LECTURER NOTES

DESIGN OF STEEL STRUCTURES

B.Tech,6TH Semester,
Civil Engineering

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COURSE CONTENT

Design of Steel Structure

B.Tech, 6th Semester, CE

Module I

Introduction, advantages/disadvantages of steel, structural steel, rolled steel section, various types of loads, design philosophy. Limit state design method, limit states of strength and serviceability, probabilistic basis for design Riveted, bolted and pinned connections, Welded connections-assumptions, types, design of fillet welds, intermittent fillet weld, plug and slot weld, failure of welded joints, welded joints vs bolted and riveted joints

Module II

Tension members, types, net cross-sectional area, types of failure, slenderness ratio, design of tension members, gusset plate.

Module III

Compression members, effective length, slenderness ratio, types of cross-section, classification of cross section, Design of axially loaded compression members, lacing, battening, design of column bases, and foundation bolts.

REFERENCES

Design of Steel structures

B.Tech, 6th Semester, CE

Books:

[1] Limit State Design of Steel structures by S.K. Duggal, Mc-Graw Hill

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Design of steel structures

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- Types of sections
- Grades of steel
- Strength characteristics
- IS code connections
- Design of tension & compression member
- steel roof truss, beams, column bases.

IRON

1. Pure iron (non alloy):→
   - It is natural metal available directly in the earth ores
   - silvery white in colour.
   - It is very soft solid (such that it can be cut by knife) having high ductility.
   - It is not used in any structural element since it directly reacts with oxygen & to form rust & reacts with moist air.
   - It is available in Fe^{2+} & Fe^{3+} forms.
2. pig iron:→
   - Basic raw iron is called pig iron (transported in the form of bricks).
   - It is also not used in any structural element since it is composed of highest carbon content 5%).
   - the pig iron can be converted into structural iron by removing excess carbon content & by adding oxygen or chemicals in molten stage
3. Cast Iron:—
   - It is the structural element (in specified shape from molten pig iron having almost same properties of pig iron).

4. Wrought Iron :→
- Lowest carbon content (0.0 - 0.1 %) of structural iron
- It has high ductility, easily converted in specified shape.
- largely used to make thin wires.

5. Steel :→
- Steel is an alloy of → (iron + carbon + chromium + copper + magnesium + nickel + silica)
- The structural element used to resist any type of load.

Carbon content

- pig iron → 4-5 %.
- cast iron → (2-4.5 %)
- cast steel → (>2 %)

→ carbon steel (less than 2 %)
- High carbon steel → (0.6 - 1.4 %)
- medium " " → 0.25 - 0.6 %.
- low " " → less than 0.25 %.
- Wrough Iron - less than 0.1 %.
- pure " - 0 %.

Introduction

properties	low carbon	Medium carbon	High carbon
carbon	Lower than 0.25 wt. %	In betn 0.25 & 0.6 wt. %	In betn 0.6 & 1.4 wt. %
some properties	- Excellent ductility & toughness. - Weldable & machinable - no amenable to martensite transformation	- lowhardenability - These steel grades can be heat treated.	Hardest, strongest & least ductile.
some appln	common products like Nuts, bolts, sheets. etc.	For higher strength such as in machinery, Automobiles & agricultural parts (gears, axels, connecting rods) etc.	used where strength, hardness & wear resistance is required. cutting tool, cable, musical wires.

→ Steel is an alloy of Fe + carbon

MILD steel (carbon content - 0.23 %.)

- when carbon is increased in steel then strength, hardness & brittleness will " but ductility will decrease.

stainless steel :→

- Alloy of iron & chromium
- chromium is 18 %. & Nickel is 8 %.
- young's modulus of steel 'E' = 2×10^5 MPa ≃ 200 GPa

$E_{Aluminium} \simeq \frac{1}{3} E_{steel}$

$= 0.7 \times 10^5$ MPa or 70 GPa

Density of steel :—

$\rho = \begin{cases} \rho_{steel} = 7850 \ kg/m^3 \\ \rho_{Aluminium} = \frac{\rho_{steel}}{3} = 2700 \ kg/m^3 \end{cases}$

Modulus of Rigidity (G) :—

$G = 0.769 \times 10^5$ MPa

poisson's Ratio (μ) :—

$$\mu = \frac{\text{lateral strain}}{\text{longitudinal strain}}$$

- μ for mild steel = 0.28
- In elastic range = 0.3
- In plastic " = 0.5

Deflection / Increase in length :—

$$\Delta L = \frac{PL}{AE}$$

$(\delta L)_{mild\ steel} = \frac{1}{3} (\delta L)_{Al}$

thermal co-efficient :—

- $\alpha_{steel} = \alpha_{concrete} = 12 \times 10^{-6} \ °C^{-1}$
- $\alpha_{Al} = 23 \times 10^{-6} \ °C^{-1}$
- steel is ductile while concrete are rubber are brittle.

Note!—

Rubber is a very brittle material, there is very little plastic deformation beyond elastic range.

Some imp. codes

IS 456 : 2000 ⟶ RCC

IS 800 : 2007 ⟶ steel (2007 - LSM, 1984 - WSM)

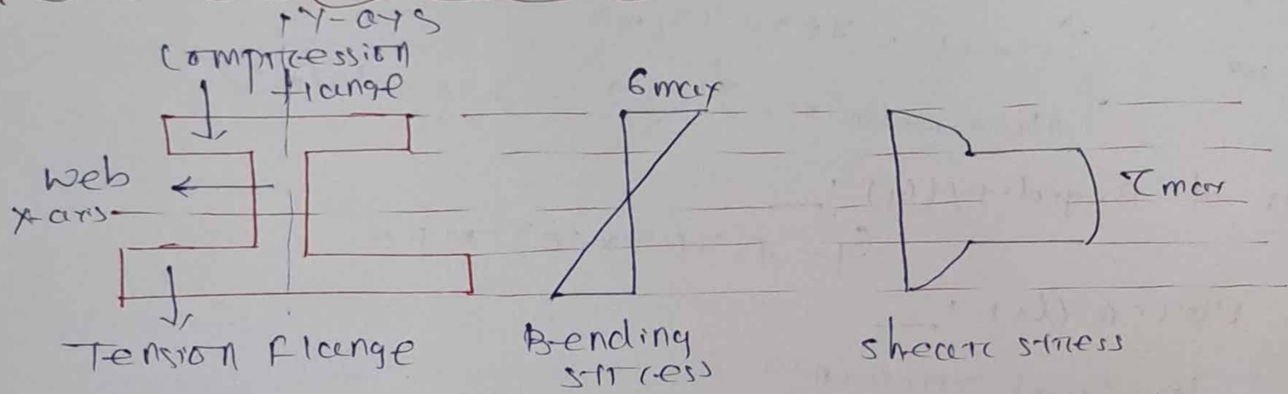
IS 1343 ⟶ Pre stress concrete

IS 10262 ⟶ Dgn. mix

IS 383 ⟶ Fine & coarse aggregate

IS 875 ⟶ Dgn. load for buildings & structures.

Standard structural steel section



→ In I-secn, the web resists SF while the flanges resist most of BM experienced by the beam.

i) ISLB 300 :—
- Indian standard light beam where overall depth = 300mm
- maxm bending stress is resisted by flange & maxm shear stress by web.
- Generally used in roof beam.

ii) ISMB :—
- Indian standard Medium flange beam generally used in floor beams.
- High moment of inertia about x-axis, so lateral buckling occurs about y-axis.

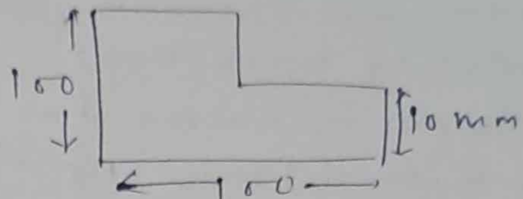
iii) ISWB :—
- Indian standard wide flange beam generally used in column.
- High moment of inertia about y-axis so they have buckling strength about y-axis.

iv) ISJB :— IS junior beam

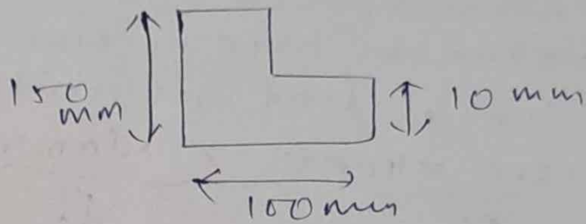
v) ISHB :— " Heavy "

Angle section

i) Equal Angle secn :→
ISA - 100×100×10
(Both legs are same; 10 = thickness of angle secn)

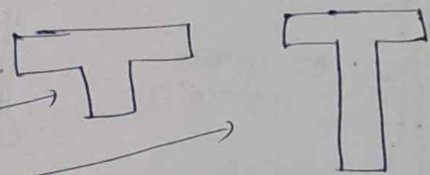


ii) Unequal angle secn :—
ISA - 150×100×10
(different leg; 10 = thickness)



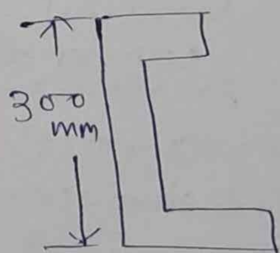
T-secn :→

i) ISHT - IS wide flange T secn
ii) ISST - IS ~~stand~~ long legged T secn



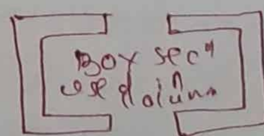
channel secn :→

they are used as purlins or columns in a roof truss which (purlin is a beam in a roof covering material). supports the roof



i) ISJC — IS Junior channel secn
ii) ISLC — IS Light " "
iii) ISMC 300 — IS Medium " " when 300 is the overall depth of channel secn.
iv) ISSC — IS special " "

BOX secn → used in column



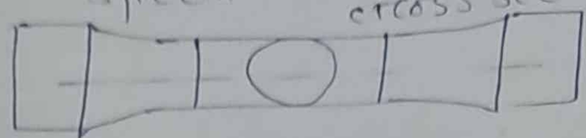
Flat secn :—

ISF → IS Flat secn
- Used in dgn. of lacing & batten.

Eg. 50 ISF 8

↓ width of plate

↓ thickness of plate.

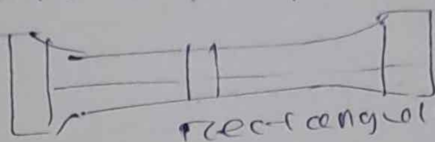


Uniaxial tension test:—

this test is of static type i.e the load is increased comparatively slowly from zero to a certain value.

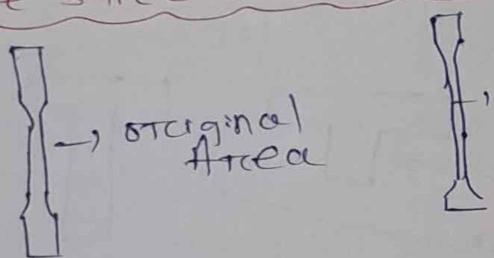
UTM → or tensile testing machine is used.

- the ends of the specimen's are secured in the grip of the testing machine.



- there is a unit for applying a load to the specimen with a hydraulic or mechanical drive.
- there must be a some recording service by which you should be able to measure the final o/p in the term of load or stress.

True stress & Nominal stress:—



1. Nominal stress-strain or conventional stress-strain diagrams:→

- stresses are usually computed on the basis of the original area of the specimen; such stresses are often referred to as conventional or nominal stresses.

2. True stress-strain diagram:—

Since when a material is subjected to a uniaxial load, some contraction or expansion always takes place.

- thus, dividing the applied force by the corresponding actual area of the specimen at the same instant gives the so called true stress.

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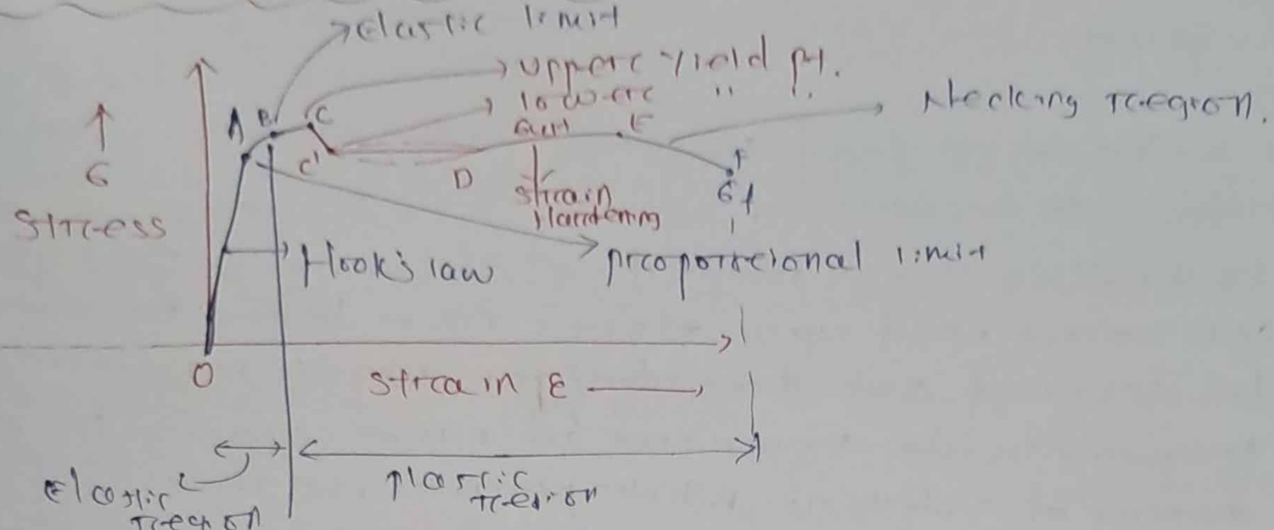
Stress-strain curve for Mild steel



OA → proportionality limit
OB → Elastic " But OB (non linear)

- the slippage of the carbon atom with in a molecular mass marginally from C to C' leads to drop down of stress.
- C → upper yield pt.
  C' → lower " " (yield stress f_y)
  For exp Fe-250 ⇒ $f_y = 250$ N/mm2
→ C'D → const. stress region called yield plateau.
- DE → Strain Hardening region → material starts offering resistance against deformation.
- EF → Necking region → drop down of stresses occur upto failure pt.
- Necking region exists only in ductile material.
- In mild steel - ABC are closer to each other, therefore it is known as linear elastic metal, & yield stress & elastic stress is taken as 250 N/mm2.
- the fracture or failure in mild steel depends upon % of carbon present in a steel.

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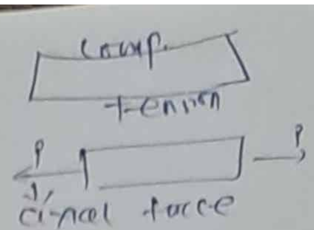
Permissible stress in steel structures



[Permissible stress = $\frac{\text{Yield stress } f_y}{\text{FOS}}$]

- It is the maxm load carried by the member without deformation.
- In working stress method, it is assumed that members can carry load upto elastic limit. hence members will be designed such that they can resist less loads as compared to the resistance of maxm capacity by proper factor of safety to whole permissible stress.

→ FOS → 1.67 → for members s.t. Axial tension or comp.
→ 1.50 → " " " Bending.

- Since in axial loading all fibers reach maxm stresses, but in bending only extreme fibers will reach maxm stresses. Hence FOS will be less for bending.

Working stress Method :→

- In the field there are always worst combination of load (DL, LL, EL, WL, etc) hence members will be designed such that they can resist more & more loads of actually we needed.
- Ultimately size & cross secn area of the member increase. Hence working/ failure stress decreases.

$$\text{Working stress} = \frac{\text{load supplied to the members}}{\text{cross sectional area}}$$

$$= \frac{\text{yield stress } f_y}{\text{FOS}}$$

Merits :→

- The members can't be failed in future having large life span.
- The dgn. is very simple.

Demerits :→

- Wt. of the stc. increases. hence it is uneconomical.

Plastic state or limit state method

The dgn. of the members may reach the plastic range i.e FOS will be desired to reach loads by considering load combinations & strength & serviceability requirements.

- Hence it is called as partial factor of safety.
- [Design load = corresponding characteristic load / partial factor of safety.]

Permissible stress in steel str.:-

1. As per WSM:—

i) max^m permissible axial stress in compression is

$\sigma_{ac} = 0.60 f_y$

- Used in the dgn. of columns & struts.
- Column is a compression member where BM exist while in case of struts, also being a compression member, BM is zero. Because strut is a component of roof trusses & roof trusses are pin jointed connection having BM = 0.

ii) max^m permissible axial stress in tension is given by $\sigma_{at} = 0.60 f_y$

- It is used in tension members.

FOS → 1.67 → axial tension & compression.
– 1.50 → Bending

iii) max^m permissible bending stress in compression is given:—

$\sigma_{bc} = 0.66 f_y$

- Used in dgn. of flexural (bending) member i.e beam, built up beam, plate girder etc.

iv) max^m permissible bending stress in tension is given → $\sigma_{bt} = 0.66 f_y$ ↑ FOS = 2.5

v) max^m permissible avg. shear stress given, $\tau_{v\,avg} = 0.40 f_y$

vi) " " max^m " " ", $\tau_{v\,max} = 0.45 f_y$
FOS = 2.2

v) maxm permissible bending stress is given by :—

$\sigma = 0.75 f_y$

Increase of permissible stress :→

- When wind & earthquake load are considered, the permissible stresses in steel stru. are increased by 33.33%
- " " , connections (rivet & weld) are ↑ by 25%

Permissible deflection in steel stru. :→

→ maxm permissible horizontal & vertical deflection given then

$\delta = \frac{span}{325}$ as per WSM

i) $\delta = \frac{span}{300}$ → supported element are not susceptible to cracking

ii) $\delta = \frac{span}{360}$ → susceptible to cracking.

Permissible stress in Gantry girder :→

~~→ for manually~~ laterally unsupported becoms to

- Gantry girders are used to carry heavy loads from place to place at the construction sites.

i) For manually operator crane, $\delta = \frac{span}{500}$

ii) For electrical " , $\delta = \frac{span}{750}$ → upto 50T or 500kN capacity.

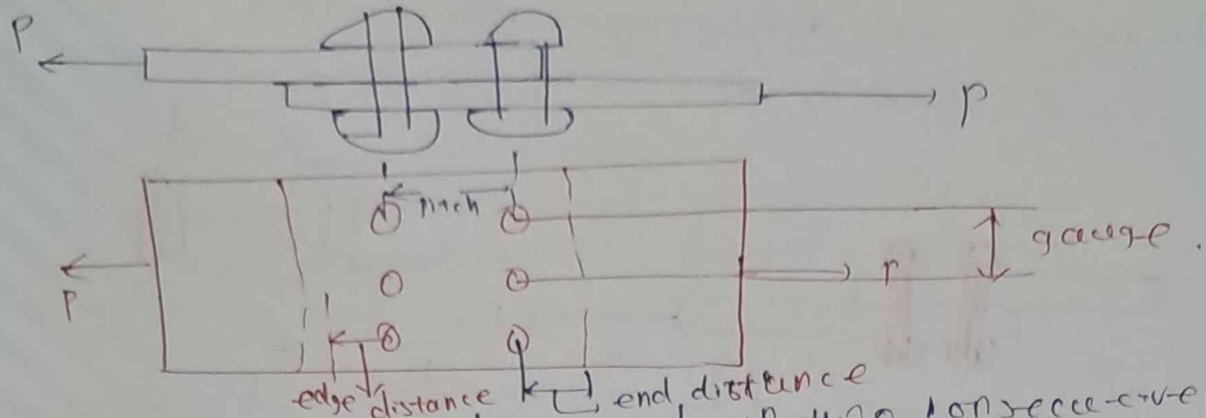
iii) " " " , $\delta = \frac{span}{1000}$ → more than 50T or 500kN

FOS for diff. stresses :→

$FOS = \frac{f_y}{f} = \frac{\text{yield stress}}{\text{working stress}}$

$= 1.67 = \frac{f_y}{0.60f}$ → axial stress ← Both tension & comp.

$= 1.50 = \frac{f_y}{0.66f}$ → bending "



1. Pitch (P) → It is the distance betn two consecutive/continuous rivets measured parallel to the dirn of force.

2. End distance! → It is the distance betn centre of rivet & edge/end of the plate element, measured parallel to the dirn of force.

3. Gauge distance → It is the distance betn two continuous rivets measured ⊥r to the force of dirn.

4. Edge distance → It is the distance betn centre of rivet & edge/end of the plate element, measured ⊥r to the force of dirn.

Types of stresses

1. Direct stress
   a) Direct tensile stress
   b) " compressive "
   c) " shear "

2. Indirect stress
   a) Bending stress → $\frac{M}{I} = \frac{\sigma_b}{y} = \frac{E}{R}$
   b) torsional shear stress, $\frac{\tau}{r} = \frac{T}{I_p}$

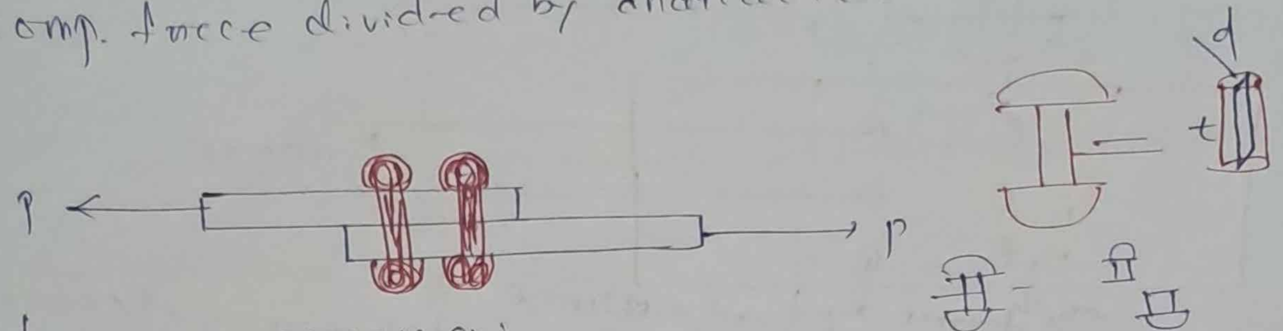
Stress! — Normal → Direct normal stress → $\sigma_d = \pm \frac{P}{A}$
Normal → Bending " → $\sigma_b = \pm \frac{M}{I} y$

shear
Primary shear stress → $\tau = \pm \frac{FAy}{IB}$
torsional " " → $\tau = \frac{T}{J} \times r$

Bearing stresses :—

It is nothing but compressive stresses developed at the surfaces of two different materials are in contact.

→ comp. force divided by characteristic area ⊥r to it.



Shearing stresses :—

two forces, equal & opposite in nature, when act tangential to the resisting secn. as a result of which the body shear off across the secn is known as shear stress.

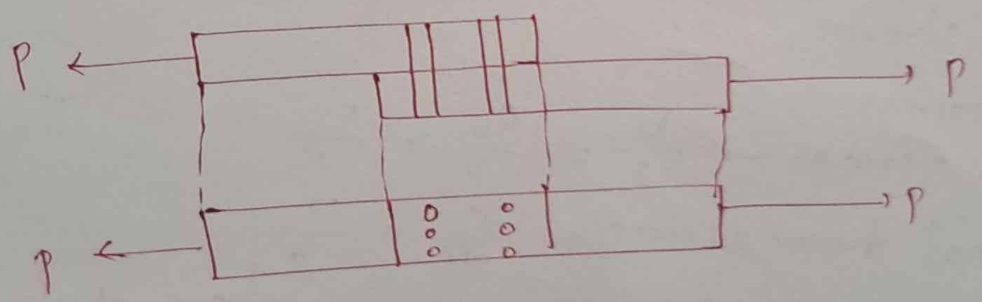
Types of Joints

1. Lap Joint :→

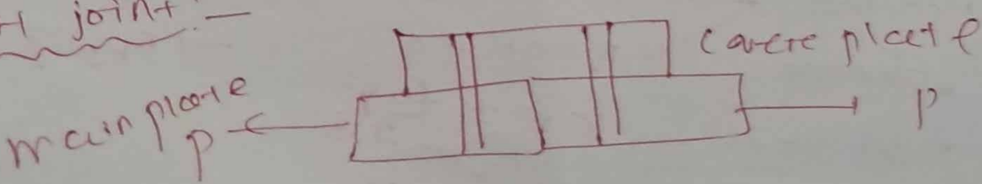
It is the least efficient joint as the line of action of two forces are not same.

– In lap joint, the rivets are subjected to single shear & bearing.

– these forces form couple & addition bending stresses are developed in the rivets.



