DESIGN OF STEEL STRUCTURES

1) Introduction
2) General Design consideration
3) Connection
4) Tension Member
5) Comp. Member
6) Beam
7) Gantry Girder
8) plate girder
9) Industrial building.

INTRODUCTION

Is codes

IS 800 : 2007 → General construction in steel
(With abendment no. 1, ~~Generally~~ January 2012)

IS 808 : 1989 → Dimensions for steel sections
(Steel Tables)

Steel :- It is an alloy of Iron having carbon content b/w $\boxed{0.1 - 1.1\%}$

- Based on carbon content, 3 types of steel.

✓ ① low carbon steel → (0.1 – 0.25 %) carbon.
② Medium carbon steel → (0.25 – 0.60 %)
③ High carbon steel → (0.6 – 1.1 %)

Deoxiders such as silicon or Aluminium are used to control dissolved oxygen during the Manufacturing process

- lower % of oxygen content is good for durability (Air in voids will be less) of steel and on the Basis of oxygen content we classify steel as

✓ (i) Killed Steel → (< 30 ppm oxygen)
(ii) Semi-Killed → (30 – 150 ppm)
(iii) Rimmed steel → (> 150 ppm oxygen)

Structural steel are generally killed or semi-killed. Carbon % in structural steel is generally < 0.25 %. (Low carbon steel) → Ductility ↑

Mild steel has a carbon content of nearly 0.1 %.

IS 800:2007 can be used for structural Mild steel or High Tension structural Steel.

Various Grades of Steel :-

Grade	Ultimate stress (MPa)	Yield Stress (MPa)
✓ E 250 (Fe 410) A	410	250
B	410	250
C	410	250
E 300 (Fe 440)	440	300
E 350 (Fe 490)	490	350
E 410 (Fe 540)	540	410

E 250 (Fe 410) A → Grade of Steel

E → char. yield stress

Fe → char. ultimate stress

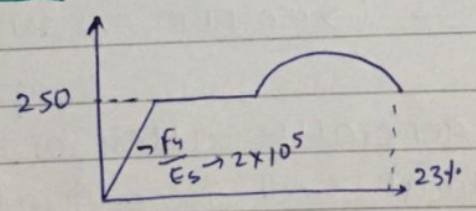
Fe 410 W → Weldability Higher

Fe → char. ultimate stress

NOTE ① Structural steel is specified according to charecteristic ultimate Tensile stress i.e f_u

It is the ultimate stress below which not more than 5% of the materials are expected to fall.

② RIF Bars in RCC are specified according to yield stress



③ Thinner the sec^n, Higher is the strength due to Higher amount of rolling, cold working, uniform rate of cooling etc.

Ex.

	$t < 20$ mm	$20 < t < 40$ mm	$t > 40$ mm
E 250 (fe410)	$f_y = 250$ MPa	240 MPa	230 MPa

$t \rightarrow$ कम, Rolling बढ़ाई, strength $\rightarrow$ बढ़ाएगा

(4) Brittle fracture occurs due to higher Tensile Stress, lower temp., thicker material, Rapid change of stresses etc.

(5) Stainless steel is low carbon steel with around 10.5% cromium by weight. strain $\rightarrow$ कम, Ductility ↑ corrosion ↓

Grade A is used for Non-critical applications i.e Normal structure when members are not prown to brittle fracture

Grade B is used for critical applications when Temp. doesn't fall below 0°C and when parts are prown to Brittle fracture or Fluctuations of stresses as in case of Bridges

Grade C has a Gurenteed low temp. upto −40°C and it shall be used for impact loading and higher chances of Brittle Fracture Ex. Ship, sea, Marine

physical properties of steel (for all Grades)

(1) Density of steel $\rightarrow$ $\rho = 7850$ kg/m^3

(2) Modulus of Elasticity $\rightarrow$ 2×10^5 MPa

(3) poisson's Ratio $\rightarrow$ $\mu = 0.3 \rightarrow$ Elastic zone
$\mu = 0.5 \rightarrow$ plastic zone

(4) shear Modulus $\rightarrow$ $G = \frac{E}{2(1+\mu)} = 0.769 \times 10^5$ MPa (Elastic zone)

(5) specific Gravity $\rightarrow$ 7.85

(6) cofficient of thermal expansion $\rightarrow$ $\alpha = 12 \times 10^{-6}$ /°c

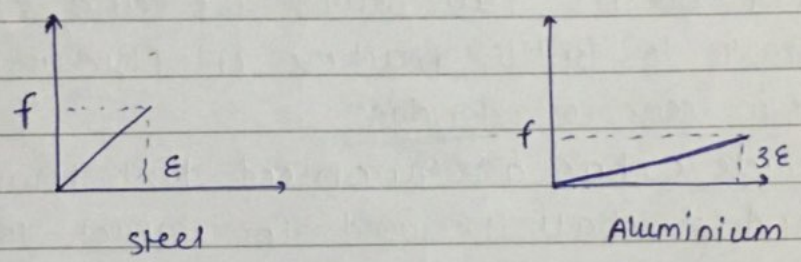
* Advantage of Steel as a Structural Material

1. High strength per unit weight
2. High Ductility, High toughness
3. uniformity ie very less quality control issues
4. Environment friendly, can be 100% recycled
5. Easy connections and faster constructions
6. Easy repair and Modifications
7. longer life is properly maintained.

* Disadvantages

1. Higher Maintainence due to corossion
2. Fire proofing cost
3. prown to buckling due to longer and slender member → (can't used more than 50% strength)
4. Fatigue

Aluminium

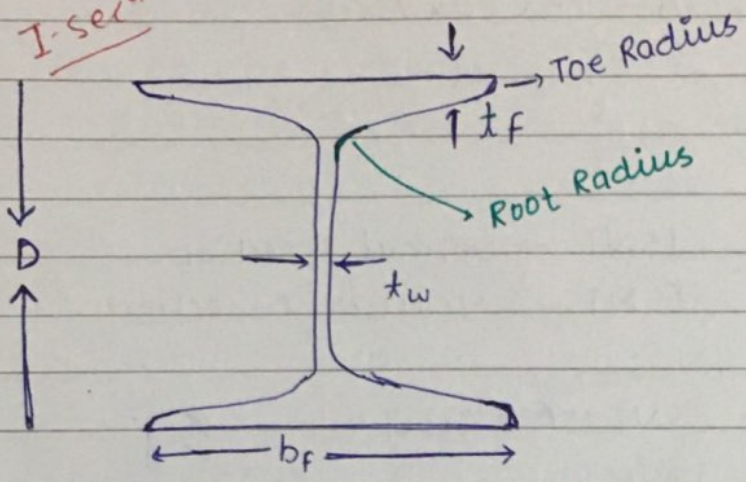


- Higher Strength to unit Wt. Ratio compare to steel
- However due to lower Modulus ($1/3^{rd}$ of steel) Bigger sec^n are required to avoid buckling
- Greater Resistance to corossion and Hence less maintainence
- Density ≈ $1/3^{rd}$ of steel. (2700-2800 kg/m^3)
- Cofficient of thermal expansion ≅ 2 × steel = 23×10^{-6} /°C
- less Ductile than Mild steel
- Doesn't have a well defined yield point and hence yield is assumed @ 0.2% proof strain → (stress) corresponding
- As it's Modulus is $1/3^{rd}$ of steel, it can absorb 3 times the energy @ same stress level compare to steel Member of same dimension provided the stress doesn't exceed the proportionality limit, Hence also used for impact loading provided Higher Deflection is allowed.

• Design concept same as steel structure.

Standared structural steel sections :-

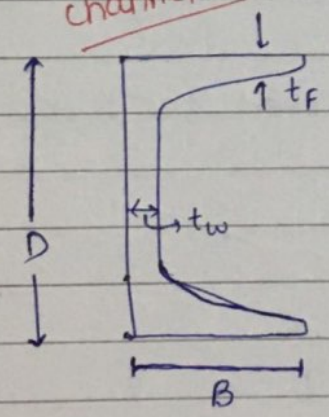
I-sec^n



ISJB → Indian standared junior Beam
ISLB → IS Light Beam
ISMB → IS Medium Beam
ISHB → IS Heavy Beam
ISSC → IS column section (Type of I section)

Ex: ISMB 300 → D = 300 mm , B/b_f = 140 mm
t_f = 13.1 mm , t_w = 7.7 mm

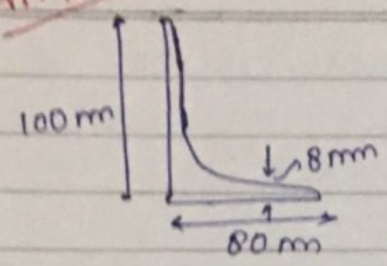
channel sec^n



ISJC → Junior channel
ISLC → Light channel
ISMC → Medium channel

Ex: ISMC 100 → D = 100 mm , B = 50 mm
t_f = 7.7 mm , t_w = 5 mm

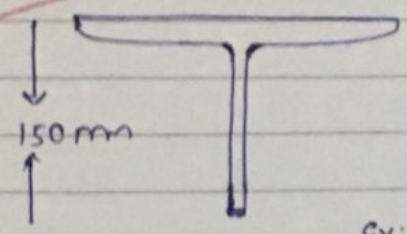
Angle secⁿ



ISA → IS equal / unequal Angle

ISA → 100 x 80 x 8

T-secⁿ



ISNT → Normal T-section

ISMT → Medium T-section

Ex: ISNT 150 @ 223.7 N/m

ISRO → Round Bars

Ex: ISRO10 (dia = 10 mm)

ISSQ → square Bars

Ex: ISSQ 10 (10 mm = side)

ISPl → plate

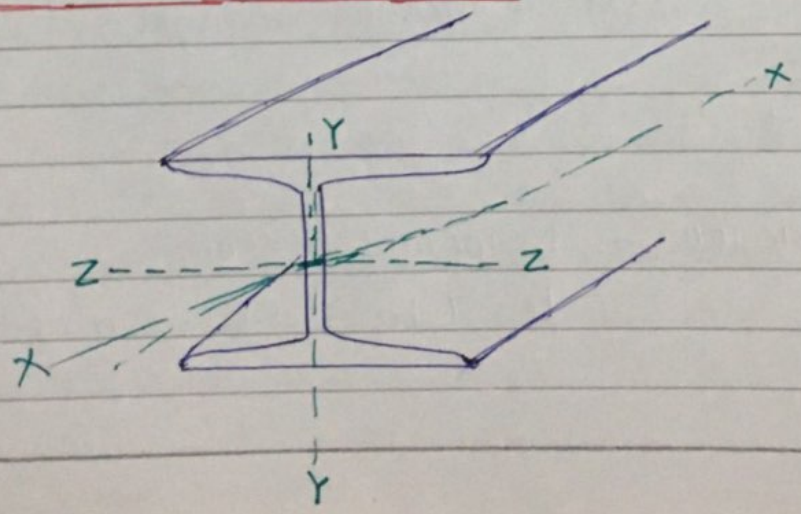
Ex: ISPl2000x1000x8 (lxbxt)

ISFl → Flat secⁿ

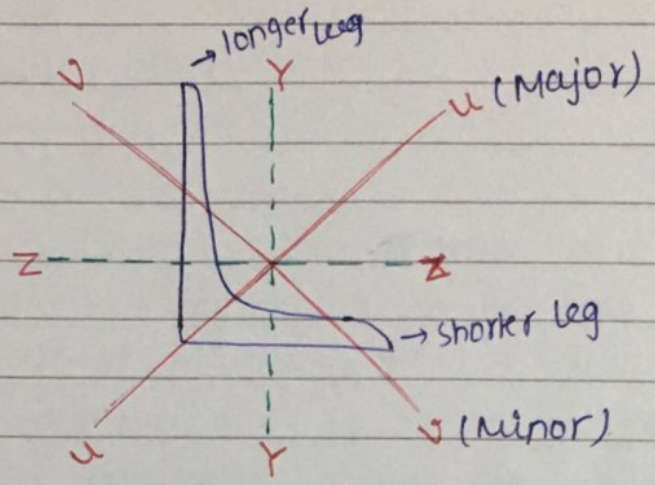
Ex: 30 ISF10 → (30 mm → width, 10 mm → thickness)

These secⁿ can be used either alone (Rolled secⁿ) or in combinations (built up secⁿ)

Convention for Member Axes :-



along length → X-X
parallel to Flange → Z-Z (Major)
⊥ to Flange → Y-Y (Minor Axis)



parallel to smaller leg → Z-Z (higher Inertia)
⊥ to smaller leg → Y-Y (lesser Inertia)
in Earlier version of the IS code i.e IS800 1984
Major Axis was denoted as X-X

GENERAL DESIGN consideration

Structure shall fulfill safety, serviceability, Economy, Aesthetic and environmental criteria.

3. Design Methods :-

(i) Elastic / WSM
(ii) plastic / ULM
(iii) LSM

(i) WSM

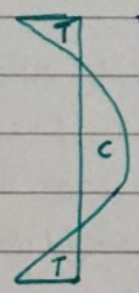
It assumes linear elastic Response and safety is ensured by ensuring working stress to be less than permissible stress.

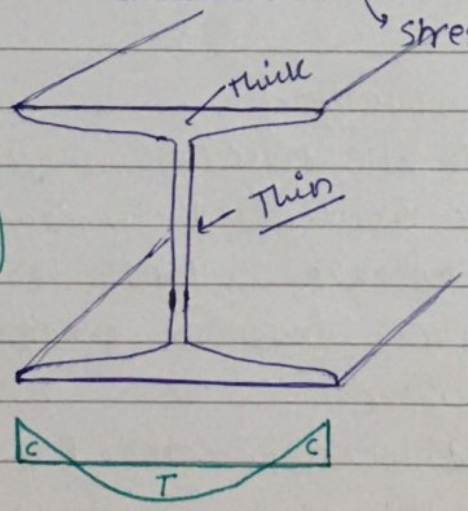
(strength of Material / FOS) = f_y/FOS

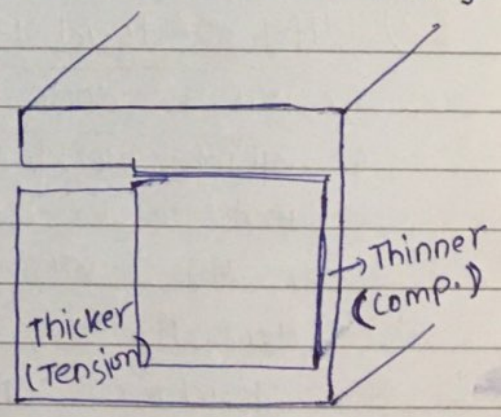
However assumptions of stress being less than permissible stress is not realistic because of stress concentration. and long term effect of creep & shrinkage, Residual stress and other secondary stresses.

stress concentration near opening.

NOTE







Flange की length ज्यादा रहेगी, web की कम due to comp. in web

- Unequal Rate of cooling due to diff^n thickness and exposed area and uneven comp. by roller will lead to generation of residual stress. The part of sec^n which cools first will have comp. because it will resist the shortening of slower cooling part. Also the slower cooling part will have Tension

- Does not utilise the reserve strength derived from ductility and redistribution of stress and hence FOS doesnot give a realistic figure or exact Margin of safety

- FOS does not have a scientific Basis and is based on experience.

- It fails to discriminate b/w diff^n types of load that are simultaneously acting but have varying degree of uncertainity.

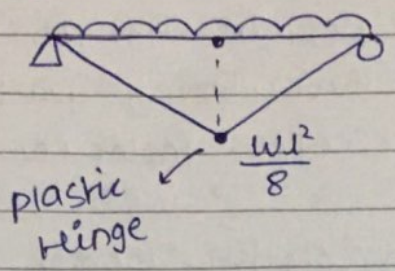
② ULM or load factor Method

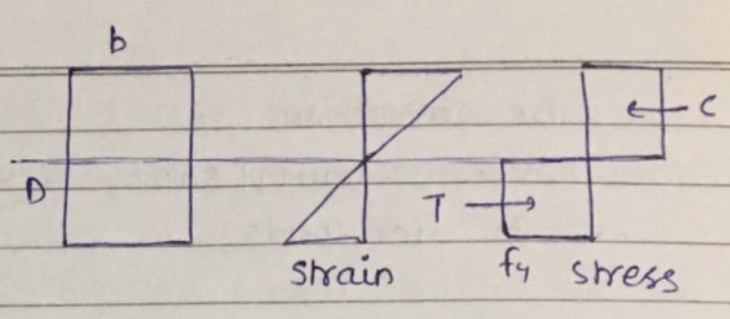
Design is done as in the case of plastic analysis in which working load multiplied by load factor is ensured to be less than the collapse load or ultimate load. However this Method does not consider serviceability cond^n like deflection, vibration etc

Also structure subjected to impact and Fatigue loading shall not be design with plastic theory. As it uses full Material strength beyond elastic limit

- Also safety for material is not considered Hence this gives smaller c/s than WSM.

