Notes by-

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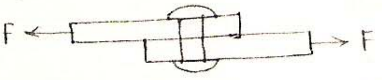
/SS-1 (I-IV)

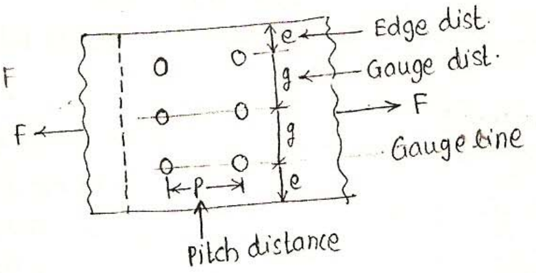
Rivet / Bolted Connections

* Types of riveted & bolted Joints :-

① Lap Joint : Plates to be connected overlapping each other.

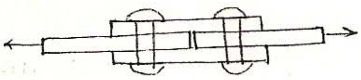


Single riveted lap Joint



chain Riveting

② Butt Joint :- Plates are connected at their faces & connection is made by providing cover plate (s) on one or both sides.



Definations :-

① Nominal dia. The dia. of shank of a rivet before rivetting. Dia. of shank is called as nominal diameter. For a bolt, dia. of unthreaded portion of shank is called as nominal diameter.

② Effective dia. / Gross dia :- The dia. of hole it fills after rivetting. For a bolt nominal dia. is same as gross dia.

③ Gross area :- ~~Ne~~ Gross area of rivet or bolt is given by its gross dia.

④ Net area :- Net area of ~~rivet~~ bolt is the area of root of thread.

⑤ Pitch :- The c/c dist. betn any two adjacent rivets.

⑥ Diagonal Pitch :- Dist. betn centres of any two ~~rivets~~ adjacent rivets in diagonal direction.

⑦ staggered Pitch :- c/c dist. betn any two consecutive rivets in a zig-zag riveting measured parallel to direction of stress in the member.

⑧ Gauge :- Normal dist betn two adjacent gauge line.

⑨ Edge dist :- Dist. betn edge of member or cover plate to the centre of nearest rivet hole.

⑩ Proof load :- Initial tension in HSFG bolt

⑪ slip factor :- Coeff. of friction in friction type joint.

⑫ Property class :- Bolts are grouped under diff. grades or properties depending upon their strength. For eg: Property class 4.6 indicates that, nominal ultimate tensile strength is 400 MPa & the nominal yield stress = 0.6x400=240 MPa.

Nominal dia. of rivet (mm)	12	14	16	18	20	22	24	27	30
Gross dia. (mm)	13.5	155	17.5	19.5	21.5	23.5	25.5	(29)	32
Min. edge dist. (mm) →	see Note*								
① For sheared or rough edge	19	25	29	32	32	38	44	51	57
② For rolled or planed edge	17	22	25	29	29	32	38	49	51
③ Min. Pitch	← 2.5 times dia. of rivet hole →								

④ Max. pitch for

a) Any two adjacent Rivets (Including Tacking Rivets) — ~~3.5 times~~ 32t or 300 mm whichever is less.

b) Rivet lying in a line parallel to force in member.
i) In Tension – 16t or 200mm – whichever is less
ii) In comp – 12t or hichever is less

} t = Thk. of outside plate

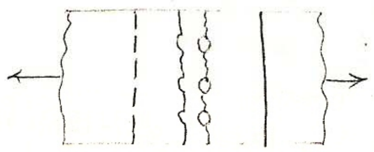
Note:- When edge dist. of a rivet or bolt in the direct^n of force it bears is less than 2x effective dia. of rivet or bolt, the per- bearing stresses of that rivet or bolt on the connected part shall be reduced by following ratio -

$$= \frac{\text{actual edge dist}}{2\times \text{eff. dia. of rivet or bolt}}$$

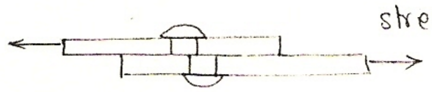
* Failure of Riveted Joint :-

① **Tearing of plates bet^n rivet hole :-** The strength of plate is reduced by rivet holes & the plates may tear off along the line of rivet holes.

This type of failure is for Tension member only.

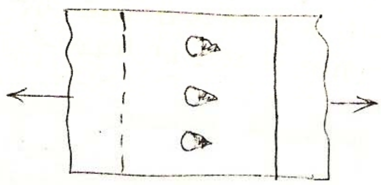


② **Shearing of Rivets :-** The rivet may fail by shearing if the shearing stress exceeds their shearing strength. In lap joints & single cover butt joints rivets are sheared at one plate only.

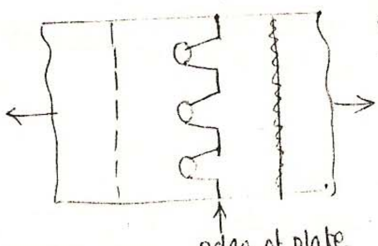


In double cover butt joint, rivets are sheared at two planes.

③ **Bearing of plate or rivet :-** The plate or rivet is crushed if comp. stresses > Bearing strength of plate or rivet.



④ **Edge cracking :-** The plate will crack at the back of rivet if it is placed very near to edge of plate. This failure is prevented by keeping min. edge dist.



First 3 types of failure determine the strength of joint.

Rivet value or strength of rivet is determined by type of failure in ② & ③ i.e. shearing & bearing of rivets.

* Strength of Riveted / Bolted Joint :-

Strength of joint per pitch = Smaller of
① Strength of plate bet^n rivet hole in Tension
② Rivet value.

① Strength of plate bet^n rivet hole in tension = $\sigma_{at} \times (p-d) \times t$

(GATE) → ② Rivet value = Smaller of bearing stress & shearing strength of rivet

Bearing strength of rivet $= \sigma_{pf} \times d \times t$ } whichever is smaller
$= \sigma_p \times d \times t$

(Aptitude) → Shearing strength of rivet $= \tau_{vf} \times \frac{\pi}{4} \times d^2$ → single shear
$= \tau_{vf} \times 2\frac{\pi}{4} \times d^2$ → Double shear.

where; σ_{at} = Allowable tensile stress in axially loaded tensile member.
σ_p = allowable bearing stress in member
σ_{pf} = Allowable bearing stress in rivets.
τ_{vf} = Allowable shear stress in rivet
p = pitch
d = eff. dia. of rivet
t = thk. of thinner plate
= thk. of main plate or total thk. of cover plates } whichever is smaller.

* **Assumptions** :-
① Tensile stress is uniformly distributed on the portions of plate betn rivets.
② Friction betn plates is neglected ⇒ Safer side
③ shearing stress is uniformly distributed on c/s of rivet
④ Rivet fill the holes completely
⑤ Rivets in groups shears load equally.
⑥ Bending stress in rivets is neglected
⑦ Bearing stress distribution is uniform & contact area = d·t
where d = dia. of rivet
t = thk. of plate.

* Efficiency of a Joint :-

$$\text{Efficiency of a Joint} = \frac{\text{strength of Joint}}{\text{original strength of member without rivet hole}}$$

∴ For max. efficiency ⇒ No. of rivet holes should be min. at critical section.

Pro: Determine rivet value of 18 mm dia. rivet connecting 10 mm plate & is in –
a) single shear
b) Double shear
The per. stress for rivet in shear & bearing are 80 MPa & 250 MPa resp.
The per. stress for rivet in shear & bearing is 250 MPa.
& for plate bearing stress is 250 MPa.

