_b = \text{Projected Contact area of rivet} \times \text{permissible stress in bearing}$$

$$= (dt) \sigma_b$$

where t = Thickness of Thinner plate (in Lap joint)

= Thickness of Thinner main plate } whichever (in Bolt joint)
Sum of cover plate thickness } is less

c) Rivet value (R_v) = Minimum of P_s & P_b

d) No. of Rivets Required (n) = $\frac{\text{Load}}{\text{Rivet value}}$

2) Strength of plate in tearing (P_t)

$$P_t = A_{net} \times \sigma_{at}$$

$$= (B - nd) t \times \sigma_{at}$$

$$\left[\begin{array}{l} B = \text{width of plate, } t = \text{Thickness of plate} \\ \sigma_{at} = \text{Allowable/Permissible tensile stress in plate} \\ = \frac{f_y}{F_{os}} = \frac{f_y}{1.67} = 0.6 f_y \end{array} \right]$$

3) Strength of joint (P_j)

$$P_j = \text{Minimum of } P_s \text{ & } P_b \text{ & } P_t$$

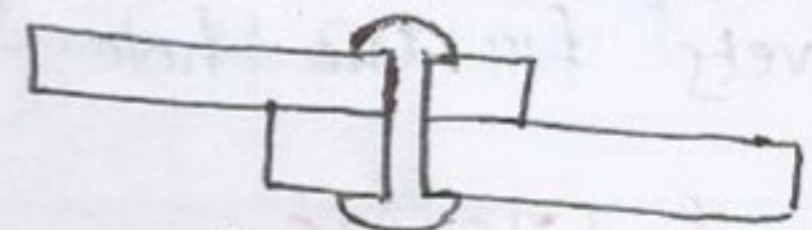
4) Strength of solid plate = $A_{gross} \times \sigma_{at} = (Bt) \sigma_{at}$

5) Efficiency of the joint (η) = $\frac{\text{Strength of joint}}{\text{Strength of solid plate}} \times 100$

Riveted joints

a) Lap joint

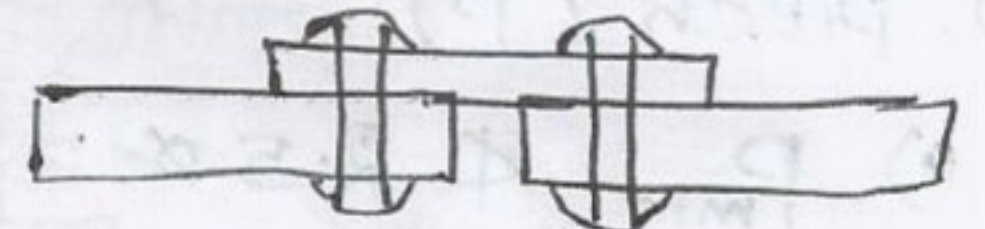
Has eccentricity of load. Thus
Causing bending on riveted connections.



(Lap joint)

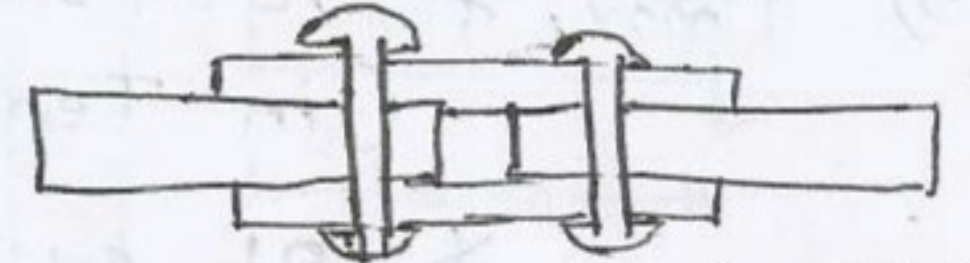
b) Butt joints

Has no eccentricity of load. Hence
no bending effect.



Single cover butt joint

Cover plate thickness $\leq \frac{5}{4} \times \text{thick of main plate}$
(For single cover)



Double cover butt joint

$\leq \frac{5}{8} \times \text{Thickness of main plate}$
(For Double cover)

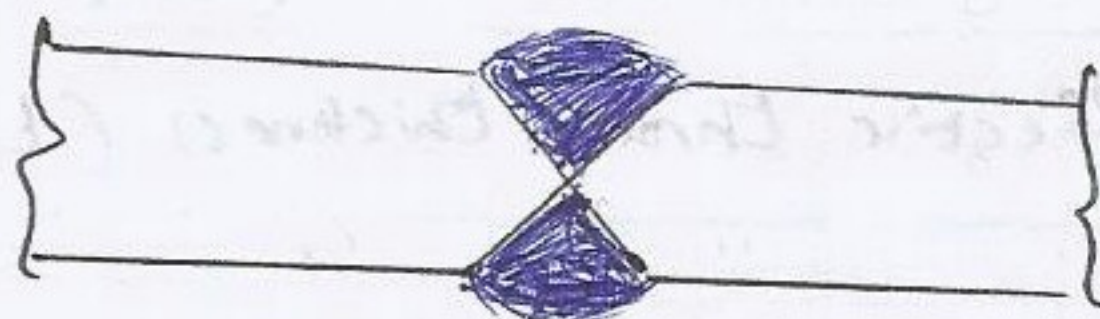


Welded Connections

1) Bolt welds (Angle b/n plates = 0°)

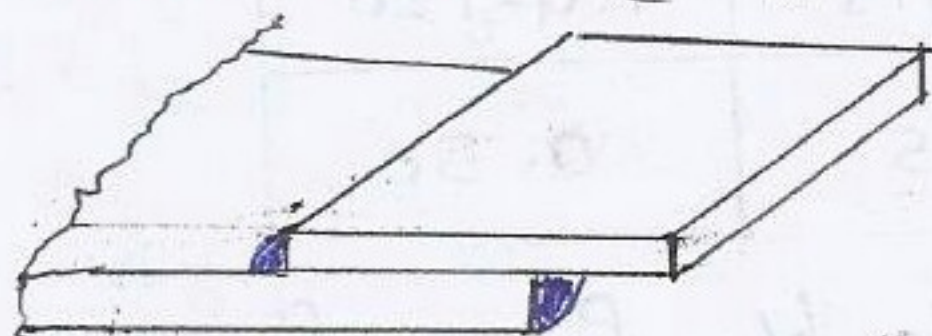


Single butt
(if $t = 12-40\text{mm}$)

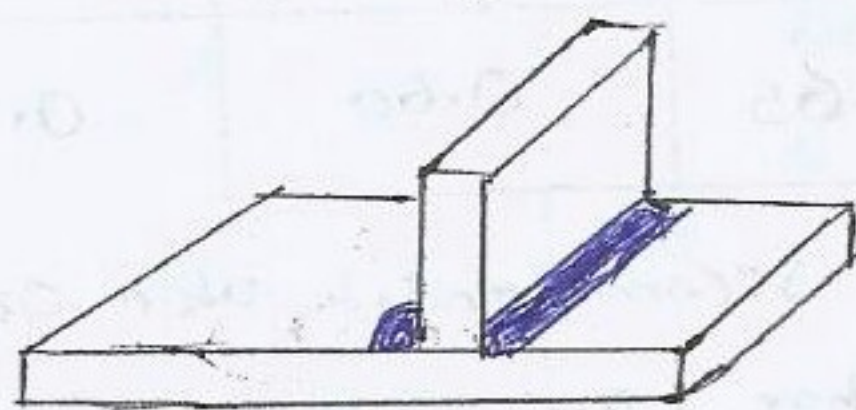


Double butt
(if $t > 40\text{mm}$)

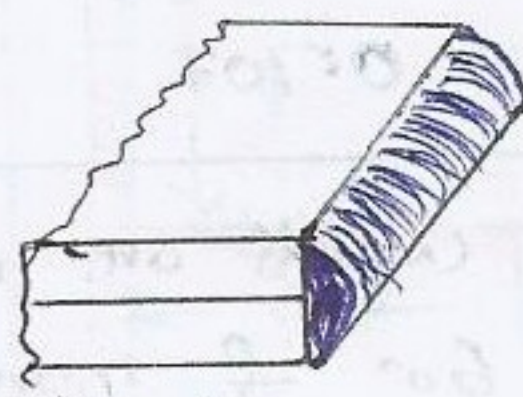
2) Fillet weld (Angle b/n plates $\neq 0^\circ$)