② ~~LSM~~:-



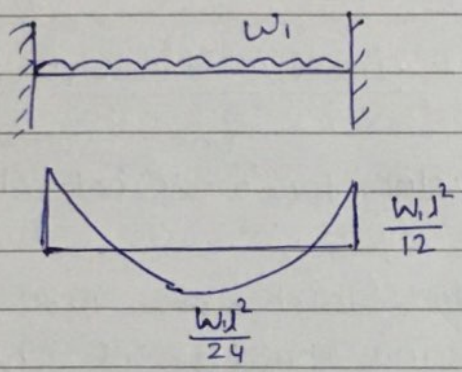


$$C = T = f_y \cdot \frac{bD}{2}$$

$$MOR = f_y \cdot \frac{bD^2}{4} = f_y \cdot Z_p$$

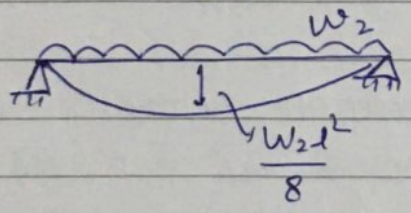
$Z_e \rightarrow$ elastic Modulus
$Z_p \rightarrow$ Plastic Modulus

$$M_e = f_y \cdot Z_e = f_y \cdot \frac{bd^2}{6}$$



$$M_p = f_y \cdot \frac{bd^2}{4}$$

$$\frac{W_1 l^2}{12} = M_p \quad \text{(Hinge @ ends)}$$



$$\frac{W_2 l^2}{8} = \frac{M_p}{2}$$

$$\Rightarrow \boxed{W_1 + W_2 = \frac{16 M_p}{l^2}}$$

③ LSM:- To avoid all deficiencies of WSM & plastic Method, LSM was proposed. partial safety factors are used for load and Material strength both based on acceptable probability of failure derived using realiability analysis (level 1)

- partial safety factor takes into account possible overloads and understrength.

Design action (considering partial safety factor for loads) $\leq$ Design strength (considering partial safety factor for Materials)

- Factored load / Design load (f_d) = charecteristic load x γ_f

$\gamma_f \rightarrow$ partial safety factors for loads. depending on loads combination and limit state being considered.

- Design strength (f_d) = $\frac{\text{charecteristic strength}}{\gamma_m}$

$\gamma_m \rightarrow$ partial safety factor for material strength.

* charecteristic load is the load which has 95% probability of not being exceeded during the life of the structure

* charecteric strength is the strength below which not more than 5% of test samples are expected to fall.

γ_f accounts for ($\gamma_f \rightarrow$ partial safety for loads)

(i) possibility of load exceeding charecteristic load
(ii) possibility of inaccurate assessment of load
(iii) Uncertainity in assessment of effect of load.
(iv) Uncertainity in the assessment of limit state being considered

γ_m accounts for ($\gamma_m \rightarrow$ partial safety factor for Material)

(i) possibility of strength falling below charecteristic strength
(ii) Reduction in Member size due to faulty construction
(iii) Reduction in strength due to fabrication & Tolerences.
(joint Match करने पर)

(iv) uncertainity in theoritical assumptions.
(v) uncertainity in the calculation of strength of Member

Limit states are the states beyond which the structure becomes unfit for use.
The limit states are classified as —
1) limit state of strength / ultimate limit state
2) limit state of serviceability

Limit state considered by IS code.

① Limit state of strength

1) strength including yielding, buckling and transformation into a Mechanism (plastic Hinge formation)
2) stability against overturning and sway
3) failure due to excessive deformation or Rupture
4) Fracture due to fatigue
5) Brittle fracture.

② Limit state of serviceability

(i) deformation & deflection (can cause damage to non-structure component and finishes but not to structural component)
2) vibration (causing discomfert)
3) repairable damage or crack due to fatigue
4) corossion, durability
5) Fire resistance

Limit state Method consider safety @ Ultimate load and serviceability @ Working load.

NOTE In practice structures are designed for critical state and check for other state However in most of cases ultimate limit state governs the design

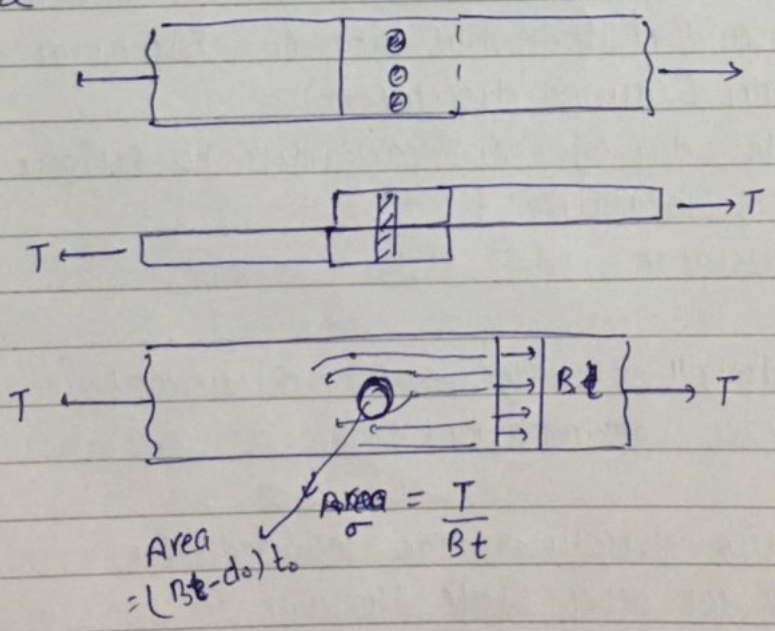
partial safety factor for material (γ_m)

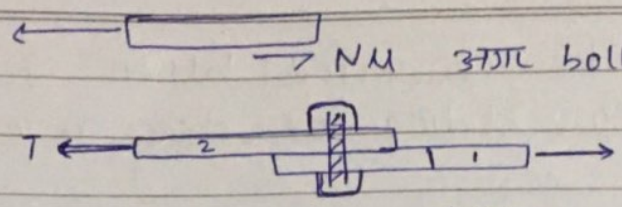
Defination	γ_m
(i) Resistance Governed by yielding (γ_{m0})	1.10
(ii) Resistance Governed by buckling (γ_{m0})	1.10
(iii) Resistance Governed by ultimate stress (γ_{m1})	1.25

For connections	Shop Fabrication	Field Fabrication
Bolt - Bearing type (γ_{mb})	1.25	1.25
Bolts - friction type (γ_{mf})	1.25	1.25
Rivets (γ_{mr})	1.25	1.25
Welds (γ_{mw})	1.25	1.5

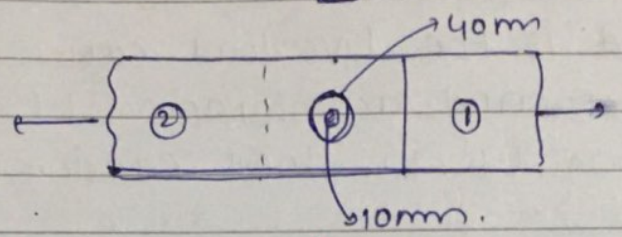
We generally consider Rupture in Net Area and yielding in Gross sec^n

Yielding is not considered @ Net sec^n i.e @ joint location as some yielding @ joint is allowed
Basically We need to check excessive deformation and Rupture



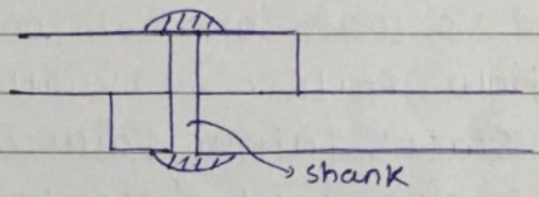


NM अगर bolt, hole से बड़ा हो.



अगर Hole, bolt से बड़ा हो
— shear से force transfer

Rivet



Partial safety factor for load (γ_f)

Combination	Limit state of strength					Limit state of serviceability			
	D.L	L.L		E.Q/W.L	A.L	D.L	L.L		E.Q/W.L
		Leading	Accom.				Leading	Accom.	
DL+LL+CL	1.5	1.5	1.05	—	—	1	1	1	—
D.L+L.L+C.L	1.2	1.2	0.53	1.2	—	1	0.8	0.8	0.8
same + EQ/W.L	1.2	1.2	1.05	0.6	—	1.0	0.8	0.8	0.8
DL+EQ/WL	1.5 or 0.9	—	—	1.5	—	1	—	—	1
DL+ER	1.2 or 0.9	1.2	—	—	—	No serviceability check required			
DL+LL+AL	1.0	0.35	0.35	—	—				

C.L → crane load

A.L → Accidental load

ER → Errection load

NOTE 0.9 value shall be considered when stability against overturning, sliding or stress reversal is critical

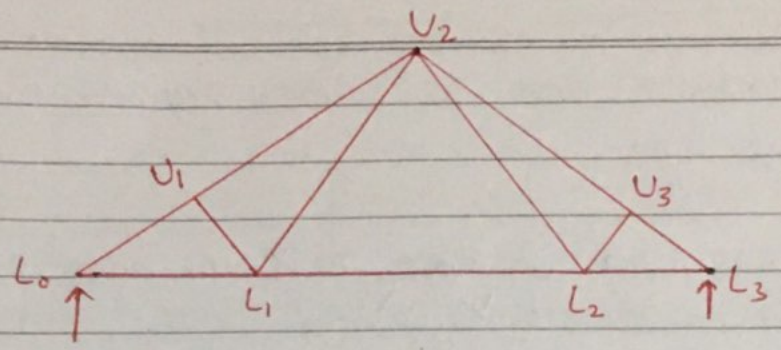
- leading live load is the live load causing the higher load effects and acommpaning L.L is the L.L except leading L.L i.e load causing lower effects.
- Impact, collissions, fire etc can be called as Accidental load.
- Wind load and EQ load are not considered simultaneously
- Although fatigue failure is usually considered as Ultimate limit state, fatigue failure design is carried out @ working load because in case of fatigue load applied is normal load or charecteristic load with higher no. of repeatations.

The charecteristic load require statistical Data in the absence of which the load can be considered from different IS code.

IS 875 (part-1) → D.L
(part-2) → L.L
(part-3) → W.L
(part-4) → snow load
(part-5) → Erection load.

IS 1893 (part-1) → EQ load

Que.



Find the design comp. and tensile force for the principle Rafter.

Member	DL(kN)	LL(kN)	WL(kN)
L_0U_1	-9.5	-9.5	32.5
U_1U_2	-8.0	-8.0	32.5
U_2U_3	-8.0	-8.0	32.5
U_3L_3	-9.5	-9.5	32.5

Member	1.5(DL+LL)	0.9 DL + 1.5 WL	1.2 (DL+LL+WL)
L_0U_1	-28.5	40.2	16.2
U_1U_2	-24	41.5	19.8
U_2U_3	-24	41.5	19.8
U_3L_3	-28.5	40.2	16.2

Design comp. force = -28.5 kN

Design Tensile force = 41.5

Permissible stress

Axial Tension | comp. = $0.6 f_y$

Bending Tension | comp. = $0.66 f_y$

in shear = $0.4 f_y$

in Bearing = $0.75 f_y$

} Structure

WSM

LSM → $\frac{f_y}{1.1}$ → all

$\frac{f_y}{\sqrt{3} \times 1.1}$ → shearing.

$\frac{f_y}{\sqrt{3} \times 1.25}$ → Bolt or Weld

BOLTED CONNECTIONS

Depending upon Rigidity of joint connections are classified as:-

① **Rigid connection / Moment connection :-**

Develop the full or significant Moment capacity of connecting Members and retain the original angle b/w the members

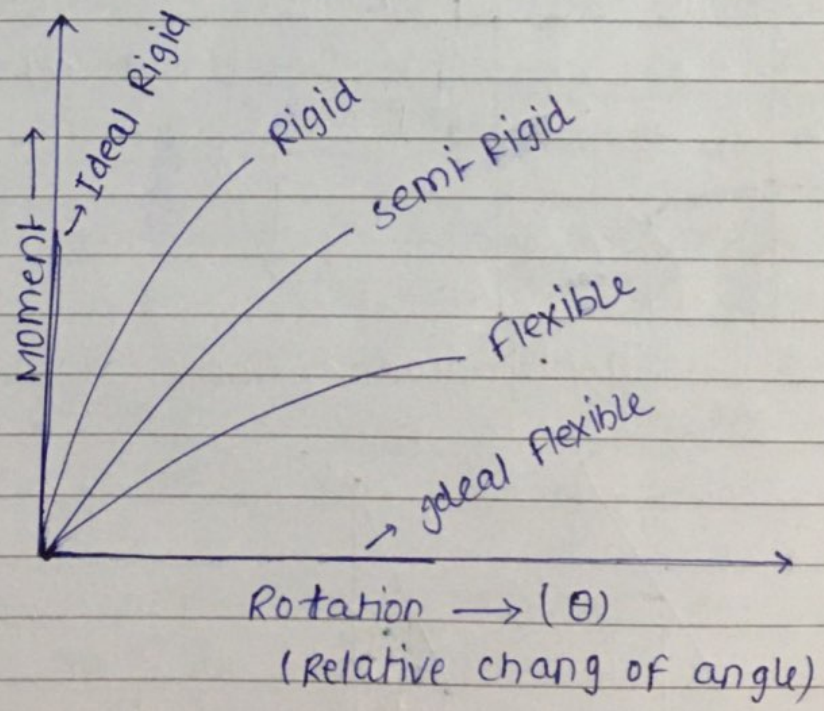
② **Simple / Flexible connection**

No moment transfer is considered b/w the connected parts and hence joints are assumed to be pin or Hinge connection

③ **Semi- Rigid connection**

Does not have sufficient Rigidity to hold the original Angle b/w the members and develop less than full Moment capacity of the connected Members.

NOTE In Reality all the connections are Semi- Rigid. However for convinience we assume either Rigid or simple conne.

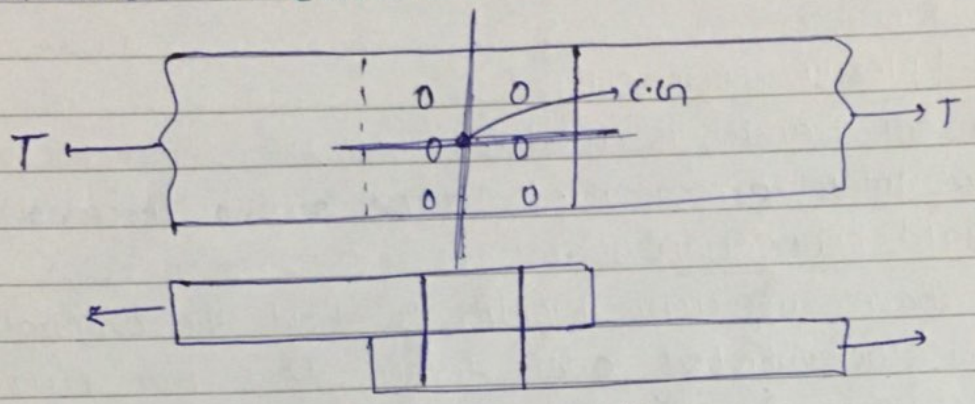


Also connections can be classified as -

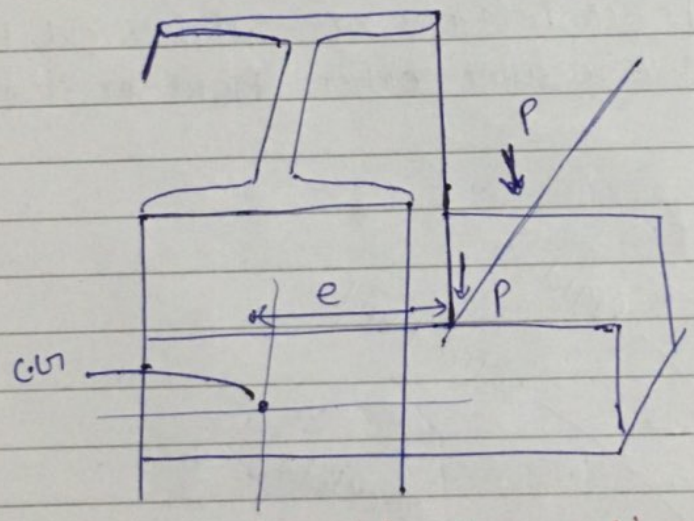
A) Direct connection B) Eccentric connection.

if the resultant load passes through the c.G of connection ⟶ Direct connection

If the resultant load does not pass through the c.G of connection ⟶ Eccentric connection.

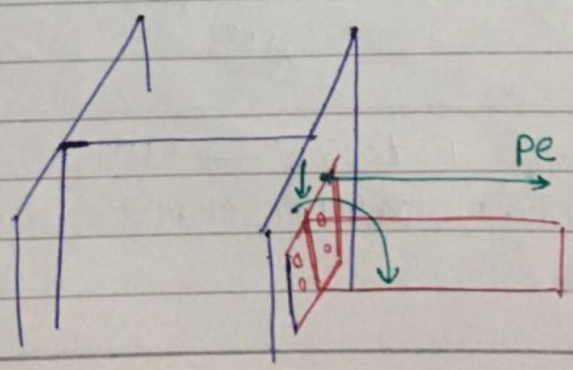


Direct connection.



Direct shear + Twisting Moment

Eccentric connection



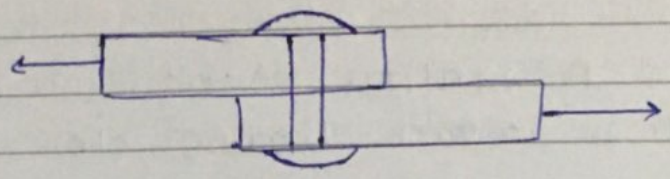
Direct shear + BM

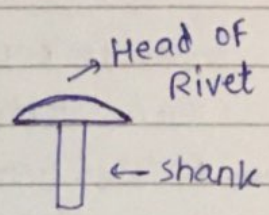
Eccentric

Various type of connections –

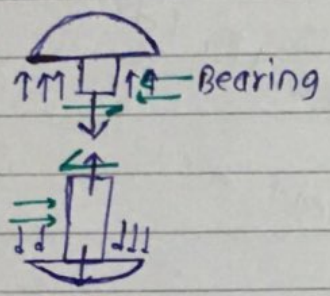
① Rivet
② Bolted
③ Welded

Rivet





Rivets are inserted in the hole made to join the member Hammering is done to make head on the other side



Rivetting → Hot rivetting (around 980°c)
Rivetting → cold rivetting

Rivetting → Field Rivetting
Rivetting → Shop Rivetting

The Nominal dia of Rivet is the shank dia under cold temp. and the Gross Dia of Rivet is taken as the Dia of Hole.