Soln :- strength of Rivet :- (d = Gross dia)
① In bearing = $\sigma_{pf} \times d \times t$
$= 250 \times (18+1.5) \times 10$
$= 48.75$ kN
② In single shear = $\tau_{vf} \times \frac{\pi}{4}(d)^2$
$= 80 \times \pi/4 \, (19.5)^2$
$= 23.89$ kN
③ In double shear = $\tau_{vf} \times 2\pi/4 \, (d^2)$
$= 47.78$ kN

a] ∴ Rivet value in single shear = Smallest of ① or ②
$= 23.89$ kN
b] Rivet value in double shear = smallest of ① & ③
$= 47.78$ kN

* **Design of Riveted Joint for axially loaded members** :-
① Dia of Rivet = $6\sqrt{\text{thk. of plate (mm)}}$
② No. of Rivets reqd. at joint = $\frac{\text{Load}}{\text{Rivet value}}$
③ Arrangement of rivet should satisfy pitch, edge dist & gauge dist. requirements.
④ Length of joint should be small to save matl. on cover plate, gusset plate etc.
⑤ No. of rivets should be increased gradually towards joint for uniform distribution of stress in the rivets.
⑥ C.G. of each rivet group should coinside with centre line of connected member.

Pro: A member of roof truss consist of two angle iron 80x50x6 mm placed back to back on both sides of an 8 mm thk. gusset plate. It carries a direct load of 71 kN. Determine the power driven field rivets reqd. for the joint.

Sol^n:- From IS : 800 - 1984 ;

$\tau_{vf} = 90$ MPa
$\sigma_{pf} = 270$ MPa } for power driven rivets.

① Dia. of rivet $= 6\sqrt{t} = 6\sqrt{6} = 14.69$ mm ≈ 16 mm.

② ∴ Gross dia. of rivet $= 16 + 1.5 = 17.5$ mm

③ strength of rivet in bearing 8 mm gusset $= \sigma_{pf} \times d \times t$

$= 270 \times 17.5 \times 8$

$= 37.8$ kN.

④ ~~sh~~ strength of rivet in double shear $= 2 \times \frac{\pi}{4} d^2 \times \tau_{vf}$

$= 2 \times \frac{\pi}{4} (17.5)^2 \times 90$

$= 43.29$ kN.

∴ Rivet value = Min. of bearing or shearing.
= 37.8 kN.

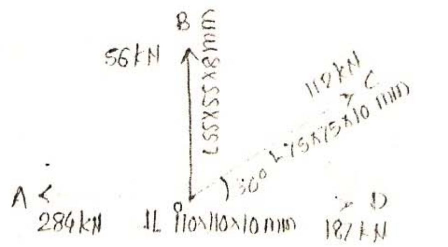
∴ No. of rivets reqd $= \frac{\text{Direct load}}{\text{Rivet value}}$

$= \frac{71}{37.8}$

$= 1.89$

≈ 2 Nos.

Pro. 2] Fig. shows a joint in the lower chord of roof truss. Design the riveted connection if permissible stresses are $\sigma_{at} = 150$ MPa & $\sigma_{pf} = 250$ MPa.



Sol^n:- Assume 12 mm thk. gusset plate.

Dia. of rivet $= 6\sqrt{10} = 18.97 \approx 20$ mm

∴ Gross dia $= 20 + 1.5 = 21.5$ mm.

① Member OB :-

Tensile load = 56 kN.

① Bearing strength $= \sigma_{pf} \times d \times t$

$= 250 \times 21.5 \times 8$

$=$ ~~25.8 kN~~ 43 kN

② strength of river in single shear $= \tau_{vf} \times \frac{\pi}{4} \cdot d^2$

$= 80 \times \frac{\pi}{4} \times (21.5)^2$

$= 29.04$ kN.

∴ Rivet value = 25.8 kN.

∴ No. of rivets reqd $= \frac{56}{29.04} =$ ~~2.17 ≈ 3 Nos.~~

$= 1.93 \approx 2$ No's.

② Member AD

① Bearing strength = $\sigma_{pf} \times d \times t$
= $250 \times 21.5 \times 10$
= 53.75 kN

② As double angle is provided,
∴ double shearing strength = $2 \times \frac{\pi}{4} \times d^2 \times \tau_{vf}$
= $2 \times \frac{\pi}{4} (21.5)^2 \times 80$
= 58.08 kN.

③ ∴ Rivet value = 53.75 kN
∴ No. of Rivets reqd = $\frac{(284-187)}{53.75}$
= 1.8
≃ 2 No's.

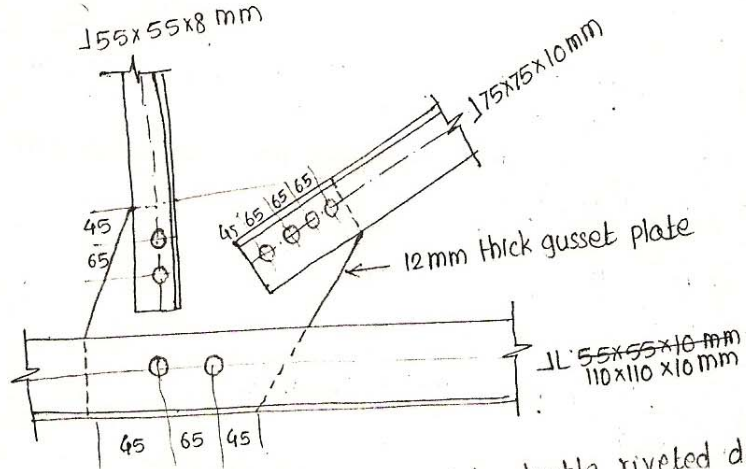
③ Member OC

① Bearing strength = $\sigma_{pf} \times d \times t$
= $250 \times 21.5 \times 10$
= 53.75 kN.

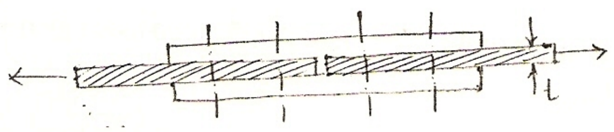
② Shearing strength = $\tau_{vf} \times \frac{\pi}{4} \cdot d^2$
= $80 \times \pi/4 \, (21.5)^2$
= 29.04 kN.

③ ∴ Rivet value = 29.04 kN.
∴ No. of rivets reqd = $\frac{112}{29.04}$ = 3.86 ≃ 4 No's.

* Adopt pitch = 3D = 3×21.5 = 64.5 mm ≃ 65 mm
Edge dist = 2D = 2×21.5 = 43 mm ≃ 45 mm.



Pro: Two plates 12 mm thick are joined by double riveted double cover butt joint as shown in fig. Using 20 mm dia. rivets, design the pitch of the rivets. Take σ_{at} = 150 MPa. Also find the efficiency of the joint.



Solⁿ:- Nominal dia = 20 mm
Gross dia = 21·5 mm

For power driven shop (pds) rivets $\sigma_{pf} = 300\,MPa$
$\tau_{vf} = 100\,MPa$

① strength of rivet in bearing $= \sigma_{pf} \times b \times d$
$= 300 \times 21.5 \times 12$
$= 77.4\,kN$

② strength of rivet in double shear $= 2 \times \tau_{vf} \times \frac{\pi}{4} \cdot d^2$
$= 2 \times 100 \times \frac{\pi}{4} (21.5)^2$
$= 72.6\,kN.$

∴ Rivet value = 72·6 kN.

For max. efficiency of joint per pitch length,=
strength of plate per pitch = 2 × Rivet value
∴ $\sigma_{at}\,(p-d) \cdot t = 2 \times 72.6 \times 10^3$
∴ $150\,(p - 21.5) \times 12 = 2 \times 72.6 \times 10^3$
∴ p = 102·16 mm
say 100 mm.

Min. permissible pitch = 2·5d = 2·5 × 21·5
= 53·7 mm < 100 mm

∴ Adopt p = 100 mm.

∴ Efficiency of joint $= \frac{150 \times (100 - 21.5) \times 12}{150 \times 100 \times 12} \times 100$
= 78·5 %.

* Welded Joint:

* Advantages of welded Joints:-
① As no holes are reqd, structural members are more effective in carring load.
② Overall wt. of structural steel reqd. is reduced.
③ Economical as less labour & matl. is reqd.
④ Looks better than rivet joint.
⑤ Faster process.
⑥ Any shape can be given.
⑦ End to end connectⁿ is possible.
⑧ Less working space is reqd.
⑨ Complete rigid joint can be provided; which can carry BM.
⑩ No noise is produced.