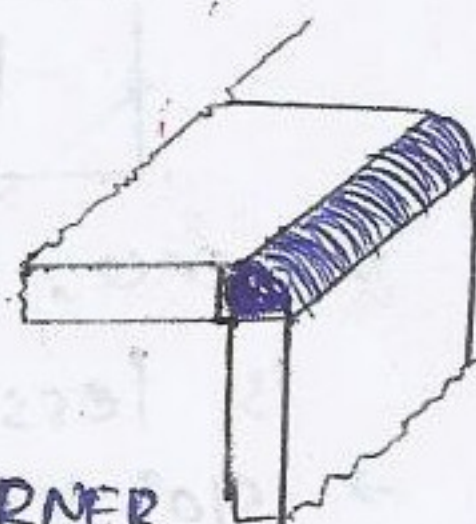
LAP



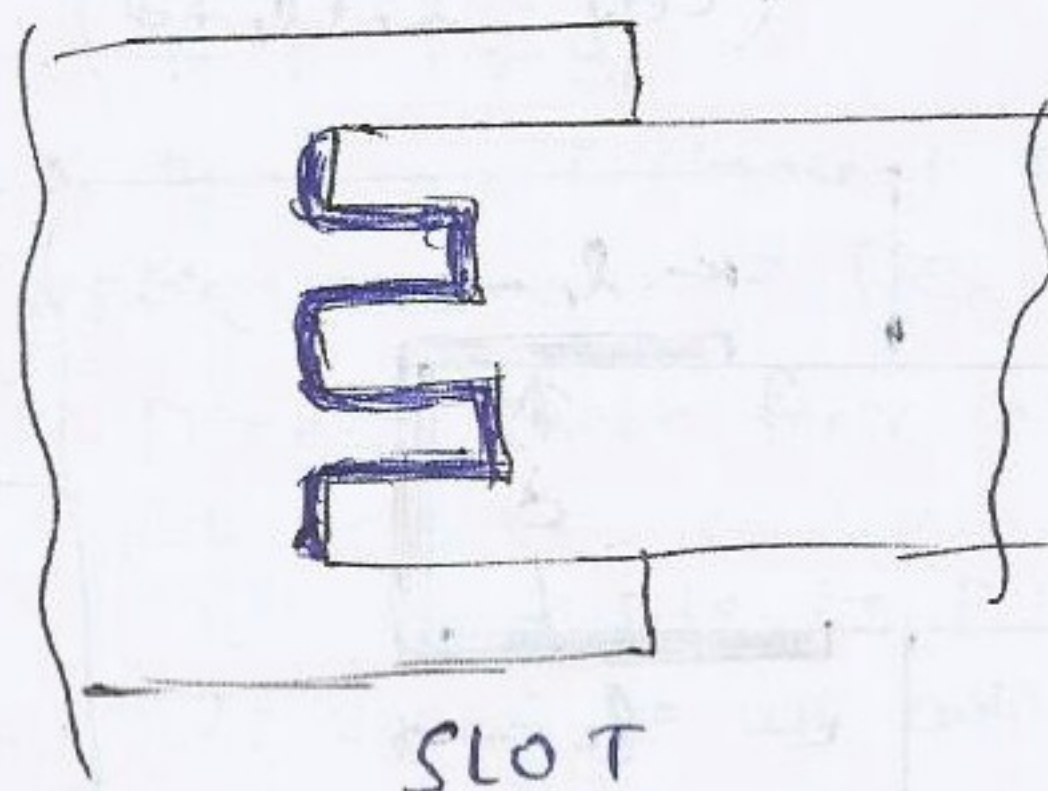
TEE



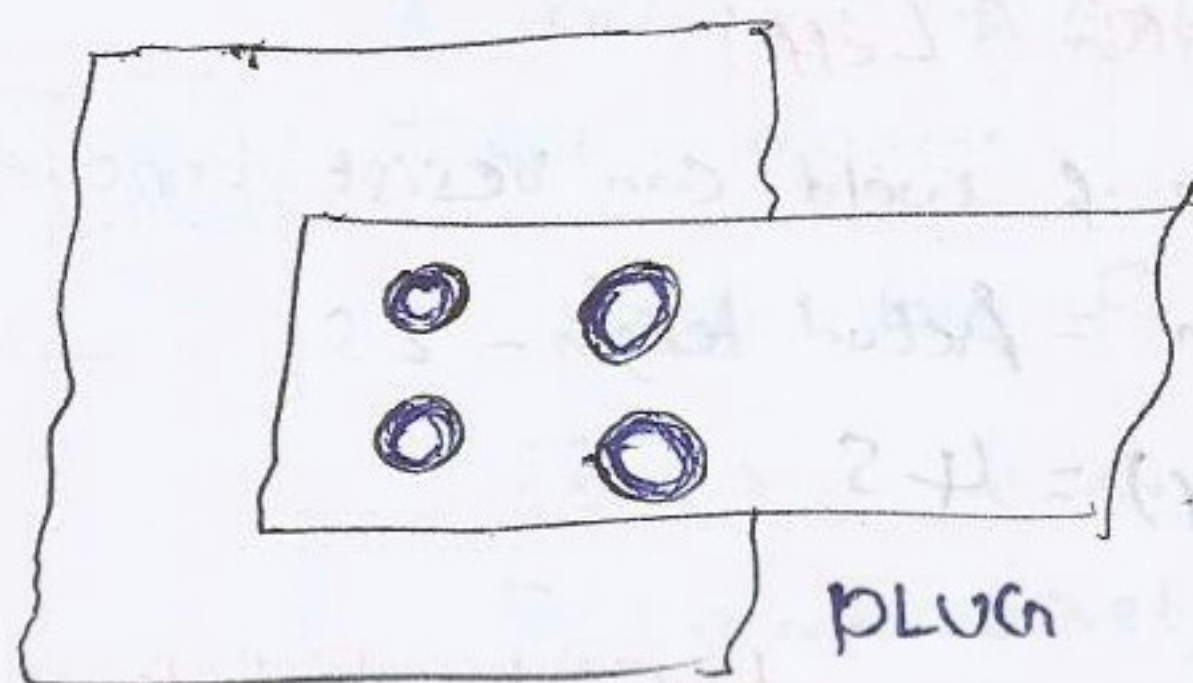
EDGE



CORNER



SLOT



PLUG

Specifications of butt weld

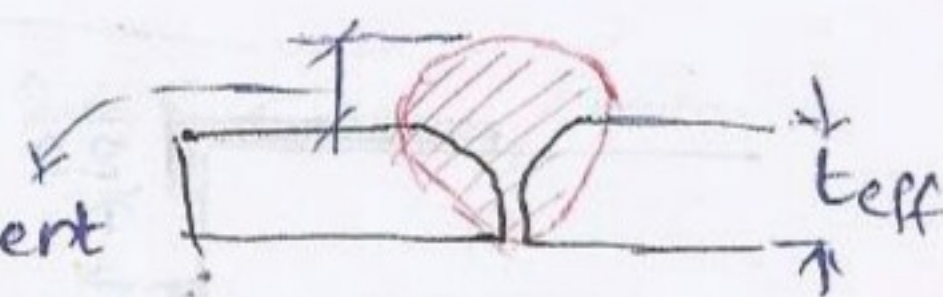
1) Additional deposition of ~~met~~ weld is called Reinforcement

→ Reinforcement under 'Fatigue' condition limited to $0.75-3\text{mm}$

2) Effective throat thickness = Height of weld - Reinforcement

(t_{eff}) & $\frac{5}{8} \times$ thickness of thinner part.

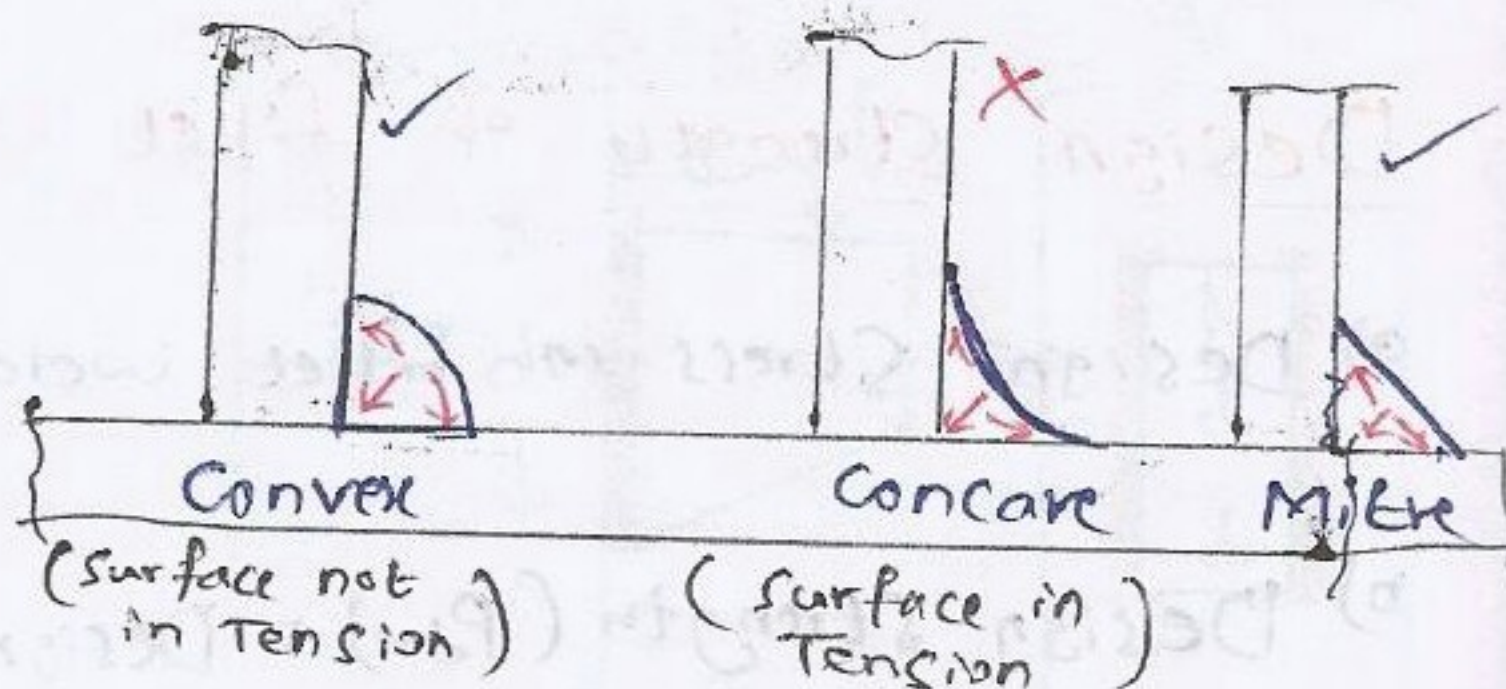
3) The size of butt weld should be specified by "Effective throat thickness."



Specifications of Fillet weld

→ Concave shape w/o not Recommended.
($\because$ weld create tension on surface of plates)

→ Concave seems large than Convex.

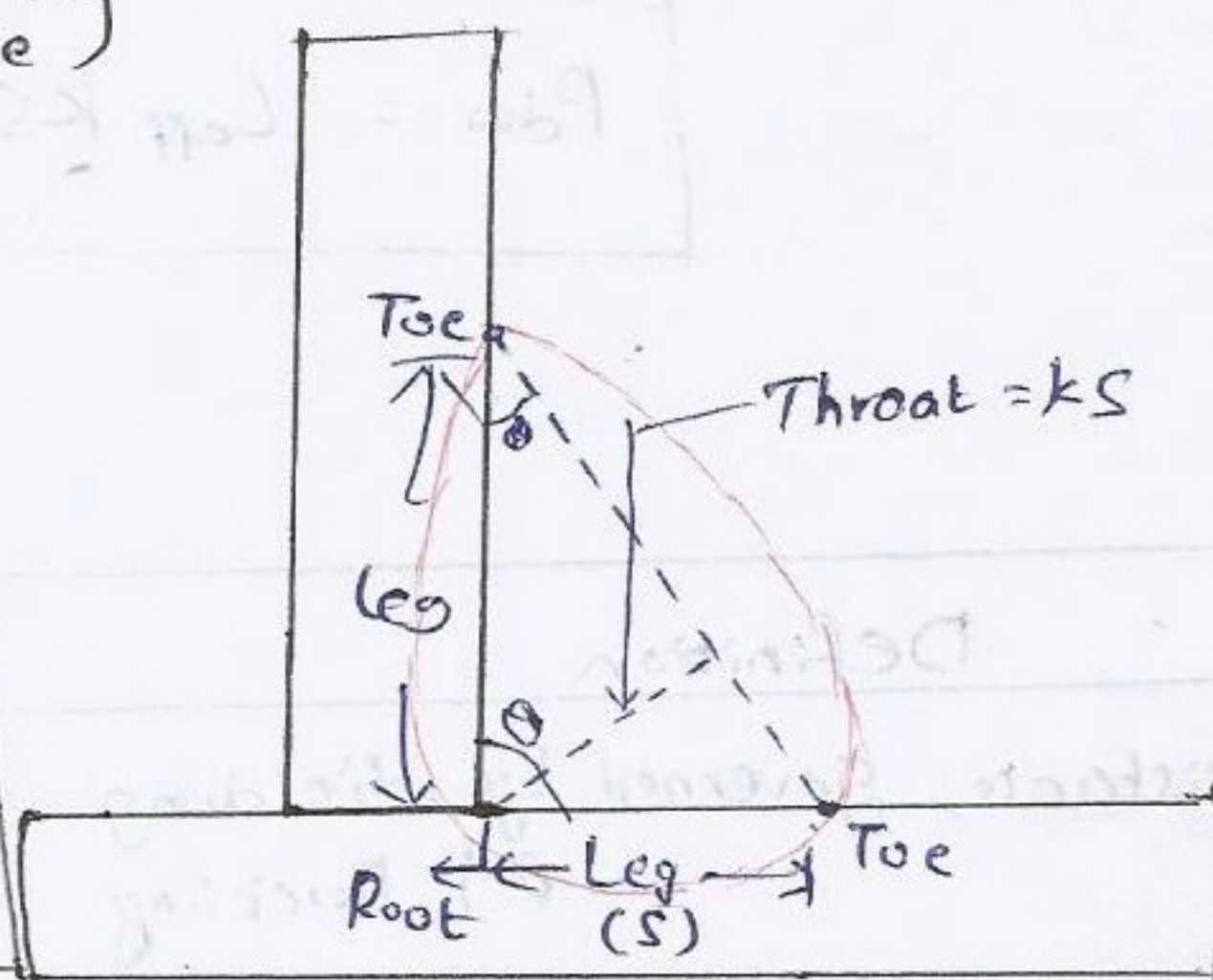


1) Size(s)