It is assumed that during Hammering action the Rivet hole is completely filled with the material so from strength calculation point of view Diameter of Rivet is taken as Diameter of Hole. and it is called Gross Dia of Rivet

For ease in construction. Rivet holes are generally made larger than the shank dia.

$(d \le 25mm)$

Gross Diameter = Dia of hole (d_0) = $d + 1.5$ mm.

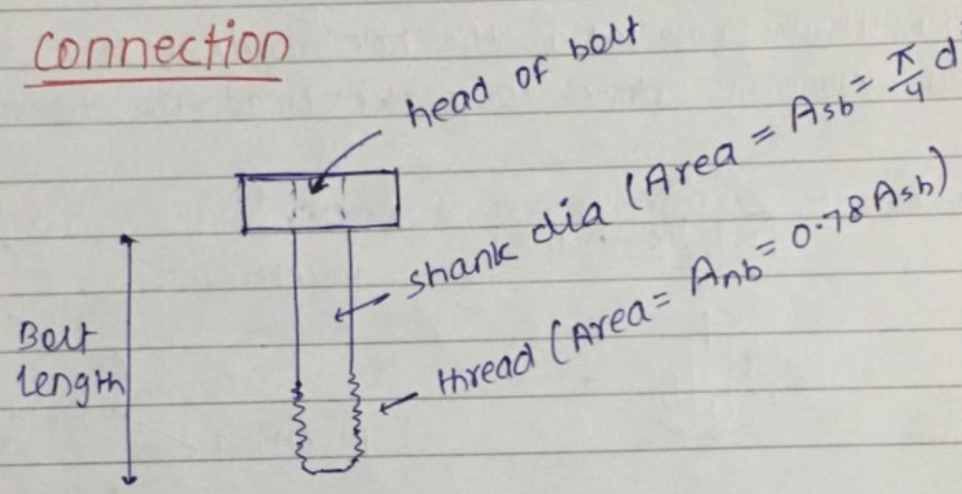
$= d + 2$ mm $(d > 25mm)$

$d \rightarrow$ nominal dia of Rivet

All formulas of Bearing type bolt are applicable for Rivets.

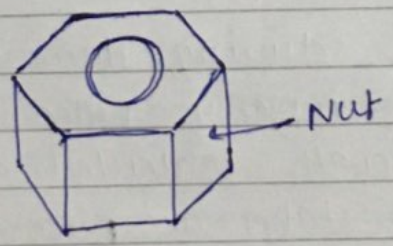
Rivets are good for fatigue loading, alternate stress, Dynamic loading, impact loading etc.

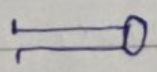
Bolted connection



$A_{sb} \rightarrow$ shank dia of bolt

$A_{nb} \rightarrow$ Net area of bolt





cotter Pin

Type of Bolt

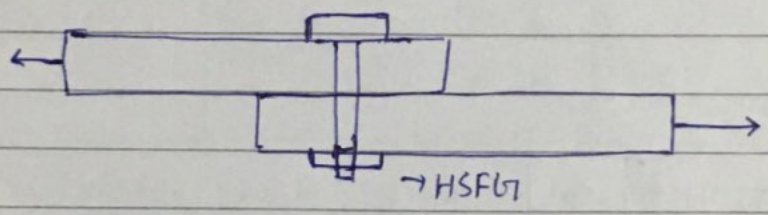
1. Black bolt / unfinished / ordinary / Rough / common Bolt
2. Turned bolt / close Tolerance Bolt
3. High strength Bolt (can be HSFG Type or Bearing Type)

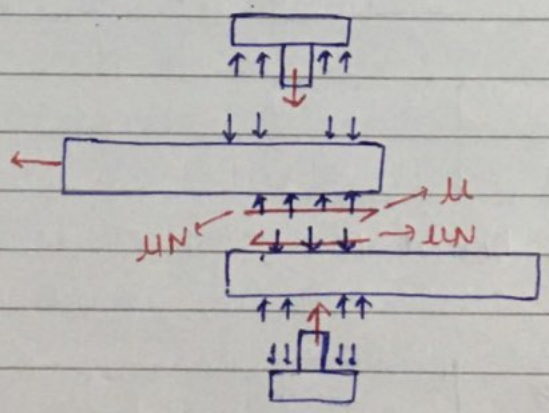
① **Black Bolts** :- These are primarily used in light structures under static load such as small Trusses, purlins etc. Also used as Temperory Fastners during eriction.

- Least expansive bolts with high tolerance, However
- it may not produce the least expansive connection.
- Not recommended for connections subjected to impact, Fatigue or dynamic load.

② **Turned-Bolts** :- Has less tolerance allowed than Black. These Bolts are used when accurate Alignment of components in there.

③ **High strength Bolts** :-





High strength bolts are less Ductile than ordinary. Bolts may be tighten until High Tensile stress is developed so that the connected parts are clamped together tightly.

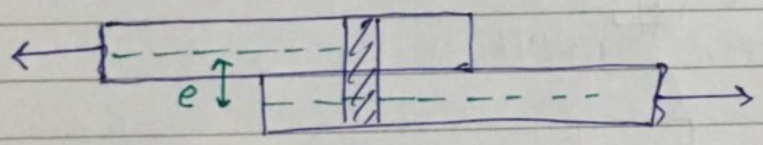
permits the load to be primarily transfer by friction

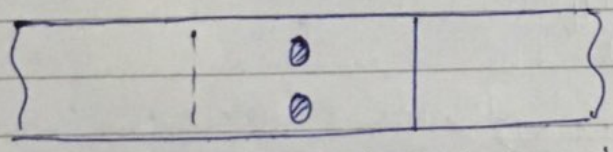
- Due to the friction the slip in the joint is eliminated and Hence the joints with HSFG Bolts are called non-slip connection. The cofficient of friction is called slip factor and the induced initial Tension is called proof load.

- Better for Bridges, siesmic loading, Fatigue etc.
- These Bolts can replace riveted connection.

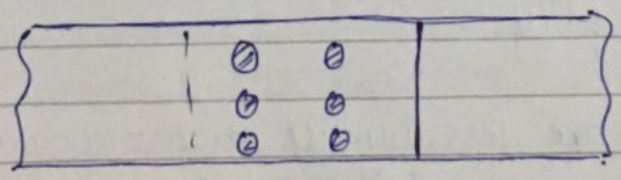
Bearing Type connection (Direct / simple connection)

Two Types of joint
1. Lap joint
2. Butt joint

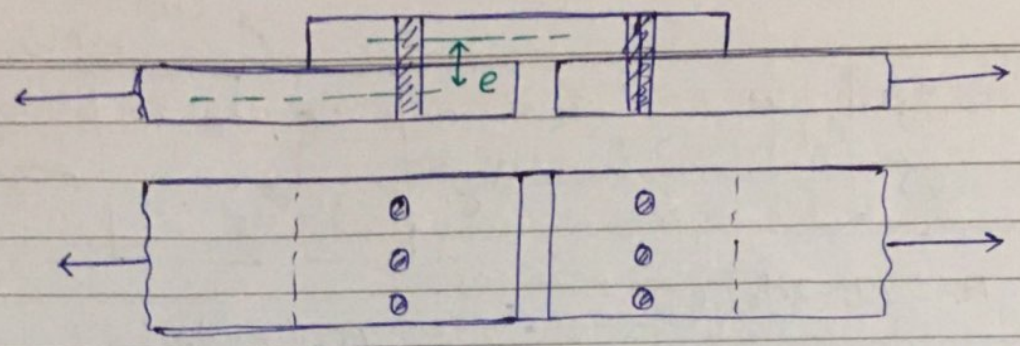




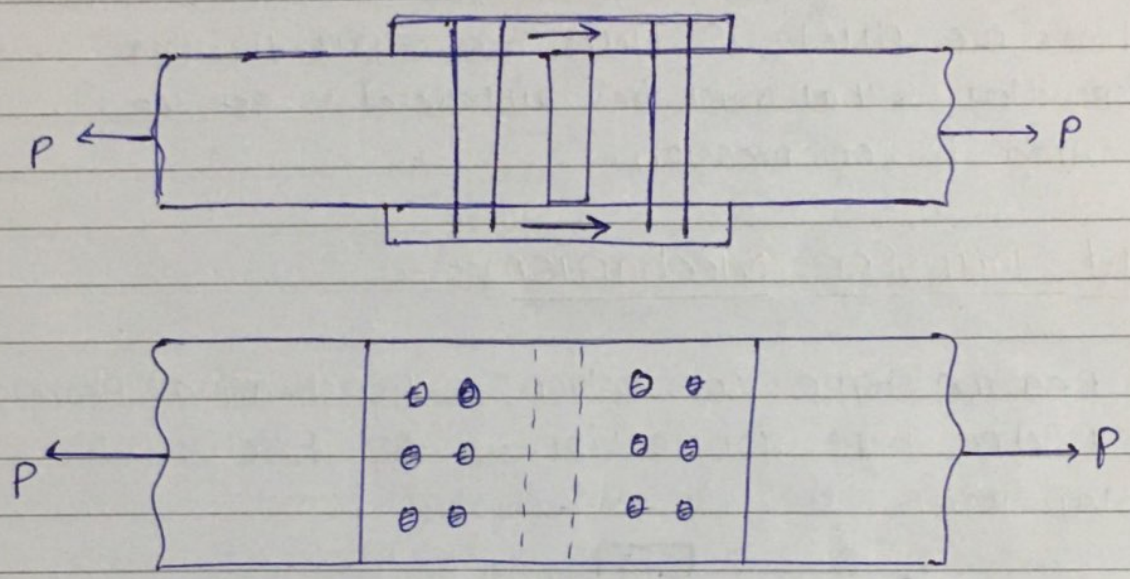
single bolted lap joint



Double bolted lap joint



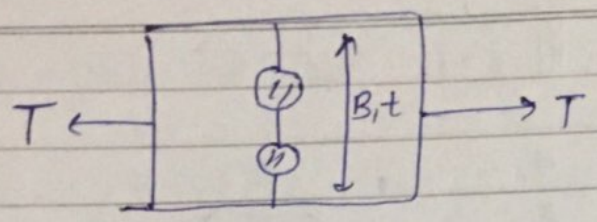
Single cover Single bolted butt joint



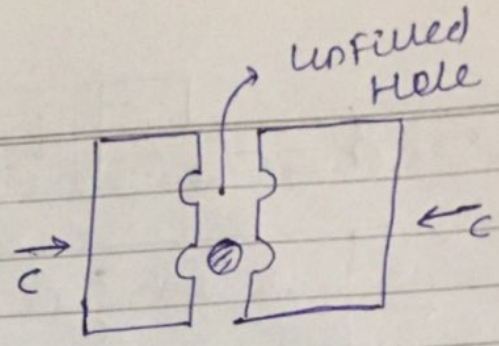
Double bolted Double cover Butt joint

Due to eccentricity in load in lap joint and single cover and butt joint bending will be generated and Hence Bolts may fails in Tension, To Minimise this effect at least two Bolts in a line shall be provided in a lap joint.

- Eccentricity of load is eliminated in Double cover Butt joint and hence this type of joints are prefered.



$A_n = (B - 2d_o)t$



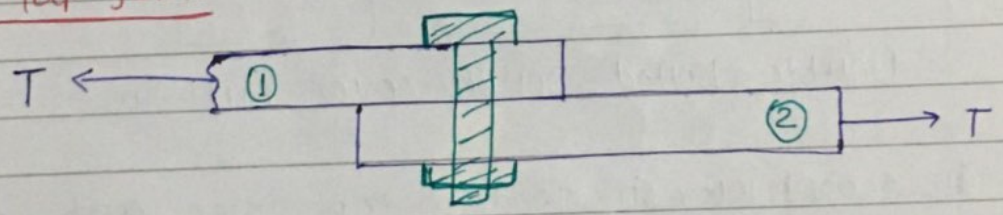
$A = (B - d_o)t$

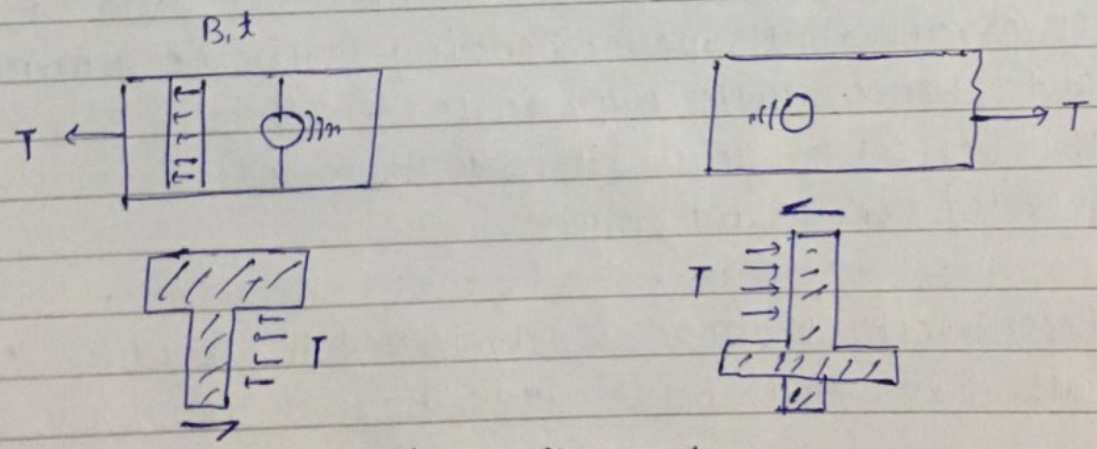
In case of Tension, Net Area is effective
In case of compression, Gross Area is effective provided All holes are filled, if Holes are unfilled, Area corresponding to that must be subtracted to get the eff. Area in compression.

Load Transfer Mechanism :-

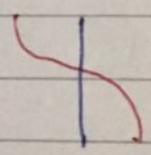
① In Bearing type connection → By shearing & Bearing
② In Friction type connection → By Friction.

In lap joint

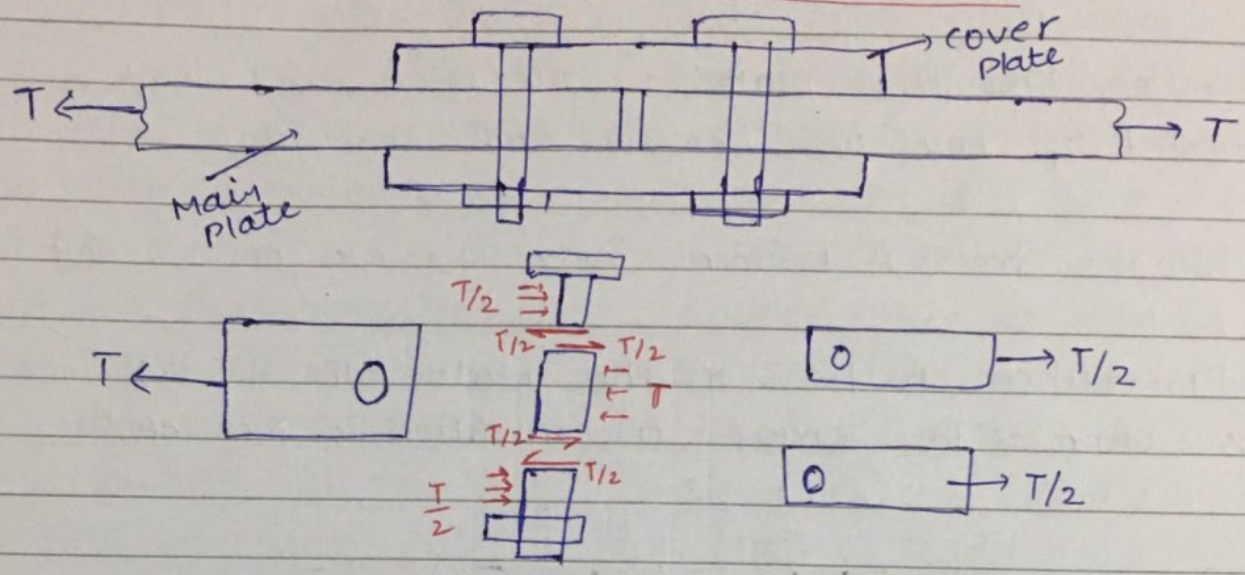




single shear plane



In double cover single bolted Butt joint



(Double shear plane)

- single shear plane in case of lap joint and single cover Butt joint
- Double shear plane in case of Double cover butt joint.
- shear capacity of a bolt in double cover bolt joint is higher than that of a bolt in lap joint

NOTE
- Bolts are designated as M16, M20, M18 (5–36 mm)
- Bolts of property class (3.6 — 12.4) are available

M20 bolt of property class 4.6

$d = 20$ mm

$f_{ub} = 400$ MPa , $f_{yb} = 400 \times 0.6 = 240$ MPa

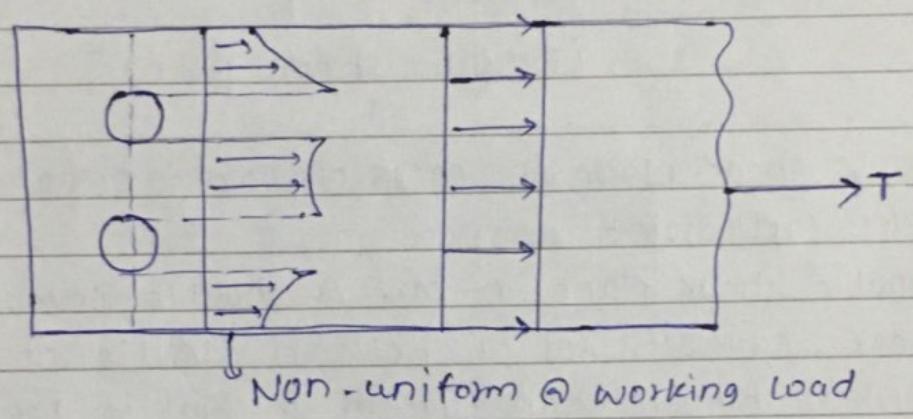
Normally we use 4.6 and 8.8

Assumptions in Bolted connection (Bearing Type):-

① friction b/w the plates is Neglected and load is resisted by bolts in shearing and Bearing.