* Disadvantages:-
① Reqd. skilled labour & electricity.
② Internal stresses & warping are produced due to unever heating & cooling.
③ Brittle
④ Defects like internal air pockets, sag inclusion & incomplete penetration are difficult to detect.

Design of fillet weld :-

① Size of fillet weld :- The length of the sides of the largest right angled triangle in c/s of fillet weld is denoted as size of fillet weld.

Min. size of fillet weld according to IS : 800 -

Thk. of thicker part	min. size
≥ 10 mm	3 mm
10 mm - ≥ 20 mm	5 mm
< 20 mm ≥ 32 mm	6 mm
< 32 mm ≥ 50 mm	8 mm first run 10 mm min

Max. size of fillet weld :- Nominal thk − 1.5 mm → Square edge of a plate.
= 3/4 Nominal thk → Toe of an angle or rounded edge.

② Throat of fillet weld :- It is the length of ⊥ar from right angle corner to the hypotenuse.

Effective thk. of throat = K × Fillet ~~weld~~. size

Values of K :-

Angle betⁿ fusion faces	60 - 90°	91 - 100°	101 - 106°	107 - 113°	114 - 120°
Constant K	0.7	0.65	0.6	0.55	0.5

* Note :- min. angle = 60
Max. angle = 120.

From Survey Notes :- For well conditioned triangle angle should not be less than 60° or more than 120°. This principle is followed here for effective connectivity.

In most cases, right angle fillet is used.

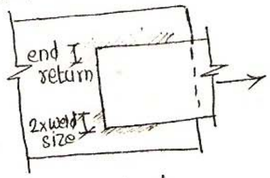
∴ Throat thk = $\frac{1}{\sqrt{2}}$ × fillet size = 0.707 × fillet size

③ Effective length of fillet weld :-

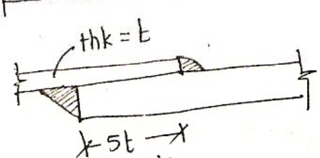
Effective length = overall length − 2 × weld size

Effective length = 4 × Fillet weld size → Load Transmiting weld
Eff. length

④ End Return :- Fillet weld terminating at end or side of a member should be returned around the corner whenever practicable for a distance not less than twice the weld size.

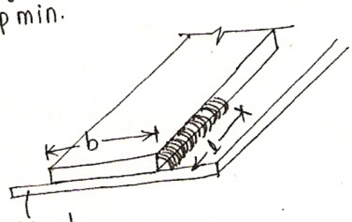


⑤ Overlap :- The overlap in a lap joint should not be less than 5 times thk. of thinner plate.



⑥ Side fillet :- In a lap joint made by a side or longitudinal fillet weld, the length of each fillet weld should not be less than ⊥ar dist. betⁿ them. (l ≮ b)

Max. ⊥ar dist = 16 × Thk. of thinner plate (b ≯ 16t)



(4) Intermittent fillet weld :- Intermittent fillet weld may be used when the length of smallest size fillet weld reqd. to transmit stress is less than the continuous length of joint.

: Any section of an intermittent fillet weld should have eff. length not less than 4x weld size or 40 mm, whichever is greater.

* clear spacing betn ends of eff. length of intermittent fillet weld :-

$\ngtr$ 12t ⇒ compression member

$\ngtr$ 16t ⇒ Tension member.

$\ngtr$ 200 mm ... any case.

where t = Thickness of thinner part jointed.

* Permissible stresses in fillet weld = 108 MPa.
* shear strength of fillet weld = $\boxed{P = Pq \cdot l \cdot t}$

Where P = strength of joint

Pq = Permissible stresses

l = eff. length.

t = throat thk. = k·s.

s = weld size

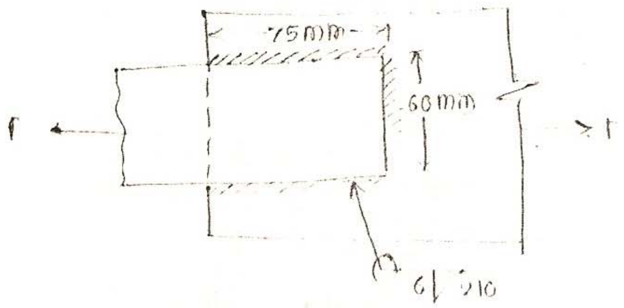
For most common case, welded surface meeting at 60-90° ∴ t = 0·7s.

$\boxed{P = 0.7 \times Pq \times l \times s}$

Per. stresses in shear & tension are reduced by 80% for field fillet weld

Per. stresses are increased by 25% if wind or earthquake load are considered.

Pro: 1] Find the safe load that can be transmitted by the fillet welded joint as shown. Size of weld = 6 mm



Soln:- Eff. length of weld = 75 + 60 + 75 = 210 mm.

∴ safe load = P = $Pq \times l \times 0.7 \times s$

$= 108 \times 210 \times 0.7 \times 6$

$= 95.26$ kN

Pro: 2] Design a suitable fillet weld to connect a tie bar 60x8 mm to 12 mm thk. gusset plate. The per. stresses in the tie bar & fillet weld are 150 MPa & 108 MPa resp.

Soln:- The joint is designed for full strength of tie bar.

∴ strength of tie bar = $150 \times 60 \times 8$ = 72 kN.

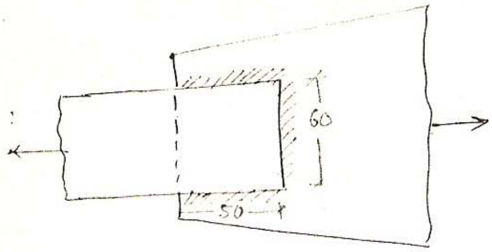
Max. size of weld = 8 - 1.5 = 6.5 mm

∴ Using 6 mm fillet weld,

eff. length of fillet weld reqd. = $\frac{P}{0.7 \times s \times Pq} = \frac{72 \times 10^3}{0.7 \times 6 \times 108} = 158.7$ mm.

∴ Provide l = 160 mm.

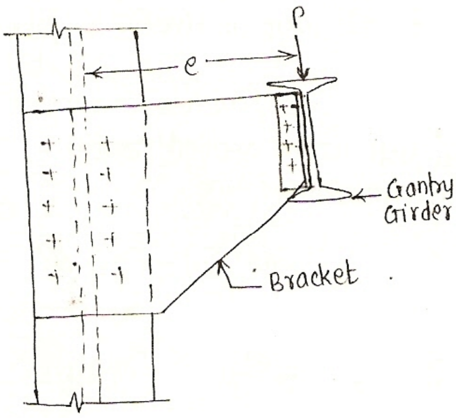
Alternatively,
weld could provided on both sides only for $\frac{160}{2} = 80$ mm.



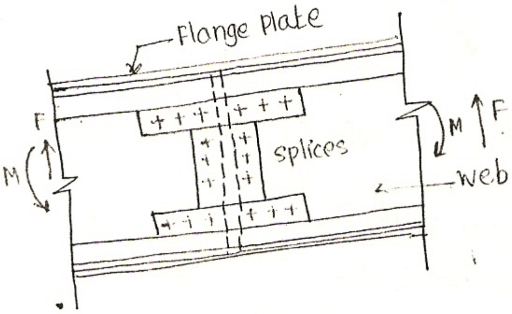
ECCENTRIC CONNECTIONS

If joint carries moments in addition to direct forces. In such cases the direct force do not pass through C.G. of joint. Therefore fasteners are designed to carry additional forces (tension or shear) due to bending.