2/ Butt joint :—



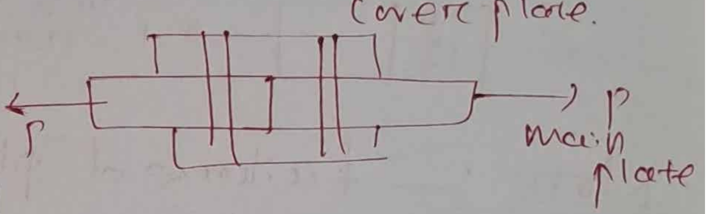
Single cover butt joint :→

The line of action of two forces is same therefore eccentricity is eliminated completely which existed in lap joint hence this joint is more efficient in carrying the forces as compared to lap joint.

- But the connection is not symmetrical.
- The rivets are s.t. single shear & bearing.
- $t_{cover} \geq t_{main}$ (so that the joint doesn't fail).

Double cover butt joint :—

- It is the most efficient joint because the line of action of two force is same & connection is symmetrical w.r.t applied load.

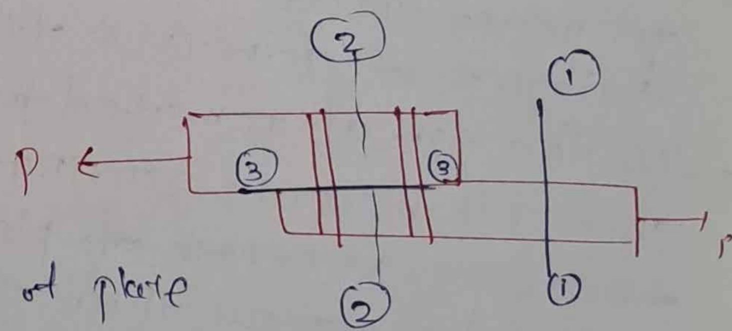


- The rivets are subject to double shear & bearing.
- Sum of thickness of cover plate $\geq t_{main}$.

Connections :→

In steel str., various types of elements are connected together using various types of connections.

i) Riveted connections
ii) Bolted "
iii) Welded "



Strength of plate :→

sec^n 1-1 → tearing strength of plate

sec^n 2-2 → Bearing " "

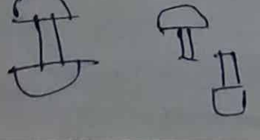
sec^n 3-3 → shear " "

Failure of Riveted joint

Failure of rivets

i) Shearing

Rivet gets cut into two or more pices.

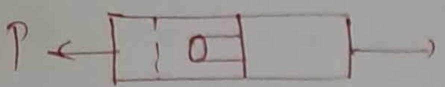


ii) Bearing

- rivet cross sec^n changes from circular to elliptical.

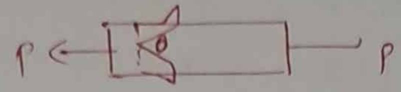
Failure of plates

i) Shearing



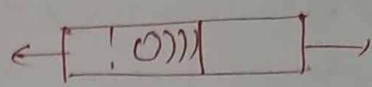
- cracks are developed parallel to the applied forces dirn.

ii) Splitting



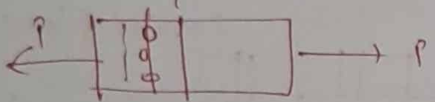
- It occurs due to diagonal tension in the plate at the rivet level

iii) Bearing



this plate is pushed forward by the rivet. this type of failure occurs generally due to insufficient end distance.

iv) tearing / tension failure of the plate:-



there is developed ⊥r to the dirn of cracks of applied force.

Note :- failure of plate:-

- shear, bearing & splitting failure of plate are due to insufficient end distance.
- to providing the proper end distance, these three failure can be prevented.
- In the dgn. of riveted joint which should consider the remaining three failure only, i.e. shear & bearing failure of rivets & tearing failure of plate.
- In the dgn. of riveted joint, we have to ensure that shear & bearing strength of rivets is more than the tearing strength of plate because rivet failure is more dangerous than the plate failure.

Plate:-

shearing
bearing } insufficient end distance
splitting

Rivet:

tearing
shearing }
bearing

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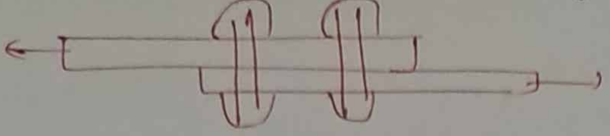
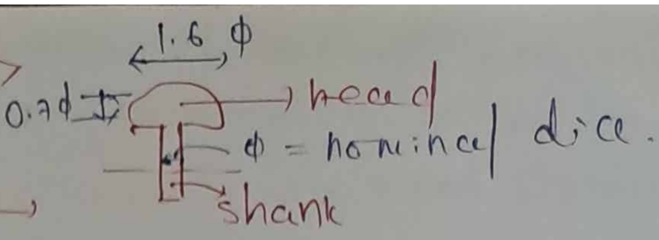
Riveted connection :->





- Rivets are inserted in the hole made to join the two members together & hammering is done to make head on other side.
- Rivets are made of mild steel. The riveting can be hot riveting (or) cold riveting.
- Cold riveting is not adopted for dia > 10 mm.
- In " " there is no gripping action but strength is better due to cold working.
- When hot rivet is used, it becomes plastic, it expands & fill the rivet hole completely in the process of forming a head at the other end. On cooling, the rivet shrinks in the length & dia. due to shortening of " shank length.
- The connected parts becomes lighter consequently resulting in tension of unpredictable amount in a shank length & some compression in plates that are connected.
- Due to reduction of dia. of shank on cooling, this small amount of space available on cooling is provided for temp. variation of unpredictable amount.
- In hot riveting, rivets are heated to 550 - 1000°c & hammering is done on other side to make head.
- According to the type of hammering we have:-
  - i) power driven rivets
  - ii) Hand " "
- Power driven rivets have better quality control & hence have a higher permissible stress.

Riveting can be done in the factory case in the field & according in these shop riveting & field riveting thus we have:—

i) power shop rivets PDS
ii) " driven field " PDF
iii) Hand " " " HDF

~~i) pow~~

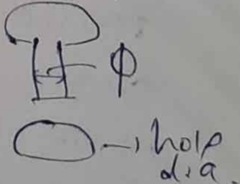
Note For shop rivet

For field rivet ○

●

N/mm2	Axial tension	shearing	bearing
PDS	100	100	300
PDF	90	90	270
HDF	80	80	250

- the nominal dia of rivet is said to be shank dia under cold condn & gross dia. of rivet is taken as dia of hole.
- the strength of a rivet is based on its gross dia. under the assumption that rivet fills the hole completely.
- For ease in connection dia. of hole is taken larger than nominal dia. of rivet thus as per IS code:—



- For ~~not~~ $\phi \leq 25mm$

  Gross dia = $\phi + 1.5mm$

  dia. hole = $\phi + 1.5$

a) $\phi = 16mm \leq 25mm$

hole dia = $16+1.5$

$= 17.5mm$

- For $\phi > 25mm$

  Gross dia. = $\phi + 2mm$ = dia of hole

- Due to many demerits, riveted connection is not in practice in modern steel connection.

Dgn. of riveted connection is same as that of bolted connection but with the following differences:—

i) the dia. of rivet to be used in the calculation is dia of hole, whereas as for bolted connection it is the nominal dia.

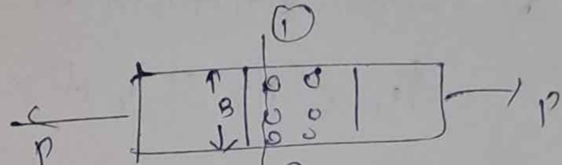
ii) the dgn. stresses are different (IS: 800: 1984) the permissible stress are reduced for bolts.

Strength of riveted joint:—

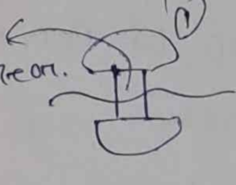
It is taken as minm of shear strength, bearing & tearing strength.

For lap joint:—

i) For entire plate:—



a) shear strength of rivets



$$P_s = n \times \frac{\pi}{4} \times d^2 \times F_s$$

where,

n → total no. of rivets at joint

F_s → permissible shear stress in rivets

$$\sigma = \frac{F}{A} \Rightarrow F = \sigma \times A$$

$F_s = 100$ MPa (WSM)

F_u = Ultimate shear stress in rivet

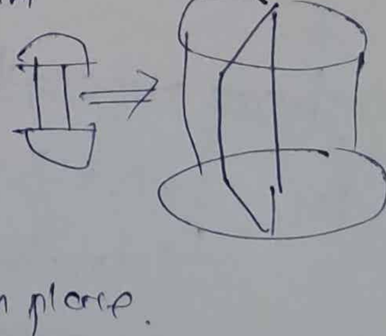
so in LSM = $\frac{F_u}{\sqrt{3} \times 1.25}$

d = gross dia. of rivet (hole dia.)

gross dia = $\phi + 1.5$ mm, $\phi \le 25$ mm

$= \phi + 2$ mm, $\phi > 25$ mm

b) bearing strength of all rivets



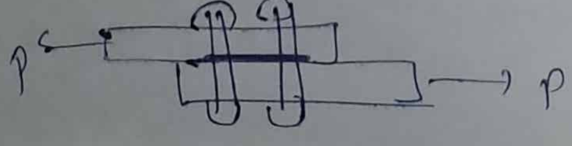
$$P_B = n \times (t \times d) \times F_b$$

where,

t = thickness of thinner main plate.

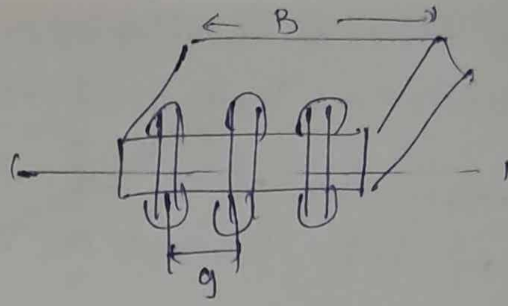
F_b = permissible shear stress in rivets (300 MPa in WSM)

d → d+



c) Tearing strength of plate :—

$P_t = (B - n_1 d) t \times F_t$



where,

n_1 → total no. of rivets at sec1

B → width of the plate

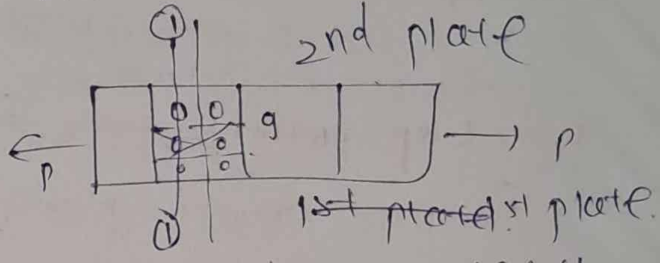
F_t → permissible tensile stress in rivets

(Axial = $0.6 f_y = 0.6 \times 250$) = 150 MPa)

ii) For Gauge length / pitch length

a) shear strength of rivets

$P_{s1} = n \times \frac{\pi}{4} \times d^2 \times F_s$



n → total no. of rivets at joint in crossed gauge length

~~f_s → perm~~

b) Bearing :—

$P_{B1} = n_1 \times (t \times d) \times F_b$

c) tearing :—

$P_{t1} = (g - d) \times t \times F_t$

when pitch distance is given then $P_t = (p - d) t \times F_t$

g → gauge length.

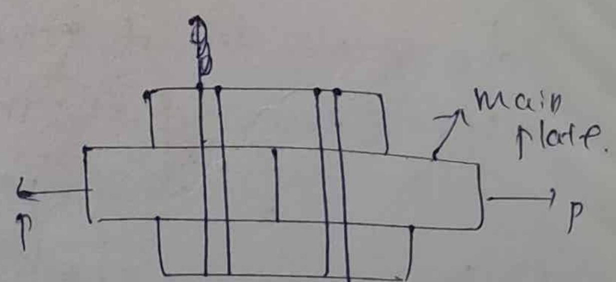
Double cover butt joint :—

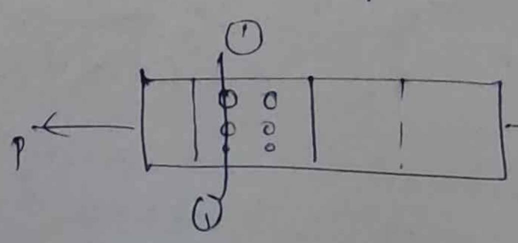
i) For entire width of plate

a) shear strength of rivets

$P_{s1} = 2 \times n_1 \times \frac{\pi}{4} \times d^2 \times F_s$

↳ double shear





b) Bearing strength of rivet

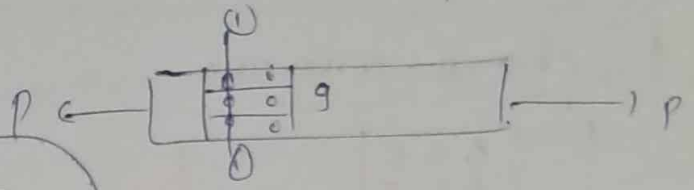
$P_B = n \times (t \times d) \times F_b$

↳ min of (thickness of thinner mainplate, sum of cover plate thickness)

c) Tearing strength ~~Here → n = total no. of rivets cut for~~

$$P_t = (B - n.d)\,t \times f_t$$

n → total no. of rivets at critical secn 1-1
t → min of (thickness of thinner main plate, sum of cover plate thickness)



ii) For Gauge length

a) shear:—

$$P_{s1} = 2 \times n \times \frac{\pi}{4} \times d^2 \times f_s$$

→ n = total no. of rivets at joint in crossed gauge length (here 2)

b) Bearing:—

$$P_{B1} = n_1 \times (t \times d) \times f_b$$

→ total no. of rivets at joint in crossed gauge length.

c) Tearing:—

$$P_{t1} = (g - n.d)\,t \times f_t$$

→ t: thickness of thinner main plate

n → total no. of rivets at in critical secn 1-1 in crossed gauge length (here 1)

* No. of rivets required at a joint, = $\frac{\text{total force at a joint}}{\text{Rivet value}}$

$$\Rightarrow n = \frac{F}{R_v}$$

* efficiency of joint, $\eta = \frac{\text{Least value of } P_s, P_b, P_t}{\text{Strength of solid plate}} \times 100$

P_s = shearing strength of joint
P_b = bearing " "
P_t = tearing " plate

* efficiency for entire plate:—
we have to ensure that P_t is less because rivet failure is more dangerous.

For entire plate:—

$$\eta = \frac{\text{Least value of } P_s, P_b, P_t}{\text{Strength of solid main plate}} \times 100$$

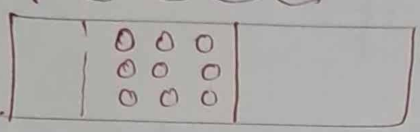
$$= \frac{(B - n.d) \times t \times f_t}{B \times t \times f_t} \times 100 = \frac{(B - n.d)}{B} \times 100$$

For Gauge length :—

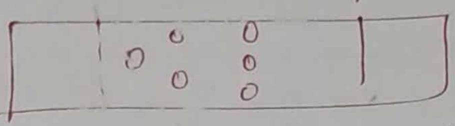
$$\eta = \frac{(g-d)\times t \times f_t}{g \times t \times f_t} \times 100 = \frac{g-d}{g} \times 100$$

Arrangement of rivets

1. chain riveting



2. Diamond

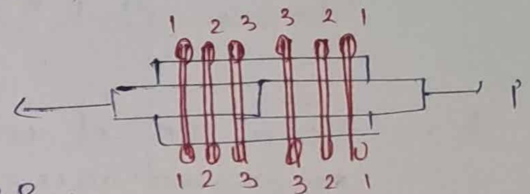


- In chain riveting the rivets are arranged as shown :—
- 1-1, 2-2 & 3-3 shows secⁿ on either side of the joint.

1-1 → critical secⁿ main plate → outer most secⁿ

3-3 → " " of cover " → Inner " "

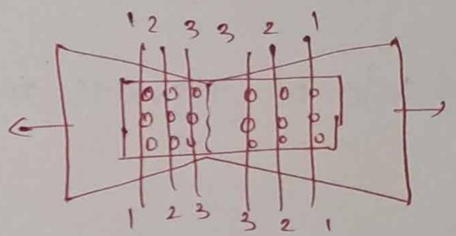
Strength of main plate :—



$P_{1-1} = (B-3d)\times t \times f_t$

$P_{2-2} = (B-3d)\times t \times f_t + 3R_v$

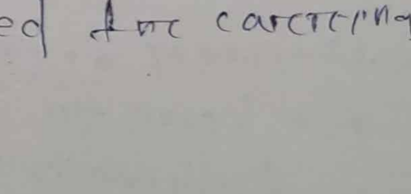
$P_{3-3} = (B-3d)\times t \times f_t + 6R_v$



Strength of cover plate :—

$P_{3-3} = (B-3d)\times t \times f_t$

$P_{2-2} = (B-2d)\times t \times f_t + 3R_v$



- In diamond pattern :—

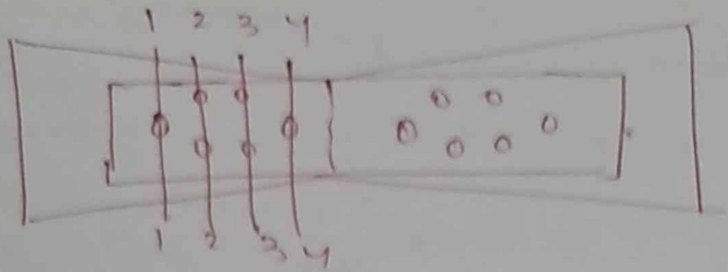
secⁿ 1-1, 2-2 & so on has to be checked for main plate in carrying a required load but for cover plate the last secⁿ is checked for carrying a required load.

Strength for main plate :—

$P_{1-1} = (B-d)\times t \times f_t$

$P_{2-2} = (B-2d)\times t \times f_t + R_v$

$P_{3-3} = (B-3d)\times t \times f_t + 3R_v$



- In triangular square pattern of riveting, sec^n 1-1 & 2-2 is checked for main plate in carrying a required load.
- For cover plate, 4-4, main plate acts as for cover plate is supposed to be checked for cover plate, also sec^n 3-3 & 2-2 is also checked for safety.

Strength for main plate :-

$P_{1-1} = (B-d) \times t \times f_t$

$P_{2-2} = (B-2d) \times t \times f_t + R_v$

$P_{3-3} = (B-2d) \times t \times f_t + 3R_v$

$P_{4-4} = (B-d) \times t \times f_t + 5R_v$

(provision is valid for machine cut elements)

Specifications As per IS 800-1984 :-

Min^m end & edge distance = 1.5 × gross dia. of rivet

- this recommendation is provided to prevent 3 types of failure in plates :-
  - i) splitting failure of plate
  - ii) shearing " "
  - iii) Bearing " "

→ For hand driven elements :-

min^m (edge distance & end distance) = 1.7 × gross dia. of rivet

→ But for analysis & dgn. purpose = 2 × gross dia. of rivet.

Pitch

- Minm pitch of rivet = 2.5 × ϕ → nominal dia of rivet
- Maxm pitch of rivet weld :— t → thickness of thinner plate

In compression

12t or 200 mm

(minm of two)

In tension

16t or 200 mm

(minm of two)

- The maxm pitch provision is provided to ensure the prevention of buckling betn the connection.
- The maxm pitch provision is provided to ensure the prevention of separation of plates betn connection.

Note

If the rivets are staggered (not in the same line) & of the gauge distance smaller than 75 mm, then above recommended values in comp. & tension zone for maxm pitch are increased by 50% i.e,

For comp.

maxm pitch = 18t or 300 mm (minm)

For tension

= 24t or 300 mm (minm)

* Gauge length (g) → ≯ 100 + 4t or 200 mm

maxm edge distance should not exceed 12tε

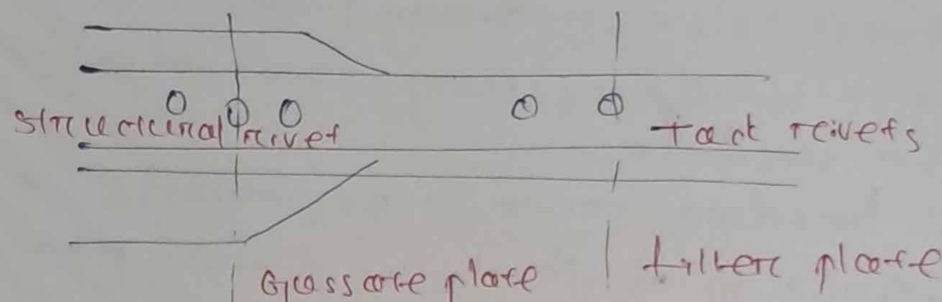
where, $\varepsilon = \sqrt{\frac{250}{f_y}}$

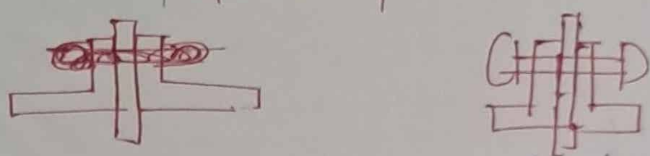
- When the member are exposed to corresponding maxm edge distance should not be greater than 40 + 4t.

Tack rivets

they are the rivets used to make the structural component as a single unit.

- they don't carry any load because we consider tack rivets not as a structural unit i.e provided as the
location of gusset plate.





- the max^m pitch provided in the case of tack rivet when two angle sec^{n}s are placed back to back to gusset plate as:—
  - 1000mm in case of tension
  - less than 600mm — comp.

  } Both valid for angle & channel sec^n

* When two plates are attached to a gusset plate back to back, then the max^m pitch is taken as
  - 32t or 300mm (min^m)

Unwin's formula

$$\phi = 6.04\sqrt{t}$$

t → thickness of thinner plate

- It is used when dia of rivet is not known.

Note

- For field rivet, the permissible stress is reduced by 10%
- the permissible stress in rivet under wind load cond^n as per IS800 can be ↑ by 25%
- " " under wind & earthquake load cond^n → ↑ by 25%

- When thickness of cover plate is not given, then the thickness of cover plate should not be ≮ $\frac{5}{8}$ of main (thinner) plate

Assumptions in dgn. of riveted joint

- the applied axial load is assumed to be sheared by all the rivets equally.
- the tensile stress (0.6 fy), shear stress (0.4 fy) & bearing stress at their respective centres are assumed to be uniform.
- the effect of bending stress is neglected.
- Grip length is the sum of thickness of two plates.
  - i) Grip length (l_g)
    $l_g \not> 5\phi$ → LSM
  - ii) $l_g \not> 8\phi$ → WSM
- the friction force betn the plates is neglected.
- $(g-d) t f_t \le n R_v$ (most imp. consideration).

1. steel is mainly an alloy of → Iron & carbon
2. Disadvantage of steel → Fire & corrosion resistance
3. elastic modulus of steel → 2×10^5 N/mm²
4. Unit mass of steel → 7850 kg/m³
5. poisson's ratio of steel → 0.3
6. structural steel normally has carbon content less than → 0.6 %
7. when Manganese is added to steel → improves strength & hardness of steel
8. property of high carbon steel → reduced ductility
9. minm % of chromium & nickel added to stainless steel → ~~0.5 %~~ 10.5 %, 0.5 %

(10) the effect of increase content of sulphur & phosphorous in steel → affects weldability.

(11) Correct criteria to be considered while designing:—
structure should be adequately safe, should have adequate serviceability.

(12) serviceability:— It means that the structure should performs satisfactorily under different loads without discomfort to user.

(13) Analysis → determination of axial forces, BM, S.F etc.

(14) Mainly adopted for dgn. for steel stc. as per IS code → limit state Method

(15) general construction of steel → IS 800

(16) permissible stress = $\frac{\text{yield stress}}{\text{FOS}}$.

(17) In WSM,
working stress ≤ permissible stress

(18) load factor = $\frac{\text{ultimate load}}{\text{working load}}$.

(19) the eff. length of a compression member of length 'L' held in position & restrained in direc at one end & effectively restrained in direc but not held in position at the other end is → L

(20) ~~load fac~~ ~~the ratio~~
In elastic limit, $\frac{\text{shearing stress}}{\text{shearing strain}}$ = shear modulus of elasticity

(21) A beam is defined as a structural member s.t → transverse loading

(22) A rivetted joint can fail in → { Tearing of plate only, shearing of rivet, bearing of rivet }

(23) ~~Gross dia~~ = 14 mm ≤ 25 mm.