S = Minimum leg length ($\because$ Leg = distance b/n Root - Toe)

Minimum Size

Thickness of thinner part	Min Size (mm)
$t \leq 10$	3
$10 < t \leq 20$	5
$20 < t \leq 32$	6
$32 < t \leq 50$	8 (1st run) 10 (2nd run)



Maximum Size = $t - 1.5$ (Flat plates)

= $\frac{3}{4}t$ (Rolled sections)

2) Effective Throat Thickness (t_{eff})

It is the shortest/1st distance b/n the Root of fillet weld and the face of the diagrammatic weld (Line joining the toes)

Minimum effective throat thickness (t_{eff}) = 3mm

Maximum " " " " " " = $0.707t = \frac{1}{\sqrt{2}}t$

* Effective throat thickness = KS

Angle	60-90°	91-100	101-106°	107-113°	114-120°
K	0.707	0.65	0.60	0.55	0.50

→ Hence, Fillet welds are not recommended when angle b/n fusion face is less than 60° & more than 120°

→ 90° is the most recommended fillet weld.

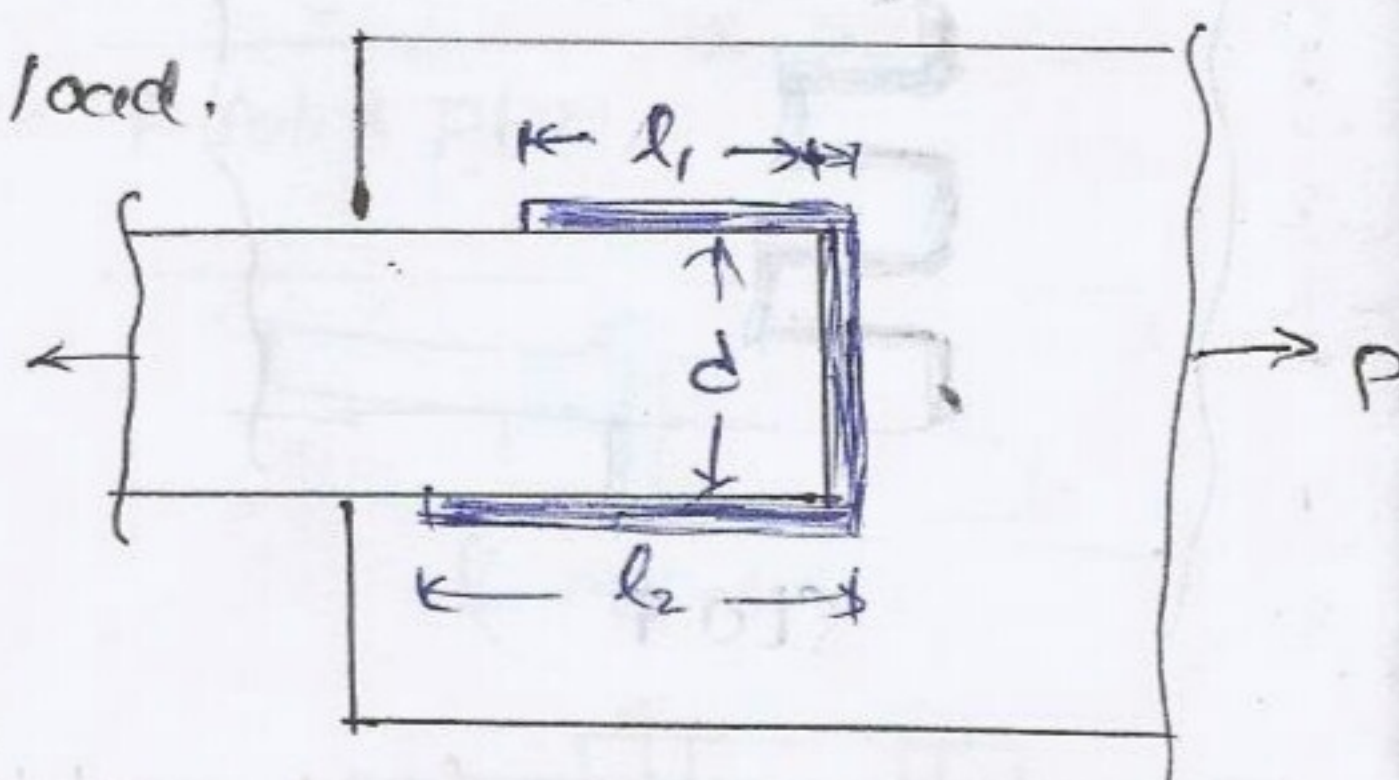
$$(L_{eff} = l_1 + l_2 + d)$$

Effective length (L_{eff})

Minimum length of weld can resist tensile load.

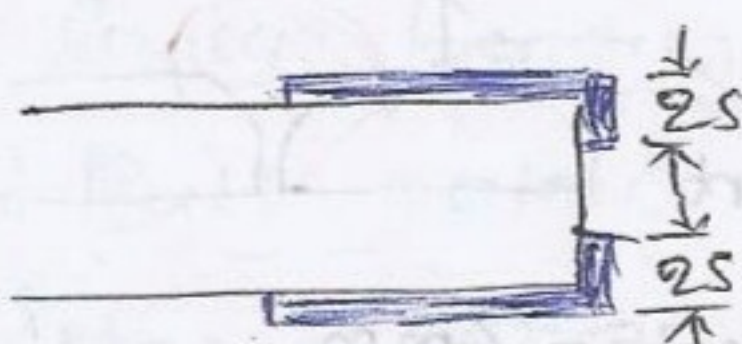
Effective length = Actual length - $2S$

Minimum (L_{eff}) = $4S$



a) End Return

b) Side Fillet



(To resist heavy stresses)
Concentrated @ ends

End return & $2S$

(i) l_1 & d & l_2 & d

→ if $l < d$, Plug weld will be provided

(ii) $d \neq 16t$

→ As 'd' increases, non-uniformity in stress also increases.

Effective Area (A_{eff}) = $L_{eff} \times t_{eff} = (0.7S L_{eff})$

Design strength of Fillet weld (P_{dw})

a) Design stress in Fillet weld (f_{wd}) = $\frac{f_{wn}}{\gamma_{mw}}$

b) Design strength (P_{dw}) = Design stress $\times$ Eff. Area

$$P_{dw} = L_{eff} KS \frac{f_u}{\sqrt{3} \gamma_{mw}}$$

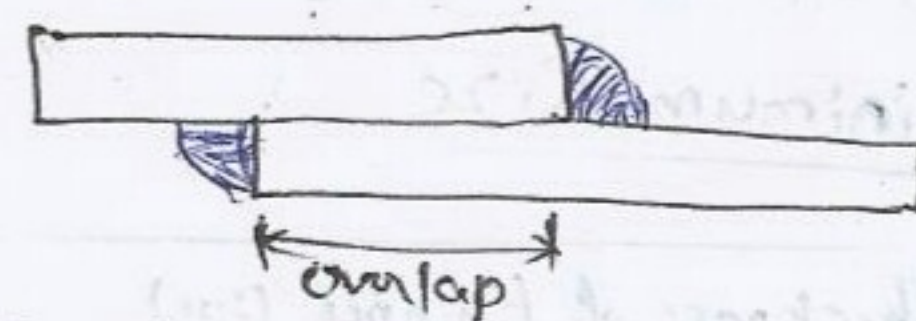
f_{wn} = Nominal strength of weld

$$f_{wn} = \frac{f_u}{\sqrt{3}}$$

γ_{mw} = partial safety factor

= 1.25 (Shop weld)

= 1.50 (Site weld)



Overlap < $4t$ } which ever is less
40mm

Definition

Partial safety factor

(i) Resistance, governed by Yielding (or) Buckling (γ_{mo})

1.10

(ii) Resistance, governed by Ultimate stress (γ_{mi})

1.25

(iii) Resistance of Connection by Bolts & Rivets (γ_{mx})