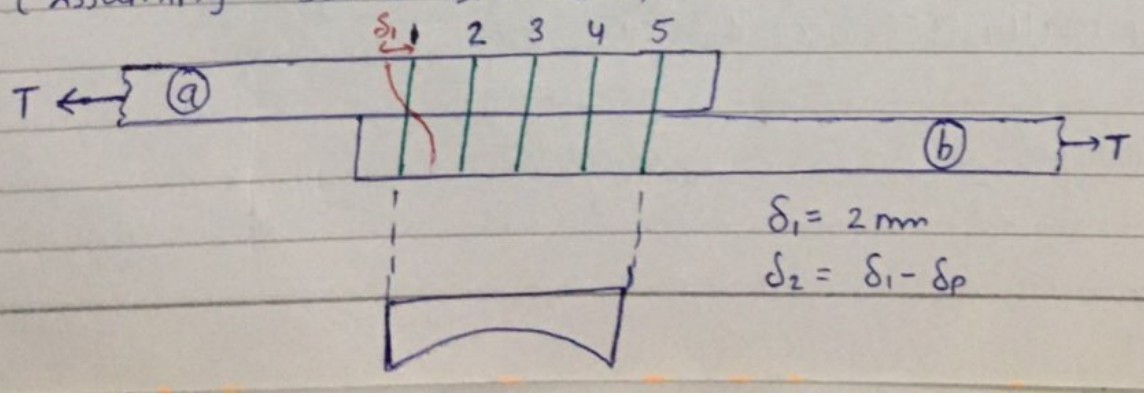
② shear stress is uniform over the c/s of the Bolt

③ Distribution of stress in the plate b/w the bolt hole is uniform i.e stress concentration is Neglected



The above assumption is not valid @ working load condition. However @ ultimate load due to High Ductility of steel, redistribution of stress will be there and uniform stress distribution may be assumed.

④ Bolts in a Group subjected to direct load share the load equally i.e Bolts are equally stressed.
(Assuming same Dia bolts)

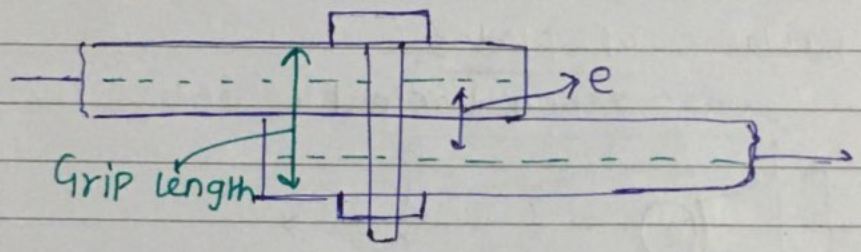


NOTE For short length of joint, the forces in bolt will be redistributed by plastic Action and hence the bolt will share the shear force equally.

However in a long joint, the shear force is not evenly distributed among the bolts and the bolts at the ends of a joint resist the highest amount of SF. This may lead to progressive joint failure called UnButtening effect. To account for this IS code recommends reducing in the shearing capacity of bolts for long joints.

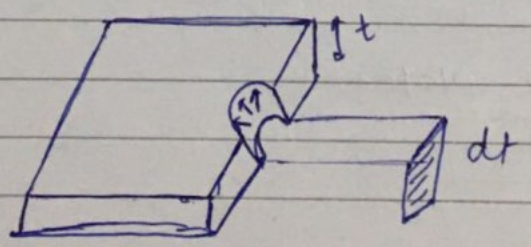
However in ∞ Rigid plate the stress in all bolts will be same even in the elastic condition.

⑤ Bending stress developed in Hole is Neglected.



if the Grip length is more correction factor is introduced to reduce the ↑ shear stress permissible

⑥ Bearing stress is assumed to be uniform over the Nominal contact Area b/w the plate and the Bolt.

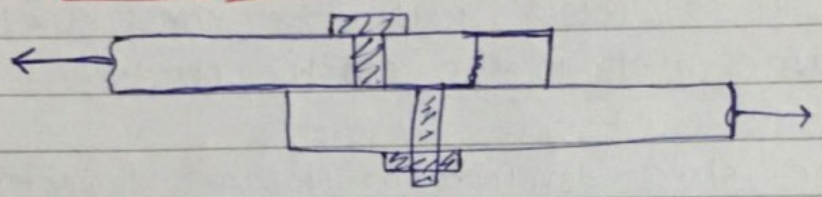


d → Nominal dia of bolt

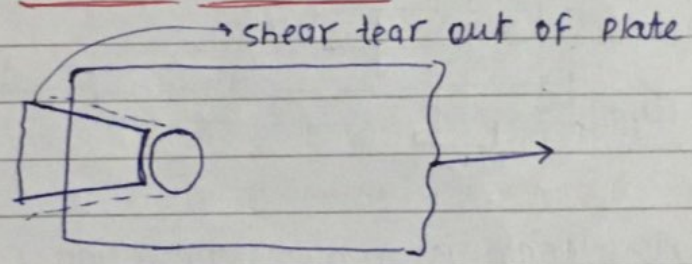
(7) if Nothing is specified then the area for resisting single shear shall normally be taken as the area of root of thread (A_{nb})
for Double shear, take one shear plane at shank area and other @ root of thread.

Failure of bolted connections

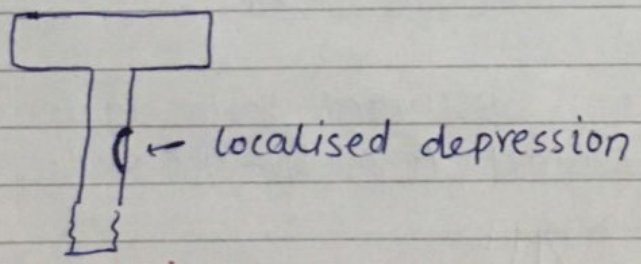
(I) Shear failure of bolt :-



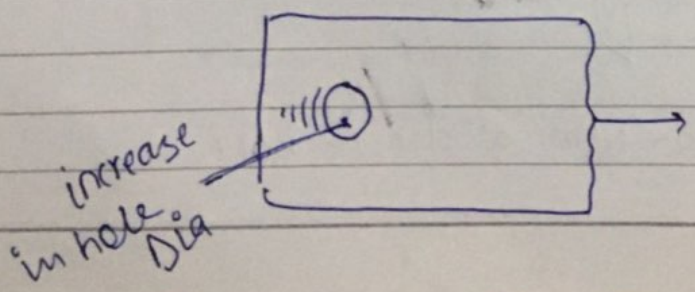
(II) shear failure of plates.



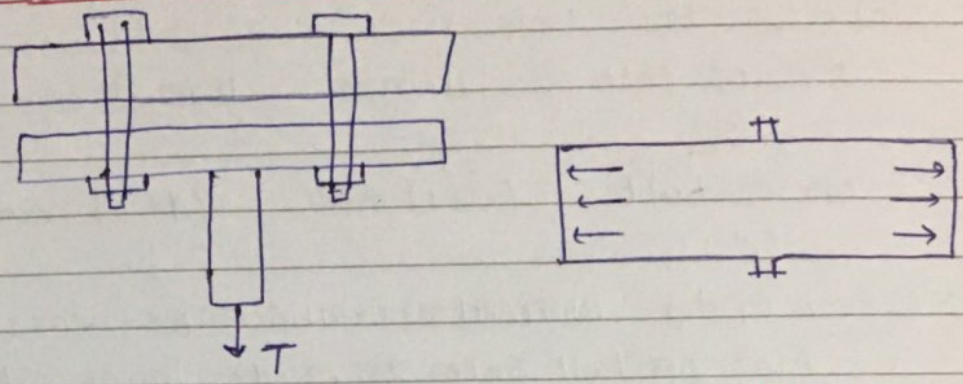
(iii) Bearing failure of bolts



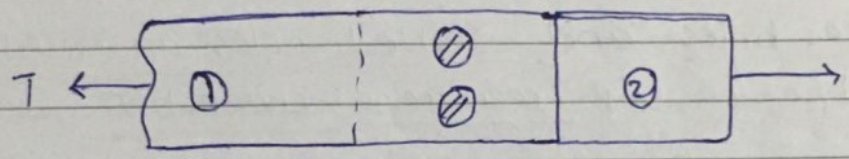
(iv) Bearing failure of plates.

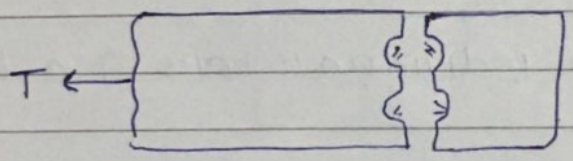


(V) Tension failure of bolts :-

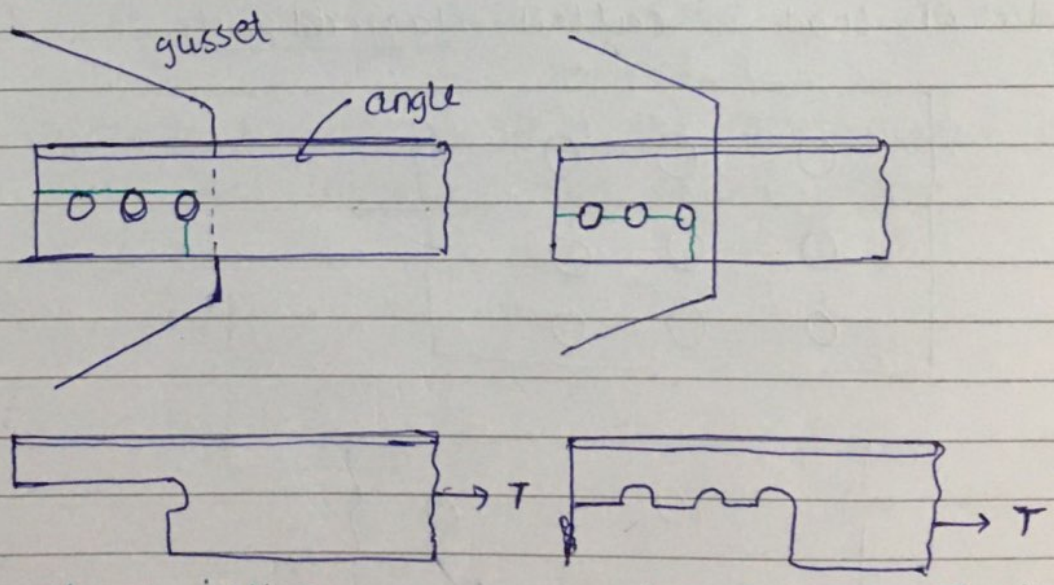


(Vi) Tension failure of plate / Tearing of plate / Rupture of plate





(Vii) Block shear failure :-



shear yielding + Tensile rupture

shear Rupture + Tensile yielding

Codal Recommendations :-

1) Dia of bolt hole

Dia of bolt (d)	≤ 14 mm	16 to 24 mm	>24 mm
Dia of ~~bolt~~ hole (d_o)	$(d+1)$ mm	$(d+2)$ mm	$(d+3)$ mm.

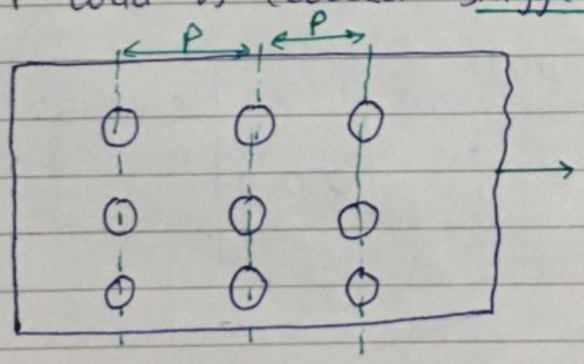
NOTE • Under Normal situation i.e No dynamic loading Dia of bolt hole is made larger than bolt dia to facilitate erection and to allow for inaccuracies in fabrication.

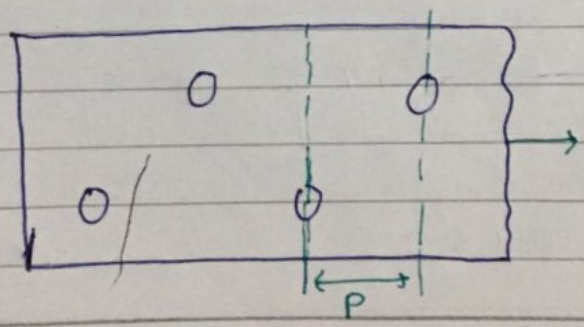
• oversize holes and slotted holes are used in friction type or slip resistance connection

2) pitch (P) :-

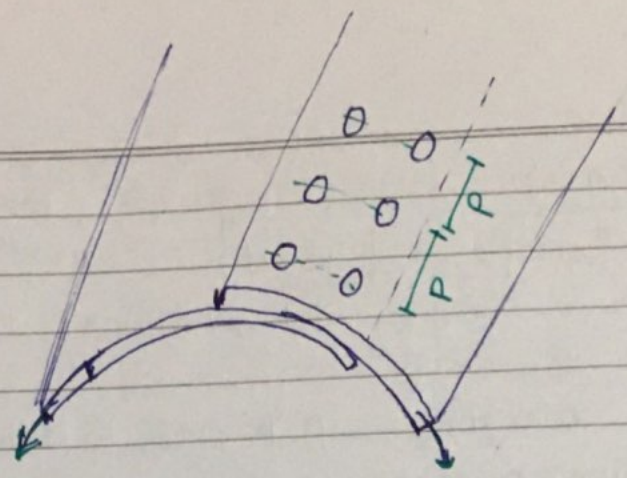
c/c distance b/w individual bolts in a line in the dir^n of load

Distance b/w any two conjucutive bolts in a zig-zag pattern of bolts measured parallel to the dir^n of load is called staggered pitch



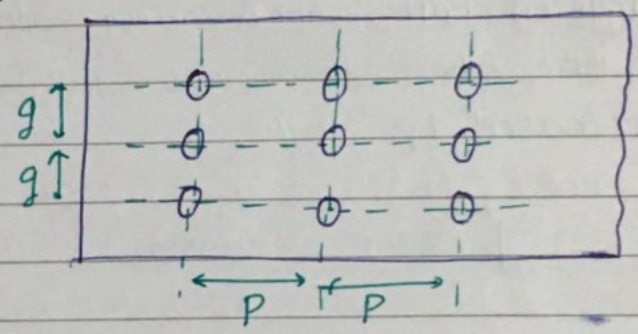


NOTE



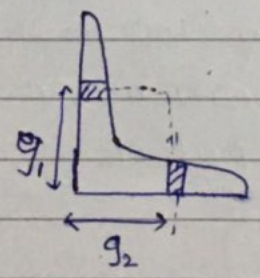
For wide plate pitch is also defined as c/c distance of bolts measured along the length of the member or the connection when per pitch strength is given, it is in the transverse dir^n of the load.

gauge (g) :-



The spacing b/w adjacent parellel line or bolt transverse to the dir^n of load is called Gauge

The distance b/w the back of the rolled section and the first bolt lying is also called gauge



Min. pitch and Gauge

min. pitch and Gauge = $\boxed{2.5 \times d}$

- To avoid overlapping
- To place Washer.
- To give space for Machine

$d \rightarrow$ nominal dia of bolt

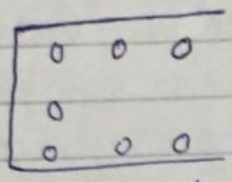
Max. pitch

(i) For Tension member $p \ngtr$ Min. $(16t, 200\ mm)$

(ii) For compression member $p \ngtr$ Min. $(12t, 200\ mm)$

$\rightarrow$ chances of Buckling, Hence it is Less

(iii) For lines near the edge $p \ngtr$ Min. $\begin{cases} 4t + 100\ mm \\ 200\ mm. \end{cases}$



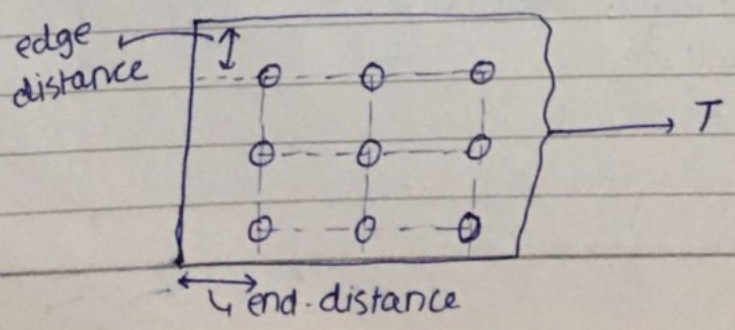
(iv) For staggered bolts, when gauge doesn't exceed 75 mm, the above distances specified in (i), (ii) & (iii) can be increased by 50%

- Max. Gauge $g \ngtr$ Min $(32t, 300\ mm)$

t in all above cases $\longrightarrow$ Thickness of thinner plate

③ End and Edge Distance

- End distance is the distance in the dir^n of stress from the center of the hole to the end of the element
- Edge distance is at Right angle to the stress.
- Bolt hole should not placed very close to the edge to avoid failure in shearing, bearing and to provide space for bolt head, washer nut etc.