$d = \phi + 1.5 = 14 + 1.5 = 15.5$

the gross dia. of a 14mm nominal dia rivet is 15.5 mm.

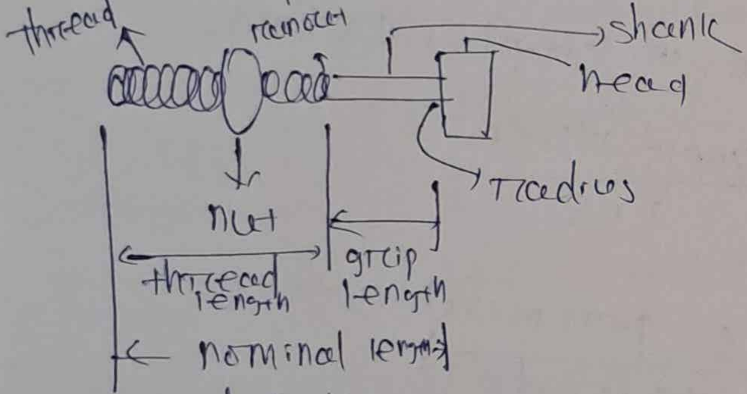
(24) the strength of field rivets as compared to shop rivets is → 90%.

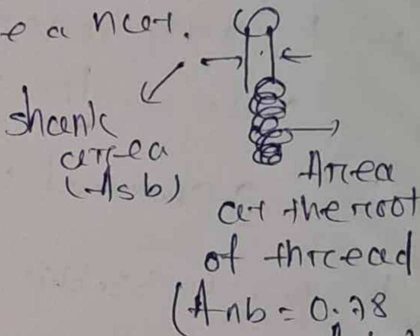
(25) the maxm c-to c distance betn rivets in a tension member of thickness 10 mm is → 160 mm

(26) minm pitch of rivets shall not be less than (where d is gross dia of rivet) → 2.5d

2. Bolted connections

- A bolt is a metal pin with a head at one end & a shank threaded at the other end to receive a nut.





Types of Bolts

(1) Black bolt / ordinary bolt / unfinished bolts:—

It is the least expensive bolts, used for light str. s.t. static loads & for secondary members such as purlins, bracing etc.

- they are not recommended for connections s.t. impact load, vibration & fatigue.
- the bolts are available from 5mm to 36mm in dia. & are designated as M5 to M36.
- Bolt of property class 4.6 means:—
  i) ultimate strength of bolt (f_{ub}) = 400 MPa
  ii) yield " " , (f_{yb}) = 0.6×400 = 240 MPa

(2) Turned bolts / close tolerance bolts:—

It has small tolerances & are used in no slip connection. they are mainly used machines & under dynamic loading cond's.

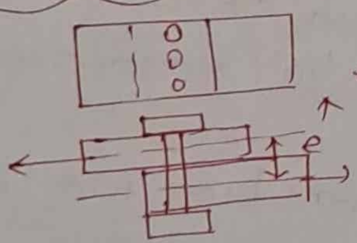
Types of bolted joints :-

1. Lap joint.
   1. single bolted lap joint
   2. Double " "
2. Butt joint.
   1. Double cover single bolted butt joint
   2. " " double " " "
   3. single " single " " "

3) High strength bolt

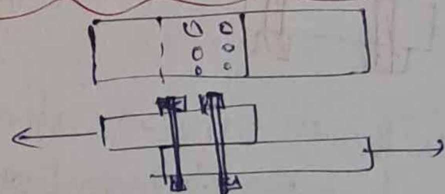
- they are available from 16 mm to 36 mm in diameter.
- the most commonly used bolts are of 8.8 S (or) 10.9 S property class. S → stands for high strength.
- this bolts may be tightened until they have many high tensile stresses so that the connected parts are tightly clamped together betn the bolt head & nut & friction developed betn the plate surfaces s.t. clamping force.
- the high strength bolts with specified initial tension are known as high strength friction grip (HSFG) bolt.

Single bolted lap joint

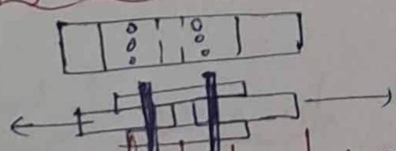


eccentricity of lap joint.

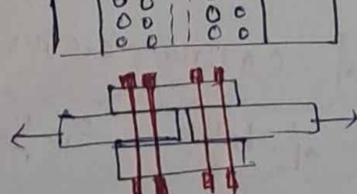
Double bolted lap joint



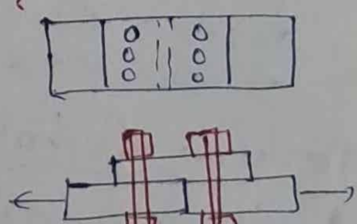
Double cover single bolted butt joint



Double cover double bolted butt joint



Single cover single bolted butt joint.



~~Types of bolted joints~~

- Double cover butt joint eliminates the eccentricity hence bending is eliminated.
- the load in the lap joint has eccentricity hence a couple is formed which causes undesirable bending in the connection & bolts may fail in tension.
- To minimize the effect of bending in lap joints atleast two bolts in a line must be provided.

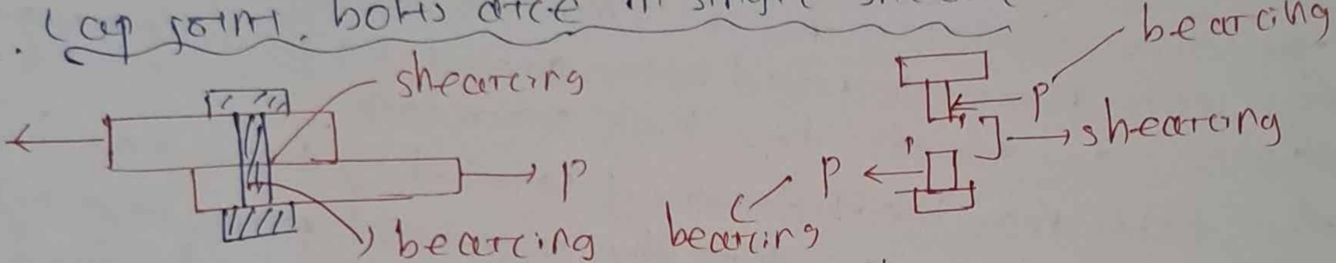
Load Transfer Mechanism

→ Load transfer from one connected part to another depends on the type of connection.

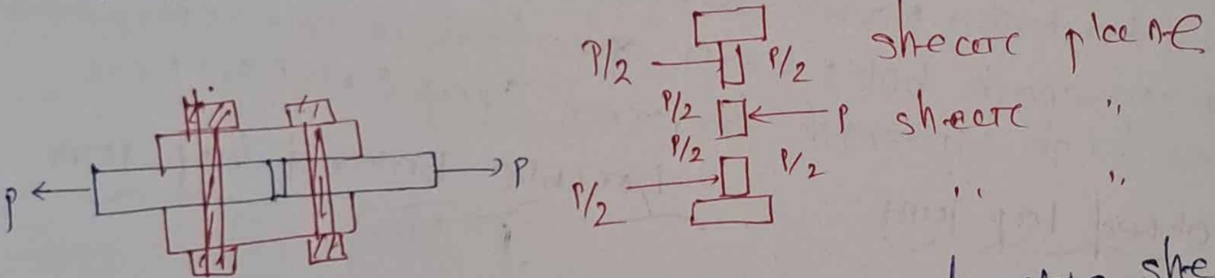
- In bearing type connection using ordinary bolts, the load transfer is by shearing & bearing.
- In slip critical / slip resistance connection using HSFG bolts, the load transfer by friction.

→ Transfer of forces in lap joint & butt joint

1. Lap joint, bolts are in single shear



2. Butt joint, bolts are in double shear :-

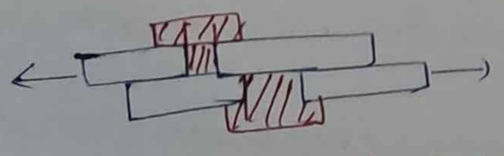


→ Bolts with single shear plane & double shear plane are called single shear bolt & double shear bolt respectively.

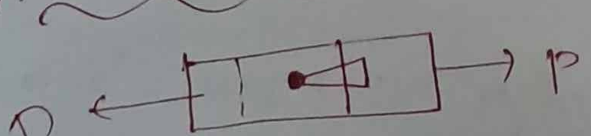
→ shear capacity of a bolt in a double cover butt joint is double that of a bolt in a lap joint because of two shear planes.

Failure of bolted connections

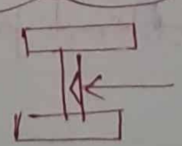
1. Shear failure of bolts



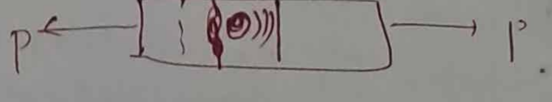
2. Shear failure of plate



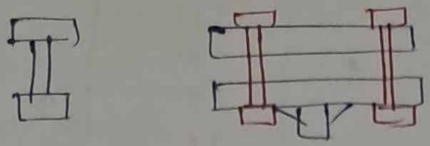
3. Bearing failure of bolts



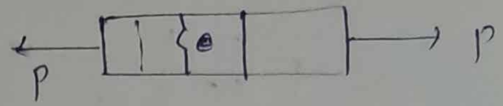
4. Bearing failure of plate



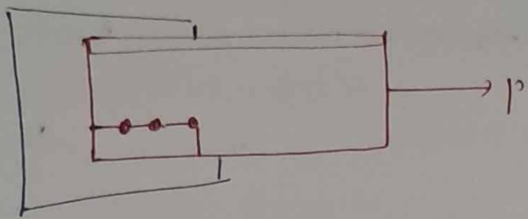
5. Tension failure of bolts

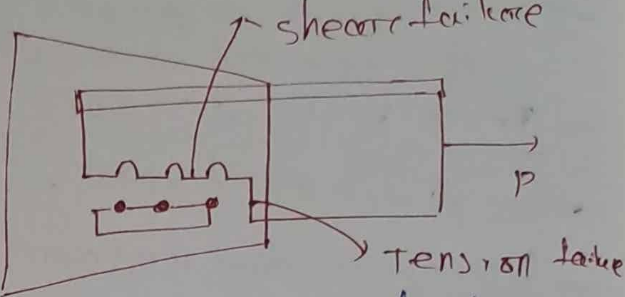


6. Tension failure of plate



7. Block shear failure





→ It is a limit state that combines tensile failure on one plane & shear failure on another plane.

→ Block shear failure may occur when material bearing strength & bolt shear strength is high.

Specifications of bolted connections

1. Dia. of bolt holes :→

→ Under normal situation, dia of bolt hole is made larger than the shank area to facilitate erection.

In mm

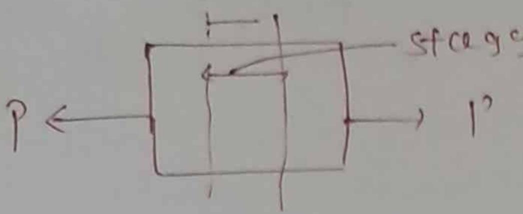
nominal dia of bolt	12mm	14mm	16mm	20 mm	24	30	36
Dia of hole	13	15	18	22	26	33	39
	+1 mm		+2 mm			+3 mm	

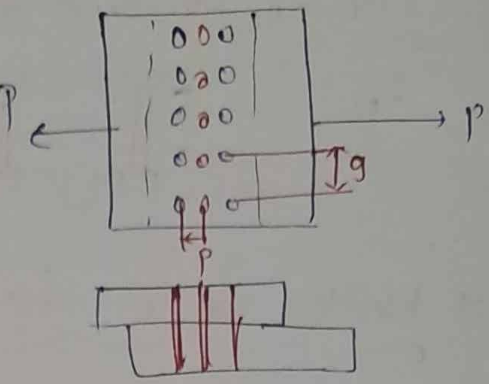
Q. In calculating the area to be deducted for bolts of 36mm dia. the dia. of hole shall be taken as →

36 mm → nominal dia. → $\phi > 24$mm

dia. of hole = $\phi + 3$ mm = 36 + 3 = 39mm

2. Pitch (p) & Gauge (g)





→ Minm pitch & gauge = 2.5 d
 d = nominal dia. of bolt

→ Pitch is the distance betn two consecutive bolts in the dirn of force.

→ Gauge is the distance " " " ⊥r to the dirn of force.

- Bolts are kept apart at a sufficient distance & a minm pitch is ensured due to:—
  - i) To prevent bearing failure of members betn the two bolts.
  - ii) To permit eff. installation of bolts.

maxm pitch

- It is desireable to place the bolts sufficiently close together for the following reasons:—
- To reduce the length of connection x gusset plate i.e, to have a compact joint.
- To have uniform stress in bolts.

1. For tension member → p ≯ 16t or 200mm
2. For comp. " → p ≯ 12t or 200mm
3. For rows near the edge → p ≯ 4t + 100mm or 200mm
4. Staggered pitch → When the gauge doesn't exeed 75 mm, the pitches in 1, 2, & 3. may be increased by 50%. when bolts are regularly staggered.
5. maxm gauge → g ≯ 32t or 300 mm

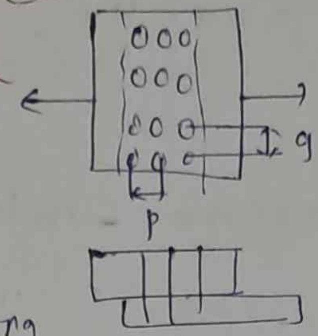
~~maxm pitch~~

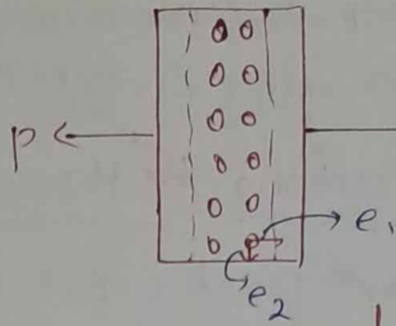
Note

For wide plates, pitch is defined as centre to centre distance of bolts measured along the length of the connection.

~~3. Edge and end distance~~

Pitch (p) & Gauge (g)





e_1 → edge distance
e_2 → end "

maxm pitch

It is assumed that the load on the joint is shared equally among all the bolts, in short length joints the force in bolts will be redistributed by plastic action & hence the bolt will share load equally.

- However in long joints (> 15 times dia. of bolt) the shear deformation is not uniform.
- the bolt at the ends of the joints are heavily stressed resulting in a progressive failure called 'unbuttoning'. thus a compacting joint is desirable.

(3) Edge & End distance

- Distance from the centre of bolt hole to the adjacent edge of the member at right angles to the dirn of stress is called edge distance.
- → Distance from the centre of bolt hole to the adjacent edge of the member in a dirn of stress is called end distance.
- Bolt holes should not be placed too near the edge due to:
  - i) the failure of plate in tension may be take place.
  - ii) the steel of the " opposite to the hole may be bulge & crack.

→ Minm end or edge distance = $1.7\,d_0$
(for sheared or hand flame cut edges).

→ minm end or edge distance = $1.5\,d_0$
(for machine cut & plained edges)
d_0 = dia. of bolt hole

→ maxm end or edge distance = $12\,t\,\varepsilon$

$\varepsilon = \sqrt{250/f_y}$, t = thickness of thinner plate

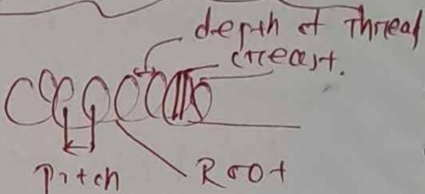
- When the members are exposed to corrosive environment, maxm edge distance ≯ 40 mm + 4t.

Tacking Bolts

- maxm pitch = 32t or 300 mm → when plates are not exposed to weather.
  = 16t or 200 mm → " " " exposed to weather

* For two members placed back to back, the maxm pitch of tacking bolts ≯ 1000 mm.

(154) **Assumptions in the analysis of simple Bolted joints**



- Friction betn the plates is neglected & load is resisted by bolts, in bearing & shearing.
- In case of bolts if threads occurs in the plane of shear, the area at the eff. area resisting shear is taken as the area at the root of thread.
- However if " don't occur in plane of shear, eff. area is the cross-secn area of the shank.
- The applied load is equally resisted by all the bolts.
- Distribution of stress on the portion of plate betn the bolt holes is uniform, i.e., stress conc. around the hole is neglected.
- This assumption is made for ease in calculation. However at the time of collapse this assumption would be actually valid.
  * The bending stress in the bolt is neglected.
  * " " " is assumed to be uniform over the nominal contact area betn the plate & bolts.

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Strength of bolt in bearing type connection

Dgn. strength of bolt = min { Dgn. shear strength of bolt ; " bearing " " " }

$V_{db} = \min \begin{cases} V_{dsb} \\ V_{dpb} \end{cases}$

$V_{dsb} = \dfrac{V_{nsb}}{\gamma_{mb}}$ → nominal shear strength of bolt
γ_{mb} → 1.25 → partial safety factor for material

$V_{dpb} = \dfrac{V_{npb}}{\gamma_{mb}}$ → nominal bearing strength of bolt

for safety,

$V_{sb} \le V_{db}$ (also called bolt value)

└→ Factored shear load in one bolt.

A) shear strength of bolt

$$V_{dsb} = \frac{f_{ub}}{\sqrt{3}\,\gamma_{mb}} (n_n A_{nb} + n_s A_{sb}) \beta_{lj} \beta_{lg} \beta_{pk}$$

where,

V_{dsb} → Dgn. shear strength of bolt

f_{ub} → ultimate " " "

γ_{mb} → 1.25

n_n → no. of shear planes with threads intercepting the shear plane

n_s → " " " " without " " " " "

A_{nb} → net tensile area of bolt (or) area at the root of thread = 0.78 A_{sb}

A_{sb} → nominal shank area of bolt.

$\beta_{lj}, \beta_{lg}, \beta_{pk}$ → Reduction factors taking into the effect of long joints, large grip length & packing plates.

i) For single shear case:-

$$V_{dsb} = \frac{f_{ub}}{\sqrt{3} \times 1.25}(1 \times A_{nb})$$

- Unless specified V_{dsb} would be calculated corresponding to shear plane intercepting the thread.

ii) For double shear case:-

$$V_{dsb} = \frac{f_{ub}}{\sqrt{3}\,\gamma_{mb}}(1 \times A_{nb} + 1 \times A_{sb})$$

- Under the assumption that one of the shear plane is intercepting the root of thread & other is intercepting the shank.

B) Bearing strength of bolt

$$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}}$$

$$= 2.5 \times k_b \times d \times t \times f_u$$

Where,

d = nominal dia. of bolt

t = summation of thickness of plate elements experiencing bearing in same dirn

2.5 is const. whose value corresponds to hole elongation about 6mm.

$$k_b = \min^m \text{ of } \left\{ \frac{e}{3d_0}, \frac{p}{3d_0} - 0.25, \frac{f_{ub}}{f_u} \times 1 \right.$$

e → end distance

p → pitch

d_0 → dia. of bolt hole

f_{ub} → ultimate strength of bolt

f_u → " " plate.

→ Bearing failure in the bolts is possible only if the bolts used are of very low grade & plate joint are of high grade which is not possible.

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Therefore the bearing strength of bolted connection is a fern of strength of connected plates & the arrangement of bolts rather than grade of bolts.

- The bearing strength is also a fcn of bolt hole.
- The " " is calculated where is for standard bolt holes. for oversized & short slotted holes it is 0.7 times the bearing strength & the slotted holes it is 0.5 times of the " ".

c) Tensile strength of bolt

$$T_{db} = \frac{T_{nb}}{\gamma_{mb}} = \frac{0.9 f_{ub} A_{nb}}{1.25}$$

→ net tensile area of bolt = $0.78 A_{sb}$

$$T_{db} \leq \frac{f_{yb} A_{sb}}{\gamma_{m0}}$$ (γ_{m0} → 1.1)

(16) Dgn. strength of plate

- The plate may fail by:—
  - i) shearing of plate:— It can be avoided by keeping sufficient edge distance.
  - ii) yielding of plate:— For gross secn yielding:— $T_{dg} = \frac{f_y A_g}{\gamma_{m0}}$ (A_g → Gross area of solid plate; γ_{m0} → 1.1)

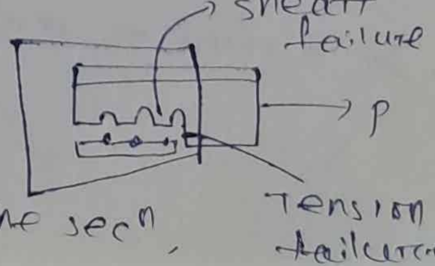
    T_{dg} → dgn. tensile strength in gross secn yielding
  - iii) Rupture of plate:— For net secn rupture:— $T_{dn} = \frac{0.9 f_u A_n}{\gamma_{m1}}$

    Where,

    T_{dn} = dgn. tensile strength in rupture.

    A_n = eff. net area of secn

    - factor 0.9 is introduced as there is no reserve strength beyond ultimate strength
  - iv) Block shear failure:—
    - When a block material within the bolted area breaks away from the remainder portion of the secn.



The failure is termed as block shear failure.

- This occurs along a path involving tension in one plane & shear on a ⊥ plane.
- It occurs, one of the surface (stronger one) fractures while the other yields.
- Yielding occurs on gross area while fracture on net area.

$T_{db} = \min \begin{cases} \text{shear yielding \& tension rupture} \\ \text{'' rupture \& '' yielding} \end{cases}$

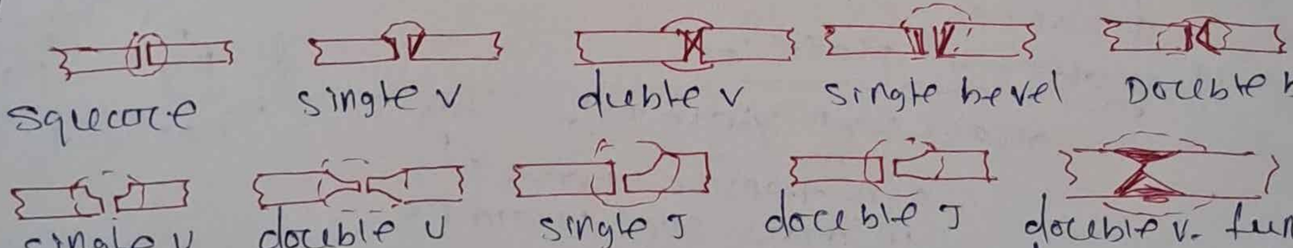
$= \dfrac{0.9 f_u A_{vn}}{\sqrt{3}\,\gamma_{m1}} + \dfrac{f_y A_{tg}}{\gamma_{m0}}$

$\dfrac{f_y A_{vg}}{\sqrt{3}\,\gamma_{m0}} + \dfrac{0.9 f_u A_{tn}}{\gamma_{m1}}$

- It occurs in joints made with high strength bolts, where few bolts are required for making the connection.
- When sufficient end distance is not provided, plates may shear out.

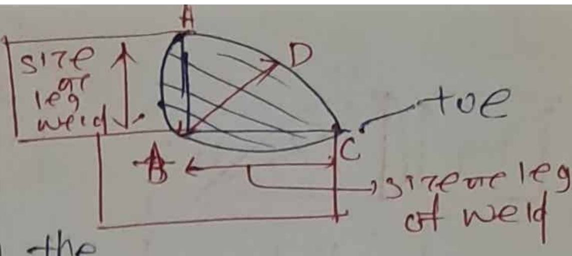
3. Welded connections :→

1) Butt weld



- This type of weld is used when the members are in same plane.
- It is also termed as groove weld.
- It is used to join structural members carrying direct comp. & tension.
- It is used to make tee- joint & butt- joint.

2. Fillet weld :—



- AB or BC → size of weld
- BD → throat thickness (t_t)

- This type of weld is used when the members to be connected overlap each other.
- A fillet weld is a weld approximately triangular cross section joining two surfaces approx. at right angles to each other in lap joint or tee joint.
- The part of weld which is assumed to be eff. in transferring the stress is called throat.
- It is assumed that fillet weld always offers resistance in the form of shear.
- The dgn. of only is done only for shear in fillet weld.

[t_t = minm dimension in fillet weld.]

- The eff. length of fillet weld should not be less than 4 x thickness of weld (s)

  $L_{eff} = 4 \times$ thickness of weld (s)

- The size of normal fillets shall be taken as the minm weld leg size.
- It should not be used if the angle betn fusion faces is less than 60° & greater than 120°.
- In weld, angle should be betn 60° to 120°.