1.25

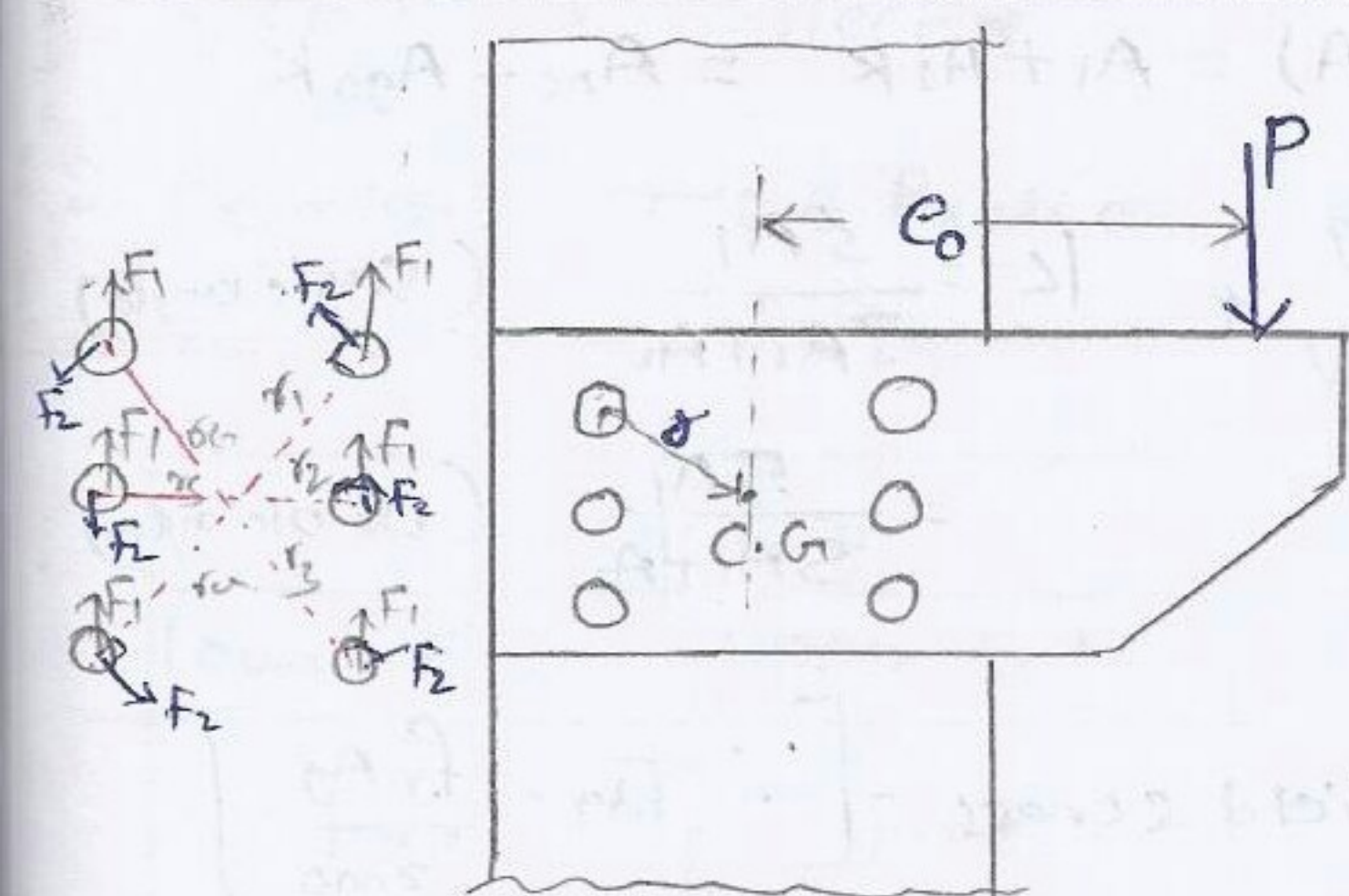
(iv) Resistance of Connection by weld (γ_{mw})

1.25 (Shop weld) & 1.50 (Site weld)

Eccentric Connections

* Bracket Connection Type-1 (Load acting in the plane of Connection)

Bolted Bracket Connections



n = No. of rivets

$x_1, x_2, x_3 \dots x_n$ = Distance of rivet from C.G.

Twisting moment = $P e_0$

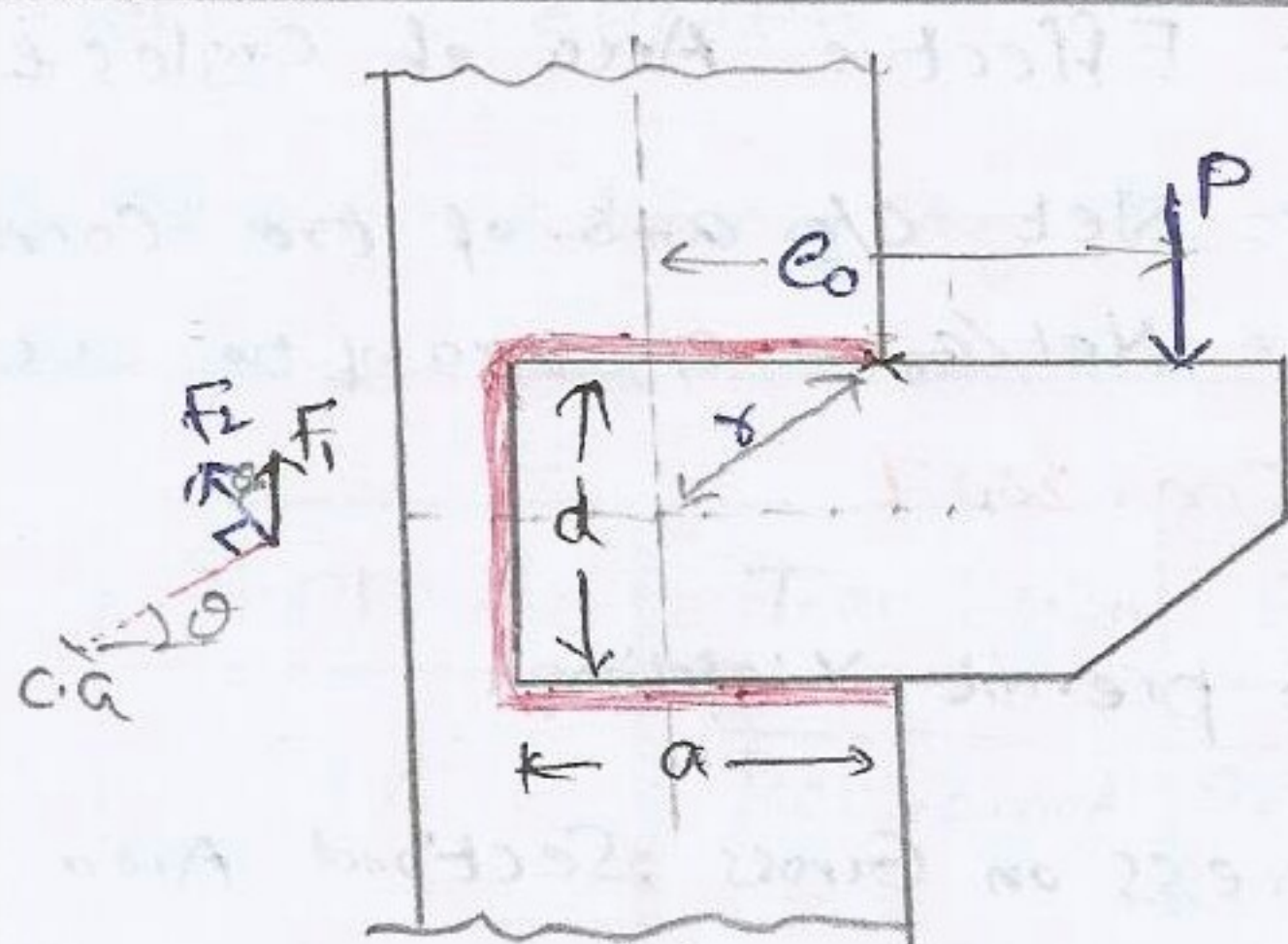
→ F_1 = Direct shear force in each bolt
(It acts vertical upward direction ↑
(i.e. opposite to P)
→ Is equal to all bolts.

→ F_2 = Shear force in rivet due to moment ($P e_0$)

$$F_1 = \frac{P}{n}$$

$$F_2 = \frac{(P e_0) x}{\sum x^2}$$

Welded Bracket Connections



x = Distance of Extreme weld from C.G.

Twisting moment = $P e_0$

θ = Angle b/w F_1 & F_2

$F_R \neq F_{wd}$

($\because F_2 \propto x$, i.e. F_2 will max at Extreme rivet/weld)

$$F_R = \sqrt{F_1^2 + F_2^2 + 2 F_1 F_2 \cos \theta} \text{ /mm}$$

$$F_1 = \frac{\text{Load}}{\text{Length of weld}} = \frac{P}{(2a + d)}$$

$$F_2 = \frac{(P e_0) x}{I_p} \quad (\because I_p = \text{Polar m. I/mm})$$

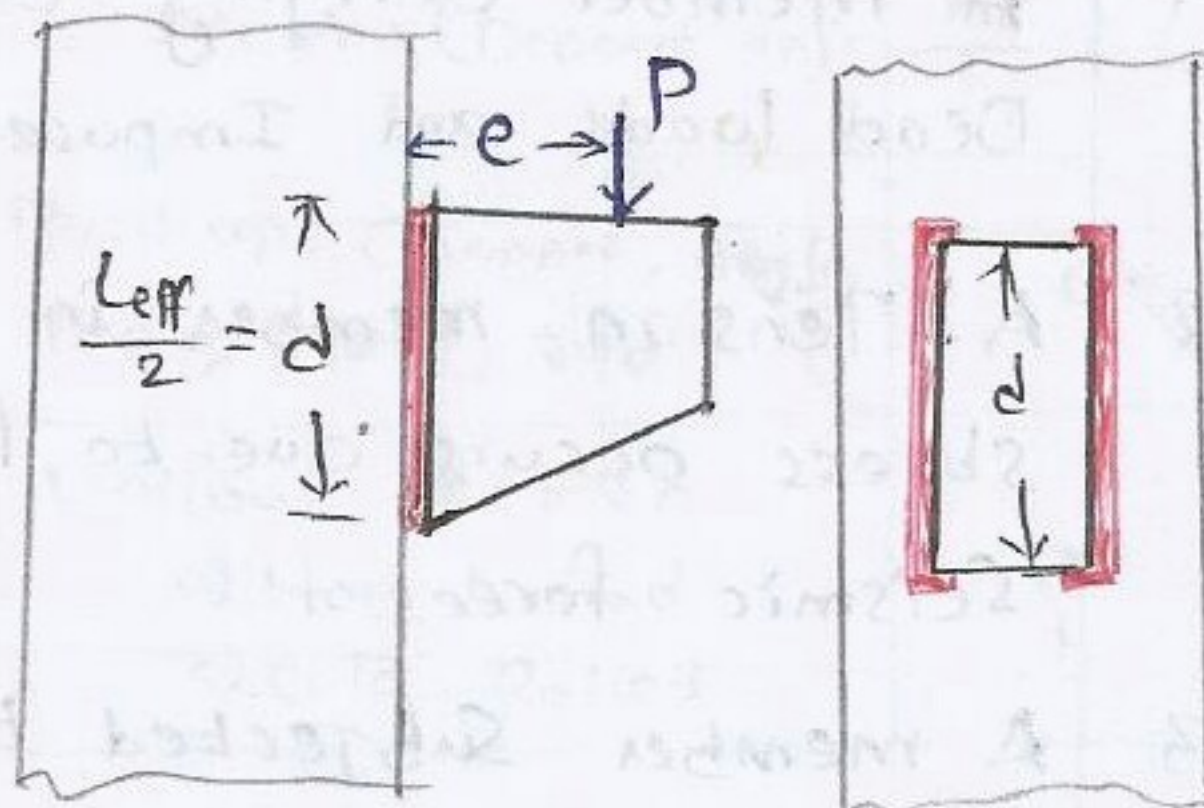
* Bracket Connection Type-2 (Load not lying in the plane of weld, i.e. L_r)

$$q = \frac{\text{Load}}{\text{Eff. Area of weld}} = \frac{(P/2)}{L_{eff} \times L_{eff}}$$

($\because$ each side weld will take half load)

$$f_b = \frac{M}{I} \times y = \frac{(P e_0) \times (\frac{L_{eff}}{2})}{\frac{2}{12} \times (\frac{E L^3}{12})} = \frac{3 P e_0}{L_{eff} \times (L_{eff})^2}$$

$$q_R = \sqrt{q^2 + f_b^2}$$



Tension members (or) Tie

IS: 800-1984

Permissible shear stress in axial tension (σ_{at}) = $0.6 f_y$ ($\because \frac{f_y}{1.67}$)