Also Max^m Edge distance is limited to reduce the chances of corroison due to Moisture.

Min Edge / End distance = $1.7 d_o$ for sheared / hand Flame cut edges

= $1.5 d_o$ For rolled or Machine Flame cut plane edges

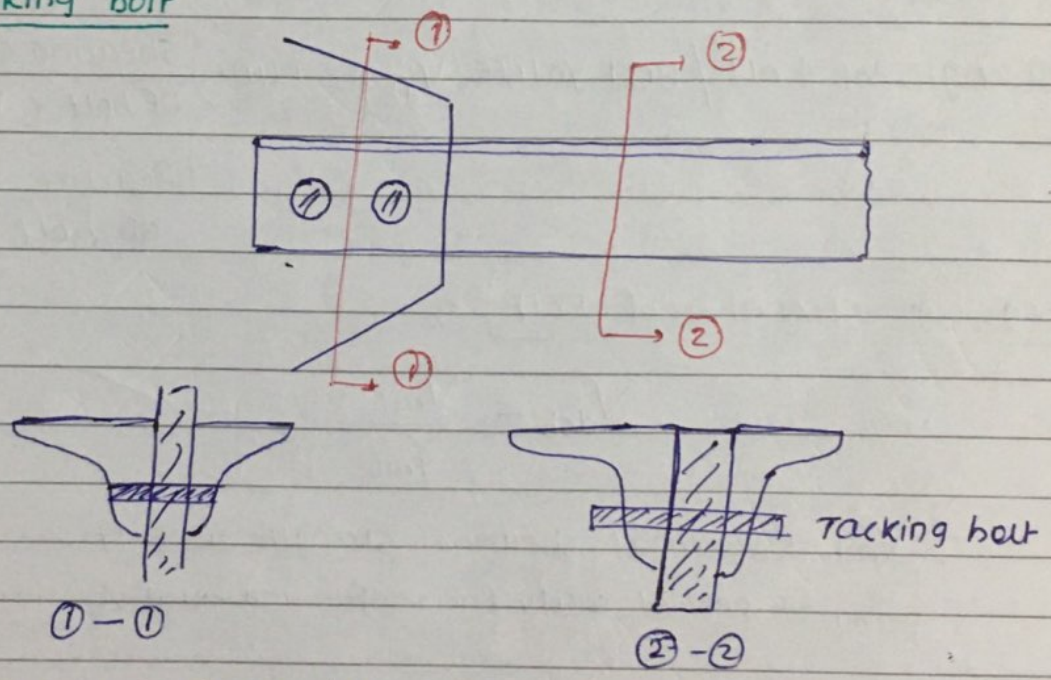
Max. Edge / End distance = $12 t \varepsilon$

$t \rightarrow$ thinner plate (mm)

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

When the members are exposed to corrossive environment the Max. edge distance $\ngtr$ $40m + 4t$

4) Tacking bolt



- The purpose of tacking bolt or stich is to equalize the stress in the two components, and to ensure that two components behave as one unit
- in comp. Member it also reduces chances of Buckling
- Tacking bolts aren't subjected to calculate stress.

(i) In General, Max. pitch = Min $\begin{cases} 32t \\ 300\,mm \end{cases}$

When connection is not exposed to weather

$$= \text{Min} \begin{cases} 16t \\ 200\,m \end{cases}$$

When connection is exposed to weather

(ii) For two members placed back to back Max. pitch of tacking bolt should not exceed 1000mm for Tension Member and 600 mm → comp. member

Bearing type connection

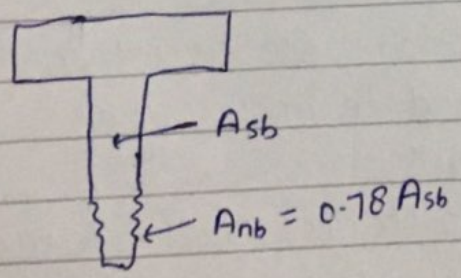
Strength of bolt / Bolt Value (V_{db}) = Min. $\begin{cases} \text{shearing strength of bolt } (V_{dsb}) \\ \text{Bearing strength of bolt } (V_{dpb}) \end{cases}$

① **Shearing strength of Bolt**

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

V_{nsb} → nominal shearing strength of bolt

γ_{mb} → partial safety factor for material, bearing type bolt (1.25)



$$V_{dsb} = \frac{f_{ub}}{\sqrt{3}\,\gamma_{mb}} \left[n_n A_{nb} + n_s A_{sb} \right] \times \beta_{lj} \times \beta_{lg} \times \beta_{pk}$$

f_{ub} → ultimate Tensile strength of bolt
A_{nb} → net shear area of bolt @ thread
A_{sb} → nominal shank Area of bolt $\left(\frac{\pi}{4} d^2\right)$
n_n → no. of shear plane through thread
n_s → no. of shear plane throug shank

for single shear plane $n_n = 1$, $n_s = 0$
for Double shear plane $n_n = 1$, $n_s = 1$

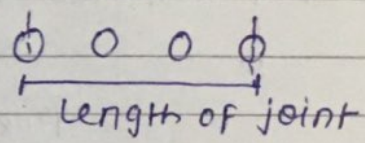
β_{lj}, β_{lg}, β_{pk} are reduction factors taking into account the effect of long joint, large Grip length and packing plate respectively.

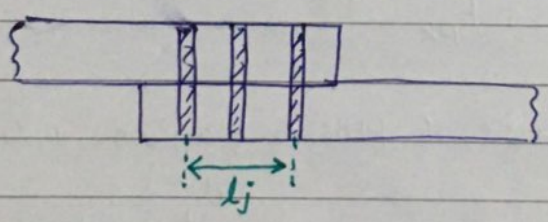
For long joints (β_{lj}) • apply when $l_j > 15d$

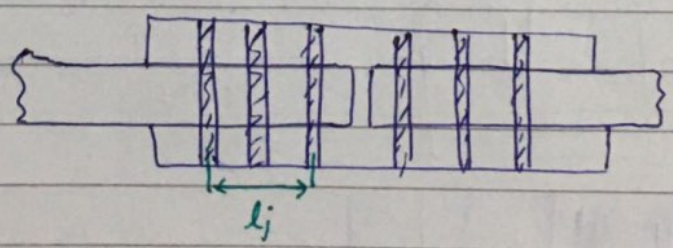
$$\beta_{lj} = 1.075 - \frac{l_j}{200d}$$

d → dia of bolt

Also $0.75 \leq \beta_{lj} \leq 1$





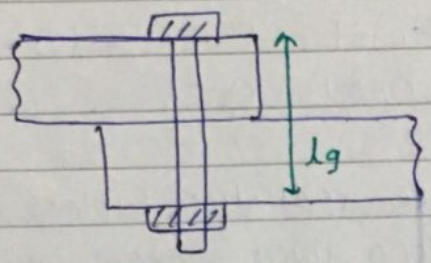


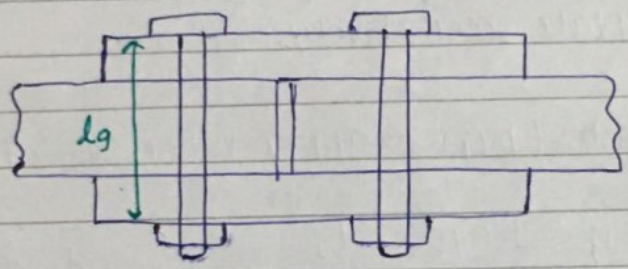
l_j → c/c distance b/w the extreme bolts in the dir^n of load transfer

For large grip length (β_{lg}) • apply when $l_g > 5d$

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

$\beta_{lg} \not> \beta_{lj}$ & $l_g \not> 8d$

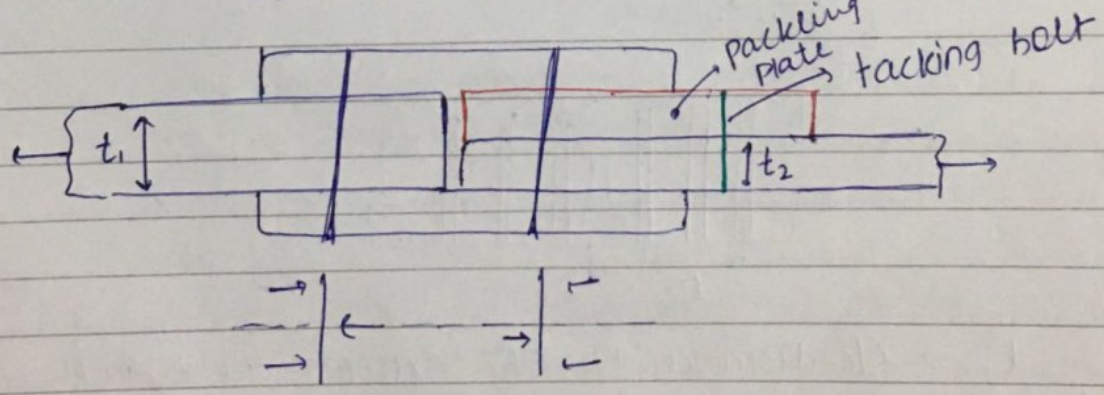




For packing plate (β_{PK}) • apply when $t_{PK} > 6mm$

$$\beta_{PK} = 1 - 0.0125\, t_{PK}$$

$t_{PK} \rightarrow$ thickness of thicker packing plate in mm.



② Bearing strength of Bolt :-

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

f_u → plate

$$K_b = \text{Min}\left\{ \frac{e}{3d_0},\ \frac{p}{3d_0} - 0.25,\ \frac{f_{ub}}{f_u},\ 1 \right\}$$

d → nominal dia of bolt

t → sum of thicknesses of plate experiencing bearing in the same dir^n

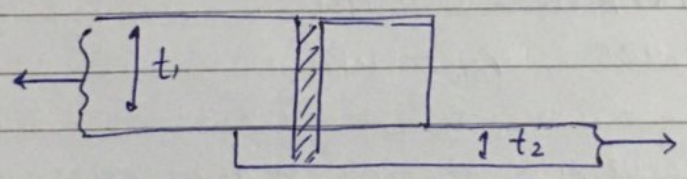
e → end distance in the bearing dir^n

p → pitch distance in the bearing dir^n

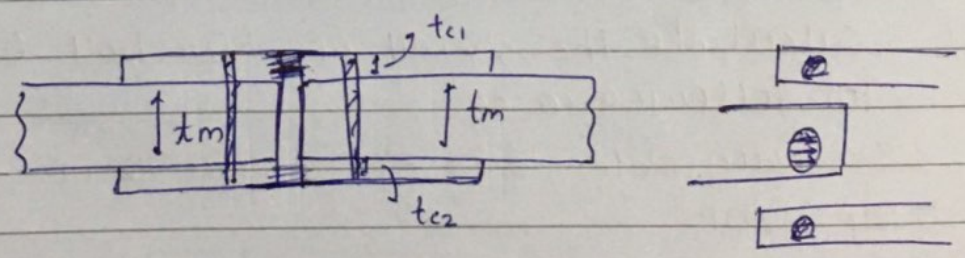
d_0 → Dia of bolt hole

f_{ub} → ultimate Tensile strength of bolt

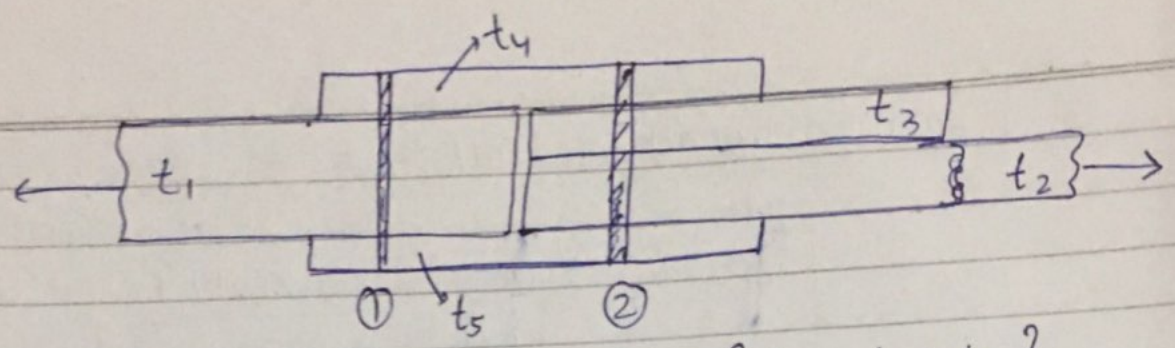
f_u → ultimate Tensile strength of plate



$$t = \text{Min}(t_1, t_2)$$



$$t = \text{Min}\{ t_{c1} + t_{c2},\ t_m \}$$



for Bolt ①, $t = \text{Min}\{t_1, t_4 + t_5\}$

for Bolt ②, $t = \text{Min}\{t_2, t_4 + t_5\}$

③ **Tensile strength of bolt :-**

Net section rupture $\boxed{T_{dn} = \frac{0.9 f_{ub}}{\gamma_{mb}} A_{nb}}$

Gross section yielding $\boxed{T_{dg} = \frac{f_{yb}}{\gamma_{mo}} A_{sb}}$

$A_{nb} \rightarrow$ net Area $= 0.78 A_{sb}$

$\gamma_{mb} \rightarrow 1.25$, $\gamma_{mo} \rightarrow 1.1$

The yielding criteria is checked so that excessive yielding doesnot result in non-functional joint before Rupture.

Que. Calculate the strength of M20 bolt Grade 4.6 for following cases

The Main plates to be joined are 12 mm thick and Fe410

i) lap joint

ii) single cover butt joint with cover plate 10 mm thick

iii) double cover butt joint with thickness of each cover plate 8 mm.

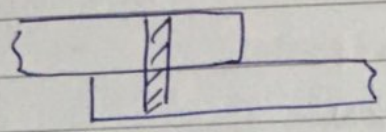
$f_u = 410$ mpa $f_y = 250$ mpa

$f_{ub} = 400$ mpa $f_{yb} = 240$ mpa $\{400 \times 0.6)$

$d = 20$ mm $d_o = 20+2 = 22$ mm

Main plate $t = 12$ mm

(i) <u>lap joint</u>



single shear case

a) $V_{dsb} = \frac{f_{ub}}{\sqrt{3}\,\gamma_{mb}} [n_n A_{nb} + n_s A_{sb}]\, \beta_{Li}\, \beta_{Lg}\, \beta_{PK}$ (assume 1, $L_g < 5d$, 1)

$V_{dsb} = \frac{400}{\sqrt{3} \times 1.25} \left[1 \times 0.78 \times \frac{3.14}{4} \times 20^2 + 0 \times \frac{3.14}{4} \times 20^2\right] 1 \times 1 \times 1$

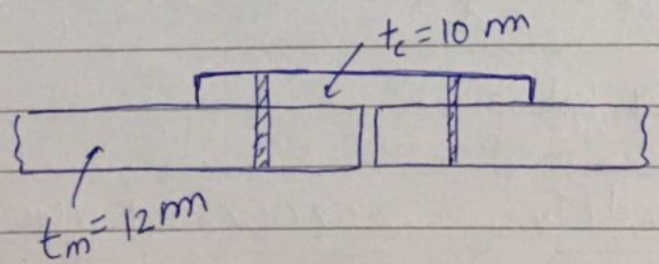
$V_{dsb} = 45.27$ kNm

b) $V_{dpb} = \frac{2.5\, k_b\, d\, t\, f_u}{\gamma_{mb}}$ $k_b = \min\left\{ \frac{e}{3d_o}, \frac{p}{3d_o} - 0.25, \frac{f_{ub}}{f_u}, 1 \right.$ (e: $1.5d_o$, p: $2.5d$)

$V_{dpb} = \frac{2.5 \times 0.5 \times 20 \times 12 \times 410}{1.25}$ $k_b = \{ 0.5$

$V_{dpb} = 98.4$ kN

(ii)



M 20
$d = 20mm$
$d_o = 22\ mm$
Fe 410 , class 4.6

$V_{db} = Min[V_{dsb}, V_{dpb}]$

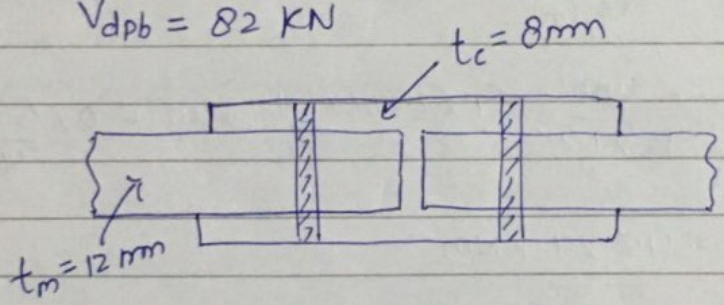
single shear case

$V_{dsb} = 45.27\ KN$

$V_{dpb} = \frac{2.5\ k_b d t f_u}{\gamma_{mb}} = \frac{2.5 \times 0.5 \times 20 \times 10 \times 410}{1.25}$

$V_{dpb} = 82\ KN$

(c)



M20
$d = 20\ mm$
$d_o = 22\ mm$

Fe410 , class → 4.6

$V_{dsb} = \frac{f_{ub}}{\sqrt{3}\gamma_{mb}} [n_n A_{nb} + n_s A_{sb}]$

$= \frac{400}{\sqrt{3} \times 1.25} [1 \times \frac{\pi}{4} \times 20^2 \times 0.78 + 1 \times \frac{3.14}{4} \times 20^2]$

$= 103.27\ KN$

$V_{dpb} = \frac{2.5\ k_b d t f_u}{\gamma_{mb}} = \frac{2.5 \times 0.5 \times 20 \times 12 \times 410}{1.25} = 98.40\ kN$

$\Rightarrow V_{db} = (\text{bolt value}) = Min(103.27, 98.40)$
$= 98.40\ kN$

NOTE Generally in case of single shear, shearing capacity govern and in case of Double shear bearing capacity governs.