[$t_t = K \times s$ (size of weld)]

Table-22 value of K for diff. angles betn fusion faces clause (10.5.3.2)

Angle betn fusion faces →	60°-90°	91°-100°	101°-106°	107°-113°	114°-120°
const., K →	0.70	0.65	0.60	0.55	0.50

Ex :— $t_t = (s) \sin 45^0$

$= 0.75$

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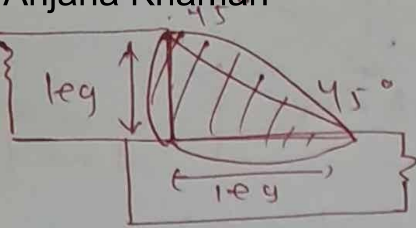
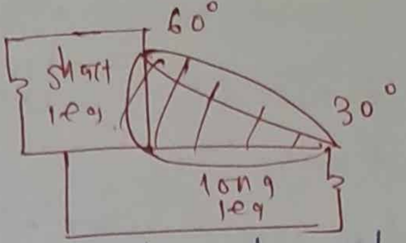
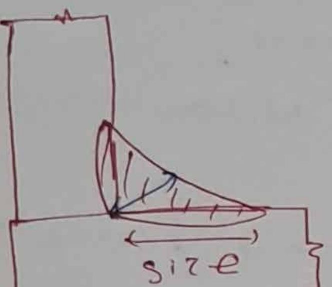




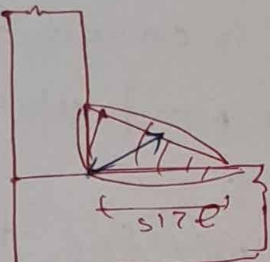
→ When the cross secn of fillet weld is 90°, isosceles Δ, it is known as a standard fillet weld. It is generally used.

→ When the cross secn of fillet weld is 30° & 60° Δ, it is known as special fillet weld.

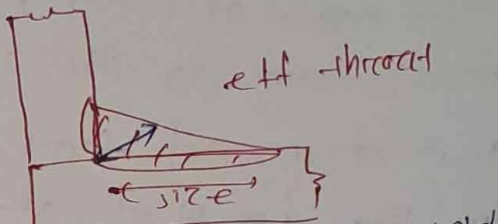
→



concave



convex

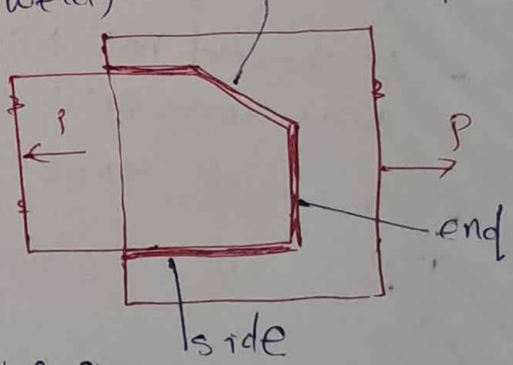


Mitre fillet weld

- A fillet weld is termed as concave fillet weld or convex " " or mitre depending on the weld face in concave or convex or approximately flat.
- It is termed as normal fillet weld or deep penetration fillet weld depending upon the depth of penetration beyond the root is less than 2.4 mm or more respectively.

i) side fillet weld (longitudinal fillet weld)

It is stressed in longitudinal shear, i.e. a fillet weld, the axis of which is parallel to the dirn of the applied loads.



ii) End fillet weld (transverse)

- It is a stressed in " shear i.e, a fillet weld, the axis of which is at right angle to the dirn of the applied load.

iii) Diagonal fillet weld

- It is a fillet weld the axis of which is inclined to the dirn of the applied load.
- It is placed on the sides or end of the base metal & it is s.t. shear along with tens. or comp. & usually bending.

IS recommendation

b. size of fillet

Minm size of the weld :-

- If the thickness of thicker part :-

  upto 10 mm → 3 mm

  betn 11 mm to 20 mm → 5 mm

  21 mm to 32 mm → 6 mm

  above 32 mm → 10 mm

  more than 50 mm → special precaution like preheating will have to be taken.

- when minm size of fillet weld > thickness of thinner part then minm size of weld = thickness of thinner part.

- It depends up the thickeness of thicker plate.
- If thickness is not given, then we assume, S = 3mm

thickness of thicker plate (mm)	minm size of weld (mm)
0 - 10	3
11 - 20	5
21 - 32	6
> 32	8 (if > 50, then 10mm)

maxm size of weld

- It is also depends upon ~~size~~ of thickness of thinner plate.

case-1

In square edge

maxm size = thickness of thinner plate - 1.5mm

case-2

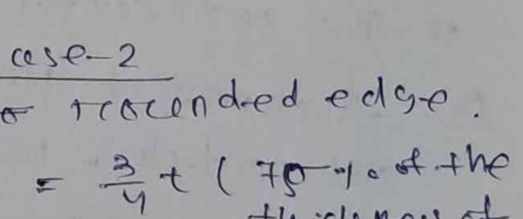
~~Atto~~ rounded edge.

$= \frac{3}{4}t$ (75 % of the thickness of thinner plate)

Effect length of weld

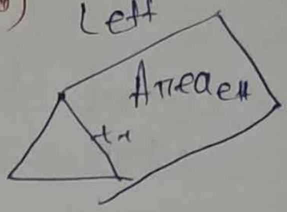
- It depends upon the size of weld.

eff length = Actual length - 2 × size of weld



- It should not be less than 4 times the size of weld.

Effective cross section Area of weld (throat area)



Area eff = $L_{eff} \times t_t$

Load carrying capacity of weld or shear strength of weld, $P = f_s \times L_{eff} \times t_t$

= permissible shear stress × eff. area of weld

t_t → throat thickness

L_{eff} → eff. length of weld.

f_s → permissible shear stress = 1000 MPa (WSM)

Fu → ultimate tensile stress in weld metal.

$LSM = \frac{f_u}{\sqrt{3} \times 1.25}$ (1.25 for shop weld) (1.5 field ")

max pitch, P = 12t or 200mm → comp. zone.

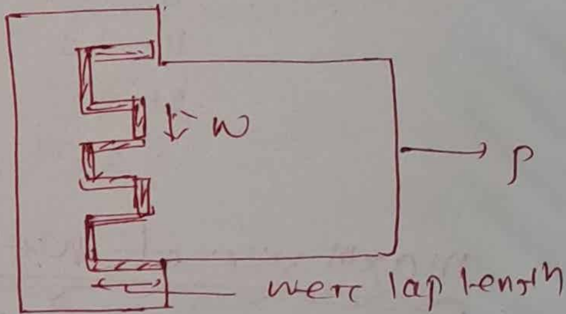
max pitch, P = 16t or 200mm → tens. "

Slot welding

W = width of slot

= max^m of (25mm × 3t)

- If overlap length is limited, then the slot welding is done by making slots in the connection plate.



Q thickness of plate, t = 8mm

slot width = W = 3t = 3 × 8 = 24mm

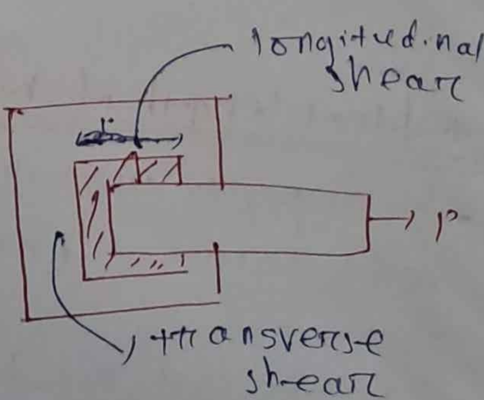
W = max^m (25m × 3t)

= max^m (25mm × 24mm) = 25mm (Ans)

longitudinal & transverse shear

longitudinal: load & length of weld are same dirn.

transverse: load & length of weld are ⊥r to each other



Note

Strength of transverse filled weld is about 30% more than longitudinal fillet weld.

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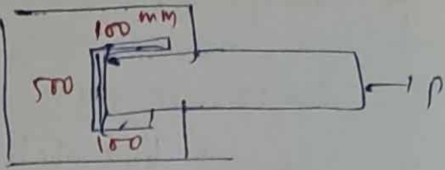
- A fillet welded joint of 6 mm size is shown.
- the welded surface meet at 60° to 90°.
- $F_s = 108$ MPa → permissible stress



$t_t = 0.7s = 0.7 \times 6 = 4.2$ mm

Safe load that can be transmitted by the joint,

$P = F_s \times t_t \times l_{eff}$

$= 108 \times 4.2 \times 250 = 113.4$ kN.

Compression Members

- Structural members s.t. axial compression or compressive force.
- their dgn. is governed by strength & buckling.
- most commonly used comp. member is column.
- other comp. members are strut, truss, frame, etc.

Column

- It is a structural member mainly s.t. comp.
- BM can also exist in this member.
- It is used for comp. of frame, i.e RCC & steel frame.

Strut

- It is a comp. member whose BM is zero because it is used in roof truss as a comp member.

Truss

- B.M = 0 → everywhere.
- It is a stu. in which all the members are either s.t. tension or comp.

Frame

- It is a stu. which is s.t. BM also in addition to tension & comp.

Stanchion

- The vertical comp. member in RCC building is called column while for a steel building it is called stanchion

Boom

- The principal comp. member in a frame is called boom.

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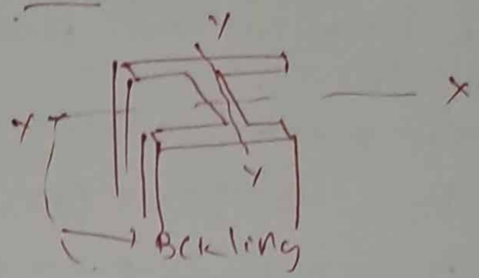
Methods of failure of column :—

1→ squashing
2→ local buckling
3→ Flexural "
4- torsional "
5- flexural torsional buckling



Squashing

When the length is relatively small & the component plate elements are prevented from local buckling, then the column will be able to attain its full strength i.e. squash load.

[squash load = yield stress × Area of cross-secⁿ]

Local buckling

- Failure occurs by buckling of one or more individual plate elements.
  - flange or web locally prior to overall buckling
  Ex!— flange or web of column.

Flexural buckling

- In this mode, failure of the member occurs by excessive deflection in the plane of weaker principal axis.
- In the fig., x-x & y-y axis are shown. Ixx > Iyy,
  so the resistance about y-y axis is less as compared to x-x axis. Hence buckling will occur about y-y axis.

Torsional buckling

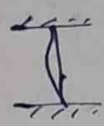
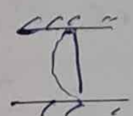
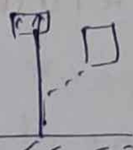
- this type of failure is caused by twisting about longitudinal axis of member.
- ~~this type~~ It can occur only in doubly symmetrical cross secⁿ with very slender cross sectional elements.

Flexural torsional buckling

- It is caused by combination of flexural & torsional buckling.
- the member bends & twists simultaneously.
- This type of failure occur only in unsymmetrical cross sections & singly symmetrical " " .

Effective length

- In SOM, we use theoretical value, & in dgn. we use IS recommended value.
- For laced columns, above values are ↑ by 5%.
- For battened " " " " " 10%.
- eff. length is slightly ~~the~~ larger than the theoretical value to account for the lack of 100% fixity at support.

Sl. No.	Degree of End Restraint of compression member	fig.	Theo. value of eff. length	Reco. value of eff. length
1.	effectively held in position & restrained against rotation in both ends.		0.50l	0.65l
2.	Held in position at both ends, against rotation in one end		0.70l	0.80l
3.	position in both ends but not restrained against rotation		1.00l	1.00l
4.	Held in position & restrained in one end, & the other " against rotation but not held in position.		1.00l	1.20l
5.	Held in position & restrained in one end, & at the other partially restrained against rotation but not held in position.		–	1.50l
6.	effectively held in position at one end but not restrained against rotation, & at the other end restrained against rotation but not held in position		2.00l	2.00l
7.	eff. held in position & restrained against rotation at one end but not held in position nor restrained against rotation at the other end.		2.00l	2.00l

Note

→ load capacity of a column depends upon the end condn's
strongest column is both end fixed.

- the most efficient cross secn is resisting compression is (thin hollow circular secn or '4 angle box sec') because for a given cross-secn area MOI is maxm so load carrying capacity is maxm.

- the most efficient cross secn in resisting BM is I-secn because for a given cross secn area, secn modulus & plastic modulus are maxm for I-sec.

- As per IS 800, the permissible axial compressive stress is given by Rankin's merchant formula:-

$$\sigma_{ac} = \min \text{ of } \begin{cases} 0.6 f_y \\ \dfrac{0.6 f_y}{\left[(F_{cc})^n + (f_y)^n\right]^{1/n}} \end{cases}$$

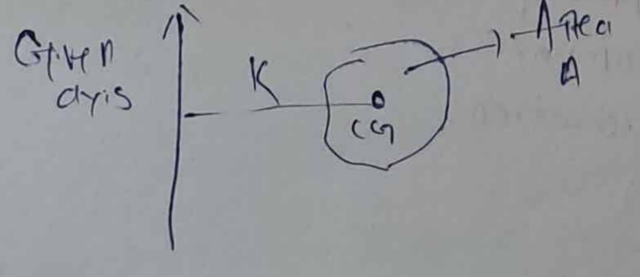
$$f_{cc} = \frac{\pi^2 E}{\lambda^2}$$

n = imperfection factor = 1.4

Unsupported length of column:-
It is the maxm clear distance betn bottom of the floor level & top of beam.

eff. length of column:-
It is length of column betn pts. of zero moment or distance betn pts of contraflexure.

Slenderness ratio $= \lambda = \dfrac{\text{eff. length}}{\text{radius of gyration}} = \dfrac{L_{eff}}{r_{min}}$



(radius of gyration) It is the distance such that its square multiplied by area gives moment of Inertia about the given axis.

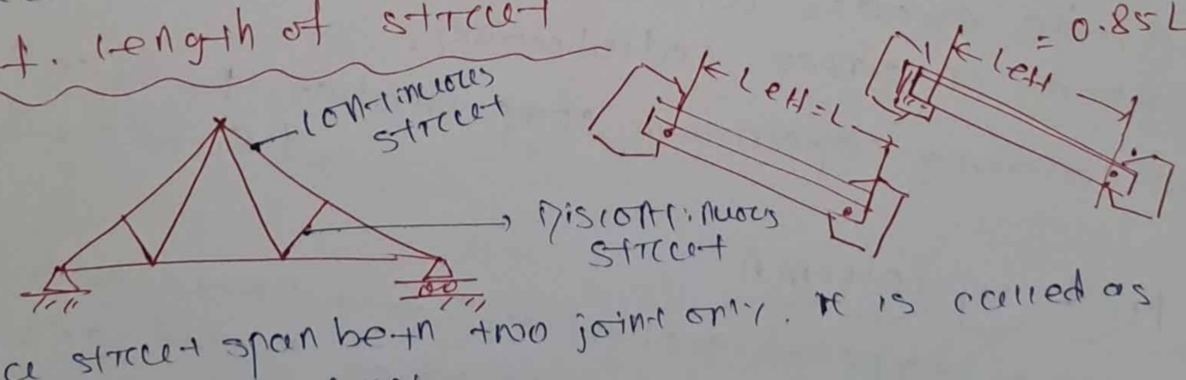
$r_{min}^2 \times A = I \Rightarrow r_{min} = \sqrt{\dfrac{I}{A}}$

Table → Maxm value of eff. slenderness ratios

Sl. no.	Member	Min^m eff. slenderness ratio (KL/r)
1.	A member carrying comp. loads resulting from dead loads & imposed loads	180
2.	A tension member in which a reversal of direct stress occurs due to loads other than wind & seismic load.	180
3.	A member s.t. comp. forces resulting only from combination with wind/earthquake actions, provided the deformation of such member doesn't adversely affect the stress in any part of the stru.	250
4.	comp. 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 eff. when s.t. possible reversal of stress into comp. resulting from the action of wind or earthquake force)	350
6.	Members always under tension (other than pre-tensioned members).	400

- Tension members, such as bracing's, pre-tensioned to avoid sag, need not satisfy the maxm slenderness ratio limits.

Eff. length of strut



- If a strut span betn two joint only. It is called as discontinuous strut.
- If a strut span ~~bet~~ over more than two joints then it is called continuous strut.

- If a single angle discontinuous strut is connected by one rivet at each end, the $l_{eff} = L$

axial comp. stress $\sigma'_{ac} = 0.8$

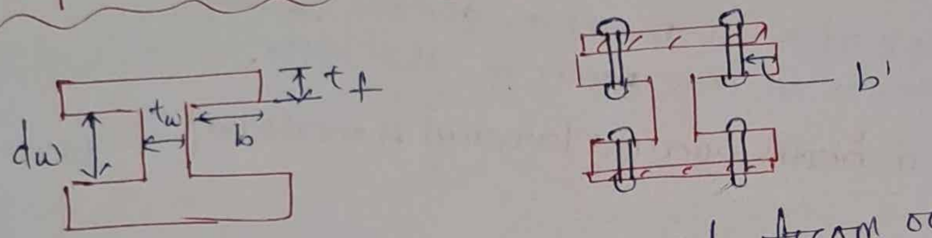
σ_{ac} → permissible stress is reduced by 20%.

- If a single angle discontinuous strut is connected by two or more rivets or weld, then $l_{eff} = 0.85l$.

$[\sigma'_{ac} = \sigma_{ac}]$

Local buckling of flange & Web plate of I-secn

b = width of flange from face of web.



- width of outstand is measured from outer line of rivet to extreme edge for built up secn.
- to prevent local buckling due to comp. as per IS 800: specify the following condn:–

$\frac{b}{t_f} \ngtr 16$ (WSM)

$\frac{d_w}{t_w} \ngtr 50$ (WSM)

$\frac{b}{t_f} \ngtr 8.4$ (LSM) → plastic secn

- If the flange & web plate dimensions exceed the above limits, the excess width should be neglected (not considering in area calculation).
- load carrying capacity in comp. member, $P = \sigma_{ac} \times A_g$

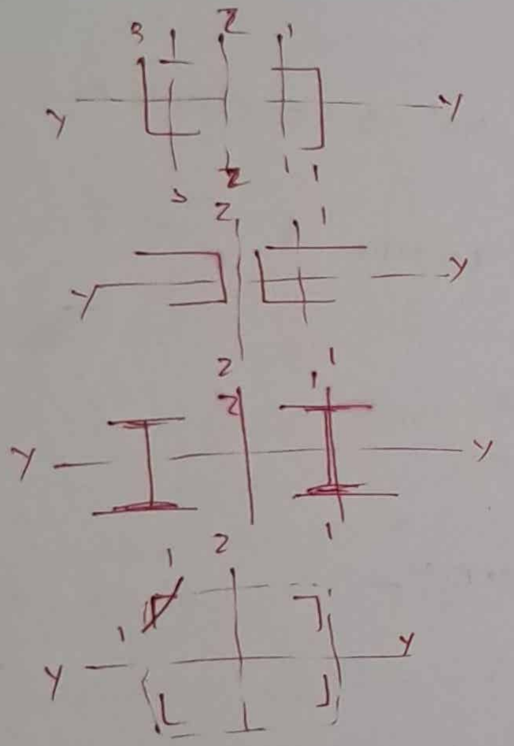
($\sigma_{ac} = 0.6 f_y$)

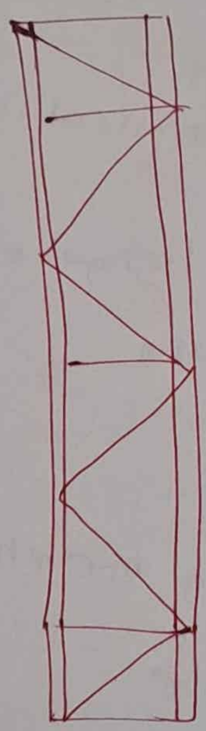
Built up column

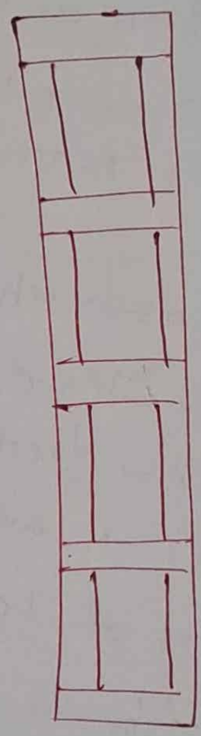
- the size & shape of rolled secn are limited because of limitations of rolling mills, so when rolled secns don't furnish the required sectional area or when a special shape or large radius of gyration is required in two different dirns, built up columns are used.

It is widely used in steel construction especially when the eff. lengths are greater & the comp. forces light.

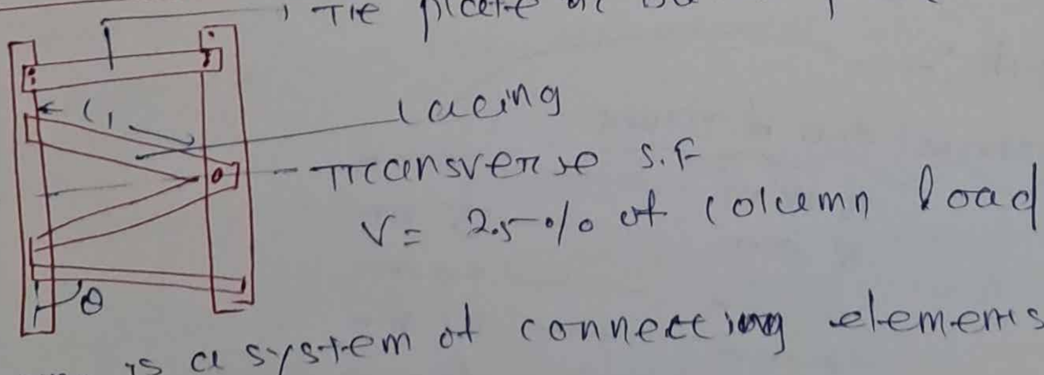
- They are composed of two or more parallel main components interconnected by lacing or batten plates.
- The greater the distance betn the chord axes, the greater is the moment of inertia of the built-up cross secn, the ↑ in stiffness, however is counterbalanced by the ↑ wt. & cost of the connection of members.
- It should be noted that built-up columns (especially battened built-up columns) are more flexible than solid columns with the same MOI.







Lacings



- Lacing is a system of connecting elements in built up column.
- It make the component of column act as a single unit.

- If the component of column are each very close to each other, then thick rivets are used to make them act as a single unit.
- If the spacing of component is more than the rivets are useless & so we use lacing or batten.

Inclined member – lacing

Horizontal " – Batten

- It is idealised as truss element, i.e., the are st. either to tension or comp.
- B.M = 0 → in lacing.
  to ensure that B.M is 0, provide only one rivet at each end as far as possible.
- $max^m \lambda = 145$ → in lacing
- Angle of lacing w.r.t vertical is 40° to 70° (welding 60° to 90°).
- If θ decreases then length of lacing ↑.
- $l_{eff} = L_1$ → single lacing
  $= 0.7 L_1$ → double "
  $= 0.7 L_1$ → welded "
- Minm thickness of lacing member :–

$$t_{min} = \frac{l_1}{40} \rightarrow \text{single}$$

$$= \frac{l_1}{60} \rightarrow \text{double.}$$

- Minm width :–

nominal dia of rivet	
16 mm	50 mm
18 mm	55 mm
20 mm	60 mm
22 mm	65 mm

- It depends upon the nominal dia. of rivet.
- In case of welding, width of lacing bar is 50mm

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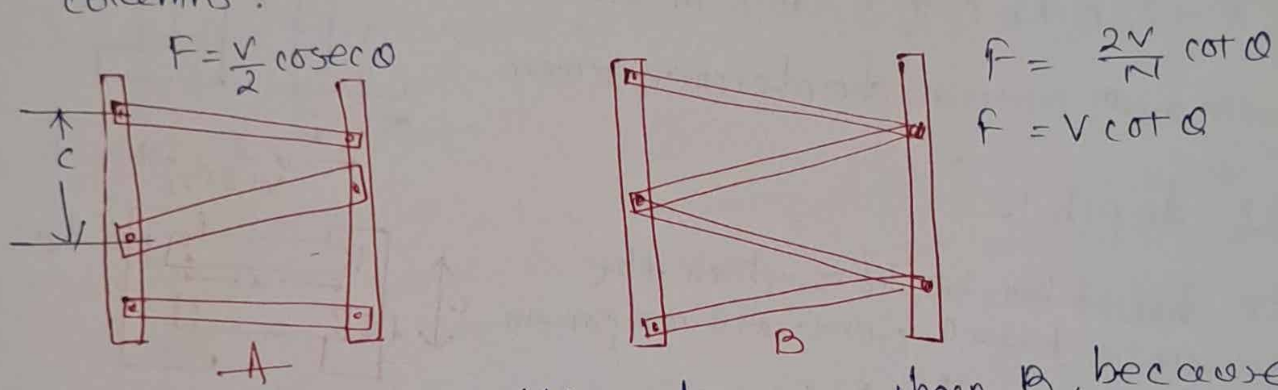
* To prevent buckling of column component b/w lacing connection:—

$\lambda_{component} \ngtr 50$
$\ngtr 0.7\,\lambda_{column}$

For tack rivets:—

$\lambda_{component} \ngtr 40$
$\ngtr 0.6\,\lambda_{column}$

- At the end of lacing system, at top & bottom, tie plates are provided (tie plate is called batten plate).
- these batten plate prevent distortion of built up columns.