Net Effective Area of angles & Tees (A) = $A_1 + A_2 k = A_{nc} + A_{go} k$

A_1 = Net c/s area of the connecting leg
 A_2 = Net/Gross c/s area of the outstanding leg $k = \frac{3A_1}{3A_1 + A_2}$ (Single angle)

IS 800-2007

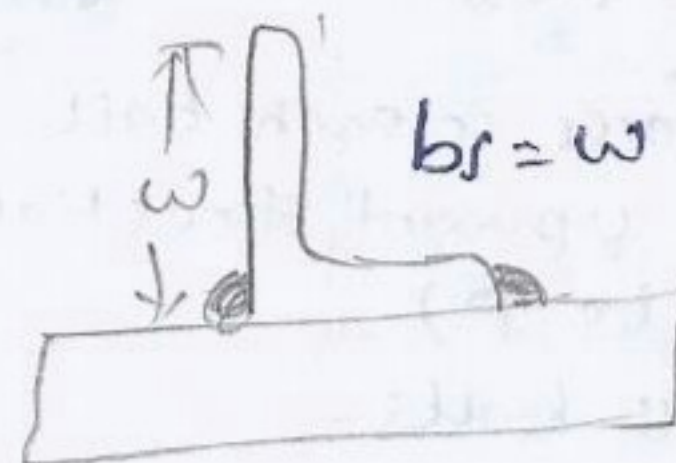
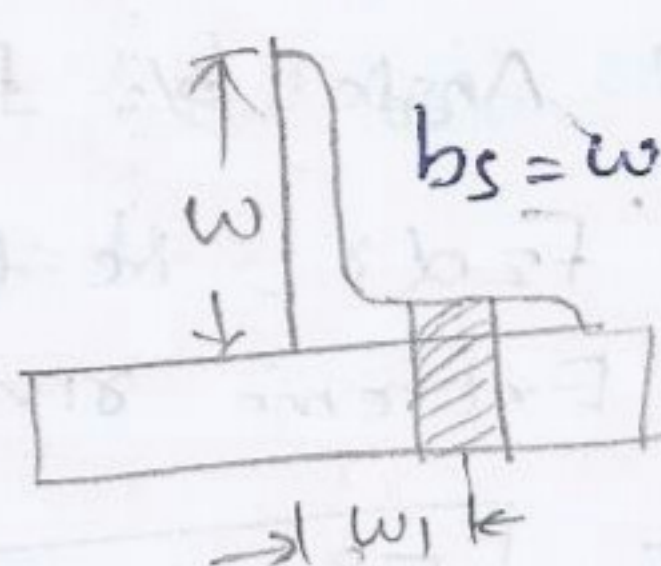
$= \frac{5A_1}{5A_1 + A_2}$ (Double angle)

a) To prevent Yielding

Stress on Gross Sectional Area $\nless \nless$ Yield stress [$\because T_{dg} = \frac{f_y A_g}{\gamma_{mo}}$]

b) To prevent Rupture

Stress on Net sectional Area $\nless$ ultimate stress [$\because T_{dn} = \frac{0.9 f_u A_n}{\gamma_{m1}}$]



(b_s = shear leg width)

c) Failure by Block Shear

May fail in the direction of rivets $\perp$ to the load direction

Minimum Gusset plate of thickness = 12mm

	Member	λ_{max}
1	A member Carrying Compressive loads resulting from Dead loads and Imposed loads	180
2	A Tension member in which a reversal of direct stress occurs due to loads other than Wind (or) Seismic forces	180
3	A member Subjected to Compression forces resulting only from combination with Wind/Earthquake actions, Provided the deformation of such member does not adversely affect the stress in any part of the structure	250
4	Compression flange of a beam against lateral torsional buckling	300
5	A member normally acting as a Tie in a roof truss (or) A bracing system not considered effective when subjected to possible reversal of stress ~~ratio~~ into compression resulting from the action of Wind (or) Earthquake forces	350
6	Members always under Tension (other than pre-tensioned members)	400

To avoid Sag, Tension members (such as bracings, pre-tensioned) need not satisfy λ_{max} limits

Compression members

Tension Member

1. In tension Net area of the section is Effective

2. Permissible stress is fixed ($\sigma_{at} \equiv 0.6 f_y$)

Compression member

1. In compression Gross Area of the section is effective

2. Permissible stress is Vary $\sigma_{ac} \leq 0.6 f_y$

End Condition		Effective length
Lower End	Upper End	
Fix	Free	$2L$
Giant	Giant	$2L$
Hinge	Hinge	L
Fix	Giant	$1.2L$
Fix	Hinge	$0.8L$
Fix	Fix	$0.65L$

Support	Translation	Rotation
Fix	Restrained	Restrained
Free	Free	Free
Hinge (or) Pin	Restrained	Free
Giant	Free	Restrained

Type of Member

Effective length
Can Assume $\frac{L}{n}$

1. Compression members in Trusses

$0.7 - 1.0L$

2. Double angle discontinuous struts Connected back to back on opposite side of the gusset plate

$0.7 - 0.85L$

Buckling class	a	b	c	d
Imperfection Factor (α)	0.21	0.34	0.49	0.76

$$\text{Yield stress Ratio } (\epsilon) = \sqrt{\frac{250}{f_y}}$$

Cross-section	Buckling class
I-section (Depend on)	$\frac{h}{b_f}$ & t_f
Built up, Channel, Angle, Tee, plate, and	C
Hollow section a) Hot Rolled b) Cold Rolled	a b

Lacings

- 1) Lacing System shall be uniform throughout the length of the column
- 2) Slenderness Ratio (λ) $\nless 145$ or $= 1.05 \lambda$ (For shear deformation effects)
- 3) Effective Length (L_{eff}) = $1.0L$ (Single Lacing Riveted)
 $= 0.7L$ (Double Lacing Riveted/Welded)
- 4) Minimum Radius of gyration (r_{min}) = $\frac{t}{\sqrt{12}}$
- 5) Angle of Inclination = $40-70^\circ$
- 6) Thickness of Lacing Flat = $\frac{L_{eff}}{40}$ (Single Lacing)
 $= \frac{L_{eff}}{60}$ (Double Lacing)
- 7) Minimum width of Lacing $> 3\phi$ (ϕ = Nominal dia of bolts, when bolted connection)
- 8) Lacing should be designed atleast to resist

$$\text{Transverse Shear } (V_t) = 2.5 \% \text{ axial force in columns (Single lacing)}$$

$$= \frac{1}{2} V_t \text{ (Double Lacing)}$$

$$= \text{Bending Shear} + V_t \text{ (Subjected to bending also)}$$

$$a) \text{ Compressive force in Lacing} = V_t \operatorname{cosec} \theta \text{ (Single Lacing)}$$

$$= \frac{1}{2} V_t \operatorname{cosec} \theta \text{ (Double Lacing)}$$

$$10) \text{ Maximum spacing of Lacing} = 0.7\lambda \left. \begin{array}{l} \text{which} \\ \text{even is} \\ \text{less} \end{array} \right\} \left(\because \lambda = \text{overall slenderness Ratio} \right)$$

Battens

Battened column has same strength of Laced column, but are uneconomical & are used therefore rarely.

- 1) Battens are 'not Recommended' when load is Eccentric
- 2) Battens provided at the ends of column are called End battens
 $\rightarrow$ All other battens are called Intermediate battens

Battens are designed similar to Lacing - **Except**

$$1) \text{ Effective slenderness Ratio } (\lambda_e) = 1.10 \lambda$$

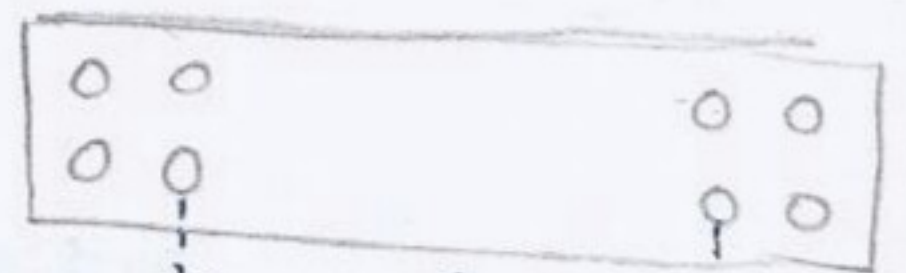
$$2) \text{ Thickness of batten} \nless \frac{S}{50} \quad \left(S = \text{innermost transverse distance b/n Centroids of bolts} \right)$$

$$3) \text{ Design Shear Force } (V_b) = \frac{V_t C}{N S} \quad (C = C/c \text{ b/n battens})$$

$$\text{Design Bending moment } (M_b) = \frac{V_t C}{2N} \quad (N = \text{No. of battens})$$

$$4) \text{ Shear stress } \left(\frac{V_d}{A} \right) < \frac{f_y}{\sqrt{3} \gamma_{mo}} \quad \left[\begin{array}{l} A = td \\ Z = \frac{I}{y} = \frac{td^2}{6} \end{array} \right]$$

$$\text{Bending stress } f_b = \frac{M_d}{Z} < \frac{f_y}{\gamma_{mo}}$$



Beams and Beam Columns

A structural member subjected to transverse loads (Loady $\perp$ to its longitudinal axis) is called a Beam.

Various other name depending upon its location/position

1. Joists Beams provided in building to support the roof.
2. Girder A large beam supporting a number of joists.
3. Spandrels Exterior beams at the floor level of buildings which carry part of the floor load & that of the exterior wall.
4. purlins Beam which carry roof loads in trusses
5. Rafter A roof beam usually supported by purlins.
6. Lintels Beams supporting the load of masonry over openings.
7. Girt A horizontal beam spanning the wall columns of industrial buildings used to support wall coverings.

IS 800-1984

Permissible Comp Stress in bending Compression (σ_{bc}) = $0.66 f_y$ ($\because \frac{f_y}{1.5}$)

Maximum deflection in beam $\neq \frac{\text{span}}{325}$

IS 800-2007

B.M is resisted by Flanges & S.F is resisted by web only.