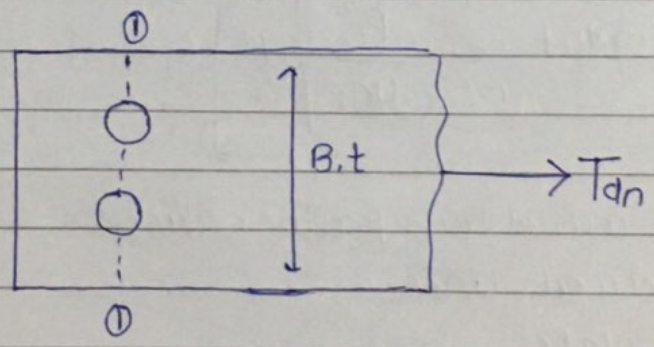
Strength of plate :-

The plate may fail by ① shear failure of plate
② Bearing failure of plate
③ Tensile failure of plate or Net secⁿ Rupture
④ Yielding of plate / Gross secⁿ Yielding
⑤ Block shear failure @ joint

① shearing and bearing failure of plate may be avoided by selecting proper end and pitch distances.

② Bearing failure can be also controlled by limiting bearing capacity of bolt

③ Tension | Rupture | Tearing of plates



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

$$A_n = (B - 2d_o)t$$

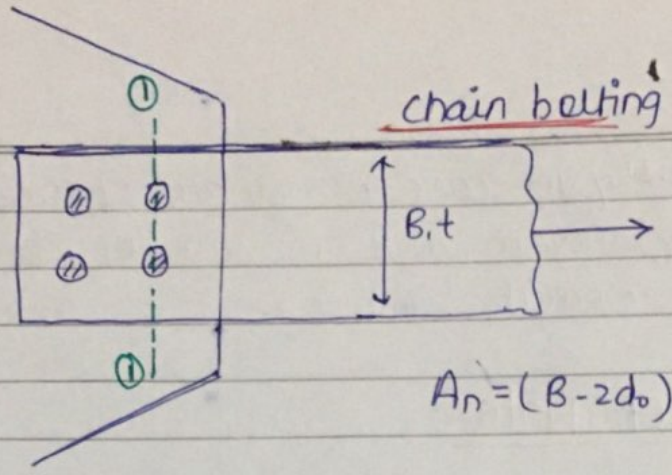
$f_u \rightarrow$ ultimate strength of plate

$T_{dn} \rightarrow$ Design Tensile strength of plate in Rupture

$\gamma_{m1} \rightarrow$ partial safety factor @ ultimate stress (1.25)

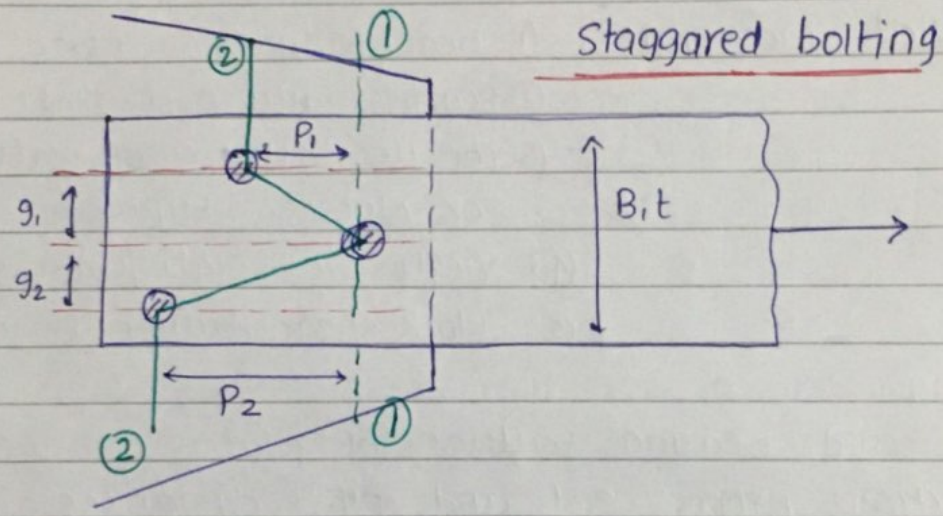
$A_n \rightarrow$ Net effective area of member

a)



$$A_n = (B - 2d_o)\,t$$

b)



$$A_{n(1-1)} = (B - d_o)\,t$$

$$A_{n(2-2)} = \left[B - 3d_o + \frac{P_1^2}{4g_1} + \frac{P_2^2}{4g_2}\right] t$$

$$\boxed{A_n = \left[B - nd_o + \sum_{i=1}^{m} \frac{P_i^2}{4g_i}\right] t}$$

$n \rightarrow$ no. of bolts along the failure line
$m \rightarrow$ no. of inclined lines
$B \rightarrow$ width of plate
$t \rightarrow$ thickness of plate

④ Yielding of plate

→ Yielding of plate is generally not considered at joint location

→ Due to excessive yielding of the member in Gross section, the structure may be non-Functional.

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

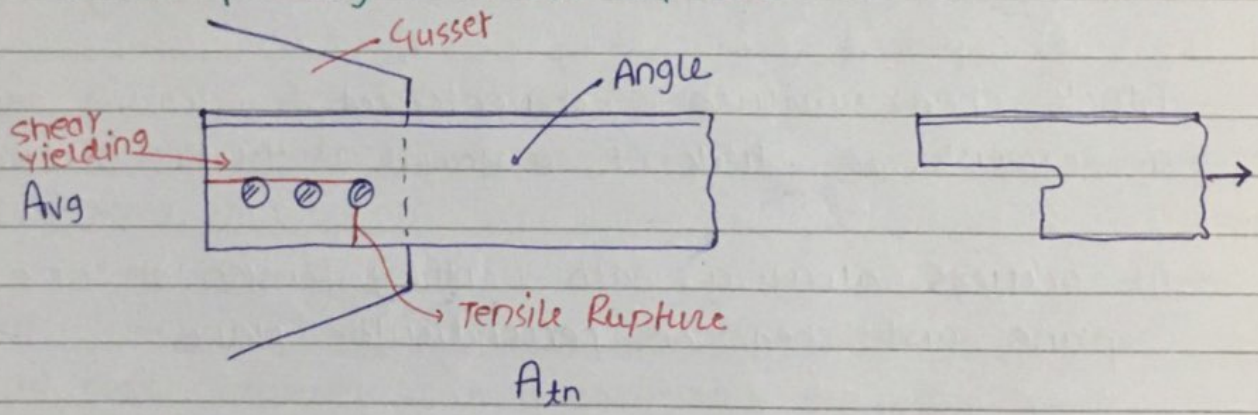
→ Design strength in yielding

γ_{m0} → partial safety Factor in yielding (1.1)

f_y → Yield strength

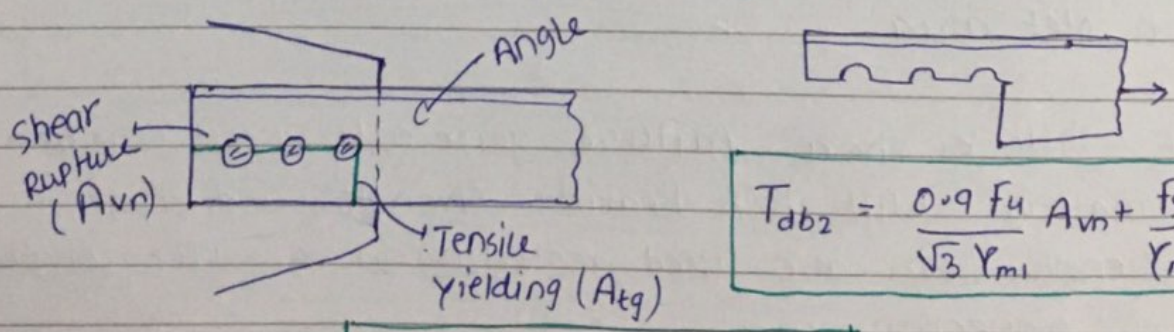
⑤ Block shear failure

(i) shear yielding + Tensile Rupture



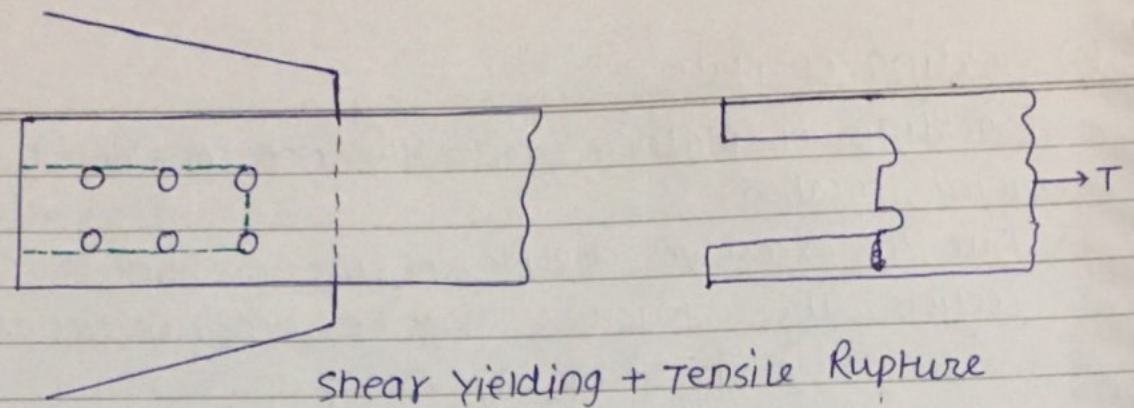
$$T_{db_1} = \frac{f_y}{\sqrt{3}\,\gamma_{m0}} A_{vg} + \frac{0.9 f_u}{\gamma_{m1}} A_{tn}$$

(ii) Shear Rupture + Tensile yielding

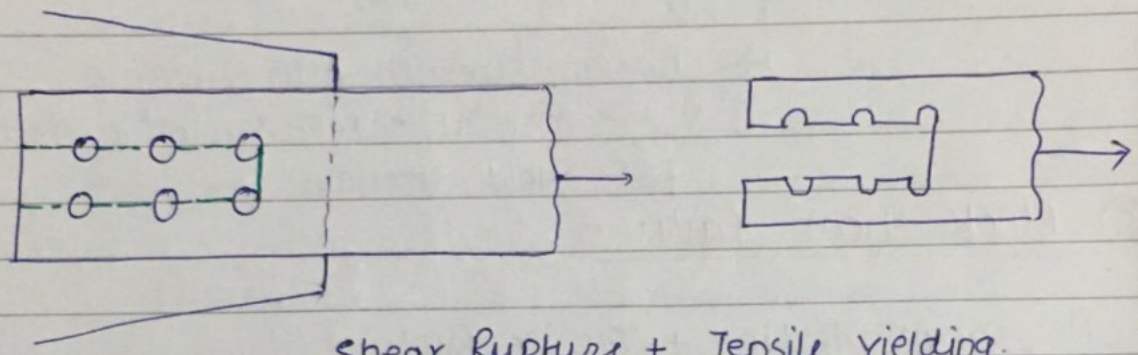


$$T_{db_2} = \frac{0.9 f_u}{\sqrt{3}\,\gamma_{m1}} A_{vn} + \frac{f_y}{\gamma_{m0}} A_{tg}$$

$$T_{db} = \text{Min.}\{T_{db1}, T_{db2}\}$$



Shear Yielding + Tensile Rupture



Shear Rupture + Tensile Yielding.

- Block shear failure is charecterized by tearing out of a segment or block of material at the end of member
- It occurs along a path involving tension on one plane and shear on perpendicular plane
- For block shear failure to occure, one of the plane rupture while other yields
- Yielding always occur in Gross Area and rupture on Net area.

NOTE Block shear failure generally ocurs when material with high bearing strength and high strength bolts are used resulting in smaller length of connection.

Efficiency of joint :- $\eta_{joint} = \dfrac{\text{strength of joint}}{\text{strength of solid plate without reduction of bolt Area}}$

$$\text{Strength of joint} = \text{Min.} \begin{cases} \left.\begin{array}{l}\text{Shearing of bolt} \\ \text{Bearing of Bolt}\end{array}\right\} n \times V_{db} \\ \text{Tearing of plate} \\ \text{Block shear failure} \\ \text{Shearing of plate} \\ \text{Bearing of plate} \\ \text{failure of cover plate} \end{cases}$$

$$\text{Strength of solid plate} = \text{Min.} \begin{cases} \dfrac{0.9 f_u}{\gamma_{m1}} \cdot A_g \\ \dfrac{f_y}{\gamma_{m0}} \cdot A_g \checkmark \end{cases}$$

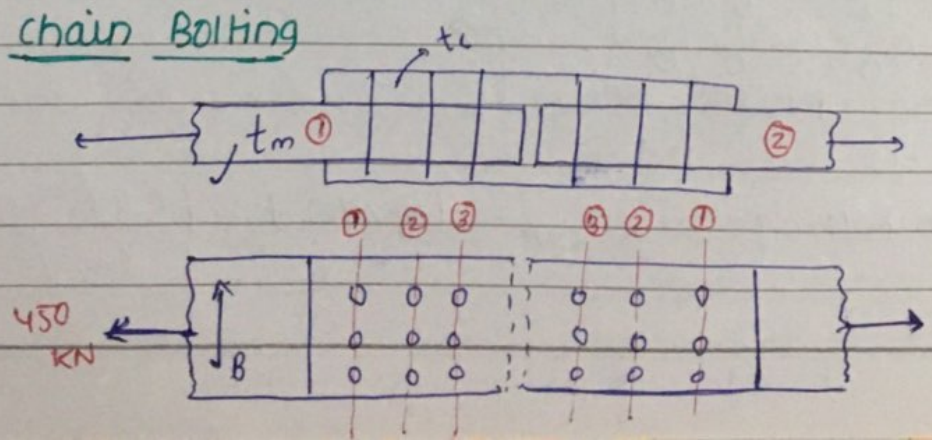
The strength of solid plate will be generally Governed by Yielding.

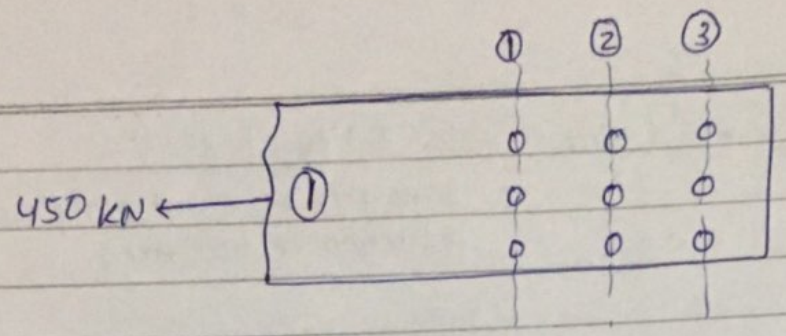
For efficient utilization of material failure of bolt and plate should be simultaneous.

Arrangement of Bolts :-

By considering diff^n arrangement of bolts, tearing strength of plate can be changed and hence efficiency of joint can be changed.