- Arrangement 'A' is better than B, because if one rivet fails, spacing of lacing member doesn't change in A while in B, spacing will be doubled. Hence there will be possibility of buckling of connection in B.

Forces in lacing member:—

- lacing system is designed to resist a transverse SF of $V = 2.5\%$ of column load.
- the transverse S.F V is shared by lacing system both side equally, so the transverse S.F on each lacing bar is $\frac{V}{2}$.

* 2 denotes no. of parallel planes.
* $F = \frac{V}{2\sin\theta}$ — single — two parallel force system force in each lacing bar
* $F = \frac{V}{4\sin\theta}$ — double

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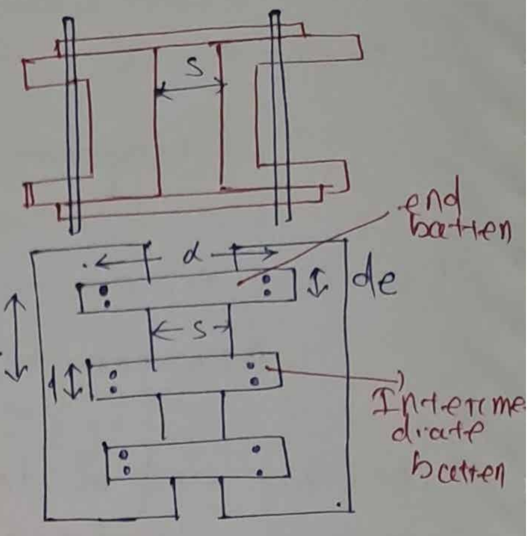
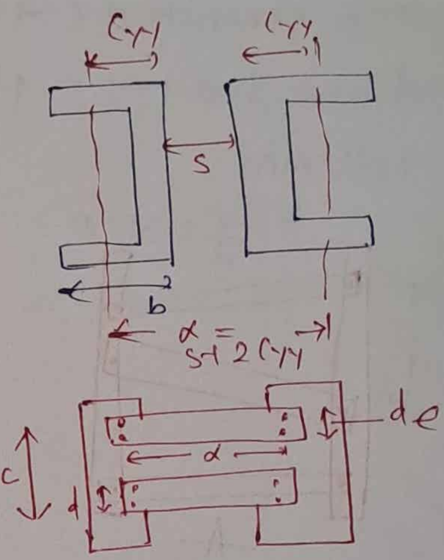
Battens



- It behave like very small beam member & S.T. BM.
- the eff. length of battened column should be ↑ by 10%.
- min^m no. of battens provided = 4
- Provided batten on opposite faces such that one should be the mirror image of other.



- eff. slenderness ratio,

  $\lambda = 1.10\,\lambda_0$ (↑ by 10% in battens)

  λ_0 → max^m actual slenderness ratio

- eff. depth :—
  - It should not be less than the distance betn centroid component members. $d_e \nless \alpha$
  - It should not be less than twice the width of one component in plane of batten (b).

    $d_e \nless 2b$.

- thickness of batten (t) :— $t \geq \frac{\alpha}{50}$
- to prevent local buckling of individual component betn the battens, following cond^ns are satisfied :—
  - $\frac{c}{r_{yy}} < 50$
  - $\frac{c}{r_{yy}} < 0.7\,\lambda_{column}$
- For Intermediate batten,

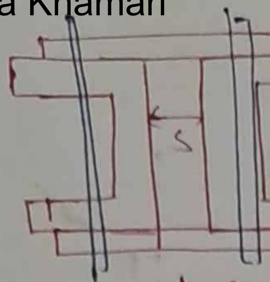
  $d_e > \frac{3}{4}\alpha$

  $d_e > 2b$

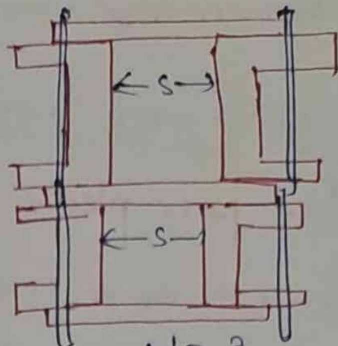
- For end batten,

  $d_e > \alpha$

  $d_e > 2b$



N = 2

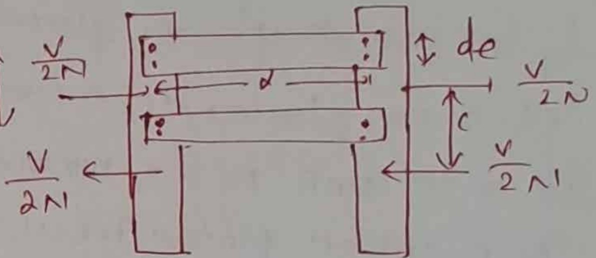


N = 3

→ Transverse S.F V is shared by parallel planes (N) equally
i.e. transverse S.F each batten = $\frac{V}{N}$

N = no. of parallel planes

- Batten should be designed to carry BM & SF arising from the transverse S.F. V, which is 2.5% of total axial load on comp. member.



- The transverse S.F is equally divided in all the parallel planes N in which there are shear resisting elements such as battens or continuous plates battens.
- Battens should be able to resist the longitudinal shear & moment arising from transverse shear V.

Longitudinal shear $(V_1) = \frac{Vc}{Nd}$

V → transverse S.F → 2.5% of P
N → no. of parallel plates of battens.

Longitudinal moment $(M) = \frac{Vc}{2N}$

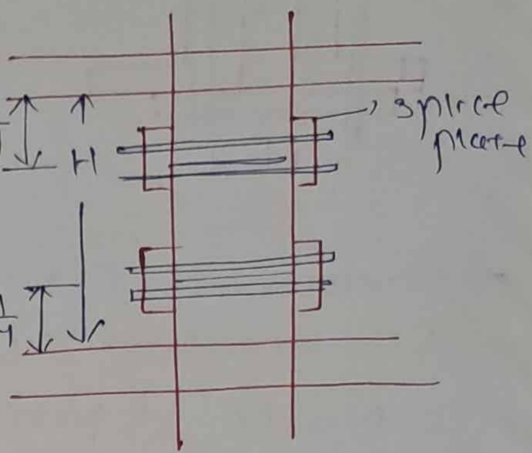
Column ~~spacing~~ splicing

- Splicing of column is done to ↑ the length of column.
- The most suitable location for column splicing is at a suitable location of $\frac{H}{3}$ to $\frac{H}{4}$ from the top & bottom level floor.
- When the column end are machined then it is assumed that 50% of load is transferred by direct bearing action & remaining 50% of the load is transferred through splice & its connection.

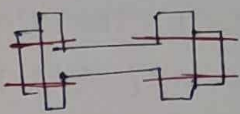
P → $\frac{P}{2}$ → Bearing action

→ $\frac{P}{2}$ → splice end connection

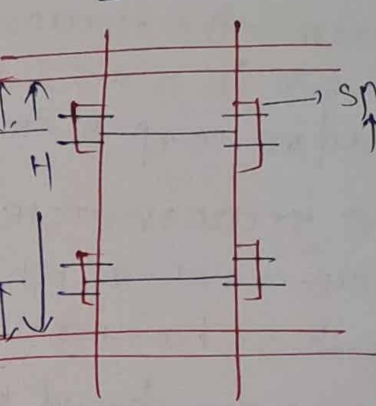
$\frac{P}{4}$ → each splice per plate



- If a column is s.t. to a comp. load P, the $\frac{P}{2}$ is transferred by bearing action & remaining $\frac{P}{2}$ is taken by splicing plate, $\frac{P}{4}$ by each splicing plate (since 2 splicing plates are used).

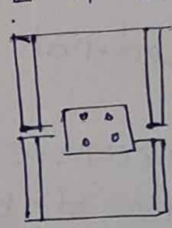


- If a column is s.t moment M also, then splice plate must also resist additional force of $\frac{M}{H}$, so maxm force in splice plate is maxm load for each splice plate = $\frac{P}{4} + \frac{M}{H}$



<u>Note</u>

If the column ends are not smooth, for complete bearing, entire load is assumed to be transferred to the bottom column through splice plate & connection only.



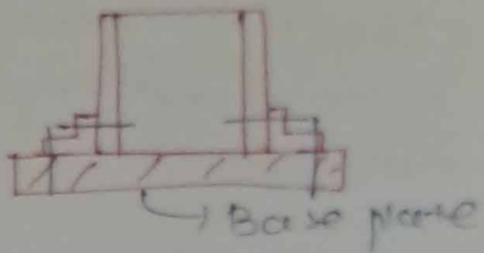
- Hence maxm force in each splice plate when column ends are not smooth.

$\frac{P}{2}$ → Each splice per plate

maxm load for " " " " $= \frac{P}{2} + \frac{M}{H}$

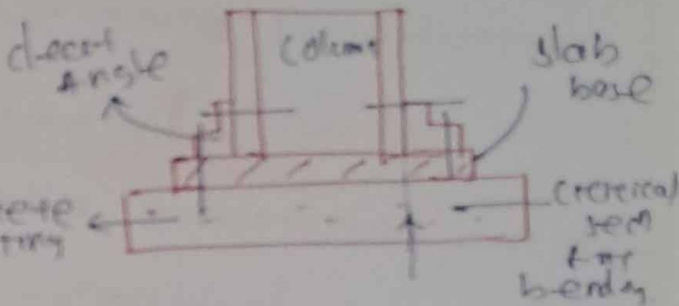
- If s.f is also acting at a column splice, a <u>web splice</u> must be done, both side of the web as shown.
- the rivet in web splice are s.t. double shear & bearing
- the rivet in flange splice are s.t. single shear & bearing.

Column base

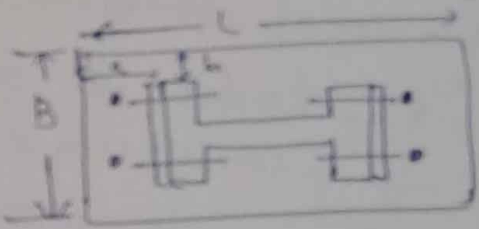


- It is a base plate used to reduce the bearing pr. on the concrete footing.
- It transfers the load to the concrete footing, preventing the punching shear of footing.
- If the column load is less, slab base is used.
- If the column load is more/heavy, then gusseted base is used.
- If the soil is weak, grillage foundn is used.

Slab base



- Permissible tensile stress of bolt used in column base is 120 MPa.
- In slab base, base plate & cleat angle are used.
- If column load is axial, then thickness of base plate is given by —



$$t = \sqrt{\frac{3w}{\sigma_{bs}}\left(a^2 - \frac{b^2}{4}\right)} \rightarrow WSM$$

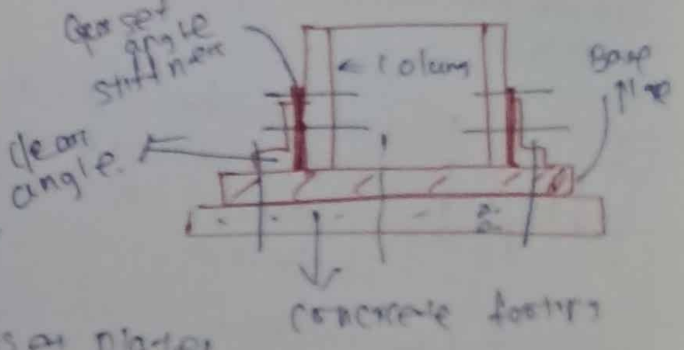
$$t = \sqrt{\frac{2.5w}{\sigma_{bs}}\left(a^2 - 0.3\, b^2\right)} \rightarrow LSM$$

w → upward pr. on base plate in N/mm2

a, b → greater & smaller projection of base plate beyond column edge.

σ_{bs} → permissible bearing comp. stress in base plate.

Gusset Base



→ Adopted when the load is large.
- Axial load accompanied by BM.
- Having eccentric loadings.
- Area can be ↑ by adding gusset plates.

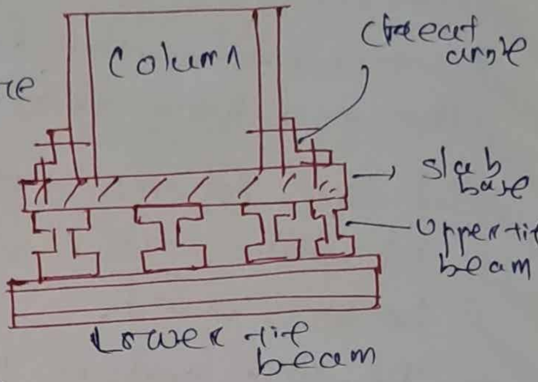
 ...rce transferred 50%. by fasteners.

- critical secⁿ for BM is at toe of Gusset plate.

core / kernel :– It is the small portion at cross section within which load is applied, tension will not be developed.

Grillage foundⁿ

- Adopted when loads on columns are extremely heavy.
- the bearing capacity of soil for the area of gusset / slab is not enough.
- Distribution area is very large.



Beams

→ It is a structural member s.t. transverse load.

Flexural formula :– $\frac{M}{I} = \frac{\sigma}{y} = \frac{E}{R}$

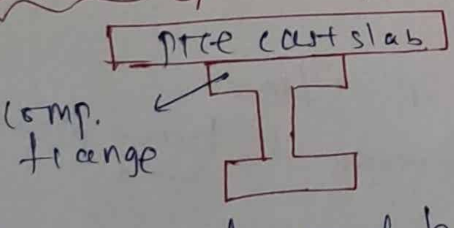
R → Radius of curvature

$\frac{1}{R}$ → curvature.

Moment is directly proportional to curvature.

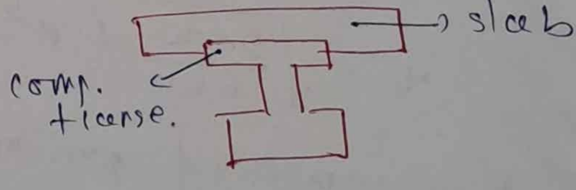
$M \propto \frac{1}{R}$.

Laterally unsupported / unrestrained beam (L.U.B)

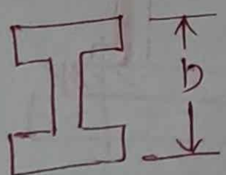


- If the comp. flange of beam is not restrained against lateral moment, then it is called as L.U.B or .

Laterally supported / restrained beam



- If the comp. flange of beam is restrained against lateral moment, then it is called as laterally supported beam or laterally restrained beam.



→ In the dgn. of beam we assume that the depth of web is resisting shear is taken as overall depth of the beam.

$\left[\tau_{va} \text{ calculate} = \frac{V}{D t_w} \right]$

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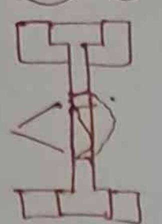
Web buckling

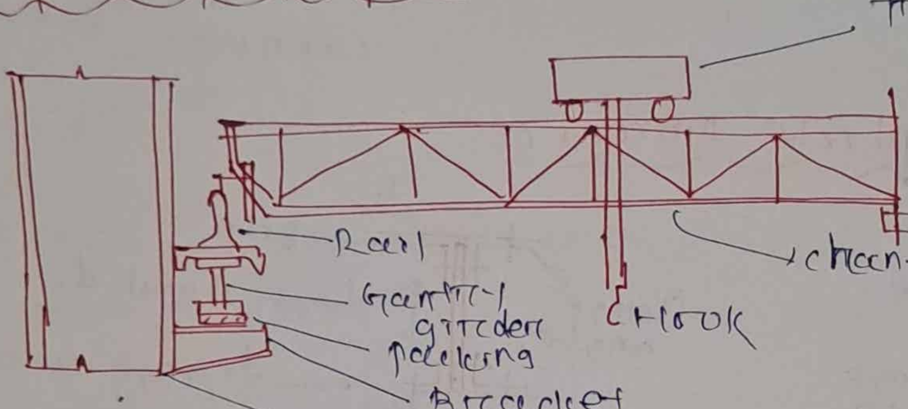


- It occurs due to diagonal comp. in web / concentrated load which is produced due to s.f in web.
- It occurs betn toe of fillet weld.

Web crippling / local buckling of web

- Web near the portion of the stress conc. tends to fold over the flange is called web crippling.
- Developed due to large amount of bearing stress.
- Stress conc. occurs at the junction of web & flange.

Gantry Girder



It is s.t :—
i) Gravity loads (i.e DL + L.L + wt. if crane truss + wt. of trolly, etc).

ii) Lateral load (due to starting or stopping of crab)
iii) longitudional " (" " " " " of crane)
- these three loads are mutually ⊥r to each other.
- Horizontal & vertical loads are applied simultaneously.
- Horizontal & vertical loads are ↑ by 10%. allowable stress in gantry girder are ↑ by 10%.

- Additional load due to impact.

Impact allowance

- Type of loading

Vertical load

1. E.O.C ——→ 25% maxm static wheel load
2. M.O.C ——→ 10% " " " "

Horizontal force transverse to rail

1. E.O.C ——→ 10% wt. of trolley & wt. lifted
2. M.O.C ——→ 5% " " " "

Horizontal force along the rail ——→ 5% of static wheel load.

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Permissible stress in Gantry Girder

- these are laterally unsupported beams to carry heavy loads from place to place at the construction sites.

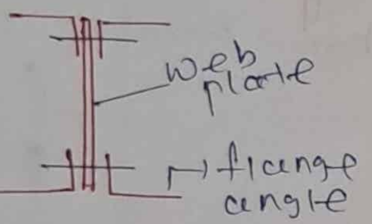
maxm permissible defn

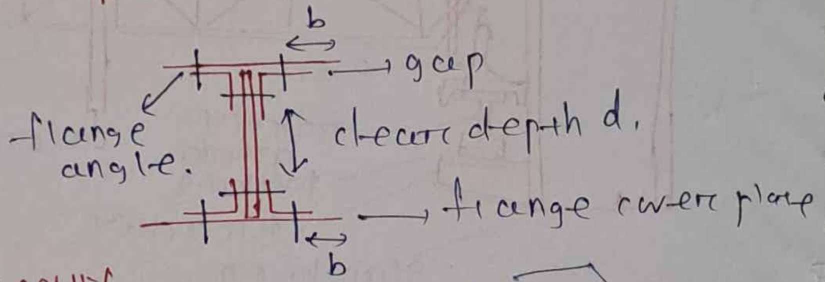
1. For $\delta = \frac{span}{500}$ → for manually operated crane.

2. $\delta = \frac{span}{750}$ → electrically operated crane capacity upto 50T or 500kN

3. $\delta = \frac{span}{1000}$ → " " more than capacity 50T or 500kN

plate Girder





- A plate Girder is usually thought as flexural member whose cross secn is composed of plate elements, flange plate, angle & web equivalent.

longitudional stiffness

transverse stiffness

- If both ap beam can't withstand applied load then plate girder are used.

- It consists of flange plate, flange angle & web plate.

Comp. flange

- It consists of flange plate, flange angle & web equivalent.
- Web equivalent is the web area embedded betn two flange angle.
- In comp. zone flange, web equivalent is taken as $\frac{area\ of\ web}{6}$ or $\frac{a_w}{6}$.

Tension flange

- It consists of flange plate, angle & web equivalent.
- In tension zone, web equivalent is taken as $\frac{a_w}{8}$.

- Plate girder is assumed that entire S.F is taken by web plate & BM is taken by flange (to ensure that web takes only S.F, gap of 5mm will be maintained betn flange plate & web plate so that direct bearing action betn flange plate & web plate is avoided).
- The load is transferred from flange plate to web plate through flange angles only.
- Width of outstand in comp. flanges → $b \ngtr 16 t_f$, $b \ngtr \frac{256 t_f}{\sqrt{f_y}}$
- " " " tension " → $b \ngtr 20 t_f$
- Economical depth of web plate (which is corresponding to minm wt. being minm cost).

$$d = 1.1\sqrt{\frac{M}{6_{bc} \times t_w}}$$

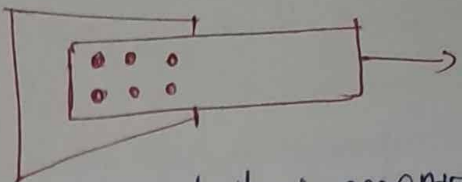
- Self wt. of plate girder is assumed as:-

$$w = \frac{W}{300} \text{ kN/m} \rightarrow \text{WSM}$$

$$= \frac{1.5W}{300} \text{ kN/m} \rightarrow \text{LSM} = \frac{W}{200} \text{ kN/m}$$

- If $\frac{d_1}{t_w} < 85$, web buckling due to shear will not happen, so stiffeners are not provided. The web will be unstiffened.
- If $\frac{d_1}{t_w} > 85$, vertical " " provided to prevent buckling of web due to diagonal comp. which is developed due to S.F.
- If $\frac{d_1}{t_w} > 200$, Horizontal stiffeners are provided above NA as they prevent buckling web due to bending comp. stress.
- If $\frac{d_1}{t_w} > 250$, then additional horizontal stiffeners are provided at NA
- If $\frac{d_1}{t_w} > 400$, then secn must be redesigned.
- After providing all the stiffeners, lesser clear dimension of web panel should not exceed $180 t_w$.
- Greater clear dimension of web panel $\ngtr 270 t_w$.

Tension Members (Tie)



- A structural member s.t. axial tension is called tension member or tie.
- The members & connections are so arranged that eccentricity in the connection & bending stress are not developed.

Types of failure

1. **Gross secn yielding:—** Considerable deformation of the member in longitudinal dirn may take place before it fractures, making the stc. unserviceable. Hence we must also consider yielding on gross-secn.

2. **Net secn rupture:—** The fracture of the member occurs when the net cross-secn of the member reaches ultimate stress.

3. **Block shear failure:—** A segment of block of material at the end of member shears out due to the possible use of high bearing strength of steel & high strength bolts resulting in smaller connection length.

Dgn. strength of TM:—

1. **Gross secn yielding:—**

$$T_{dg} = \frac{f_y}{\gamma_{m0}} \cdot A_g$$

T_{dg} → dgn. strength.
A_g → Gross sectional area
γ_{m0} → 1.1 → partial safety factor

- When a TM is s.t. tensile force although the net cross-secn yields 1st, the deformation with in the length of connection will be smaller than the deformation in the remainder of TM.
- It is because the net action exist with in the small length of member. Most of the length of member will have un-reduced cross-secn, so attainment of yield stress on gross area will result in larger total elongation.