Web buckling

The web in a rolled steel section behaves like a column when placed under concentrated loads. ($\because d = \text{Depth of the web}$)

Effective depth (d_{eff}) of the web depends on Restraintment of the Top Flange

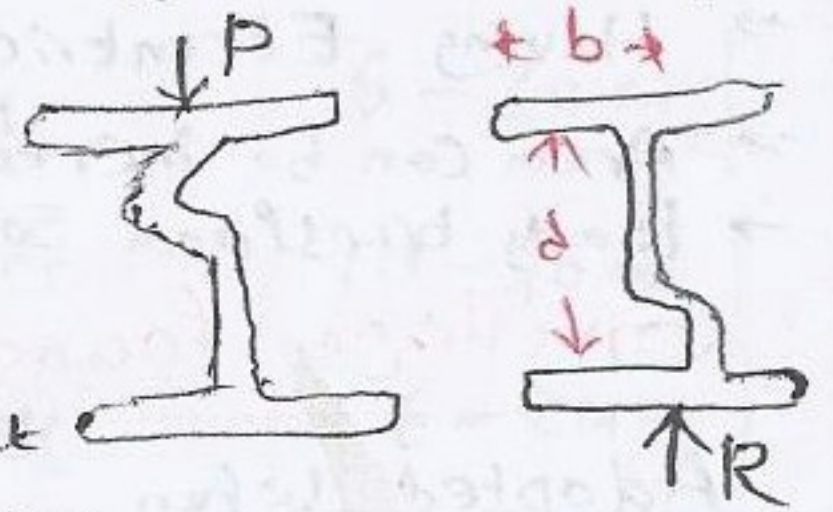
1) Fix ($\frac{d}{2}$) 2) Free ($2d$) 3) Girt (d) 4) Hinge ($\frac{2d}{3}$)

$\rightarrow$ web will buckle when

The max diagonal compression occurs at the N/A & will be inclined 45°
 ($\because$ The angle of dispersion of the load is 30° with horizontal (IS 800-1984))

Web crippling

occurs at just below the top flange & just above the bottom flange due to large amount of stresses from loads & reactions respectively.



$$\left[\begin{array}{l} R = 1.0 \text{ for plastic \& compact} \\ = Z_e / z_p \text{ for slender sections} \end{array} \right]$$

Cross-Section	C/S — before failure	Outstand Flange ($\frac{b}{t_f}$)	Web ($\frac{d}{t_w}$)
Class 1 (Plastic)	Can develop Plastic Hinge & Plastic mechanism	9.4ϵ	84ϵ
Class 2 (Compact)	Can develop plastic moment of resistance, but not ①	10.5ϵ	105ϵ
Class 3 (Semi-compact)	Extreme fiber in compression can reach Yield stress, but not ②	15.7ϵ	126ϵ
Class 4 (Slender)	Can buckle even before Yield stress	$> 15.7 \epsilon$	$> 126 \epsilon$

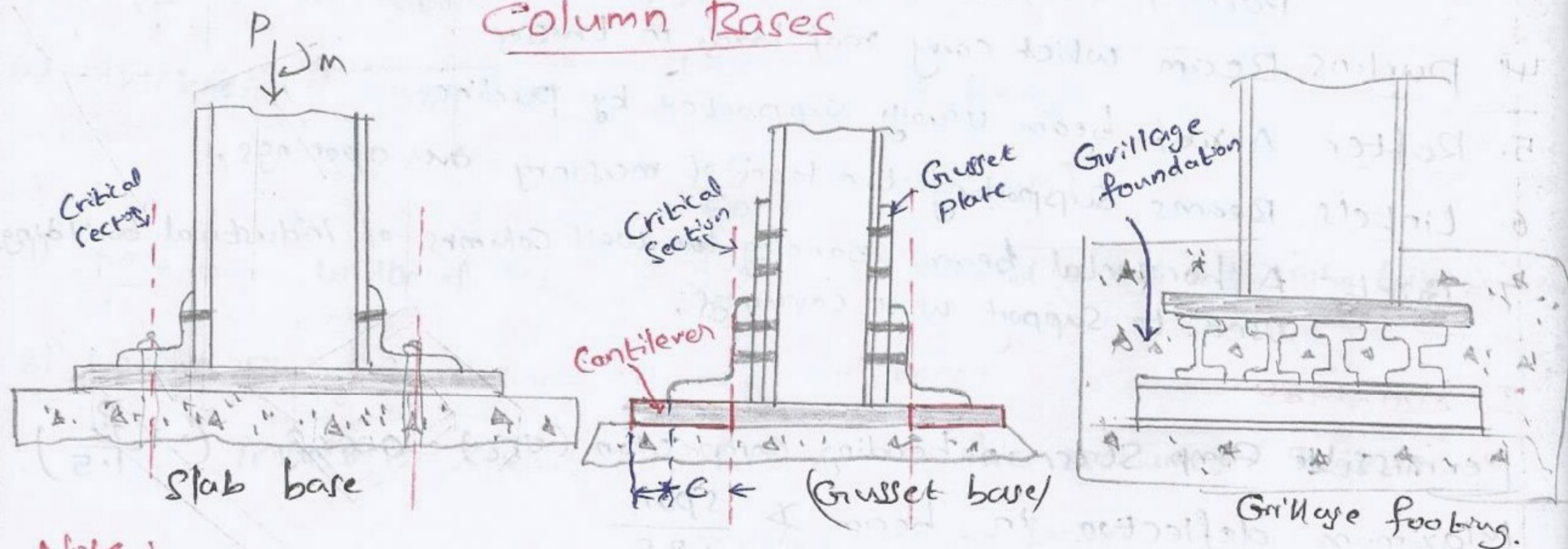
Note:

Webs shall be checked for Shear buckling, when $\frac{d}{t_w} > 67 \epsilon$

Extreme Conditions

Member	Supporting	Maximum deflection	Maximum B.M
Simply supported	Elastic cladding	Span/240	$\frac{1.2 Z_e f_y}{\delta_{mo}}$
	Brittle cladding	Span/300	
Cantilever	Elastic cladding	Span/120	$\frac{1.5 Z_e f_y}{\delta_{mo}}$
	Brittle cladding	Span/150	

Column Bases



Note!

→ For an axially loaded column, the pressure distribution b/n base plate and concrete pedestal is assumed to be uniform.

Slab base

- For small loads, slab base is used
- Angles are designed for uplift forces.
- The maximum bearing pressure $\neq 0.6 f_{ck}$

Gusset base

- Adopted when the load is large.
- Axial load accompanied by B.M.s
- Having Eccentric loading
- Area can be increased by adding gusset plate
- loady transferred 50% fasteners

Grillage foundation

- Adopted when
- Loads on column are extremely heavy
- The bearing capacity of soil for the area of gusset/slab is not enough
- Distribution area is very large

$$t_s = \sqrt{\frac{2.5W(a^2 - 0.3b^2)\delta_{mo}}{f_y}} \geq t_p$$

$$\therefore W = \text{Intensity of pressure} \\ = \frac{\text{Direct load}}{\text{Area of plate provided}} \\ 0.3 = \mu (\text{poisson's ratio})$$

$$t_g = \sqrt{\frac{2.75W}{f_y}} \times C \quad (\because \delta_{mo} = 1.10)$$

$$\therefore C = \text{Cantilever of gusset base} \\ = \frac{\text{Size of plate} - (b_f + \dots)}{2}$$

Plate Girder

Stiffeners

a) Intermediate Transverse web Stiffener

To improve the buckling strength of a slender web due to shear

b) Load Carrying Stiffener

To improve prevent local buckling of the web due to concentrated loading

c) Bearing stiffener

To prevent local crushing of the web due to concentrated loading

d) Torsion Stiffener

To provide torsional restraint to beams & girders at supports

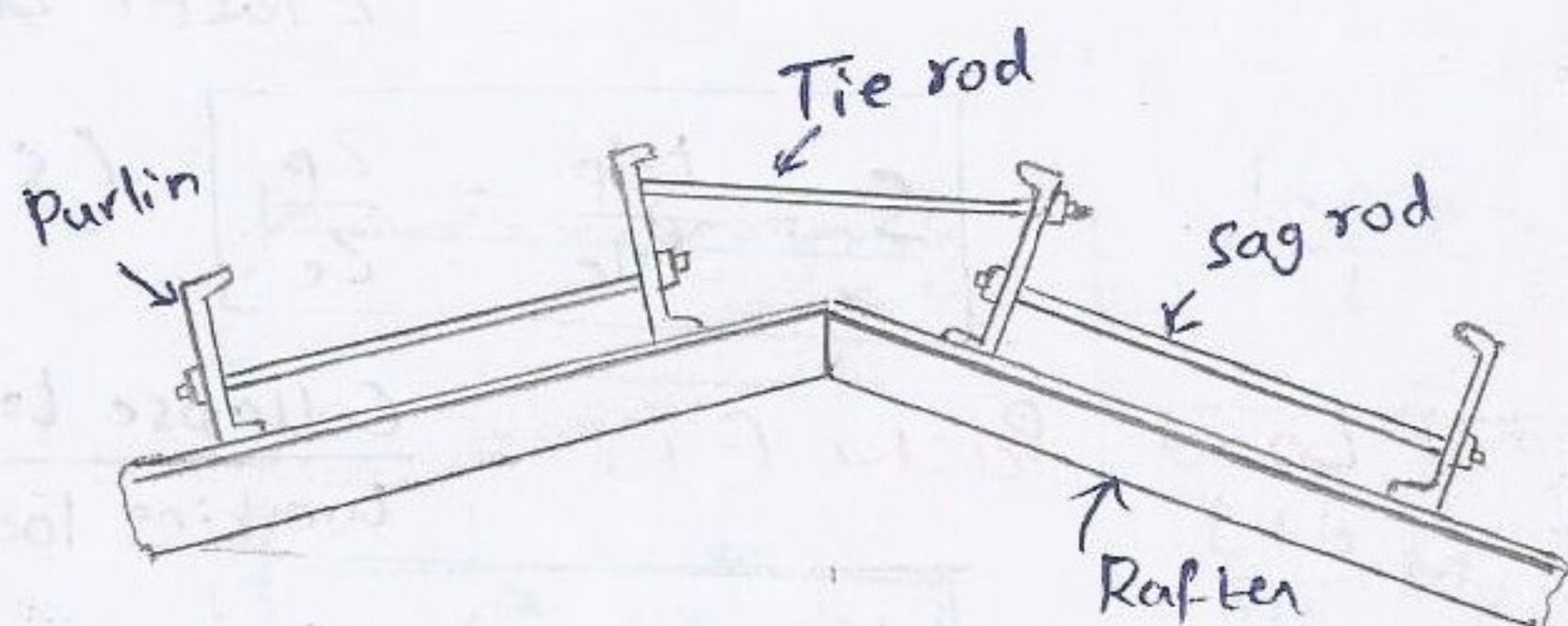
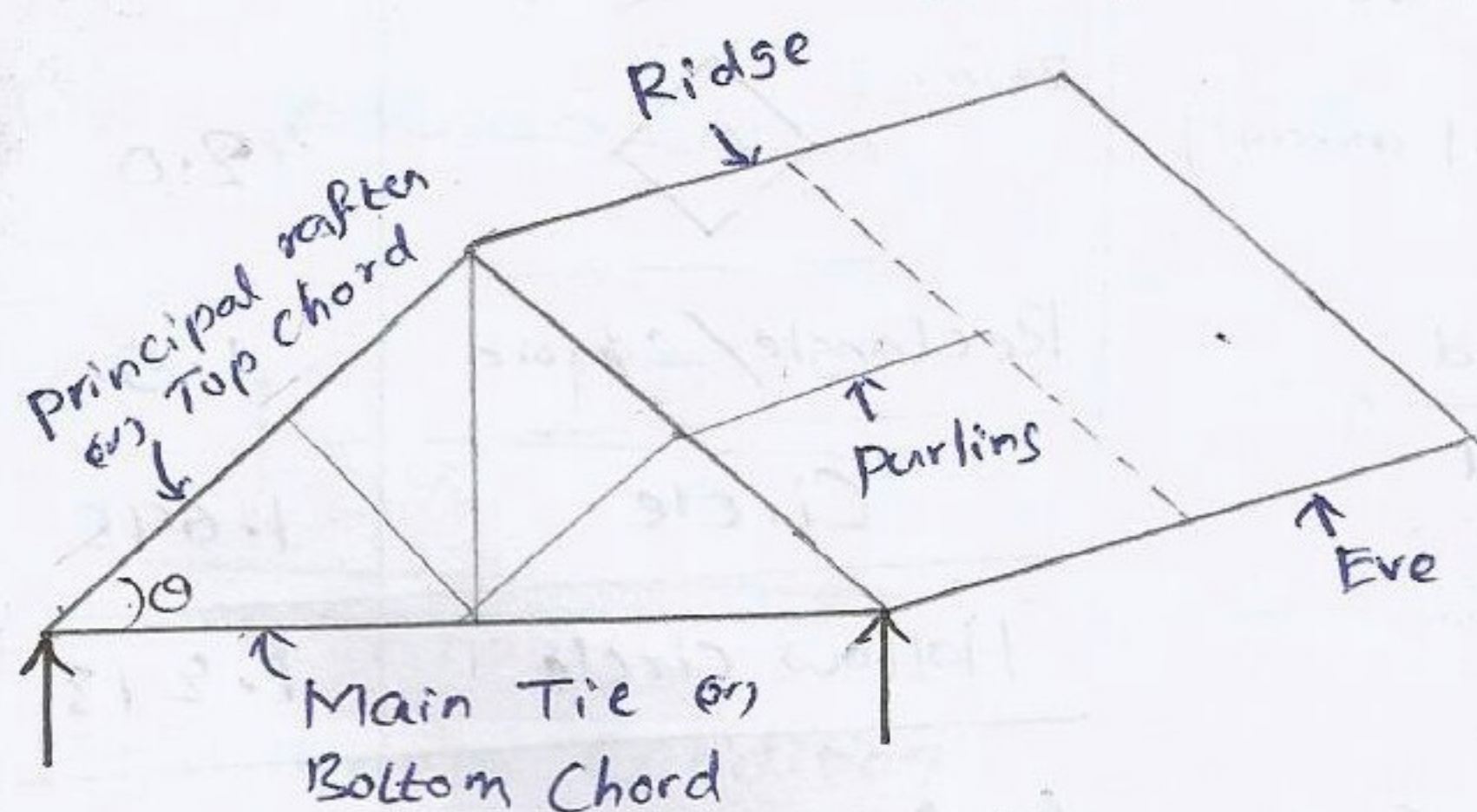
e) Diagonal Stiffener