Riveted Joints subjected to moment acting in the plane of Joint:-



Bracket Connection



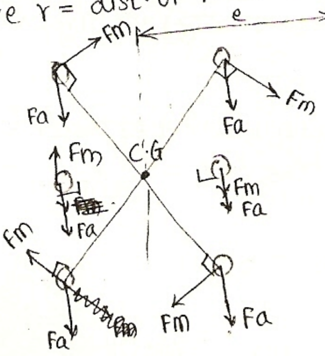
Web splice

consider a group of rivet. as shown in fig.
The rivets are to resist - ① Direct load = P
② Moment $= P.e = M$

① Force due to axial load on each rivet $= F_a = \frac{P}{\text{No. of Rivet}}$ (↓)(↓)

② Force due to moment M on any rivet, $= F_m \propto r$.

where r = dist. of rivet from C.G. of rivet group.



$\therefore F_m = k.r$
where k = constant.

Moment of force F_m @ C.G of rivet group = $F_m \times r = K \cdot r^2$

$\therefore \Sigma M @ C.G = \Sigma K \cdot r^2 = K \Sigma r^2$

$\therefore$ Resisting mmt = $\frac{F_m \cdot \Sigma r^2}{r}$

$\therefore$ equating this moment to external moment,

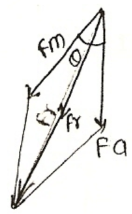
$$\therefore M = \frac{F_m \cdot \Sigma r^2}{r}$$

$$\boxed{F_m = \frac{M \cdot r}{\Sigma r^2}}$$

The direction of Fm is assumed ⊥ar to the line joining rivet & C.G. of rivet group.

$\therefore$ Resultant force Fr acting on a rivet is vectorial sum of Fa – due to direct load & Fm – due to moment.

$$\boxed{F_r = \sqrt{F_a^2 + F_m^2 + 2 \cdot F_a \cdot F_m \cdot \cos\theta}}$$



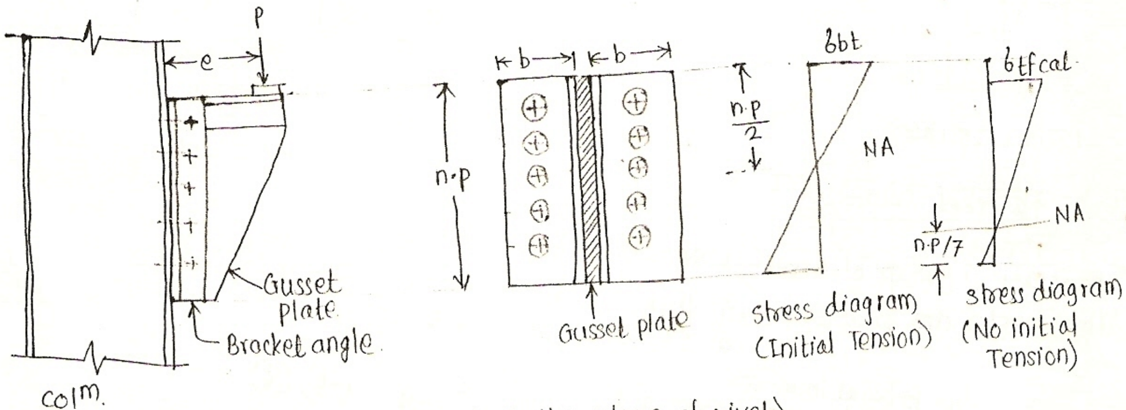
where θ = angle betⁿ Fa & Fm acting on rivet

$F_a = \frac{P}{n}$ (↓)

$F_m = \frac{M \cdot r}{\Sigma r^2}$ (⊥ar to the line joining C.G. of rivet group & rivet)

* **Rivets & bolts in Tension & shear:-**

Rivets are subjected to tension & shear both when applied moment is not in the plane of rivet, as in case of rivet connecting the bracket to the colᵐ flange as shown in fig.



Bracket type II (Moment is not in the plane of rivet)

The external force tends to rotate the connection about the NA. The location of NA depends on initial tension (if any) in rivet or bolt.

① **Initial Tension in rivets & bolts:-**

Hot driven rivets have initial tension when they cools & compress the connecting plates together.

If n = No. of rivets in each line.
m = No. of lines of rivets.
p = pitch of rivets.

$\therefore$ stresses due to moment at extreme fibre = $\sigma_{bt} = \frac{6M}{m \times b (n \times p)^2}$

$$\boxed{n = \text{No. of rivets} = \sqrt{\frac{6M}{m \cdot P \cdot R}}}$$ where $R = T_m$

∴ effective tensile load on extreme fastener = $T_m = \frac{6M}{m \cdot p \cdot n^2}$

② <u>No tension condition</u> :-

Applied moment M causes seperation of connected plates. The moment M is resisted by couple formed by the tension in the rivet above NA & comp. in connected plate below NA.

* <u>Permissible combined shear & Tensile stress</u>:-

$$\boxed{\frac{\tau_{vf\,cal}}{\tau_{vf}} + \frac{\sigma_{tf\,cal}}{\sigma_{tf}} \not> 1.4}$$

Where $\tau_{vf\,cal}$; $\sigma_{tf\,cal}$ = Calculate shear & axial tensile stress resp. in the fastener.

τ_{vf}, σ_{tf} = Allowable shear & axial tensile stress resp.

* <u>Design of eccentric connection</u>:

① <u>Eccentric load lying in the plane of Rivet : [Bracket Type I]</u>

① Assume suitable rivet dia.

② Find Rivet Value R < Shearing = $\tau_{vf} \cdot \frac{\pi}{4} \cdot d^2$; Bearing = $\sigma_{pf} \cdot d \cdot t$

③. If applied moment is small,

No. of Rivets reqd = $\frac{\text{direct load}}{\text{Rivet Value}} \times ③$ → for safty.

If applied moment is large,

No. of rivets reqd = $n = \sqrt{\frac{6 \times M}{m \cdot p \cdot R}}$

M = Applied moment
m = No. of lines of Rivets
p = pitch of Rivet
R = Rivet value.

④ check for stresses :-

a) Tensile stress at extreme rivet = $\sigma_{tf\,cal} = \frac{6M}{m \cdot p \cdot n^2 \cdot A_r}$

b) shear stress on each rivet = $\frac{\text{Direct shear load on each rivet}}{A_r = \text{Area of rivet}}$

check : $$\boxed{\frac{\tau_{vf\,cal}}{\tau_{vf}} + \frac{\sigma_{tf\,cal}}{\sigma_{tf}} \not> 1.4}$$

(II) <u>Eccentric load not lying in plane of Rivet : [Bracket Type II]</u>

① If fasteners have initial tension,

$$n = \sqrt{\frac{6M}{m \cdot p \cdot R}}$$

② If fasteners does not have initial tension,

$$n = 0.8\sqrt{\frac{6M}{m \cdot p \cdot R}}$$

Note :- Extreme rivets usually have max. stress. since $F_m = \frac{M \cdot r}{\Sigma r^2}$ & r is max. for them.

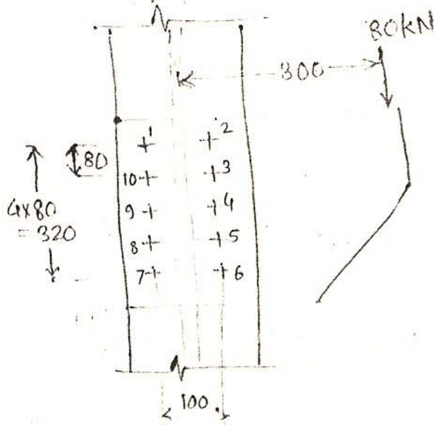
Also, angle betn Fa & Fm is minimum for extreme rivets.

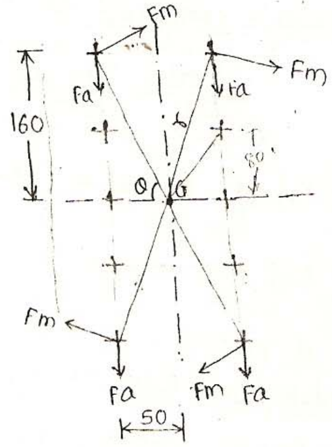
⇒ cos θ ⇒ More.

⇒ Fr = More.

Resultant. Stresses will be max.