2. **Net secn rupture (fracture):—**

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

T_{dn} → dgn. strength in rupture.
f_u → ultimate strength of material
A_n → eff. net area of c/s
γ_{m1} → 1.25

Block shear :-

- For plates :-
- For shear yield & tension fracture :-

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

- For tension yield & shear fracture :-

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

Where,

A_{vg} → minm gross area in shear along the line of force

A_{vn} → " net " " " " " " "

A_{tg} → " gross " in tension

A_{tn} → " net " " "

Slenderness ratio in TM :→

- It is the ratio of its unsupported length 'l' to its least radius of gyration.
- Maxm slenderness ratio for tension member is given as:-

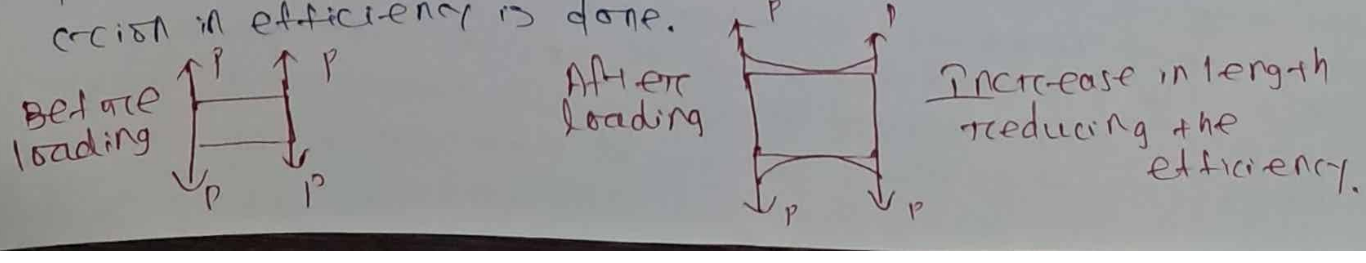
* A TM in which reversal of direct stress due to loads other than wind or seismic force occur. → 180

* A member normally acting as a tie in roof truss or a bracing system s.t. possible reversal of stresses resulting from the action of wind & earthquake forces. → 350

* Members always under tension (other than pretensioned members) → 400

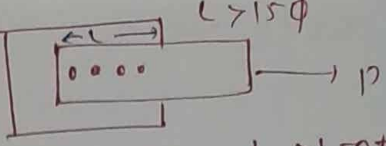
SHEAR LEG

- Non uniform straining of member due to tension is called shear leg.
- It reduce the efficiency of TM that are not connected to gusset plate.
- For reducing shear leg, lengthening of connections & reduction in efficiency is done.

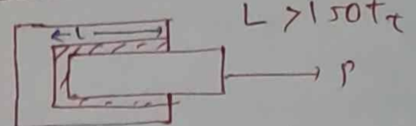


Long Joint (LJ)

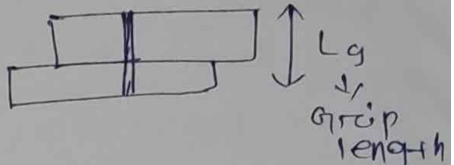
L = overlapping length.



If L > 15φ or L > 150t_{r} then it is called long joint.



- It is assumed that applied load is shared by all the rivets, but in LJ, outer rivet takes more load than the inner " so failure of rivets in long joints is sequential, beginning with those at the ends & progressing towards centre, this type of failure is termed as 'unbuttoning'.



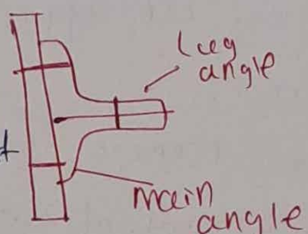
- If length of joint is more then efficiency of the TM would be less.

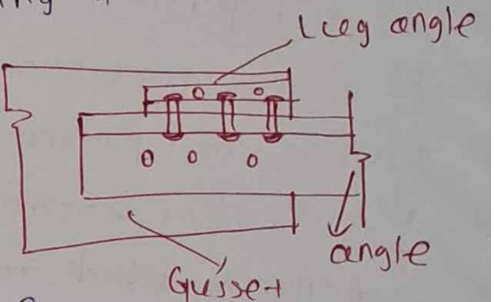
  $L_g = 5\phi$ (LSM)

  $L_g = 8\phi$ (WSM)

- If L_g increases then efficiency of joints decreases due to additional bending stress developing in rivets.

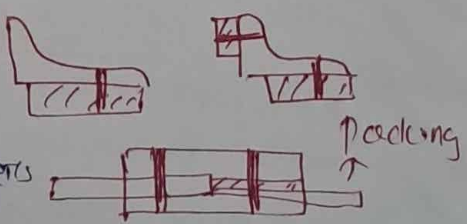
LUG ANGLES





- It is small piece of angle used to connect the outstand leg of the structural member to the gusset plate.
- The purpose of lug angle is to reduce the length of connection to gusset plate & reduce the shear leg effect.
  * shear leg effect is reduced by increasing the length of connection & by providing lug angles.
  the efficiency of TM increases.
- If lug angle are used, the efficiency of shear leg effect is decreased
- If length is increased, then shear leg is reduced. if lug angle is used then but efficiency is also decreased.
  efficiency is increased & shear leg is reduced.

SPLICES



- They are used to joint two sec^n when a joint is to be provided, i.e, those place the members at the joint where it is cut.
- When a splice occurs in an angle, channel, tee or joint sec^n, the force is received from the member through the connection on one side of joint & is transferred to the splice cover plate.
- The force is then carried through these covers across the joint & is transferred to other portion of member through the connections.

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Load carrying capacity of tension member

P_t → safe load carrying capacity

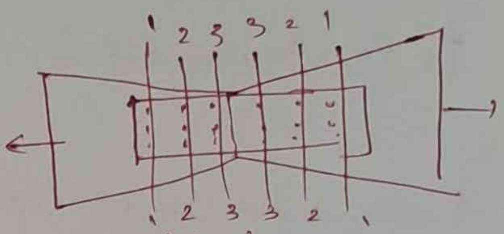
$P_t = \sigma_{at} A_{net}$ (WSM)

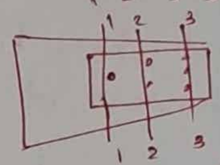
where,

σ_{at} → permissible axial tensile stress $= 0.6 f_y$

A_{net} → net effective c/s area.

$P_t = \min\left\{ A_g \frac{f_y}{1.10} \,\middle|\, A_{net} \frac{0.9 f_u}{1.25} \right\}$

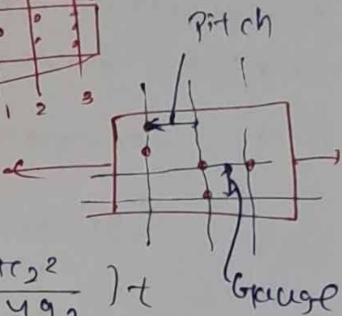




– To prevent fracture:–

$A_{eff} = 0.9 A_{net}$

$P_t = A_{eff} \frac{f_u}{1.25}$



For plate:–

calculate of A_{net}

i) chain riveting, $A_{net} = (B - 3d)t$

ii) Diamond ", $A_{net} = (B - d)t$

iii) staggered ", $A_{net} = \left(B - n_1 d + \frac{p_1^2}{4g_1} + \frac{p_2^2}{4g_2}\right)t$

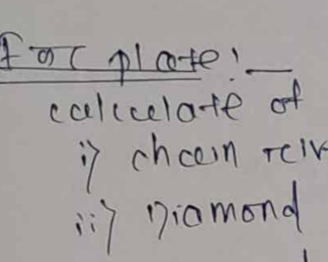
n_1 → no. of rivets along critical secn

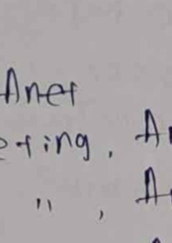
d → gross dia. or hole dia.

p_1 & p_2 → staggered pitch

g_1 & g_2 → " gauge.

For angles





$A_1 = \left(a - d - \frac{t}{2}\right)t$

$A_2 = \left(b - \frac{t}{2}\right)t$

– calculation of A_{net}

– for angle:– $A_{net} = (A_1 + k A_2)$

$k = \frac{3A_1}{3A_1 + A_2}$

↳ shear leg effect

A_1 → net area of connected leg

= gross area of " " – area of rivet hole

A_2 = gross area of unconnected leg / outstand leg.

– If two angles are placed back to back & connected to both sides of gusset plate.

$k = 1$ then $A_{net} = (A_1 + A_2)$

– It is most efficient way of connecting, then load carrying capacity is max^m.

– If the two angles don't have rivet then each angle behaves independently hence, $k = \frac{3A_1}{3A_1 + A_2}$

Dgn of TM :→

1. To find Anet required ?

$$A_{net}\ required = \frac{P}{\sigma_{at}}$$

2. ↑ Anet by 40 to 50% to get Ag required when reiveting is done.

3. ↑ Anet by 20% to get Ag required when welding is done.

4. select a suitable secⁿ & find the no. of reivets required.

5. If Ag provided > Ag required, so dgn. is safe.

Q

Check whether single angle of TM in welded steel is required that has area 475mm2 to be dgn-ed for axial tension force of 50kN, σ_{at} = 150 MPa.

A →

$$A_{required} = \frac{P}{\sigma_{at}} = \frac{50000\,N}{150\,N/mm^2} = 333.33\,mm^2$$

A provided = 475 mm^2 > 333.33 mm^2 (dgn. safe)

As per IS 800

- In case of single angles in tension connected by one leg only, the net eff. area = $a + \frac{b}{1+(0.2)\frac{b}{a}}$

- eff. area of plate girder, in tension = $A_f + \frac{A_w}{8}$
  in comp. = $A_f + A_w$

As per IS 800 for mild steel :—

Proportional limit → 190 - 220 N/mm2

yield strength → 230 - 250 N/mm2

Ultimate " → 410 - ~~370~~ 530 N/mm2

Fracture " → 250 - 300 N/mm2

Elongation of fracture → 23 - 35 %

Bearing stress → 0.75 fy

NOTE

- Vertical stiffeners are provided to prevent shear buckling of web.
- Horizontal stiffeners are " to avoid comp. buckling.

Vertical stiffners:-
- these are not designed as column.
- Minm spacing is $\frac{d_1}{3} = 0.33d$.
- Maxm " $= 1.5d$.

End bearing stiffners:-
They are designed as imaginary column with both end are fixed whose eff. length = $0.7l$.

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DESIGN OF STEEL STRUCTURES

Syllabus

Module I

- Introduction, advantages/disadvantages of steel, rolled steel section, various types of loads, design philosophy.
- Limit state design method, limit state of strength and serviceability, probabilistics basis for design, Riveted, bolted and pinned connections.
- Welded connections - assumptions, types, design of fillet welds, intermittent fillet weld, plug and slot weld, failure of welded joints vs bolted and riveted joints.

Module II

Tensions members, types, net cross-sectional areas, types of failure, slenderness ratio, design of tension members, gusset plate.

Module III

- Compression members, effective length, slenderness ratio, types of cross section, classification of cross section.
- Design of axially loaded compression members, lacing, battening, design of column bases and foundation bolts.

Module IV

Design of beams, types of c/s, lateral stability of beams, lateral torsional buckling, bending and shear strength, web buckling and web crippling, deflection, design procedure.

Module V

Plate girders - various elements and design of components Eccentric and moment connections, roof trusses.

Book

Limit state design of steel structure by S.K Duggal

IS 800:2007

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Ingredients of Cement

1) Lime
2) Silica - 20%
3) Alumina
4) Gypsum
5) Iron oxide
6) Magnesia
7) Sulphur Trioxide
8) Soda and potash

Introduction

- A steel structures is an assemblage of a group of members expected to sustain their share of applied forces and to transfer them safely to the ground.
- Depending upon the orientation of the member in the structure and its structural use, the member is subjected to forces either axial, bending or torsion or a combination thereof.
- Axial load can be either tensile or compressive and accordingly the member are called tension members or compression members.

Advantages of Steel

1) Steel members have high strength per unit weight.
2) Steel being a ductile material does not fail suddenly but gives visible evidence of impending failure by large deflections.
3) Structural steels are tough i.e they have both strength and ductile.
4) Being light steel members can be conveniently handled and transported.
5) Properly maintained steel structure have a long life.
6) The properties of steel mostly do not change with time.
7) Additions and alterations can be made easily to steel structure.
8) They can be erected as a faster rate.

9) Steel has the highest scrap value amongst all building materials
10) Steel is the ultimate recyclable material.

Disadvantages of steel

1) Steel structure when placed in exposed conditions are subjected to corrosion.
2) The use of weathering steels however in suitable application may eliminate the requirement of frequent painting.
3) Steel structure need fireproof treatment which increases cost.
4) Fatigue of steel is one of the major drawbacks. It involves a reduction in the strength when steel is subjected to large number of stress reversals and even to a large number of variations of tensile stress.
5) At the place of stress concentration in the steel section under certain conditions the steel may loose its ductility.

8/1/25

Tension Member

The axial load can be tensile members is called tension members.

Ex → tie

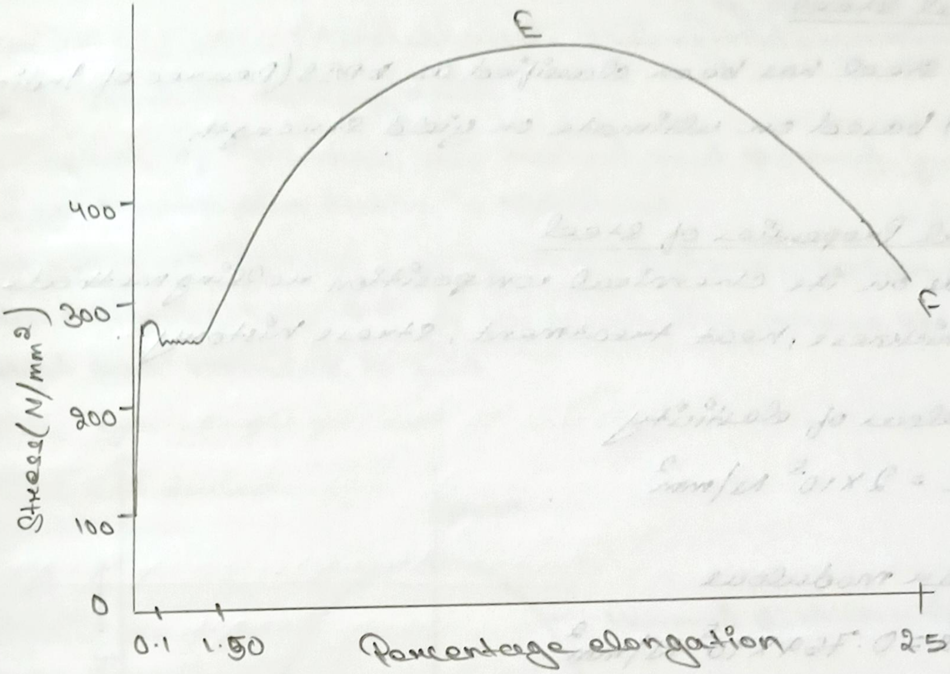
Compression Member

The axial load may be compressive member is called compression member.

Ex: → Strut
Column

Design of Structural elements

The position of elements i.e. beams, column, trusses, purlins etc are used to planning the architecture design of buildings.



(Stress strain Curve of mild Steel)

Properties of Steel

1) Carbon
2) Sulphur
3) Phosphorus
4) Manganese
5) Silicon

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Structural Steel

Structural steel has been classified as BBIS (Bureau of Indian Standard) based on ultimate or yield strength.

Mechanical Properties of Steel

It depends on its chemical composition, rolling methods, rolling thickness, heat treatment, stress history.

1. Modulous of elasticity

   $E = 2 \times 10^5 \ N/mm^2$

2. Shear modulous

   $G = 0.769 \times 10^5 \ N/mm^2$

3. Poisson's ratio (μ)

   a) Elastic range = 0.3

   b) Plastic range = 0.5

4. Coefficient of thermal expansion

   $12 \times 10^{-6} / °C$

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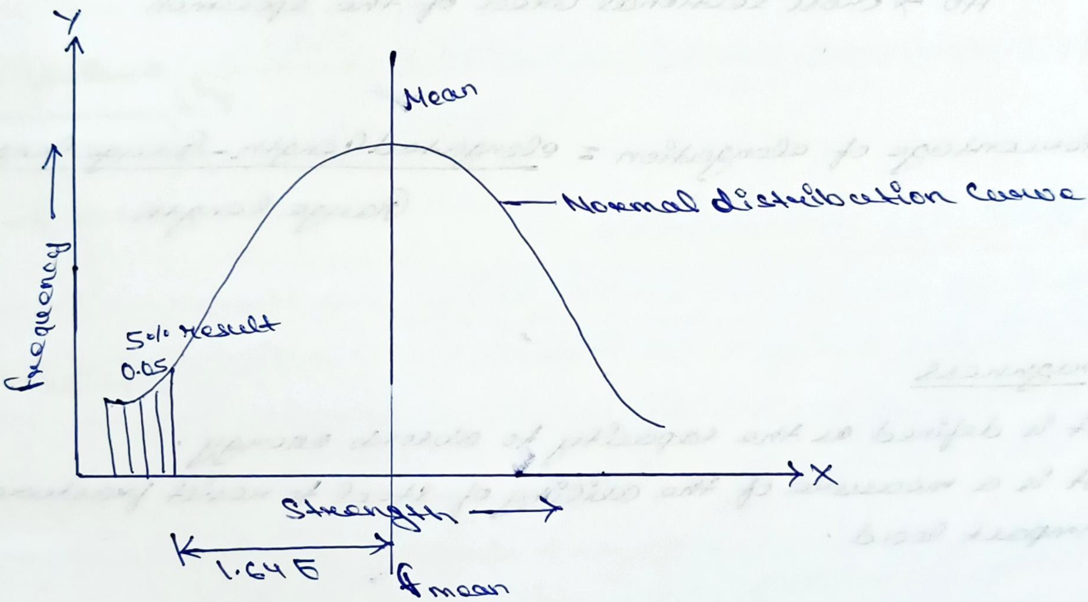
Ultimate Strength (UTS)

$$UTS = \frac{\text{Ultimate tensile load}}{\text{Cross sectional area}}$$

- It is subjected to tension, the area of cross-section of the specimen decreases and the ultimate ~~strength~~ stress would be ultimate tensile load divided by reduced area of cross section.

Characteristic Strength

- As per IS 800, the characteristic strength of steel is determine from test of steel in rolling mills.
- If a number of specimen are tested and plotted for strength, a normal distribution curve is obtained.
- The characteristic strength is the minimum value below which not more than 5% of corresponding stress of specimens tested are expected to fail



$$\text{Characteristic strength, } f_{ck} = f_{mean} - 1.64\sigma$$

where,

σ = Standard deviation

$$= \frac{\Sigma (f_{mean} - f)^2}{(n-1)^{0.5}}$$

where,

n = no. of specimens

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Ductility

- It is another prime important properties of steel.
- It is defined as the capacity of the structure or its members to undergo large inelastic deformation without significant loss of strength or stiffness.

$$\text{Gauge length} = 5.65\sqrt{A_0}$$

where,

A_0 → cross sectional area of the specimen

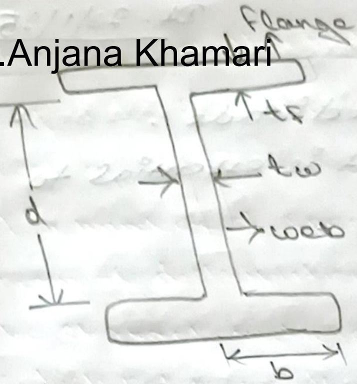
$$\text{Percentage of elongation} = \frac{\text{elongated length} - \text{Gauge length}}{\text{Gauge length}} \times 100$$

Toughness

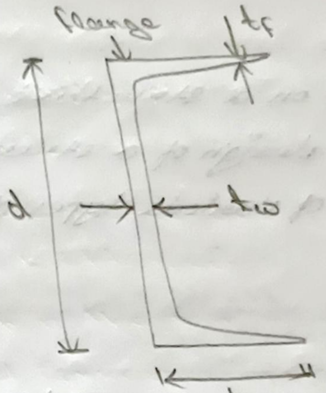
- It is defined as the capacity to absorb energy.
- It is a measure of the ability of steel to resist fracture under impact load.

Rolled Steel Section

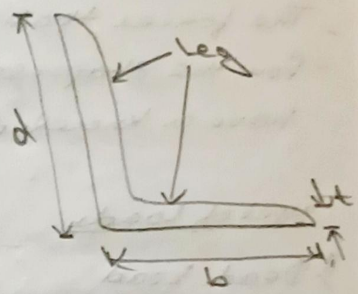
1) I Section
2) Channel Section
3) Angle Section
4) T - Section
5) Tube Section
6) Flat Section
7) Plate Section
8) Sheet Section
9) Bar Section
10) Strip Section



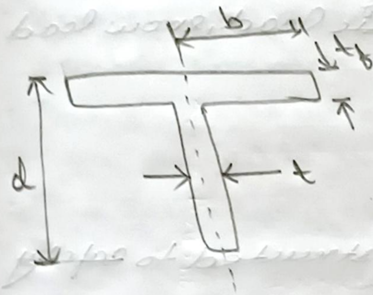
(I-section)



(Channel section).



(Angle Section)



(T section)

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Loads

- The forces that act on a structure is called loads.
- For the ~~shap~~ safe design of a structure it is essential to have a knowledge of various types of loads.

Types of Loads

1) Dead Load
2) Live Load → earth pressure, water current load, impact load, Thermal loads etc.
3) Environmental Loads :- → Wind load, ~~Sesmasic~~ Seismic load, snow load, rain load

Basic Design of Structure

Steel structure should be design and constructed to ~~safety~~ satisfy the design requirements for stability, strength, serviceablity, brittle fracture, fatigue, fire and durability with due to economy.

Design Philosophy

1) Intial ~~reading~~ yielding
2) Full ~~wildin~~ yielding
3) Tensile strength
4) Critical buckling
5) Maximum deflection permitted
6) Stress concentration
7) Fatigue
8) Brittle fracture

* Working stress method (elastic)
* Limit design method (Plastic)
* Limit State method

Working Stress Method

- It is the elastic method of design.
- The worst combination of working load is proportional to working stresses.
- The factor of safety is defined as

$$FOS = \frac{\text{Working stress}}{\text{Yield stress}} \quad \frac{\text{Yield stress}}{\text{Working stress}}$$

- The analysis methods are based on assumptions and so do not give the exact stresses.
- Structural members may temporarily be overloaded under certain circumstances.
- The stresses due to fabrication and erection are not considered in the design of ordinary structures.
- The secondary stresses may be appreciable.
- Underestimation of the future live load.
- Stress concentrations.
- Unpredictable natural calamities.

Plastic Method

- Steel is a ductile materials and the stress strain curve observe the load over the structure.
- The plastic range means large strain.

Limit State Method

- It is the similar to plastic design which consider most critical limit state and serviceability (design action ≤ design strength)

Q) The yield strength for a mild strength specimen was found to 250 N/mm^2. Taking a factor of safety of 2 find out the working stress.

Sol^n

Given data:-

For mild steel,

Yield strength = 250 N/mm^2

FOS = 2

$$\text{Working stress} = \frac{\text{Yield strength}}{\text{FOS}}$$

$$= \frac{250 \text{ N/mm}^2}{2}$$

$$= 125 \text{ N/mm}^2$$

Q) A tie bar 50 mm × 8 mm is to carry a load of 80 kN. A specimen of the same quality steel of cross section area 250 mm^2 was tested in the laboratory. The maximum load carried by the specimen was 125 kN. Find the ultimate tensile strength factor of safety in the design and the gauge length.