(i) Chain Bolting





$$T_{dn\,(1-1)} = (B-3d_o)\,t_m \times \frac{0.9\,f_u}{\gamma_{m1}}$$

$$T_{dn\,(2-2)} = (B-3d_o)\,t_m \times \frac{0.9\,f_u}{\gamma_{m1}} + 3\,V_{db}$$

$$T_{dn\,(3-3)} = (B-3d_o)\,t_m \times \frac{0.9\,f_u}{\gamma_{m1}} + 6\,V_{db}$$

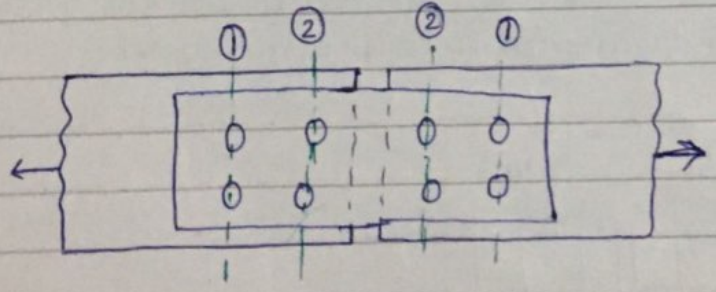
So Tearing strength of 2nd and 3rd section will be higher compare to secⁿ 1 and hence most critical secⁿ for Main plate is secⁿ 1

Max. force in cover plate will be at secⁿ 3

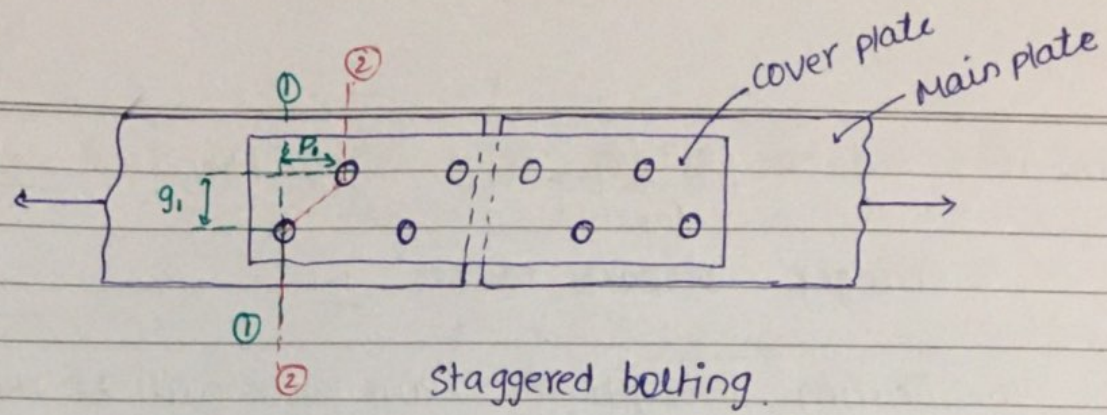
Tearing strength of cover plate @ 3 $= (B-3d_o)(2t_c)\,\frac{0.9\,f_u}{\gamma_{m1}}$

2 cover plate

(ii) staggered bolting :-



chain bolting . $T_{dn\,(1-1)} = (B-2d_o)\,t_m \left(\frac{0.9\,f_u}{\gamma_{m1}}\right)$



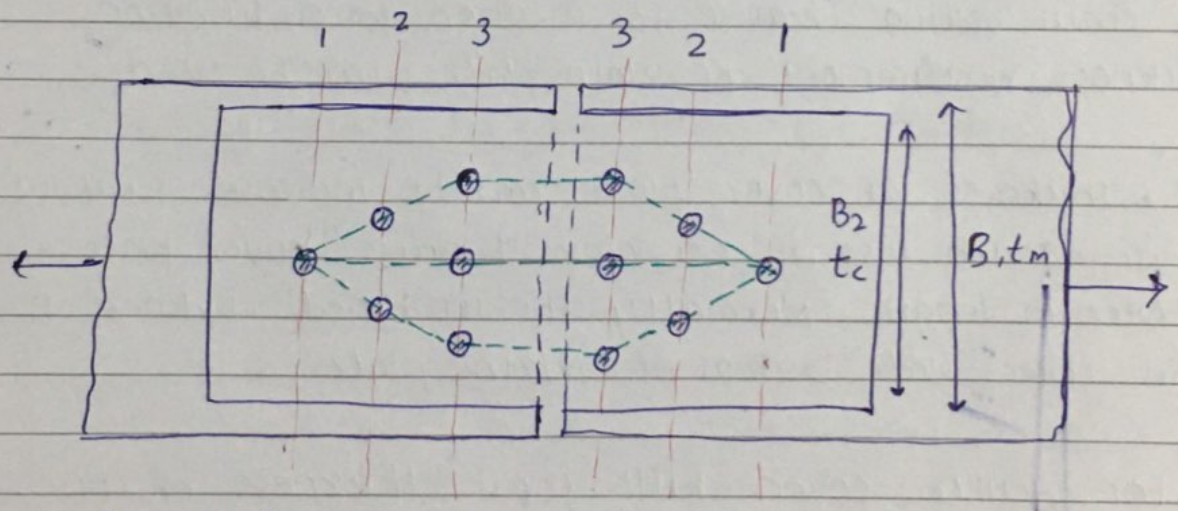
Staggered bolting.

$$T_{dn\,(1-1)} = (B-d_o)\,t_m\left(\frac{0.9\,f_u}{\gamma_{m1}}\right)$$

$$T_{dn\,(2-2)} = \left[B-2d_o+\frac{P_1^2}{4g_1}\right]t_m\left(\frac{0.9\,f_u}{\gamma_{m1}}\right)$$

Staggered bolting is better compared to chain bolting because tearing strength will be higher for staggered bolting.

(iii) Diamond bolting



For Main plate

$$T_{dn\,(1-1)} = (B_1-d_o)\,t_m\left(\frac{0.9\,f_u}{\gamma_{m1}}\right)$$

$$T_{dn\,(2-2)} = (B_1-2d_o)\,t_m\left(\frac{0.9\,f_u}{\gamma_{m1}}\right)+V_{db}$$

$$T_{dn\,(3-3)} = (B_1-3d_o)\,t_m\left(\frac{0.9\,f_u}{\gamma_{m1}}\right)+3V_{db}$$

→ if $-d_o t_m \left(\frac{0.9 f_u}{\gamma_{m1}}\right) + V_{db}$ is >0 then sec^n 2-2 is stronger compare to sec^n 1-1

→ The Tearing strength of Main plate will be the Min. of Tearing strength of sec^n ①, ② and ③

→ If sec^n 2-2 is not critical then sec^n 3-3 can't be critical.

→ For cover plate sec^n 3-3 is most critical because at that sec^n cover plate is having Max^m force and Max^m reduction of Area due to bolt hole.

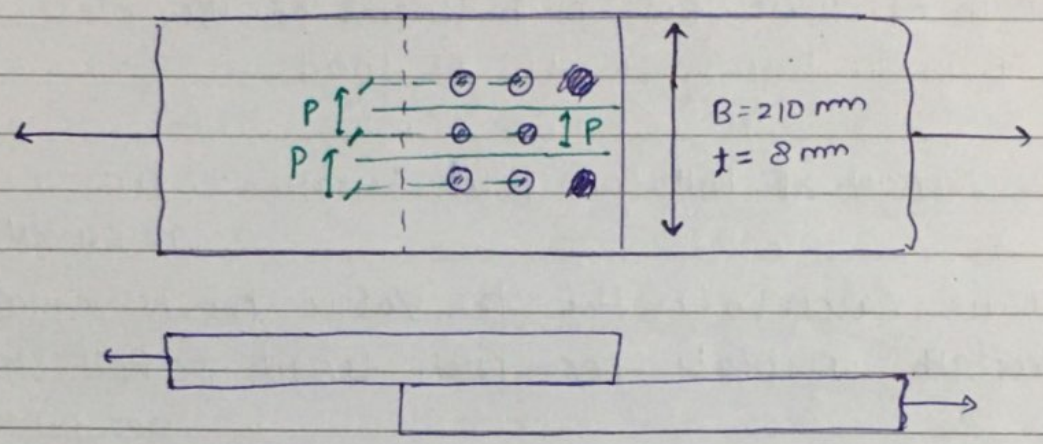
→ Diamond pattern is most efficient and economical compare to chain bolting because for a given force, width or thickness requirement of Main plate will be less.

NOTE • Thickness of cover plate can be increases without any much loss of economy because cover plate is shorter in length. Normally the combined thickness of the cover plate > that of Main plate

• For double cover butt joint thickness of one cover plate $\nless \frac{5}{8}$ times main plate thickness

• for single cover butt joint, thickness of cover plate shall not be less than $\frac{9}{8} \times$ thickness of Main plate

Que. Two plates of Fe410 Grade steel each (210x8) mm² are to be joined using 20 mm dia, 4.6 Grade to form a lap joint, the joint is supposed to transfer a factored load of 250 kN. Design the joint and determine suitable pitch for the bolt.



Fe 410, M20, Bolt class = 4.6

$f_u = 410$ MPa, $f_{ub} = 400$ MPa $\quad d = 20$ mm

$f_y = 250$ MPa, $f_{yb} = 240$ MPa $\quad d_o = 22$ mm

$P_u = 250$ kN

single shear case

$$V_{dsb} = 45.27 \text{ kN}$$

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

$$K_b = \text{Min}\left[\frac{e}{3d_o}, \frac{p}{3d_o} - 0.25, \frac{f_{ub}}{f_u}, 1\right] = 0.50$$

($e \to 1.5d_o$, $p \to 2.5d$)

$$V_{dpb} = \frac{2.5 \times 0.5 \times 20 \times 8 \times 410}{1.25} = 65.60 \text{ kN}$$

$$V_{ab} = \text{Min}[45.27, 65.60] = 45.27 \text{ kN}$$

$\Rightarrow$ No. of bolts $= \dfrac{250}{45.27} = 5.52$

provide <u>6 bolts</u>

Consider the distance b/w the bolts as p, and calculate strength of joint per pitch length of the plate
Here, p is in transverse dir^n of load.

The strength of bolts per pitch length $= 2 \times 45.27$
$= 90.54$ kN

let us calculate the p value ~~for~~ by equating strength of plate per pitch length = Bolt strength per pitch length.

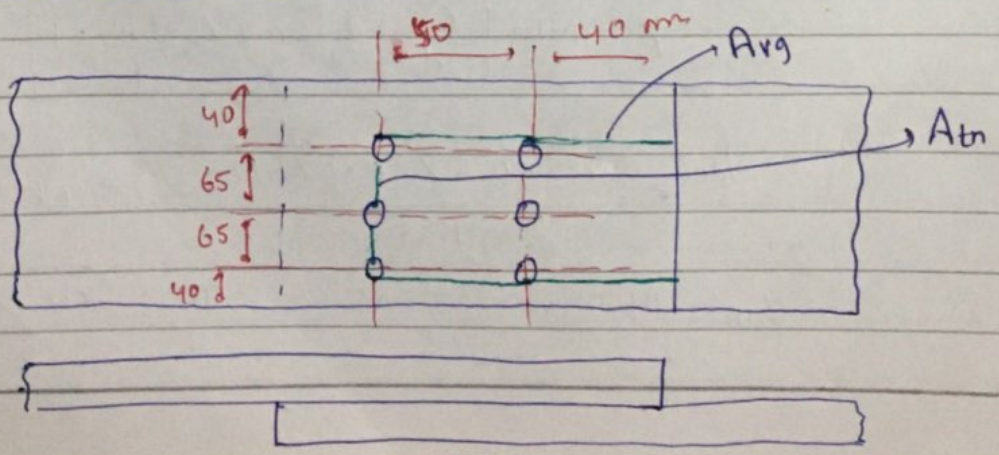
$$(P - d_o)\, t \left(\frac{0.9 f_u}{\gamma_{m1}}\right) = 90.54$$

$$(p - 22) \times 8 \times \left(\frac{0.9 \times 410}{1.25}\right) = 90.54 \times 10^3 \text{ N}$$

$$p \geq 60.33 \text{ mm}$$

Min Gauge provided shall be $2.5d = 50$ mm
adopt <u>$p = 65$ mm</u> and $e = \dfrac{210 - 2 \times 65}{2} = 40 \text{ mm} > 1.5 d_o$

<u>Block shear Failure</u>

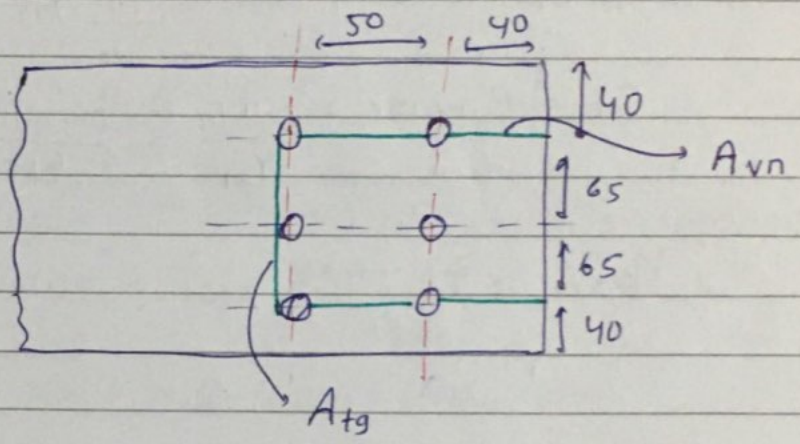


$$A_{vg} = (40+50) \times 8 \times 2 = 1440 \text{ mm}^2$$

$$A_{tn} = (2 \times 65 - 2 \times \underset{d_o}{22}) \times 8 = 688 \text{ mm}^2$$

$$T_{db_1} = \frac{f_y}{\sqrt{3}\,\gamma_{m0}} \cdot A_{vg} + \frac{0.9 f_u}{\gamma_{m1}} A_{tn}$$

$$T_{db_1} = \frac{250}{\sqrt{3} \times 1.1} \times 1440 + \frac{0.9 \times 410}{1.25} \times 688 = 392.05 \text{ KN}$$



$$A_{vn} = (40+50-1.5d_o) \times t \times 2$$

$$A_{vn} = 912 \text{ mm}^2$$

$$A_{tg} = (65+65) \times 8 = 1040 \text{ mm}^2$$

$$T_{db_2} = \frac{0.9 f_u}{\sqrt{3}\,\gamma_{m1}} A_{vn} + \frac{f_y}{\gamma_{m0}} A_{tg}$$

$$= \frac{0.9 \times 410 \times 912}{\sqrt{3} \times 1.25} + \frac{250 \times 1040}{1.1} = 391.79 > 250 \text{ KN}$$

Que. calculate efficiency of joint of the previous question.

$$\text{Strength of joint} = \text{Min} \begin{cases} n \times V_{db} \\ \text{Tearing strength of plate} \\ \text{Block shear failure} \end{cases}$$

$n \times V_{db} = 6 \times 45.27 = 271.62$ KN

Tearing strength of plate = 340.07 KN

$$\left\{ (B - 3d_0)\, t \, \frac{0.9 f_u}{\gamma_{m1}} \right.$$

Block shear failure = $\text{Min}(T_{db1}, T_{db2}) = 391.79$ kN

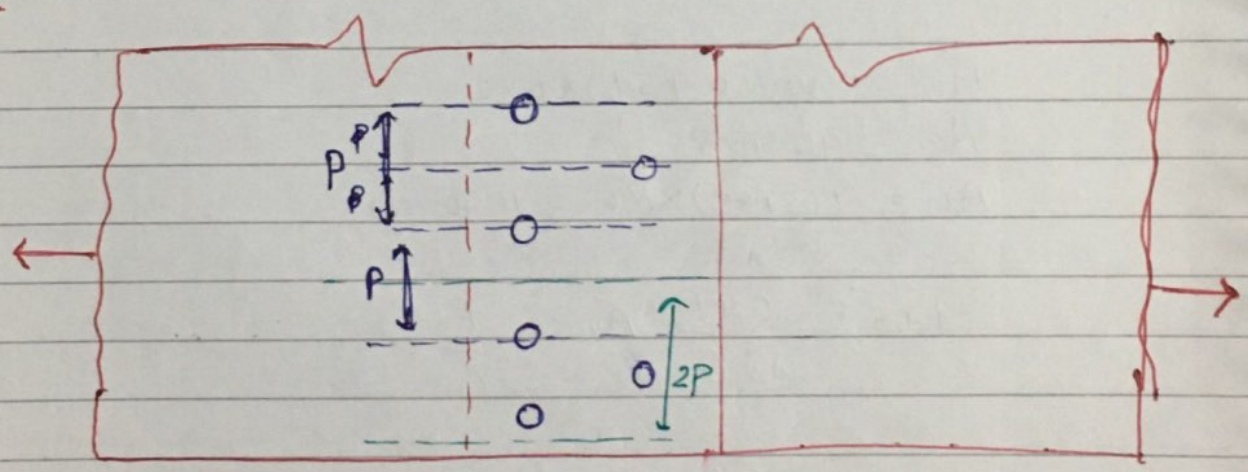
Strength of joint = 271.62 KN

Strength of solid plate = $\frac{f_y}{\gamma_{m0}} \cdot A_g = \frac{250 \times 210 \times 8}{1.1} = 381.81$ KN

Gross Rupture = $\frac{0.9 f_u}{\gamma_{mp}} = \frac{0.9 \times 410}{1.25} = 495$ kN

$$\eta_{joint} = \frac{271.62 \text{ KN}}{381.81} \times 100 = 71.14\%$$

Que.



Find p for most economical utilization of material.
$f_u = 410$ MPa, $t = 10$ mm, bolt M20 & 4.6 class.

For most economical utilization of material, the strength of repeating pattern of width 2P in tearing shall be equal to strength of bolt in shearing or bearing.