Pro:- A bracket transmit a load of 80 kN at an eccentricity of 30 cm to a colm through 10 rivets of 22 mm dia. arranged in two vertical rows 10 cm apart. The pitch of the rivets is 8 cm & the load lies in the plane of rivets. Calculate the max. stresses in rivets.





Soln:- ~~Loes~~ load lies in plane of Rivet

∴ Bracket I type connection.

Gross dia = 22+1.5 = 23.5

As extreme rivels have max. stress ;

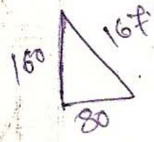
① Let $Fa = \frac{P}{n} = \frac{80\times10^3}{10} = 8\times10^3$ N.

$\theta = \tan^{-1}\left(\frac{160}{50}\right) = 72.65°$

$r = \sqrt{160^2+50^2} = 167.63$

$M = P\times e = 80\times10^3\times300 = 24\times10^6$

$\Sigma r^2 = 167.63^2\times4 + 4\times94.34^2 + 2\times50^2 = ~~148040.3~~ \ 153000$



② $Fm = \frac{M \cdot r}{\Sigma r^2} = \frac{24\times10^6\times167.63}{~~148040.3~~ \ 153000}$

$= ~~27.174\times10^3~~$ N

$= 26.295\times10^3$ N.

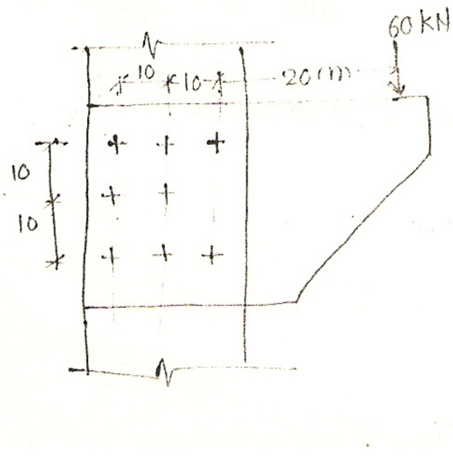
③ Resultant force = $Fr = \sqrt{Fa^2+Fm^2+2Fa\cdot Fm\cdot\cos\theta}$

$= \sqrt{8^2+26.295^2+2\times8\times26.295\times\cos(72.65)}$

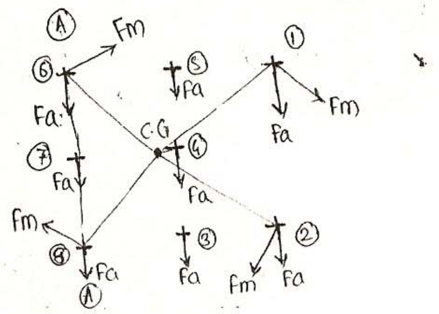
$= 29.68$ kN.

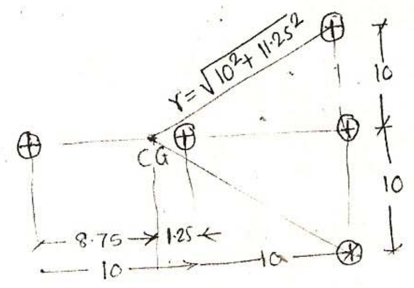
∴ Max. stress = $\frac{Fr}{c/s\ area\ (gross)} = \frac{29.68\times10^3}{\frac{\pi}{4}(23.5)^2} = 68.23$ MPa.

Pro 2] Determine the max load in the rivets of eccentric connection as shown:-



Soln:-





To locate C.G. Take moment of area @ A-A

$\therefore n \times \bar{x} = 10\times3 + 2\times20$ [n = No. of rivets = 8]

$\therefore \bar{x} = \frac{30+40}{8}$

$\therefore \bar{x} = 8.75$ cm from A-A

$n \cdot \bar{y} = 2\times10 + 3\times20$

$\therefore \bar{y} = 10$ cm.

① ∴ $Fa = \frac{\text{Direct load}}{n} = \frac{60}{8} = 7.5$ kN.

② $Fm = \frac{M \cdot r}{\Sigma r^2}$

In this case extreme rivets are ① & ②.

$r_2 = r_1 = \sqrt{10^2+11.25^2} = 15.05$ cm.

$r_3 = r_5 = \sqrt{1.25^2+10^2} = 10.08$ cm

$r_4 = 1.25$ cm

$r_6 = r_8 = \sqrt{8.75^2+10^2} = 13.29$ cm

$r_7 = 8.75$ cm

$\therefore \Sigma r^2 =$ ~~509.75 cm²~~ 1087.59 cm²

$M = 60 \times [20+10+10-8.75] =$ ~~1869~~ 1875 kN cm

$r_{max} = 15.05$ cm.

$\therefore Fm = \frac{1875 \times 15.05}{1087.59}$ (~~1869~~, ~~509.75~~) $\theta = \tan^{-1}\left(\frac{10}{11.25}\right) = 41.23°$

= ~~55.18 kN~~

= 25.95 kN.

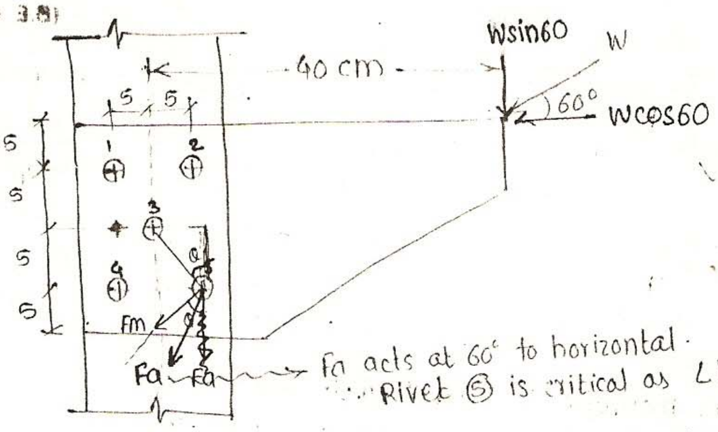
③ $Fr = \sqrt{Fa^2 + Fm^2 + 2\,Fa \cdot Fr \cdot \cos\theta}$

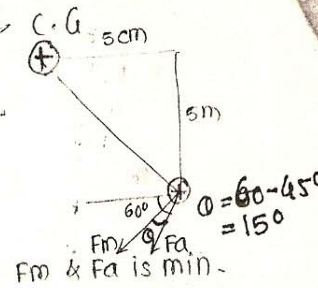
$= \sqrt{7.5^2 + 25.95^2 + 2\times7.5\times25.95\times\left(\frac{11.25}{15.05}\right)}$

= 31.95 kN

Pro: Find the max. load inclined at 60° to the horizontal, which the bracket as shown in fig, can transmit if 5-20mm dia. power driven shop rivets are used and the plates connected & are 10mm thick.

(Typical Problem)





Fa acts at 60° to horizontal.
∴ Rivet ⑤ is critical as ∠ bet^n Fm & Fa is min.

Sol^n:- Due to symmetry CG of rivet group is at rivet 3.

We resolve W in terms of $W\cos 60$ (⟷) & $W\sin 60$ (↓)

∴ ① $Fa = \frac{~~W\sin 60~~}{~~5~~}$ ~~= 0.17W~~ $\frac{W}{5}$ → Fa Resist direct load which may be inclined.

② $Fm = \frac{M \cdot r}{\Sigma r^2}$

$M = -(W\cos 60 \times 10) + W\sin 60 \times 40$

∴ $M = 29.64\,W$

$r_1, r_2, r_5, r_4 = \sqrt{5^2 + 5^2} = 7.07$ cm

$\Sigma r^2 = 4 \times 7.07^2 = 200$

∴ $Fm = \frac{29.64\,W \times 7.07}{200}$

$Fm = 1.05\,W$

③ $Fr = \sqrt{Fa^2 + Fm^2 + 2 Fa Fm \cos\theta}$ ~~cosθ = 5/7.07~~   θ = Angle bet^n Fa & Fm at rivet 5 = 15°.