Sol^n

Given data:-

Ultimate tensile strength for the steel

$$= \frac{\text{Maximum load}}{\text{Cross-sectional area}}$$

$$= \frac{125 \times 1000}{250}$$

$$= 500 \text{ N/mm}^2$$

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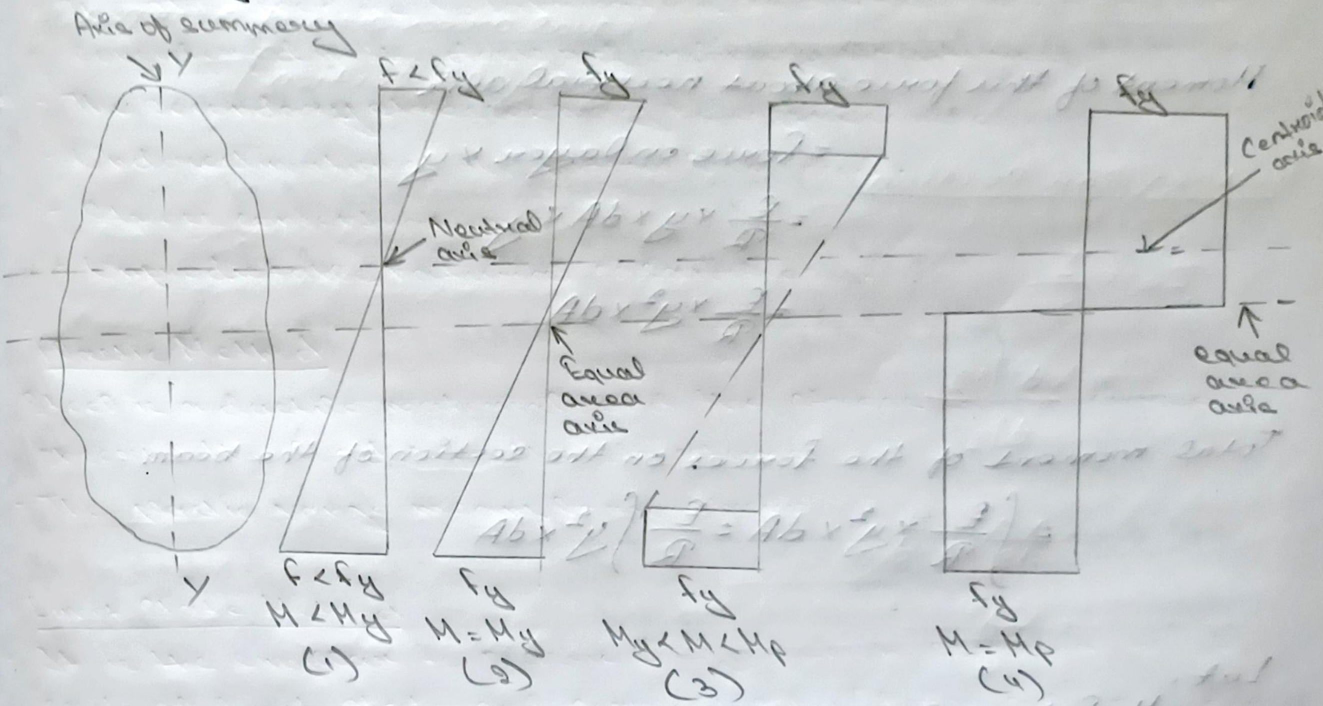
Plastic Analysis and Design

- Steel has unique physical property, ductility because of which it is able to absorb large deformation beyond the elastic limit without fracture.
- Plastic method of design are
  - * Limit design
  - * Collapse method of design
  - * Ultimate design

Bending of Beams



Development (in stages) of full plasticity of a beam section

Moment of Resistance

Due to pure bending, the layers above the neutral axis are subjected to compressive stresses whereas the layers below the neutral axis are subjected to tensile stresses. Due to these stresses, the forces will be acting on the layers. These forces will have moment about the neutral axis. The total moment of these forces about the neutral axis for a section is known as moment of resistance of that section.

$$\text{force on layer} = \frac{E}{R} \times y \times dA$$

Moment of this force about neutral axis

$$= \text{force on layer} \times y$$

$$= \frac{E}{R} \times y \times dA \times y$$

$$= \frac{E}{R} \times y^2 \times dA$$

Total moment of the forces on the section of the beam.

$$= \int \frac{E}{R} \times y^2 \times dA = \frac{E}{R} \int y^2 \times dA$$

Let,

M = External moment applied on the beam section.

for equilibrium the moment of resistance offered by the section should be equal to the external bending moment.

$$\therefore M = \frac{E}{R} \int y^2 \times dA$$

 expression $\int y^2 \times dA$ represents the moment of inertia of the area of the section about the neutral axis.

Let the moment of inertia be I.

$$\therefore M = \frac{E}{R} \times I \quad \text{or} \quad \frac{M}{I} = \frac{E}{R}$$

From the equation (7.2), we have

$$\frac{\sigma}{y} = \frac{E}{R}$$

$$\frac{M}{I} = \frac{\sigma}{y} = \frac{E}{R} \quad \text{(Bending equation)}$$

In this equation, the different quantities of are expressed in consistent units as given:

M is expressed in Nmm; I in mm^4

σ is expressed in N/mm^2; y in mm

E is expressed in N/mm^2; R in mm

3/2/25

Shape factor (S)

It is defined as the ratio of the plastic moment and the yield moment of the section.

$$S = \frac{M_P}{M_Y}$$

$$\Rightarrow S = \frac{f_y \cdot Z_P}{f_y \cdot Z_e}$$

$$\Rightarrow \boxed{S = \frac{Z_P}{Z_e}}$$

Where,

Z_e = Elastic modulus

(Elastic modulus)

... defined as the first moment of area.

$$Z_e = \frac{I}{Y}$$

where,

I = Moment of Inertia (Second moment of area)

Y = Distance of the extinct fibre distance from the neutral axis.

Z_p = Plastic modulus (It is the combine statistical moment of the cross sectional area above and below the equal area axis of a completely yeil yield section)

Rectangular Section

$$Z_e = \frac{I}{Y}$$

$$= \frac{bd^3/12}{d/2}$$

$$= \frac{bd^3}{12} \times \frac{2}{d}$$

$$= \frac{bd^2}{6}$$

$$Z_p = \frac{bd^2}{4}$$

$$S = \frac{Z_p}{Z_e}$$

$$= \frac{bd^2/4}{bd^2/6}$$

$$= \frac{bd^2}{4_2} \times \frac{6^3}{bd^2}$$

$$= 1.5$$

Load factor (F)

It is defined as the ratio of the collapse load to the working load.

$$F = \frac{P_u}{P_w}$$

$$= \frac{M_u}{M_w}$$

$$= \frac{f_y Z_p}{f Z_e}$$

$$\boxed{F = \frac{f_y}{f} \cdot S}$$

where,

$\frac{f_y}{f}$ = factor of safety

$$F.O.S = \frac{f_y}{f}$$

$$= \frac{f_y}{0.66 f_y} = 1.515$$

$(\because f = 0.66 f_y)$
↓
Permissible bending stress

Rectangular Section

$$F = 1.515 \times 1.5$$

$$= 2.2725$$

$$\simeq 2$$

* For rectangular section:-

$$S = 1.5$$

$$F = 2$$

* For I section:-

$$S = 1.12$$

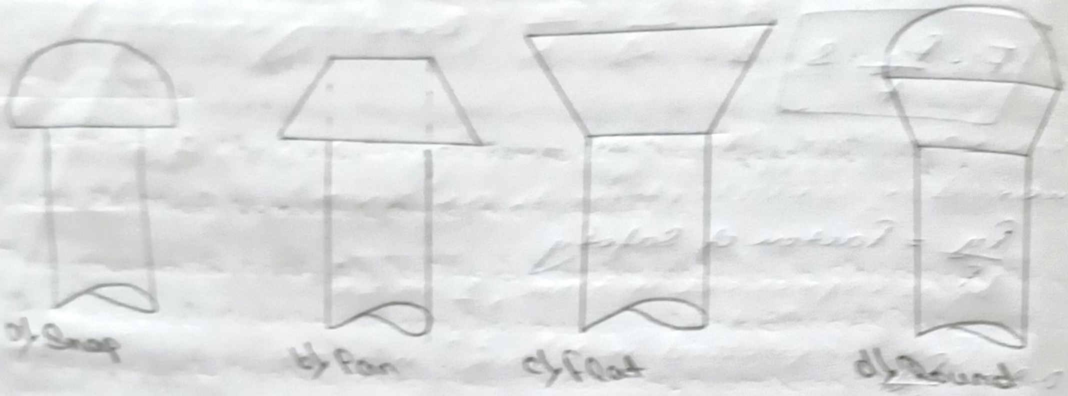
$$F = 1.7$$

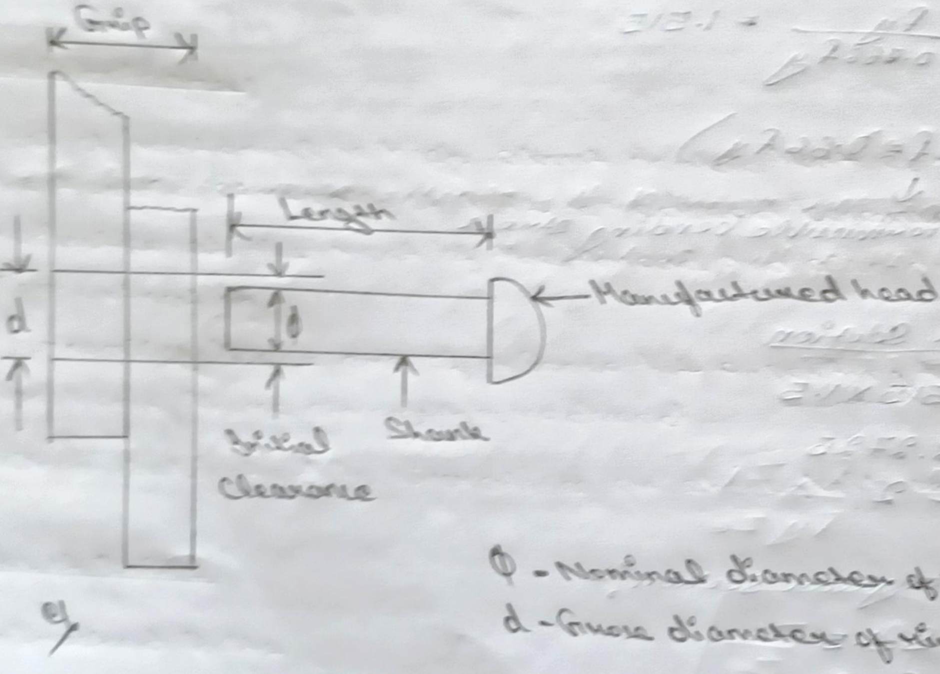
Probabilistic Basis for design for Connection

There are three types of connections:-

1) Riveted Connection
2) Bolted Connection
3) Pinned Connection

1) Riveted Connection





e)

φ - Nominal diameter of rivet

d - Gross diameter of rivet

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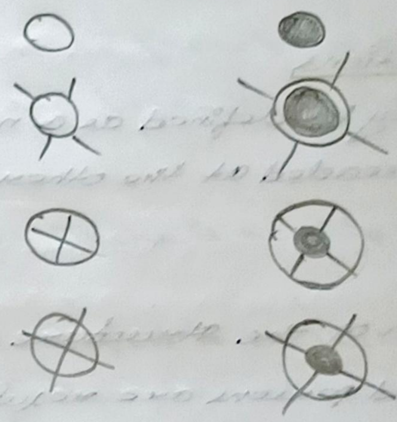
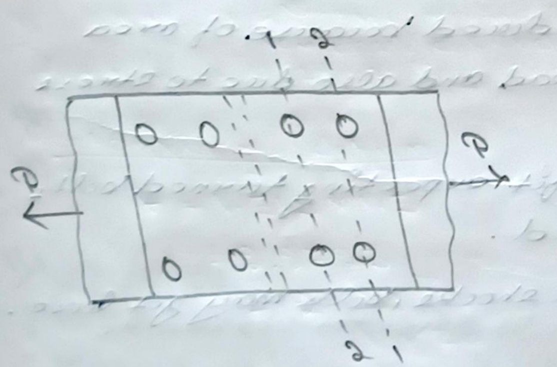
A rivet is made of a round ductile steel bar piece (mild or high tensile) called shank, with a head at one end.

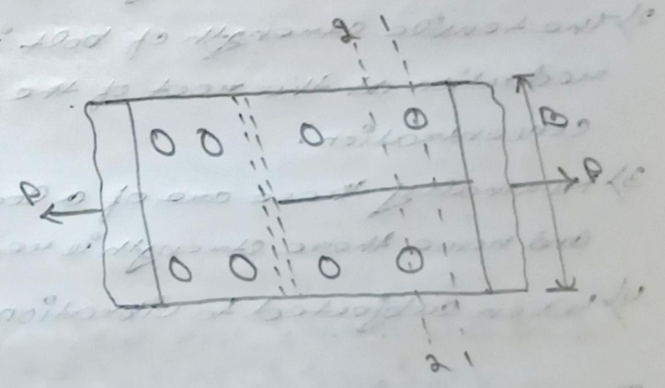
Description	Shop rivets	Field rivets
Round head both sides		
Countersunk near sides		
Countersunk far sides		
Countersunk both sides		

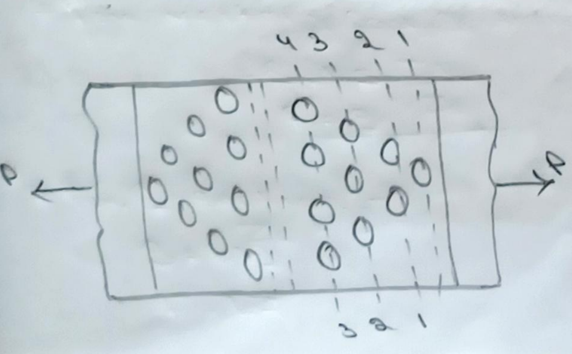


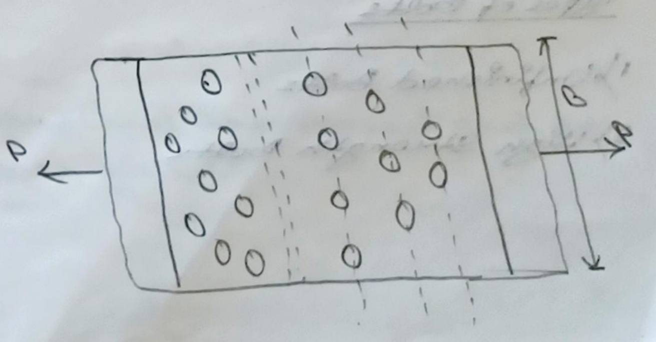
Patterns of Rivetted Joints

1) Chain
2) Staggered chain
3) Diamond
4) Staggered diamond









Design of Riveted Joint

1) The diameter of rivet to be used in the calculation is diameter of hole whereas for bolted connections it is the nominal diameter of bolt in case the holes are drilled.

2) The design stresses are slightly different those for bolts.

Bolted Connections

The bolt may be defined as a metal pin with a head at one end a shank threaded at the other end to receive a nut.

Advantages

1) The erection of the structure can be speeded up.

2) Less skilled persons are required.

3) The overall cost of bolted construction is cheaper than that of riveted construction because of reduced labour and equipment costs and the smaller number of bolts required resisting the same load.

Disadvantages

1) Cost of material is high, about double that of rivets.

2) The tensile strength of bolt is reduced because of area reduction at the root of the thread and also due to stress concentration.

3) Normally these are of a loose fit expecting turned bolts and hence there strength is reduced.

4) When subjected to vibrations or shocks, bolts may get loose.

Types of Bolts

1) Unfinished Bolts

2) High Strength bolts

1) Unfinished Bolts

Unfinished bolts are called ordinary, common, rough or black bolts. These are used for light structures subjected to static loads and for secondary members such as purlins, bracings, roof trusses etc.

2) High Strength Bolts

The high strength bolts are made from bars of medium carbon heat treated steel and from alloy steel.

Advantages

- HSFG bolts provide a rigid joint. There is no slip between the elements connected.
- Large tensile stresses are developed in bolts, which in turn provide large clamping force to the element connected.
- There is no stress concentration in holes, and therefore the fatigue strength is more.
- Few persons are required to make the connections, thus cost is reduced.
- Noise nuisance is not there and there is no danger of tossing of the rivet.
- Alterations can be done easily.

Types of Bolted Joints

1) Lap Joint
2) Butt Joint

1) Lap Joint

The two members to be connected are overlaped and connected together. Such a joint is called lap joint.

2) Butt Joint

The two members to be connected are placed end to end. Such a joint is called butt joint.

Load Transfer Mechanism

- Depending upon the type of bolts - ordinary or high strength bolts — the load transfer from one connected part to another may be by shear and bearing or by friction.
- The former being called bearing type connection and the latter the slip-critical or slip-resistant connection.
- The connection made with high-strength bolts may, however be slip-resistant or bearing type.

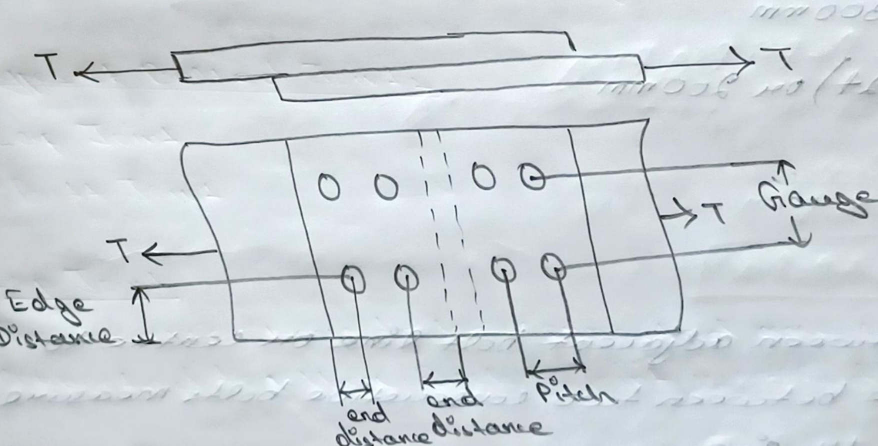
Failure of Bolted Joints

1) Shear failure of bolts
2) Bearing failure of bolts
3) Tension failure of bolts
4) Tension or tearing failure of bolts
5) Block Shear failure.

~~1) Shear failure of bolts~~

6/2/25

Specification for bolted Joints



(Double cover butt joint)

Diameter of Bolt

The large diameter of bolt are favourable in connection where shear governs because, the bolt capacity in shear o vese as square of the bolt diameter.

Spacing of Bolt hole

1) Pitch (P)

It is the distance between the centre of two consecutive bolts measures along a row of the column.

* Minimum Pitch

To prevent bearing failures of members between the two bolts. Less than 2.5 times of the nominal diameter of the bolt.

* Maximum Pitch

To reduce the length of the connection and gusset plate.

Limit of Pitch

1) ≯ 16t or 200 mm (Tension members)

≯ 12t or 200 mm (Compression member)

2) ≯ 32t or 300mm

3) ≯ (100 mm + 2t) or 200mm

4) ≯ 750 mm

Gauge (g)

The distance between adjacent bolt line or centre to centre distance between two consecutive bolts measure along the width size.

... of Connection

1) Shearing strength of bolts
2) Bearing strength of bolt
3) Tensile strength of bolt

1) Shearing strength of bolt

The nominal shear capacity of bolt

$$V_{nsb} = \frac{f_{ub}}{\sqrt{3}}(n_n A_{nb} + n_s A_{sb})$$

The nominal shear capacity of bolt for long joint will be less i.e.

$$V_{nsb} = \frac{f_{ub}}{\sqrt{3}}(n_n A_{nb} + n_s A_{sb})\beta_{lj}\,\beta_{lg}\,\beta_{pkg}$$

Where,

V_{nsb} = Nominal shear capacity of the bolt

f_{ub} = Ultimate tensile stress of the bolt

n_n = Number of shear planes with thread intercepting the shear plane.

n_s = Number of shear plane without thread intercepting the shear plane

A_{nb} = Net tensile stress area

A_{sb} = Nominal shank area

n = Number of shear plane

β_{lj} = A reduction factor to allow for the overloading of the end bolt that occurs in longer connection.

β_{lg} = A reduction factor to allow for the effect of the large grip length.

β_{pkg} = A reduction factor to account for packing plates in excess of 6 mm grade 4.6,

Bolt diameter (mm), d →	12	16	20	22	24	27	30	36
A_{nb} (mm^2) →	84.3	157	245	303	353	459	561	817

8/2/25

For the safety of joints in shear, the strength of bolt V_{sb}.

$$V_{sb} \le \frac{V_{nsb}}{\gamma_{mb}}$$

where,

γ_{mb} = Partial safety factor for the material of bolt = 1.2

$$V_{sb} = \frac{V_{nsb}}{\gamma_{mb}}$$

$$= \frac{f_{ub}}{\sqrt{3}\,\gamma_{mb}} \times (n_n A_{nb} + n_s A_{sb})\,\beta_{lj}\,\beta_{lg}\,\beta_{pkg}$$

For 4.6 grade steel bolt,

$f_{ub} = 400$ MPa

$$V_{sb} = \frac{400}{\sqrt{3} \times 1.25} \times (n_n A_{nb} + n_s A_{sb})\,\beta_{lj}\,\beta_{lg}\,\beta_{pkg}$$

$$= 184.75\, A_e\, \beta_{lj}\, \beta_{lg}\, \beta_{pkg}$$

Where,

A_e = effective area

⇒ $\boxed{A_e = n_n A_{nb} + n_s A_{sb}}$

For bolt in double shear,

$$V_{sb} = 2 \times 184.75\, A_e\, \beta_{lj}\, \beta_{lg}\, \beta_{pkg}$$

Reduction factor for Long Joint (β_{lj})

- When the length of the joint exceeds 15 d in the direction of the load the shear capacity of the joint is reduced.

$$\beta_{lj} = 1.075 - \frac{l_j}{200d} \quad (\text{for } 0.75 \le \beta_{lj} \le 1.0)$$

Where,

l_j = length of the joint (The diameter between the 1st and last row of bolt in a joint measure in the direction of load transfer)

* The reduction factor = 1

Reduction factor for large grip length (β_{lg})

- When the grip length of the bolt increases, the bolt is subjected to a greater bending moment due to shear force acting on its shank.

$$\beta_{lg} = \frac{8d}{3d + l_g}$$

Where,

l_g = Grip length

Reduction factor for Packing Plates (β_{pkg})

When the packing plates are more than 6mm in thickness, the shank of the ~~bold~~ bolt is subjected to bending which affects the nominal shear capacity of the bolt.

$$\beta_{pkg} = (1 - 0.0125\, t_{pkg})$$

Where,

t_{pkg} = thickness of the thinner packing plate in mm.

Bearing Strength of Bolt

Nominal bearing strength of bolt,

$$V_{npb} = 2.5 \, k_b \, d t f_u$$

where,

$$k_b = \text{smaller of} \begin{cases} \dfrac{e}{3d_0} \\ \dfrac{p}{3d_0} - 0.25 \\ \dfrac{f_{ub}}{f_u} \\ 1.0 \end{cases}$$

where,

d_0 = diameter of hole

e, p = end and point distance of the fastener along bearing direction.

f_{ub} = Ultimate stress of bolt

f_u = Ultimate tensile stress of the plate in Mpa

d = Nominal diameter of the bolt in mm

t = aggregate thickness

For the safety of the joint in bearing,

$$V_{pb} \leq \frac{V_{npb}}{\gamma_{mb}}$$

where,

γ_{mb} = the partial safety factor for material of bolt
= 1.25

~~$V_{pb} = 2.5 k_b$~~

$$V_{pb} = 2.5 k_b \, d t \frac{f_u}{\gamma_{mb}}$$

Tensile Strength of Bolt

The nominal capacity of bolt,

$$T_{nb} = 0.9 f_{ub} A_{nb}$$

$$< f_{yb} A_{sb} \frac{\gamma_{mb}}{\gamma_{mo}}$$

where,

f_{ub} = ultimate tensile stress of bolt

f_{yb} = yield stress of bolt

A_{nb} = net tensile stress area of bolt

A_{sb} = shank area of bolt

The bolt is safe in tension if the factored tension force,

$$T_{db} \leq \frac{T_{nb}}{\gamma_{mb}}$$

where,

γ_{mb} = the partial safety factor for material of bolt
= 1.25

γ_{mo} = Partial safety factor for material resistance governed by yielding = 1.10

$$T_{db} = 0.9 \frac{f_{ub}}{\gamma_{mb}} A_{nb}$$

Tensile Strength of Plate

Net area $A_n = (B - nd_h)t$, for chain bolting

For staggered bolting,

$$A_n = \left[B - nd_h + \sum_{i=1}^{m} \frac{p_{si}^2}{4g_i} \right] t$$

The tensile strength of the plate is given by

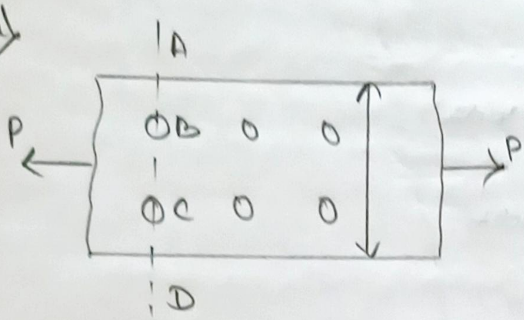
$$T_{nd} = 0.9 A_n \frac{f_u}{\gamma_{ml}}$$

f_u = the ultimate stress of material in MPa

A_n = the net effective area in mm^2

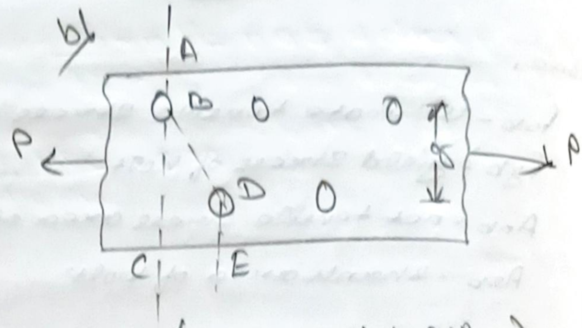
γ_{ml} = partial safety factor = 1.25

a)