To provide local reinforcement to a web under shear and bearing

f) Tension stiffener

To transmit tensile forces applied to a web through a flange

Roof Trusses



$$\text{Pitch} = \frac{\text{Rise}}{\text{Span}} \left[\begin{array}{l} \text{Pitch} < \frac{1}{12} \text{ (Small)} \\ > \frac{1}{5} \text{ (Large)} \end{array} \right]$$

$$\text{Slope} = \frac{\text{Rise}}{\text{half span}}$$

Spacing of Truss

a) Economic view

$$E = 2P + r \quad \text{(where } E, P, r = \text{Cost of Truss, Purlins \& roof Coverings) per unit area respectively)}$$

b) General view

$$\text{Spacing} = \frac{1}{4} \text{ span (if Span} \leq 15 \text{ m)}$$

$$= \frac{1}{5} \text{ span (if Span} = 15-30 \text{ m)}$$

Live load

Roof Slope	Access	Live load (of Plan Area)
$\leq 10^\circ$	Provided	1500 N/m ²
	Not provided	750 N/m ²
$> 10^\circ$	—	750 - 2(0-10°)
		400 } which ever is less

Truss	Span
King post	$\leq 6 \text{ m}$
Queen post	6 - 9 m
Palt Truss	6 - 30 m
French Truss	10 - 20 m
Howe Truss	6 - 24 m

Self weight of Roof Truss

$$= \left(\frac{\text{Span}}{3} + 5 \right) \times 10 \text{ N/m}^2$$

Plastic Analysis

The term plastic has occurred due to the fact that the ultimate load is found from the strength of steel in plastic range.

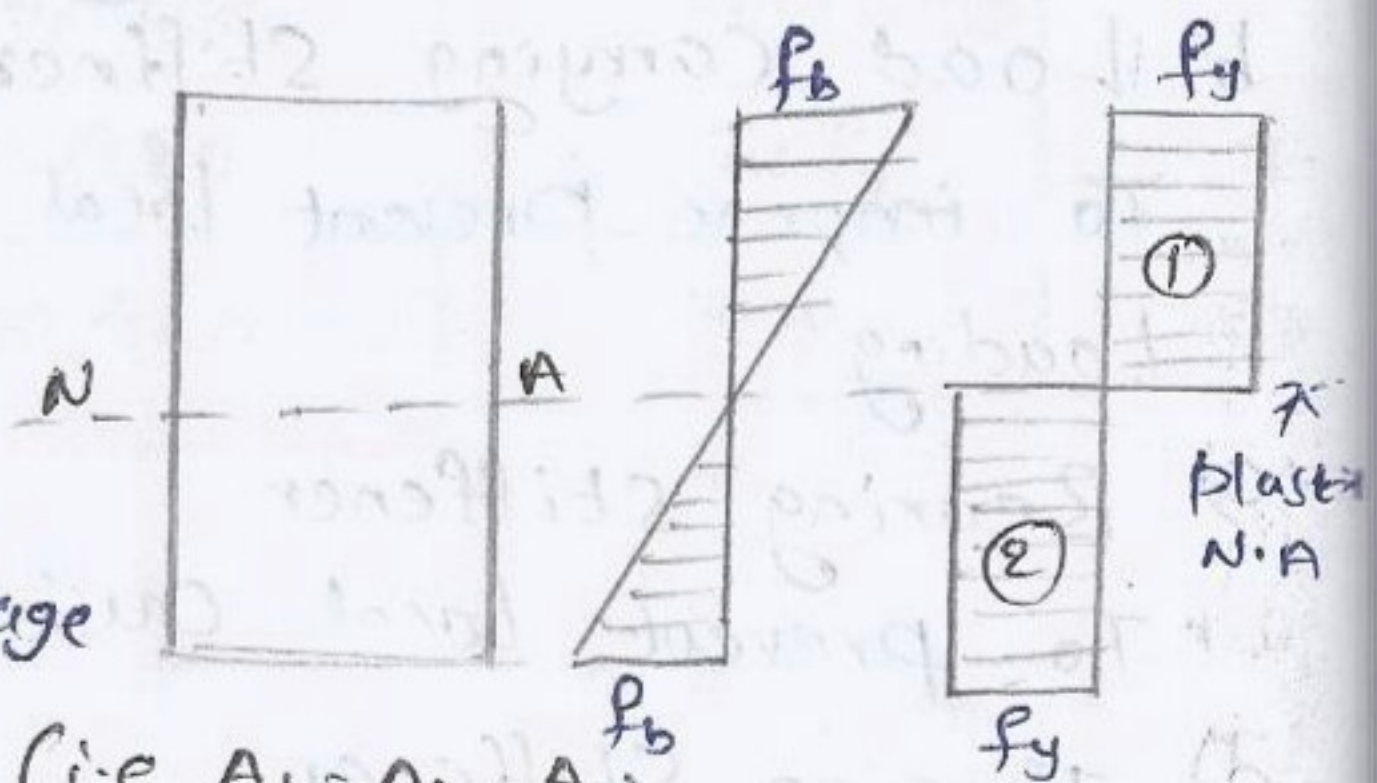
→ plastic analysis mainly design statically indeterminate structures

* Due to the increase in BM, a stage will be reached when all fibres will be yielded. The beam at this stage reached its max resisting capacity at this stage.

→ Plastic analysis will be considered at this stage

→ Plastic N.A also called Equal Area axis (i.e. $A_1 = A_2 = \frac{A}{2}$)

→ Moment of resistance of the section at this stage is called Plastic moment.



1) Plastic moment (M_p) = $f_y Z_p$

Plastic section modulus (Z_p) = $\frac{A}{2} (\bar{y}_1 + \bar{y}_2)$

2) Shape factor (S) = $\frac{\text{Plastic moment}}{\text{Elastic moment}}$

$$S = \frac{M_p}{M_e} = \frac{Z_p}{Z_e} \quad (S > 1 \text{ always})$$

3) Load factor (λ) = $\frac{\text{Collapse Load}}{\text{Working load}}$

$$\lambda = (F_o.s) \times S$$

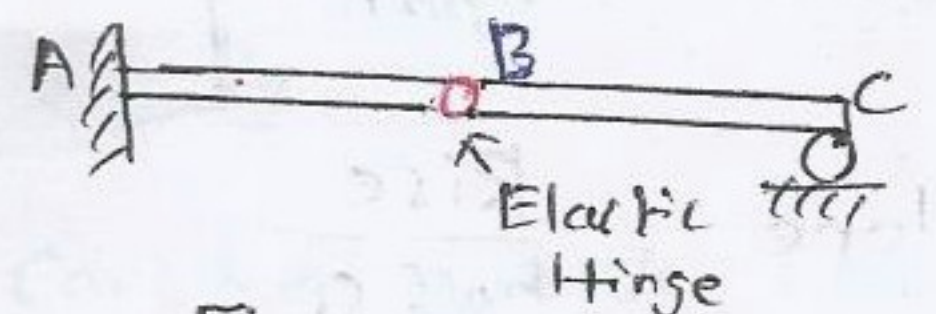
Shape	Shape factor (S)
Triangle	2.343
Triangle	2.0
Rhombus	2.0
Rectangle/square	1.5
Circle	1.698
Hollow circle	1.273

$$\lambda = \frac{P_u}{P_w} = \frac{M_p}{M} = \frac{f_y Z_p}{f Z_e} = \left(\frac{f_y}{f} \right) \times \frac{Z_p}{Z_e} = (F_o.s) \times S \quad (\because f = \text{permissible working stress})$$

Elastic Hinge

It is the location where structural member is free to rotate

→ i.e., member cannot resist BM at elastic hinge location.