∴ $= \sqrt{(0.20)^2 W^2 + (1.05)^2 W^2 + 2 \times 0.17 \times 0.05\, W^2 \times \cancel{5/7.07}\ \cos 15°}$

$=$ ~~$\sqrt{1.19 W^2}$~~ $1.24\,W$

$=$ ~~1.08 W~~

Equating rivet value with the max. force in member;

∴ Rivet value :- for 20-mm dia. pds rivets -

① In bearing $= \sigma_{bf} \times d \times t = 300 \times 21.5 \times 10 = 64.5$ kN

② In shear $= \tau_{vf} \times \frac{\pi}{4}(d^2) = 100 \times \frac{\pi}{4}(21.5)^2 = 36.3$ kN

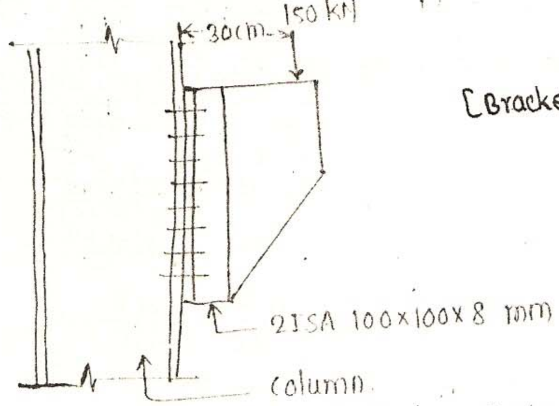
∴ ~~1.08 W = 36.3~~

∴ ~~W = 33.61 kN~~

∴ $1.24\,W = 36.3$

∴ **W = 29.17 kN**

Pro: Design the riveted connection joining the bracket angle 2 ISA 100×100×8 with the colm flange as shown:-



[Bracket Type II connection]

if 1) Rivets are power driven (hot) shop rivets
2) Rivets are power driven (cold) shop rivets. [See Book].

Soln:- case 1) Rivets are power driven (hot) shop rivets.
i.e. Initial Tension is considered.

Let $\sigma_{tf} = 100$ MPa
$\tau_{vf} = 100$ MPa

Using two rows ⇒ m = 2
Dia. of Rivet = 20 mm
∴ Gross dia = 20 + 1.5 = 21.5 ~~cr~~ mm
Pitch = 3d = 60 mm.

∴ ~~Ma~~ No. of rivets per line = $n = \sqrt{\dfrac{6M}{m \cdot p \cdot R}}$

Where M = applied moment = 150 × 30 = 4500 kN·cm
p = pitch = 60 mm = 6 cm
m = 2 = No. of rows.
R = Rivet value
$= \tau_{vf} \times \frac{\pi}{4}(d^2)$
$= 100 \times \frac{\pi}{4}(21.5)^2$
= 36.31 kN

$\therefore n = \sqrt{\dfrac{6 \times 4500}{2 \times 6 \times 36.31}}$
= 7.87
≈ 8 No's. → (NOTE) If odd fig. comes, make it even, so that pattern is symmetric

check for stresses:-

① Tensile stresses at extreme rivet = $\sigma_{tf\,cal} = \dfrac{6M}{m \cdot p \cdot n^2 \times A_r}$

$= \dfrac{6 \times 4500}{2 \times 6 \times 8^2 \times \frac{\pi}{4}(2.15)^2}$ cm

= ~~619.75~~ 9.68 kN/cm²
= ~~619.75~~ 9.68 N/mm²

② Direct shear load on each rivet $= \frac{150}{16}$ kN. ← Total No. of rivets $= m \times n = 2 \times 8 = 16$

∴ Shear stress on each rivet $= \tau_{vf\,cal} = \frac{\text{Direct shear load on each rivet}}{\text{c/s area of each rivet}}$

$$= \frac{150 \times 10^3}{16} \cdot \frac{1}{\frac{\pi}{4}(21.5)^2}$$

$$= 25.82 \text{ N/mm}^2$$

∴ Check ⇒ $\frac{\sigma_{tf\,cal}}{\sigma_{tf}} + \frac{\tau_{vf\,cal}}{\tau_{vf}} = \frac{96.8}{100} + \frac{25.82}{100}$

$$= 1.23 \not> 1.4$$

∴ O.K.

* **Butt Welded Joints loaded eccentrically**

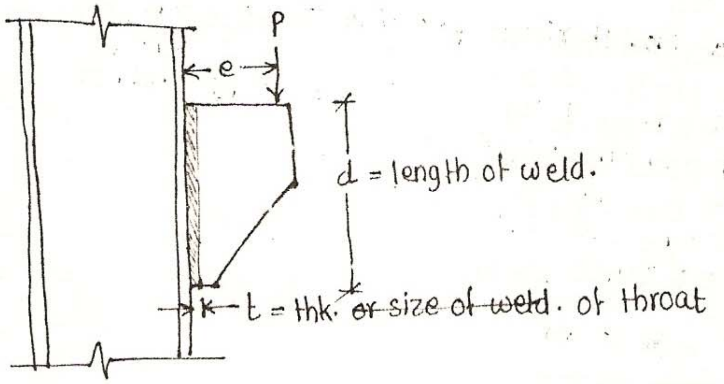
Consider a bracket connection to the flange of a col^m by a full penetration butt weld. A load 'P' is placed at an eccentricity of 'e' as shown.

Eccentric load causes :-

① shear force = P

② Bending moment = M = P×e

at the weld section.



[Bracket Type-II]

Let throat thickness = t

length of weld = d

① shear stress at weld $= \frac{P}{d \times t} = \frac{\text{Direct load}}{\text{Area.}}$ ⇒ $\boxed{P_s = \frac{P}{d \cdot t}}$

② Tensile or comp. stress due to bending at extreme fibre

$$P_b = \frac{M}{I} \times y = \frac{M \times d/2}{\frac{t \cdot d^3}{12}} = \frac{6M}{t \cdot d^2} \Rightarrow \boxed{P_b = \frac{6M}{t \cdot d^2}}$$

As per IS 800- if butt weld subjected to combined & shear & bending stresses, equivalent stress is,

$$\boxed{P_e = \sqrt{P_b^2 + 3 P_s^2}}$$

Per. value of equivalent stress is, $P_e = 0.9 f_y = 225$ MPa — 4.1.1.4

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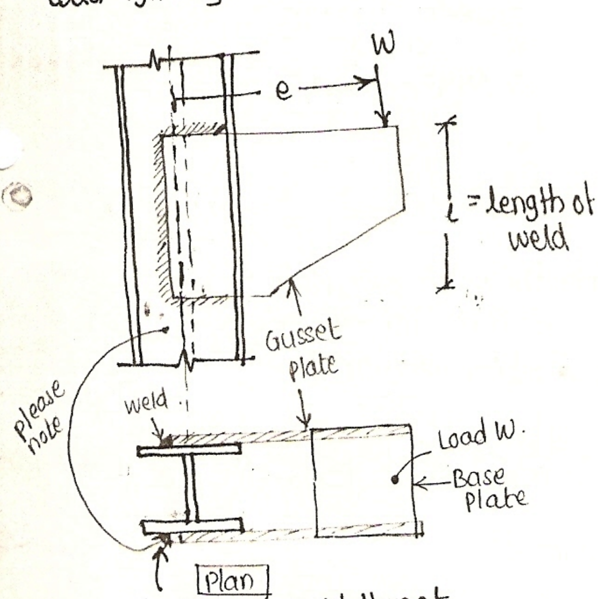
* Length of weld = $d = \sqrt{\dfrac{6M}{t \cdot P_b}}$

Where P_b = Permissible bending stress in the weld = 165 MPa.