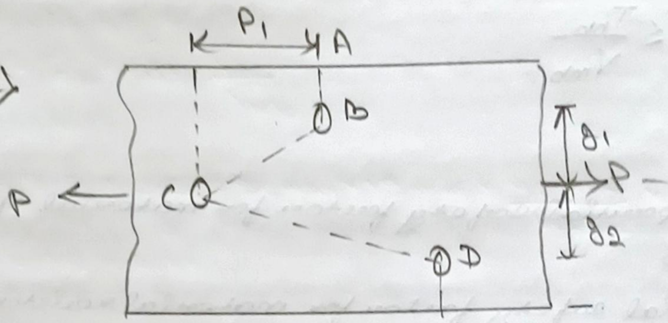
(chain bolting)

b)



(staggered bolting)

c)



(Pitch and gauge)

Strength and Efficiency of the Joint

Efficiency of a bolted joint, (η)

(η) is also called the percentage strength of the joint

$$\eta = \frac{\text{Strength of bolted joint / Pitch length}}{\text{Strength of solid plate / Pitch length}} \times 100$$

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Ex-4.1

Q) Calculate the strength of a 20mm diameter bolt of grade 4.6 for the following cases. The main plates to be jointed are 12 mm thick.

a) Lap Joint

b) Single cover butt joint: the cover plate being 10 mm thick.

c) Double cover butt joint: each of the cover plate being 8 mm thick

Soln

Given data:-

For grade 4.6,

$f_{ub} = 400$ MPa (for bolt)

For Fe 410 grade of steel,

$f_u = 410$ MPa (for steel)

$\gamma_{mb} = 1.25$ (partial safety factor for bolt)

Diameter of bolt, $d = 20$ mm

$A_{nb} = 245\ mm^2$

a) For Lap joint:-

Strength of bolt for single shear,

$$V_{sb} = A_{nb} \frac{f_{ub}}{\sqrt{3}\,\gamma_{mb}}$$

$$= 245\ mm^2 \times \frac{400\ MPa}{\sqrt{3} \times 1.25} \times 10^{-3}$$

$$= 45.26\ kN$$

From table 4.3

$d = 20$mm

$d_0 = 22$ mm

$e = 33$ mm

Assume,

$p = 50$ mm

The strength of the bolt in bearing:

$$V_{pb} = 2.5\, k_b d t \frac{f_u}{\gamma_{mb}}$$

$$k_b = \text{less of} \begin{cases} \frac{e}{3d_0} = \frac{33\,mm}{3 \times 22\,mm} = 0.5 \\ \frac{p}{3d_0} - 0.25 = \frac{50\,mm}{3 \times 22\,mm} - 0.25 = 0.51 \\ \frac{f_{ub}}{f_u} = \frac{400\,MPa}{410\,MPa} = 0.98 \\ 1 \end{cases}$$

So, $k_b = 0.5$

$$V_{pb} = 2.5\, k_b d t \frac{f_u}{\gamma_{mb}}$$

$$= 2.5 \times 0.5 \times 20\,mm \times 12\,mm \times \frac{410\,MPa}{1.25} \times 10^{-3}$$

$$= 98.4\ kN$$

∴ The strength of the bolt will be minimum in shear and bearing i.e. 45.26.

b) For single cover butt joint :-

$t = 10\,mm$

The strength of bolt in single shear = 45.26 kN

The strength of bearing:

$$V_{pb} = 2.5\, k_b d t \frac{f_u}{\gamma_{mb}}$$

$$= 2.5 \times 0.5 \times 20\,mm \times 10\,mm \times \frac{410\,MPa}{1.25} \times 10^{-3}$$

$$= 82\ kN$$

So, strength of the bolt is 45.26 kN.

c) For double cover butt joint,

$t = 12$ mm

The strength of bolt in double shear,

$$V_{sb} = 2 \times A_{nb} \frac{f_{ub}}{\sqrt{3}\,\gamma_{mb}}$$

$$= 2 \times 245 \times \frac{400}{\sqrt{3} \times 1.25} \times 10^{-3}$$

$$= 90.52 \text{ KN}$$

The strength of bolt in bearing,

$$V_{pb} = 2.5\, K_b d t \frac{f_u}{\gamma_{mb}}$$

$$= 2.5 \times 0.5 \times 20 \times 12 \times \frac{410}{1.25} \times 10^{-3}$$

$$= 98.4 \text{ KN}$$

The strength of the bolt will be minimum of the strength in shear and bearing i.e. 90.25 KN.

<u>Minimum Edge Distance of bolts</u>

Nominal diameter of bolt (mm) (d)	Diameter of hole (mm) (do or dh)	Distance of sheared or hand flame cut edge (mm)	Distance to rolled machine flame cut, sawn or planed edge, (e) (mm)
12	13 and above	20	19
14	15	26	23
16	18	30	27
18	20	34	30
20	22	37	33
22	24	40	36
24	26	44	39
27	30	51	45
30	33	56	50
Over 33 mm	Bolt diameter + 3 mm	1.7 × hole diameter	1.5 × hole diameter

Ex:- 4.2 Dt - 15/2/25

Q. A single-bolted double-cover butt joint is used to connect two plates which are 8 mm thick. Assuming 16 mm diameter bolts of grade 4.6 and cover plates to be 6 mm thick, calculate the strength and efficiency of the joint if 4 bolts are provided in the bolt line at a pitch of 45 mm.

Also determine the efficiency of the joint if two lines of bolts with two bolts in each line have been arranged to result in a double-bolted double cover butt joint.

Soln

Given data :-

For Fe 410 grade of steel,

$f_u = 410$ MPa

For bolts of grade 4.6,

$f_{ub} = 400$ MPa

For 16 mm dia bolt,

$A_{nb} = 157\ mm^2$

$\gamma_{mb} = 1.25$

$\gamma_{ml} = 1.25$

$t = 8$ mm

Plate thickness $(t) = 8$ mm

Cover plate thickness $(t_c) = 6$ mm

Bolt diameter $(d) = 16$ mm

Bolt grade $= 4.6$

From Table 4.3 :-

For 16 mm dia bolt,

$A_{nb} = 157\ mm^2$

$\gamma_{mb} = 1.25$

$\gamma_{ml} = 1.25$

 ...bolt in shear per pitch length,

$$V_{sb} = 2 \times A_{nb} \times \frac{f_{ub}}{\sqrt{3}\,\gamma_{mb}}$$

$$= 2 \times 157 \times \frac{400}{\sqrt{3} \times 1.25} \times 10^{-3}$$

$$= 58 \text{ kN}$$

$t = 8\text{mm}$

the strength of bolt in bearing per pitch length,

$$\Rightarrow V_{pb} = 2.5\, k_b d t \frac{f_u}{\gamma_{mb}}$$

$$k_b = \text{least of} \begin{cases} \frac{e}{3d_0} = \frac{3}{3 \times 18} = 0.55 \\ \frac{p}{3d_0} - 0.25 = \frac{45}{3 \times 18} - 0.25 = 0.58 \\ \frac{f_{ub}}{f_u} = \frac{400}{410} = 0.975 \\ 1 \end{cases}$$

$k_b = 0.55$

$$V_{pb} = 2.5 \times 0.55 \times 16 \times 8 \times \frac{400}{1.25} \times 10^{-3}$$

$$= 56.31 \text{ kN}$$

The net strength of plate per pitch length,

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

$$T_{dn} = 0.9 \frac{f_u}{\gamma_{m1}} (p - d_h) t$$

from table 4.2,

$$T_{dn} = 0.9 \times \frac{410}{1.25} (45 - 18) \times 8 \times 10^{-3}$$

$$= 63.76 \text{ kN}$$

Strength of the solid plate per pitch length,

$$= 0.9 \frac{f_u}{\gamma_{m1}} p t$$

$$= 0.9 \times \frac{410}{1.25} \times 45 \times 8 \times 10^{-3}$$

$$= 106.27 \text{ kN}$$

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the strength of the joint per pitch length = 56.31 kN

$$\text{Efficiency of the joint} = \frac{56.31}{106.27} \times 100$$

$$= 52.99\,\%$$

When the bolts are arranged in the rows the strength of the joint per pitch length on the basis of

a) Shear = 2×58 = 116 kN

b) Bearing = 2×56.31
= 112.62 kN

c) Net tensile strength = 63.76 kN

So strength of the joint per pitch length = 63.76 kN

$$\text{Efficiency of the joint} = \frac{63.76}{106.27}$$

$$= 59.99\,\%$$

18/2/25

Simple welded Connection (chapter)

Introduction

- When two structural members are jointed by means of welds, the connection is called a welded connection.
- Welded design offer the opportunity to achieve a more efficient use of materials.
- The speed of fabrication and erections helps compress production schedules.
- Welding saves weight and consequently cuts costs.
- No deductions are there for holes, thus the gross section is effective in carrying loads.
- Welded joints are better for fatigue loads, impact loads and vibrations.

Types

1) fillet weld
2) Groove weld
3) Plug weld
4) Slot weld
5) Spot weld

Assumptions of welded joint

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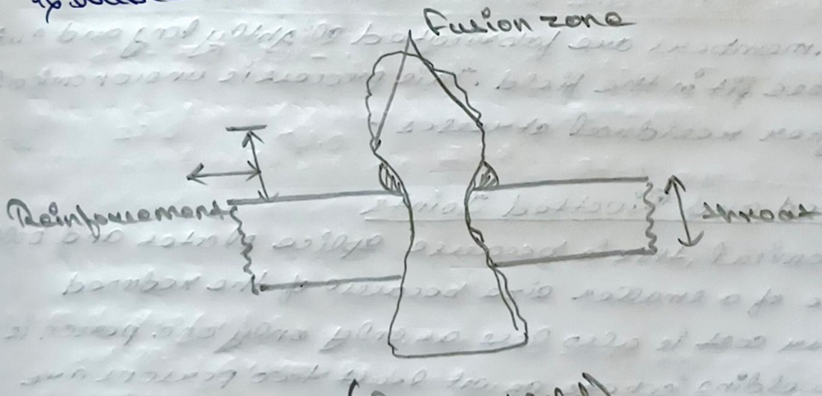
- The welding connection various parts are homogeneous, isotropic and elastic elements.
- The part connected by the weld are rigid and their force of deformation are neglected.
- Only stress due to external loads are consider.

Design of groove welds

- Groove and filleted weld are most frequently used.
- Groove welds are provided when the member to be joined are lined up.
- It requires edge preparation are costly.

Types of Groove weld

1) Square
2) Single VEE
3) Double VEE
4) Single U
5) Double V
6) Single Tee
7) Double Tee
8) Single be Ve
9) Double be Ve



(Groove weld)

Design Strength

- The design strength of the groove weld in tension or compression is governed by yield.

$$T_{dw} = \frac{f_y L_w t_e}{\gamma_{mw}}$$

where,

f_y = smaller of yield stress of the weld (f_{yw}) and the parent metal (f_y) in MPa.

L_w = effective length of the weld in mm

t_e = effective throat thickness of the weld in mm.

γ_{mw} = partial safety factor
= 1.25 for shop welding
= 1.5 for site welding

- The design strength of the butt weld in shear is also governed by yield.

$$V_{dw} = \frac{f_{yw1} L_w t_e}{\gamma_{mw}}$$

Where,

f_{yw1} = smaller of shear stress of weld ($f_{yw}/\sqrt{3}$) and the parent metal ($f_y/\sqrt{3}$)

γ_{mw} = Partial safety factor as defined above.

f_{yw} = yield stress of weld (MPa)

Design of fillet welds :-

- The fillet weld is done for members which overlap each other. For such joints the critical stress is shear stress.

Fillet weld Vs Butt weld

1) A fillet weld saves the operations of veeing and finishing the ends of members.
2) In case of butt weld, members are fabricated slightly long and cut exactly to have a close fit in the field. This process is uneconomical.
3) Butt welds have higher residual stresses.

Welded joints Vs Bolted and Rivetted Joints

1) Welded joints are economical, this is because splice plates and bolts/rivetted materials are of a smaller size because of the reduced connection length. Labour cost is also less as only only one person is required to do the welding whereas at least two persons are required for bolting and four for rivetting.

2) Welded structures are more rigid as compared to bolted/rivetted joints. In bolted/rivetted joints, cover plates, connecting angles etc. deflect along with the member during load transfer and make the joint more flexible.

3) Due to the fact that the strength of a welded joint is the same as that of the parent metals, even a smallest piece of the metal which otherwise is a scrap can be used, bringing overall economy.

4) With welding it has become possible to connect tubular sections, which are structurally very economical.

5) Due to the fusion of two metal pieces jointed, a continuous structure is formed which gives a better architectural appearance than bolted/rivetted joints.

6) Alterations can be done with less expensed in case of welding as compared to bolting/riveting.

7) The process of welding is quicker in comparison to bolting/riveting.

8) The process of welding is silent, whereas in the case of riveting a lot of noise produced.

9) In welding less safety precautions are required for the public in the vicinity, whereas a hot rivet may toss and injure the persons working.

10) As slice plates, bolts/rivets etc are not used, the details and drawings of welded structures are easier and less time consuming.

20/2/25

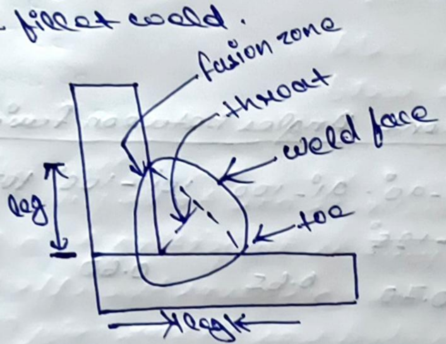
Design of fillete welds

Size(s):-

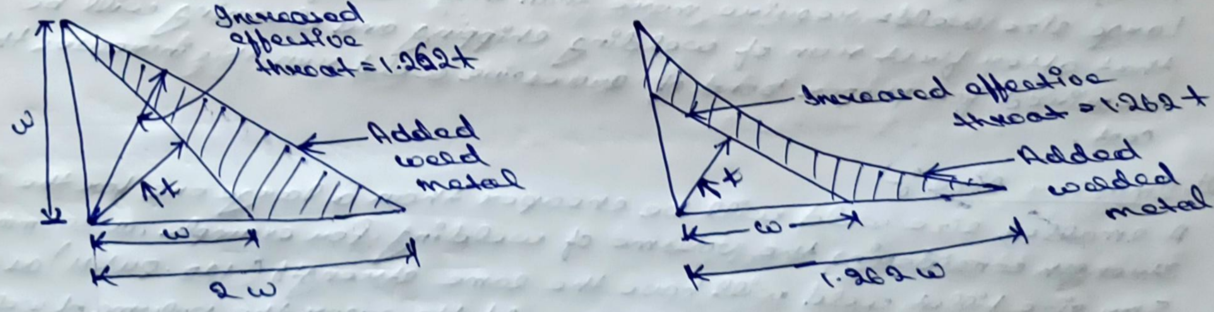
The minimum leg length of the weld.

Leg length:-

The leg length is the distance from the root the toe of the toe of the fillet weld.



- It is measured by the largest right angle triangle which can be inscribed within the weld.



Size of weld

(a) **Maximum size of weld**

It is obtained by subtracting 1.5mm from the thickness of the thinner member to be joined.

(b) **Minimum size of weld**

The weld size should not be less than 3mm

Table 5.4

Minimum size of weld

Thickness of thicker member		Minimum size (mm)
Over (mm)	Upto and including (mm)	
0	10	3
10	20	5
20	32	6
32	50	8 first run 10

Table 5.5

Values of Constant K for Different Angles between fusion faces

Angle between fusion faces	60°–90°	91°–100°	101°–106°	107°–113°	114°–120°
K	~~0.70~~ 0.70	0.65	0.60	0.55	0.50

~~114°–125°~~

Minimum size

1) **Large size:-**

Large size welds require more than one run of welding which means that after first run of welding chipping and cleaning will be required for proper bond of successive weld run this increases the cost.

2) **Smaller size:-**

A smaller size weld will be cheaper than a large one for the same strength considering the volume of welding. For example, a 300mm, 5mm size fillet weld will have the same strength (198.84kN) as a 150 mm long and 10 mm size fillet weld. However the volume of weld metal for a 10mm weld will be $\left[\frac{1}{2}\times 10^2 \times 150 = 7500\,mm^3\right]$

which is about twice that of 5mm size weld,

$$\left[\frac{1}{2}\times 5^2 \times 300 = 3700\,mm^3\right]$$

Effective throat thickness

The shortest distance from the root of the fillet weld to the face of the diagrammatic weld (line joining to the e)

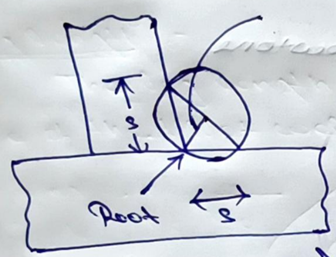
- Effective throat thickness should not be less than 3mm.
- It should not exceed 0.7 t or 1.0 t

where,
t = thickness of thinner plate of elements being welded.

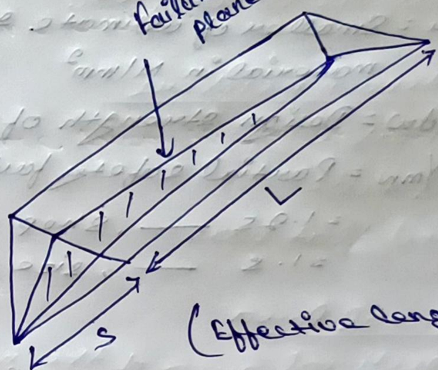
Effective throat thickness
= k × size of weld
= k × s

where,
s = size of weld in mm
k = constant

Throat = ks
= 0.707s



(Throat of weld)



(Effective length of weld)

* Fillet weld is not recommended if the angle between fusion force is less than 60° or more than 120°

For long joints the design capacity of weld is reduced by a factor,

$$\beta = 1.2 - \frac{0.2 l_j}{150 t_t} \leq 0.1$$

where,
l_j = length of joint in the ~~decrease~~ direction of force transfer.
t_t = throat size of the weld

Effective Area

It is equal to effective length of weld multiplied by effective throat thickness.

Design strength

Design strength of a fillet weld,

$$f_{wd} = \frac{f_{wn}}{\gamma_{mw}}$$

f_{wn} = nominal strength of fillet weld

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

The design strength of a fillet weld is based on its given by,

$$P_{dw} = L_w t_t \frac{f_u}{\sqrt{3}\gamma_{mw}}$$

or,

$$= L_w K s \frac{f_u}{\sqrt{3}\gamma_{mw}}$$

where,

L_w = effective length of weld in mm.

t_t = throat thickness in mm

s = size of weld in mm

f_u = Smaller of ultimate strength of the weld and material in N/mm²

P_{dw} = Design strength of weld in connections

γ_{mw} = Partial safety factor

= 1.25 —, shop weld

= 1.5 —, site weld

4/3/25

Ex-5.1

Two plates of 16 mm and 14 mm thickness are to be joined by a groove weld as shown in fig. Ex-5.1. The joint is subjecting to a factored tensile force of 430 KN. Due to some reasons the effective length of the weld that could be provided was 175 mm only. Check the safety of the joint if.

a) Single V groove weld is provided.

b) Double V groove weld is provided

Assume the plates to be shop welded (γ_{mw} = 1.25)

Solⁿ

Given:-

a) Single V groove weld:-

Tensile force = 430 KN

For Fe 410 grade of steel,

f_y = 250 MPa

Throat thickness,

$T_e = \frac{5}{8} t$

$= \frac{5}{8} \times 14\,mm$

$= 8.75\,mm$

For shop weld ;

$\gamma_{mw} = 1.25$

Effective length of weld ,

$L_w = 175\,mm$

Strength of the weld ,

$T_{dw} = L_w t_e \frac{f_y}{\gamma_{mw}}$

$= 175 \times 8.75 \times \frac{250\,MPa}{1.25}$

$= 175 \times 8.75 \times \frac{250 \times 10^{-3}}{1.25}\,kN/mm^2$

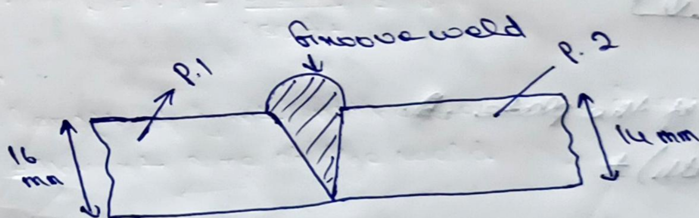
$= 306.25\,kN < 430\,kN$ thickness (t) (not ok)

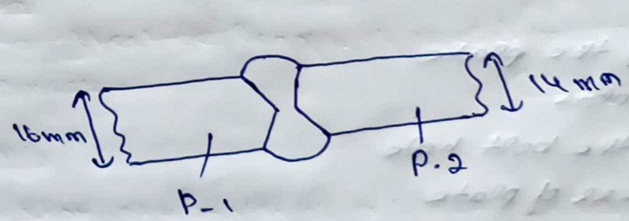
b) Double-V groove weld :-

$T_{dw} = L_w t_e \frac{f_y}{\gamma_{mw}}$

$= 175 \times 14 \times \frac{250 \times 10^{-3}}{1.25}\,kN/mm^2$

$= 490\,kN > 430$ (ok)





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MODULE - II

TENSION MEMBER

Introduction

- A structural member subjected to two pulling tensile forces applied at its ends is called a tension member.
- The stress in an axial load, tension member can be determined by axial load with its cross-sectional area.

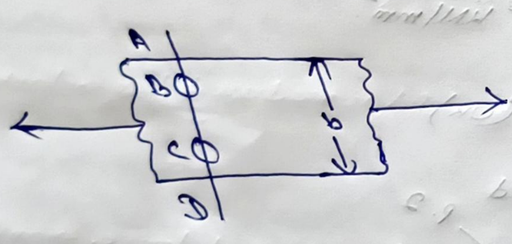
- e.g
  1) Trusses in building and bridge
  2) Communication and satellite to use
  3) Ties in lattice girders
  4) Bracing system in multistorey building
  5) Industrial building etc

Types of tension members

1) Wires and cable
2) Bars and rods
3) Plates and flat bars

Net sectional Area

- It is the sectional area of the members minus the sectional area of the maximum number of holes.



Net sectional area of the plate, A.

$$A_n = (b - n d_h) t$$

where,

b = width of the plate
n = no. of bolts
d_h = dia of the bolt hole
t = thickness of plate

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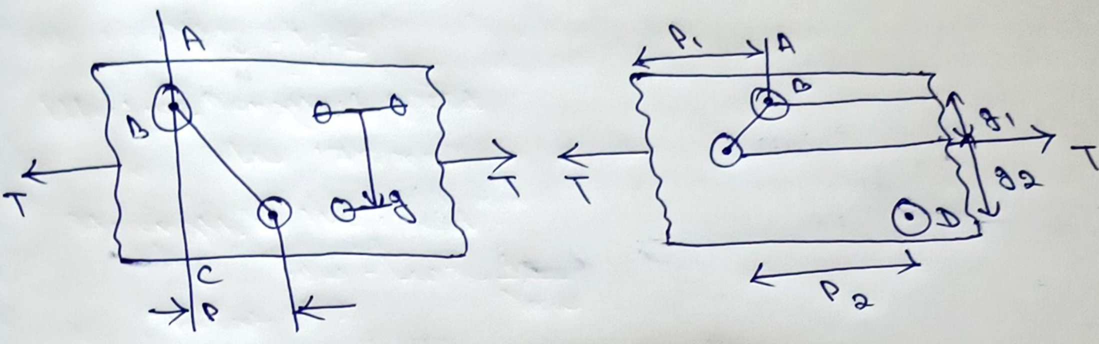
$$A_n = A_g$$

where,

A_g = Gross-cross sectional area of the plate



$$A_n = A_g - \left[\text{sectional area of the hole} - \left(\frac{P_1^2 t}{4g_1} + \frac{P_2^2 t}{4g_2}\right)\right]$$

$$= bt - \left[ndht - \left(\frac{P_1^2 t}{4g_1} + \frac{P_2^2 t}{4g_2}\right)\right]$$

$$= bt - ndht + \frac{P_1^2 t}{4g_1} + \frac{P_2^2 t}{4g_2}$$

$$= \left(b - ndh + \frac{P_1^2}{4g_1} + \frac{P_2^2}{4g_2}\right)t$$

If, $P_1 = P_2 = P$

$g_1 = g_2 = g$

then,

$$A_n = \left(b - ndh + \frac{P^2}{4g} + \frac{P^2}{4g}\right)t$$

$$= \left(b - ndh + \frac{2P^2}{4g}\right)t$$

$$\Rightarrow A_n = \left(b - ndh + \frac{P^2}{2g}\right)t$$

where,

P = Pitch distance

g = Gauge distance