$$[B M_B = 0]$$

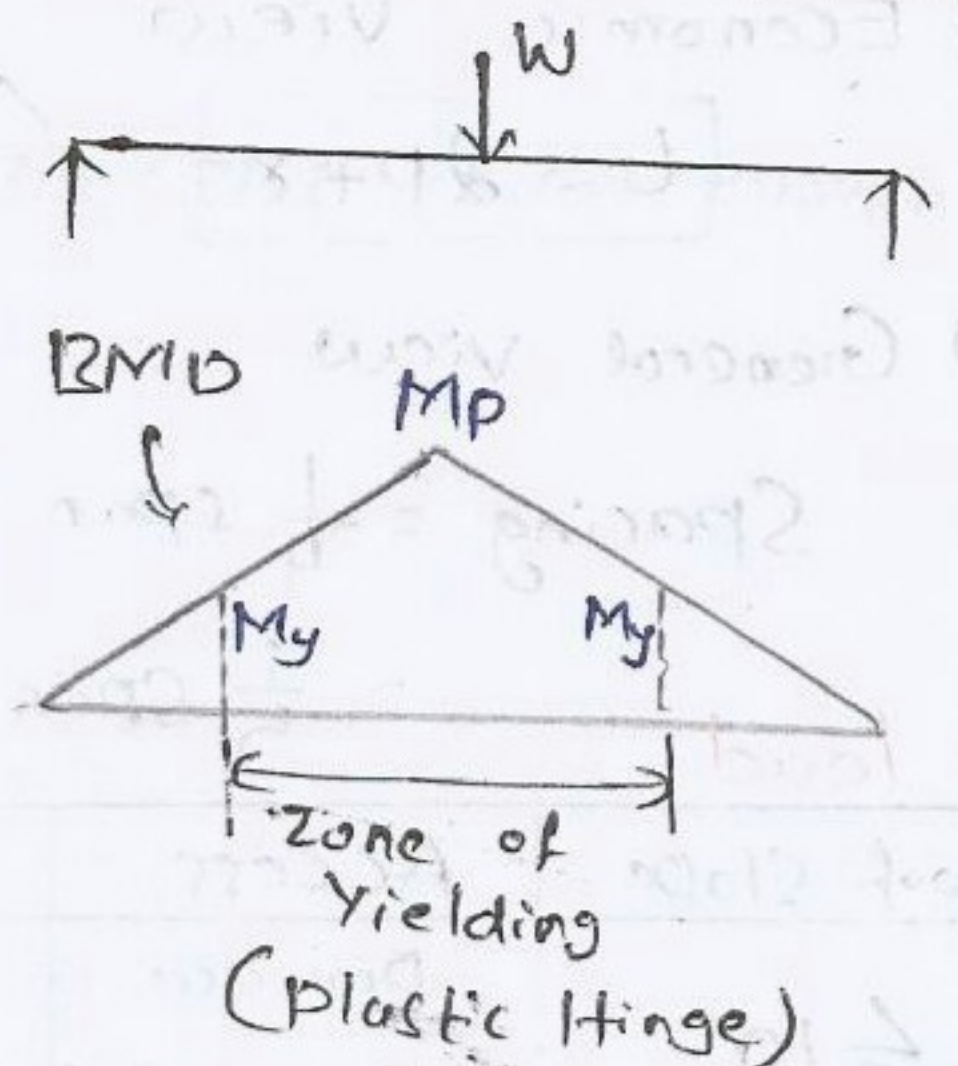
Plastic Hinge

Plastic hinge is not a point, but a zone of yielding where moment value lies M_y to M_p

→ At plastic hinge location moment capacity ($\frac{M}{EI} = \infty$)

i.e. Curvature $\frac{1}{R} = \text{Infinity}$

→ B.M at plastic hinge $\leq M_p$

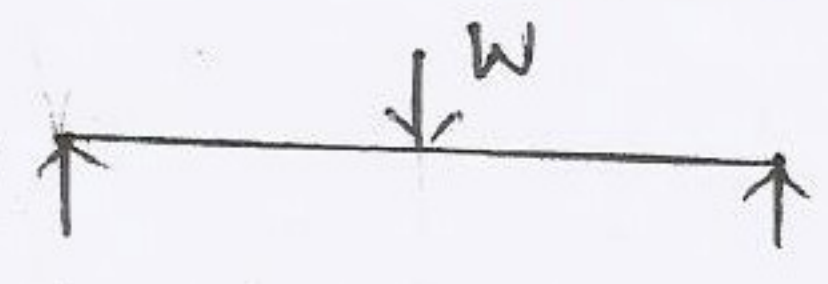
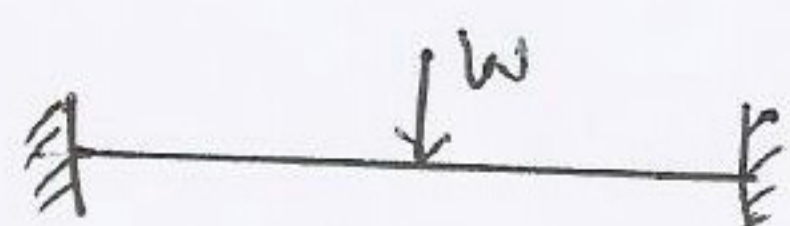
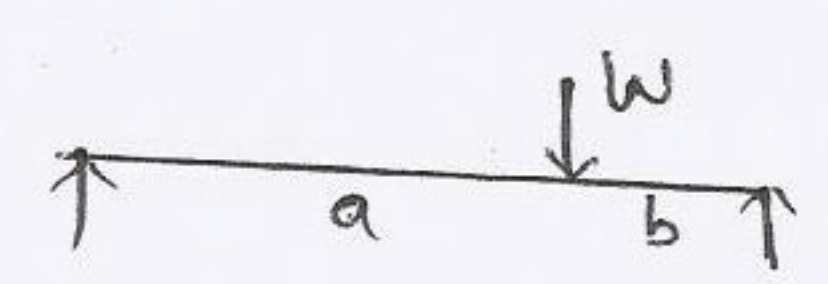
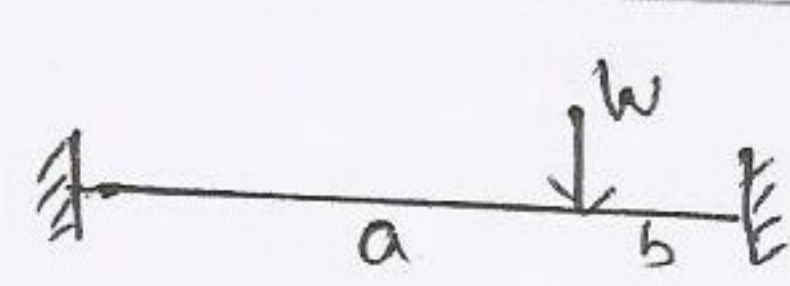
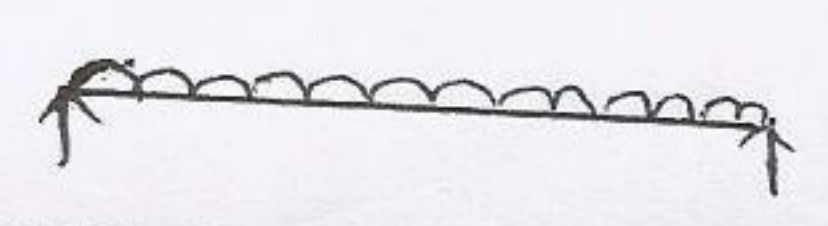
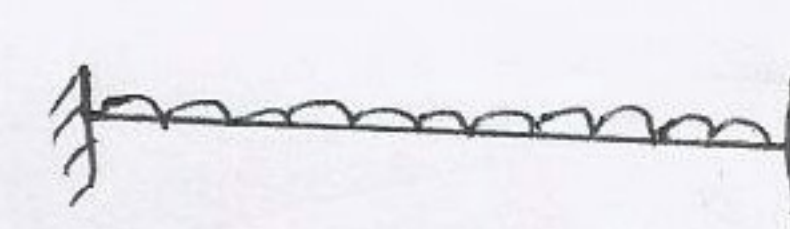
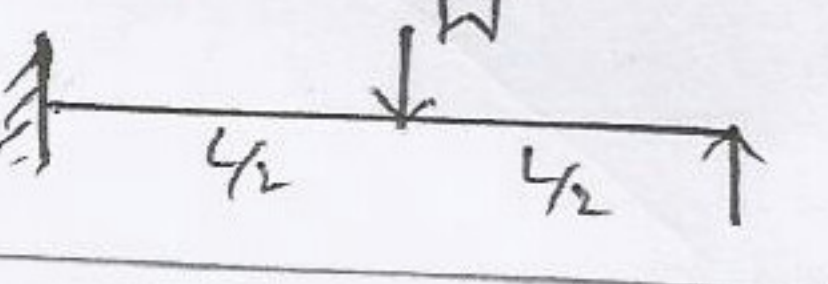
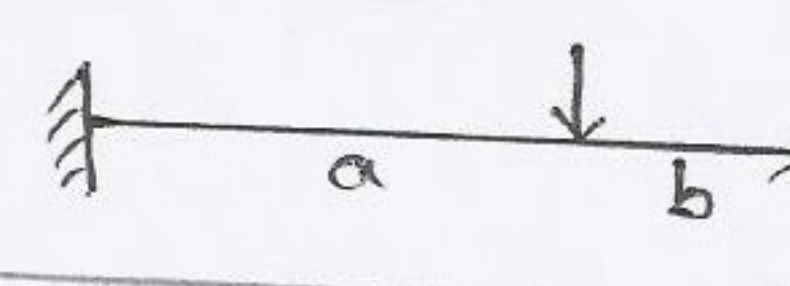


$$\text{Plastic hinge} = \frac{L}{3} \quad \text{for above condition}$$

Location of Plastic Hinge

1. At points of action of concentrated loads
2. At sections of sudden change in geometry (i.e. $\boxed{2EI} \rightarrow \boxed{EI}$)
3. At fixed ends
4. At points of Zero Shear in span
5. When two members meet at a section, A plastic hinge will be formed in the member of Lesser ($2p$)
6. When 3 or 4 members meet at a point, plastic hinge formed in All members.

Collapse Load (P_u)

	P_u	P_u	
① 	$\frac{4 M_p}{L}$		$\frac{8 M_p}{L}$
② 	$\frac{L}{ab} M_p$		$\frac{2L}{ab} M_p$
③ 	$\frac{8 M_p}{L}$		$\frac{16 M_p}{L}$
④ 	$\frac{6 M_p}{L}$		$\frac{L+b}{ab} M_p$

Plastic Analysis

Plastic Analysis

1) Equilibrium Condition

$$\sum F_x = 0, \sum F_y = 0 \text{ \& } \sum M = 0$$

2) Yield Condition

B.M at any point should not exceed the full plastic moment at that point.

i.e. $\boxed{M \leq M_p}$

3) Mechanism or Continuity Condition

($\therefore$ Mechanism = The Condition of Segments of the beam b/w the plastic hinges) are able to move without increase of load

i.e The structure at collapse must be Capable of deforming as a mechanism due to the formation of plastic hinges.

i.e The ultimate load is reached when a mechanism forms.

Basic Theorems in Plastic Analysis

Usually, it is not possible to satisfy all the three conditions of Plastic analysis in one operation.

a) Static Theorem (Lower Bound theorem)

Represents the lower limit of collapse load to the true ultimate load. and has a $(F.O.S)_{max}$. It satisfies Equilibrium & Yield Conditions

b) Kinematic Theorem (Upper Bound theorem)

Represents the upper limit of collapse load to the true ultimate load.

& has a $(F.O.S)_{min}$. It satisfies Equilibrium & Mechanism Conditions