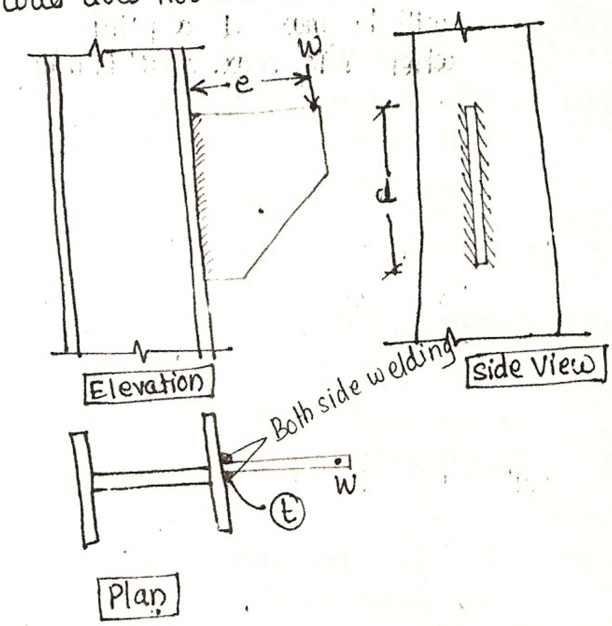
* Fillet welded Joint loaded eccentrically

Bracket Type I — Load lying in the plane of weld



t = Thk. of ~~weld~~ throat

Bracket Type – II — Load does not lies in the plane of weld.



Consider a bracket connection to the flange of colm by a fillet weld as shown in fig.

let l = Length of weld.
t = Thk. of ~~weld~~ throat.

Fillet weld is subjected to —
① Direct shear (load) (W)
② Torsional moment (M) = W·e

①∴ Vertical shear (stress) at weld;

$$P_s = \frac{W}{l \cdot t}$$

② Torsional (stress) due to moment; at "any point" in the weld;

$$P_b = \frac{M \cdot Y}{I_p}$$

Where M = Torsional moment = W·e

Consider a bracket connection to the flange of colm by fillet weld on both sides of the bracket as shown in fig.

Let t = throat thickness
l = Total length of weld = 2×d

Fillet weld is subjected to —
① Vertical shear (load) (W)
② Horizontal shear (load) due to bending at extreme fibre

① Vertical shear (stress) at weld;

$$P_s = \frac{W}{2d \cdot t}$$

② Horizontal shear ~~to~~ stress due to bending at extreme fibre;

$$P_b = \frac{M}{I} \cdot y = \frac{W \times e \times d/2}{2 \times t \times \frac{d^3}{12}} = \frac{6We}{2td}$$

r = Dist. of the "point" from C.G. of weld.

Ip = Polar MI of weld group = Ix + Iy.

∴ Resultant stress due to direct shear & torsional moment is,

$$P_r = \sqrt{P_s^2 + P_b^2 + 2 P_s \cdot P_b \cdot \cos\theta}$$

$P_r \ngtr$ Per. stresses in fillet weld = 108 MPa.

GATE (Note) Resultant stress P_r will be max. at a point wher r is max. & 'θ' is min.

[same as Riveted connection]

∴ Resultant stress;

$$P_r = \sqrt{P_s^2 + P_b^2}$$

$P_r \ngtr$ Per. shear stress in fillet weld = 108 MPa.

From design point of view,

depth of weld = $\sqrt{\frac{6 \times W \times e}{2 \times t \times p_q}}$.

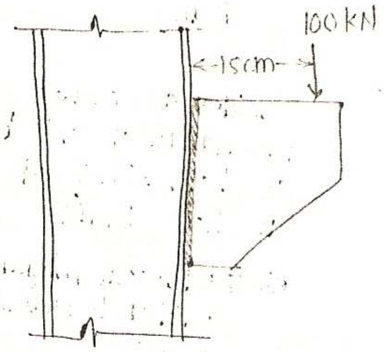
① 6 Ps
② Pb } Per. stresses.
③ Pr
④ Per. stresses.
⑤ design → n or d

Weld
① Fillet
② Butt
Rivet
I, II

Prob Design a butt weld to connect a 12 mm thick bracket plate to the flange of a colm. the bracket is to transform transmit a load of 100 kN at an eccentricity of 15 cm & steel conforms to IS : 226-1975.

Soln :- Use full penetration butt weld.

Permissible stresses in bending = P_b = 165 MPa.



① Approximate length of weld reqd = $d = \sqrt{\frac{6 \times M}{t \times p_b}}$

$= \sqrt{\frac{6 \times 100 \times 150}{12 \times 165}}$

= ~~2.13 cm~~ 213.2 mm

Try for depth of 22 cm.

② Shear stress at weld = $P_s = \frac{W}{d \times t}$

$= \frac{100 \times 10^3}{220 \times 12}$

$= 37.88 \text{ N/mm}^2$

③ Max. bending stress in the weld = $P_b = \frac{6M}{t \times d^2}$

$= \frac{6 \times 150 \times 100 \times 10^3}{12 \times 220^2}$

$= 154.96\ N/mm^2$

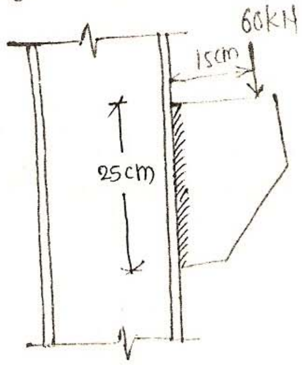
④ Equivalent stress, $P_e = \sqrt{P_b^2 + 3 P_s^2}$

$= \sqrt{154.96^2 + 3 \times 37.88^2}$

$=$ ~~159.5~~ $168.28\ N/mm^2 \not> 225\ MPa$

$\therefore$ O.K.

Pro: 2] Determine the size of fillet weld reqd. to join a bracket plate with the flange of colm as shown in fig.



Soln:- Let throat thk. = t mm.

① Max. shear stress at weld $= P_s = \frac{W}{2 \times d \times t}$

$= \frac{60 \times 10^3}{2 \times 250 \times t}$

$= \frac{120}{t}$ ~~(kN/cm²)~~ N/mm^2

② Horizontal shear stress due to bending at extreme fibre,

$P_b = \frac{6We}{2 \cdot t \cdot d^2} = \frac{6 \times 60 \times 10^3 \times 150}{2 \times t \times 250^2}$

$= \frac{432}{t}\ N/mm^2$

③ Resultant stress at extreme fibre;

$P_r = \sqrt{P_s^2 + P_b^2}$

$= \sqrt{\left(\frac{120}{t}\right)^2 + \left(\frac{432}{t}\right)^2}$

$= \frac{448.36}{t}$ MPa.

④ Since P_r = Permissibe stress = P_q

$\therefore \frac{448.36}{t} = 108$

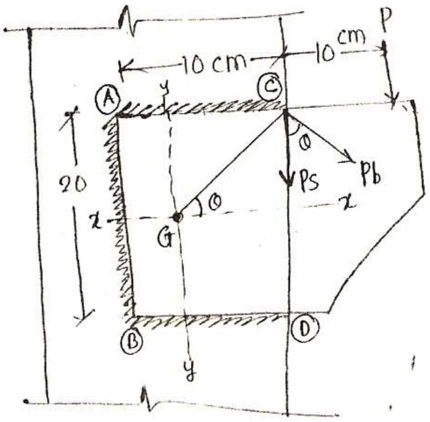
$\Rightarrow t = 4.15\ mm$

But $t = 0.707S$

$\therefore S = 5.87\ mm$

Provide size of weld = 6 mm

Pro 3] Fig. shows an eccentric welded connection with 6mm fillet welds. Determine the greatest load 'P' per bracket plate which can be applied on the connection if the shear stress in the weld is not exceed 108 MPa.



Soln :- To calculate C.G, taking mmt of weld sectn about line AB

$\therefore (20+10+10)\bar{x} = 10\times5 + 10\times5$

$\therefore \bar{x} = 2.5$ cm

$\bar{y} = 10$ cm

$\therefore e = 20 - 2.5 = 17.5$ cm

l = length of weld = 40 cm = 400 mm

t = throat thk = $0.707\times6 = 4.24$ mm

① Vertical shear stress due to load $P = \frac{P}{l\cdot t} = \frac{P}{40\times.424} = \frac{P}{16.96}$ kN/cm²

② Torsional stress due to momet at extreme point C & D = $P_b = \frac{M}{I_P}\times r$

where $M = P\times e = 17.5P$ kN·cm

$I_{xx} = A\cdot h^2 + \frac{b\cdot d^3}{12} = 2\times l\times t^2 + \frac{t\times l^3}{12}$

$= 2\times10\times0.42^2\times10^2 + \frac{0.42\times20^3}{12}$

$I_{xx} = 1120$ cm⁴ [Neglecting Mt of weld AC & BD]

& $I_{yy} = 2\left[\frac{bd^3}{12} - Ah^2\right]$

$= 2\left[\frac{0.42\times20^3}{12} - (0.42\times10)(5-2.5)^2\right]$

$= 175$ cm⁴

$\therefore I_p = I_{xx} + I_{yy} = 1295$ cm⁴

$\therefore P_b = \frac{P\times17.5\times12.5}{1295} = \frac{P}{5.92}$ (kN/cm²)

angle betn P_s & $P_b = \theta = \tan^{-1}\left(\frac{10}{10-2.5}\right) = 53.13°$

③ $\therefore P_r = \sqrt{P_s^2 + P_b^2 + 2P_s\cdot P_b\cdot\cos\theta}$

$= 0.21P$ kN/cm² $\cdot$ = Per. stress in weld = 108 MPa = 10.8 kN/cm²

$\therefore P = 51.4$ kN

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[Summary]

① Strength of Rivet [Axially load]

ⓐ In bearing = $\sigma_{pf} \times d \times t$ ⇒ $\sigma_{pf} = 250$ MPa
ⓑ In single shear = $\tau_{vf} \times \frac{\pi}{4} \cdot d^2$ ⇒ $\tau_{vf} = 80$ MPa
ⓒ In double shear = $2 \times \tau_{vf} \times \frac{\pi}{4} \cdot d^2$

} Rivet Value = Smallest of ⓐ, ⓑ, ⓒ

* No. of $(6\sqrt{t})$ dia. rivets reqd = $\frac{\text{Direct load}}{\text{Rivet Value}}$ ⇒ t = Thick. of thinner plate.

* Gross dia = effective dia = (Nominal dia + 1.5) mm
* Min. pitch = $2.5 \times d$
* Max. pitch = $16 \times t$ or 200 mm → Tension member } lesser value
  = $12t$ or 200 mm → comp. member
  t = Thk. of outside plate.

Max. pitch = $32t$ or 300 mm → any (Including tacking)

* Failure of rivet joint ⇒ ① Tearing of plate
② shearing of Rivet
③ Bearing of plate or rivet
④ Edge cracking → Prevented by edge dist. requirement.

② strength of welded Joint (Without eccentricity)

① Min. size of fillet weld :- t = 10 mm ⇒ 3 mm
t = 11-20 ⇒ 5 mm
t = 21-32 ⇒ 6 mm
t = 33-50 ⇒ 8 mm first run + 10 mm

② Max. size of fillet weld := (Nominal thk. − 1.5 mm) → sq. edge plate
= 3/4 Nominal thk. → rounded edge.

③ Throat thickness = $0.707 \times$ size of fillet weld.
④ Effective length of weld = l = overall length − $2 \times$ size of weld
= $4 \times$ Fillet weld size.

⑤ End return = $2 \times$ size of weld. (min.)
⑥ min. overlap = $5 \times t$ t = thk. of thinner plate.
⑦ clear spacing betn ends of eff. length of intermittent fillet weld
≯ 12t ⇒ comp. member
≯ 16t ⇒ Tension member
≯ 200 mm.
} Same as Pitch requirement in Rivet
t = thk. of thinner plate.

⑧ shear strength of fillet weld = $P = P_q \times l \times t$
P_q = Per. stress in fillet weld = 108 MPa.
80 % reduction for shear & tension field fillet weld.
i.e. $0.2 \times P_q$.

* Eccentric Riveted Joint (Bracket type 1.)
[Moment acting in the plane of Joint]

① Force due to axial load on each rivet = $F_a = \frac{P \; W}{\text{No. of Rivet}}$

② Force due to moment M on any rivet situated at farthest distance 'r' from CG of group of weld;

$$F_m = \frac{M \cdot r}{\Sigma r^2}$$

③ Resultant load = $F_r = \sqrt{F_a^2 + F_m^2 + 2F_a \cdot F_m \cos\theta}$

Where θ = Angle betⁿ Fa & Fm.

For calculating stresses; r ⇒ max, θ ⇒ min } select farthest rivet from CG such that angle betⁿ Fa & Fm is min.

⇒ Fr ⇒ Max.

* Bracket Type - II - Moment not acting in the plane of joint

Design Problem ⇒ Assume
$\sigma_{tf} = 100$ MPa
$\tau_{vf} = 100$ MPa
m = No. of rows = 2
d = 20 mm
pitch = 3d = 60 mm.

① No. of rivets per line = $n = \sqrt{\frac{6M}{m \cdot p \cdot R}}$ ← fastners have initial tension

$= 0.8\sqrt{\frac{6M}{m \cdot p \cdot R}}$ ← fastners does not have initial tension.

M = applied moment
m = No. of rows
p = pitch
R = Rivet value.

② check for stresses,

$$\frac{\sigma_{tf\,cal}}{\sigma_{tf}} + \frac{\tau_{vf\,cal}}{\tau_{vf}} \ngtr 1.4$$

$\sigma_{tf\,cal}$ = Tensile stresses at extreme rivet = $\frac{6M}{m \cdot p \cdot n^2 \times A_r}$

$\tau_{vf\,cal}$ = Shear stress in each rivet = $\frac{\text{Direct shear load on each rivet} = W/n}{\text{c/s area of each rivet}}$

* Design of Bracket Type - I

~~① Find C.G. of group of rivet~~ by taking moment at face of line of rivet.

① Assume suitable dia. of rivet.

② Find Rivet value.

③ If applied mmt is small, $n = \frac{3 \times \text{direct load}}{R}$

④ If applied mmt. is large; $n = \sqrt{\frac{6M}{m \cdot p \cdot R}}$

check for stresses. [same procedure as above]

① Butt welded Joint loaded eccentrically [Must be Type I]

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Design : ① Approx. length of weld reqd = $d = \sqrt{\frac{6M}{t \times p_b}}$

② Shear stress at weld = $P_s = \frac{W}{d \times t}$

③ Max. bending stress in weld = $P_b = \frac{6M}{t \times d^2}$

④ Equivalent stress = $P_e = \sqrt{3P_s^2 + P_b^2} \ngtr 225$ MPa.

② Fillet welded Joint loaded eccentrically [Type I] [Moment in the Plane of Joint]

① Vertical shear stress at weld = $P_s = \frac{W}{l \cdot t}$

② Torsional stress due to moment at any point 'r' from C.G. of weld situated at farthest dist = $P_b = \frac{M \cdot r}{I_P}$

M = Torsional mmt = W·e
r = Dist. of "critical" pt. from C.G. of weld.

I_p = Polar MI
= $I_{xx} + I_{yy}$

③ Resultant stress = $P_r = \sqrt{P_s^2 + P_b^2 + 2P_s \cdot P_b \cdot \cos\theta} \ngtr 2$

θ = ∠ betn P_s & P_b.
For $P_r \Rightarrow$ max,
r ⇒ max
θ ⇒ min.

③ Fillet welded joint loaded eccentrically, (Type II) Moment does not lies in the plane of joint

① Max. shear stress at weld = $P_s = \frac{W}{2 \times d \times t}$ as ~~two brac'~~ 2 lines of weld.

② Horizontal stress due to bending at extreme fibre,

$$P_b = \frac{6We}{2 \cdot t \cdot d^2}$$

③ Resultant stress at extreme fibre ;

$$P_r = \sqrt{P_s^2 + P_b^2} \ngtr \text{Per. stress} = 108 \text{ MPa}.$$