$V_{dsb} = 45.27$ KN (for M20, 4.6)

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

$$K_b = \min\left\{ \frac{e}{3d_o}, \frac{p}{3d_o} - 0.25, \frac{f_{ub}}{f_u}, 1 \right\} = 0.50$$

($e \to 1.5d_o$, $p \to 2.5d$, $f_{ub} \to 400$, $f_u \to 410$)

$$V_{dpb} = \frac{2.5 \times 0.5 \times 20 \times 10 \times 410}{1.25} = 82 \text{ KN}$$

$V_{db} = \min\{45.27, 82\}$
$= 45.27$ KN

Strength of all the bolts in one repeating unit
$= 3 \times 45.27 = 135.81$ KN

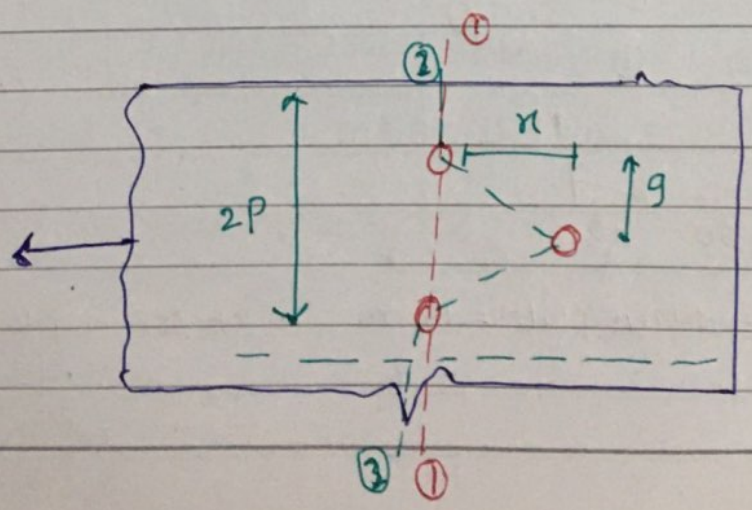
Strength of plate in Rupture (2p width)

$$T_{dn\,(1-1)} = (2p - 2d_o)\, t\, \frac{0.9 f_u}{\gamma_{m1}}$$

For most economical utilization

$$(2p - 2d_o)\, t\, \frac{0.9 f_u}{\gamma_{m1}} = 135.81 \times 10^3$$

$p = 45.00$ mm.



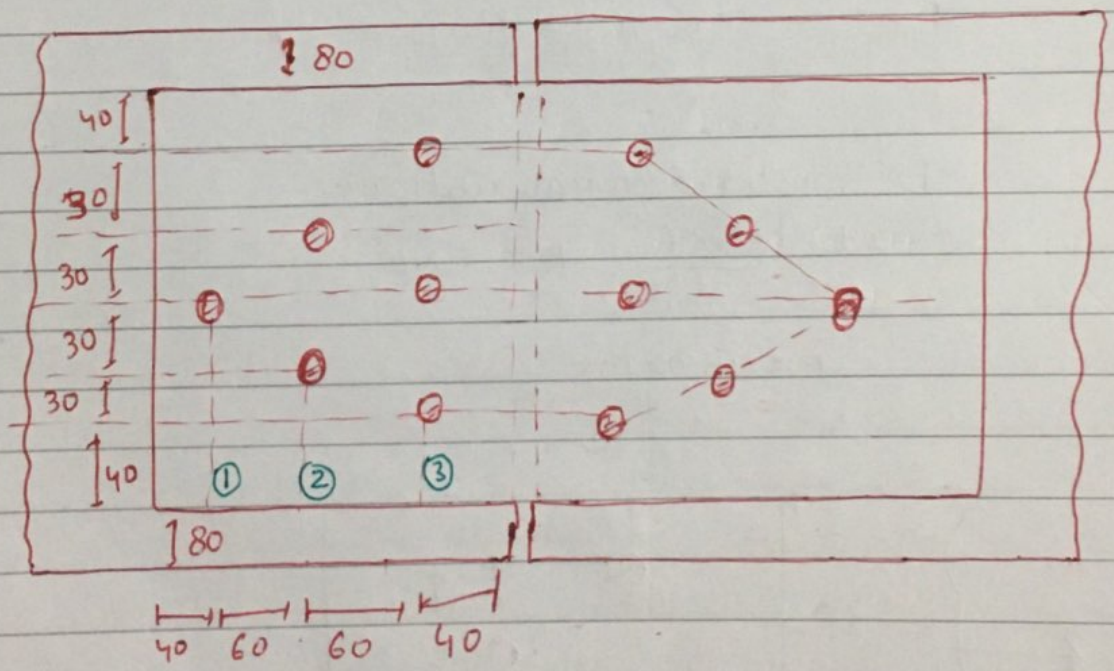
In this case the pitch/Gauge shall be min. of 2.5d considering the Gauge distance b/w two parellel lines of bolt. Min. Gauge = 2.5d = 50 mm so in this case the distance 2p shown in figure shall be 100 mm.

If we conside failure along 2-2 then

$$T_{dn(2-2)} = \left(B - 3d_0 + \frac{2x^2}{4g}\right) t \frac{0.9 f_u}{\gamma_{m1}}$$

$$= \left(50 - 3\times22 + \frac{2\times50^2}{4\times50}\right)\times10\times\frac{0.9\times410}{1.25} = 174.16 \text{ kN}$$

Que: Find the Max Force that can be transferred through double cover butt joint as shown in the figure below. Also find the efficiency of joint. Bolts → M20, 4.6 plate Fe410



thickness of Main plate = 16mm each corner plate = 10m

In double cover butt joints bolts are in double shear

shearing strength of bolt $= \frac{45.27}{0.78} \times 1.78 = \underline{103.31 \text{ kN}}$

$$V_{dsb} = \frac{f_{ub}}{\sqrt{3}\,\gamma_{mb}} \left[n_n A_{nb} + n_s A_{sb} \right] \beta_{lj}\, \beta_{lg}\, \beta_{PK}$$

joint length = c/c distance b/w end ~~distance~~ bolts.

$= 2 \times 60 = 120 < 15d$

$\beta_{lj} = 1$

l_g i.e grip length $= 2 \times 10 + 16 = 36 \text{ mm} < 5d$

$\beta_{lg} = 1$

No packing plate $= \beta_{PK} = 1$

$$V_{dsb} = \frac{400}{\sqrt{3} \times 1.25} \left[1 \times 0.78 \times \frac{3.14}{4} \times 20^2 + 1 \times \frac{\pi}{4} \times 20^2 \right] \times 1 \times 1 \times 1$$

$V_{dsb} = 103.30 \text{ kN}$

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

$$K_b = \text{Min} \left\{ \frac{e}{3d_0}, \frac{p}{3d_0} - 0.25, \frac{f_{ub}}{f_u}, 1 \right\} = 0.606$$

($e \to 40$, $p \to 60$, $d_0 \to 22$, $f_{ub} \to 400$, $f_u \to 410$)

$$V_{dpb} = \frac{2.5 \times 0.606 \times 20 \times \min(2 \times 10, 16) \times 410}{1.25}$$

$V_{dpb} = 159.01 \text{ kN}$

$V_{db} = \text{Min}\{103.3, 159.01\} = 103.3 \text{ kN}$

so force capacity $= 6 \times 103.3 = 619.8 \text{ kN}$ —(i)

$$T_{dn(1-1)} = (B-d_o)t \cdot \frac{0.9 \times f_u}{\gamma_{m1}}$$

$$= (360-22) \times 16 \times 0.9 \times \frac{410}{1.25} = 1596.44 \text{ KN}$$

$$T_{dn(2-2)} = (B_m - 2d_o)t \times \frac{0.9 f_u}{\gamma_{m1}} + V_{db}$$

$$= (360 - 2 \times 22) \times 16 \times 0.9 \times \frac{410}{1.25} \times 103.3 \times 10^3$$

$$= \boxed{1595.84 \text{ KN}} \quad * \quad \text{——(ii)}$$

$$T_{dn(3-3)} = (B_m - 3d_o)\, t \times \frac{0.9 f_u}{\gamma_{m1}} + 3V_{db}$$

$$= (360 - 22 \times 3) \times 16 \times 0.9 \times \frac{410}{1.25} + 3 \times 103.3 \times 10^3$$

$$= 1698.52 \text{ KN}$$

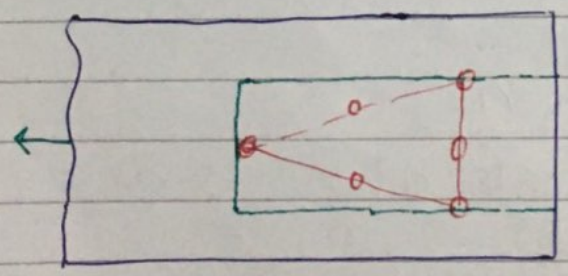
We can also consider possible staggered failure plane

Tearing strength of cover plate

$$T_{dn(3-3)} = (B_c - 3d_o)\, t_c \, \frac{0.9 f_u}{\gamma_{m1}}$$

$$= (200 - 3 \times 22) \times 2 \times 10 \times \frac{0.9 \times 410}{1.25} = 791.2 \text{ KN}$$

(iii)



$$A_{vn} = \left(40 + 2 \times 60 + \frac{d_o}{2} - d_o\right) \times t_m \times 2 = 4768 \text{ mm}^2$$

$$A_{tg} = 4 \times 30 \times 16 = 1920 \text{ mm}^2$$

$$T_{db1} = \frac{0.9 f_u}{\sqrt{3}\,\gamma_{m1}} A_{vn} + \frac{f_y}{\gamma_{m0}} \times A_{tg} = 4373.$$

($f_u \to 410$, $\gamma_{m1} \to 1.25$, $f_y \to 250$, $\gamma_{m0} \to 1.1$)

$T_{db1} = 1248.9$ kN

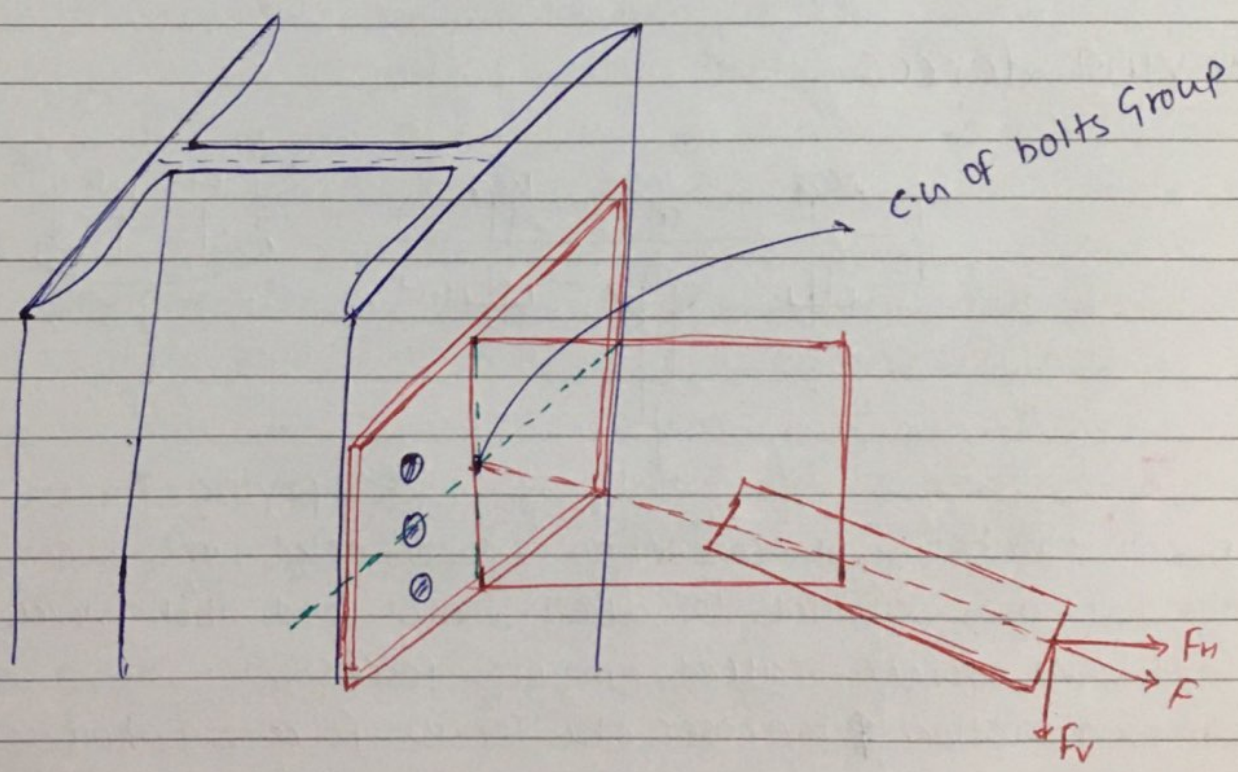
Generally no need to check block shear failure.

Strength of joint = Min. $\{619.8, 1595.84, 791.2\}$

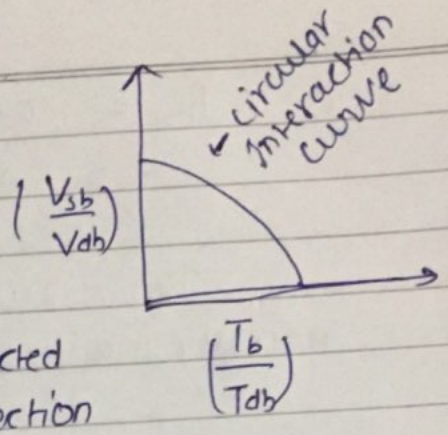
$= 619.8$ KN

$$\text{efficiency of joint} = \frac{619.8}{\frac{f_y}{\gamma_{m0}} B_m t_m} = \frac{619.8}{\frac{250}{1.1} \times 360 \times 16} \times 100 = 47.34\%$$

Bolt Subjected to combined shear & Tension



$$\left(\frac{V_{sb}}{V_{db}}\right)^2 + \left(\frac{T_b}{T_{db}}\right)^2 \leq 1$$

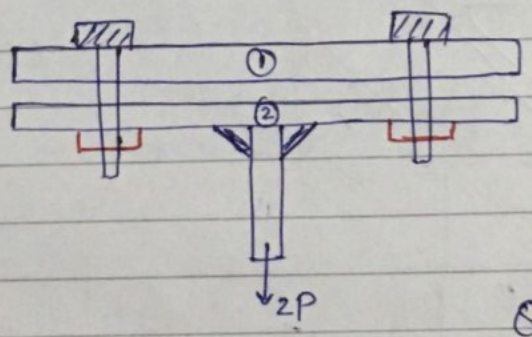


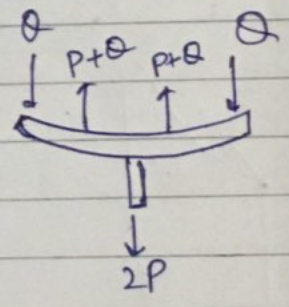
In most cases when bolts are subjected to both shear and Tension, the connection is loaded eccentrically However in some cases the member force can pass through the centroid of connection as shown above

$V_{sb} \rightarrow$ Factored shear force in bolt
$V_{db} \rightarrow$ shear capacity or Bearing capacity Design (Bolt Value)

$T_b \rightarrow$ Factored Tensile force in bolt
$T_{dn} \rightarrow$ Design Tension capacity of bolt.

prying forces





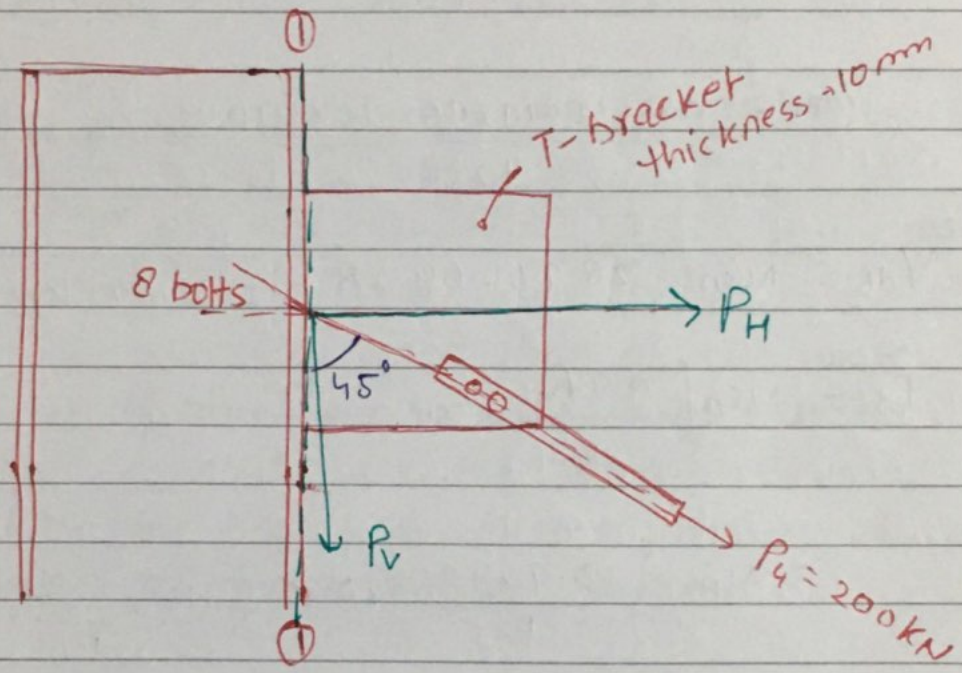
$Q \rightarrow$ Prying force

Due to Flexibility of connection i.e connected part when the bolts are subjected to direct tensile load, bolts develop additional forces called prying forces.

- prying action increases the Tensile force in bolt
- it can be reduced by using thick place or by limiting the lever Arm. i.e the distance b/w bolts

* In case of ∞ rigid plate, pyring force = 0

Que check the safety of the joint shown below 8 no. of M16 bolts of 4.6 used for making the connection @ sec^n ①-①. Neglect the effect of prying action. Also find the no. of 16 mm dia bolts of Grade 4.6 to connect the double Angle sec^n (8mm thick angle) with the Web of the T-bracket



the bolt @ sec^n ① are in direct shear and direct tension

verticle force due to P_4 $\quad P_V = 200 \sin 45 = \frac{200}{\sqrt{2}}$ kN

$P_H = 200 \cos 45 = \frac{200}{\sqrt{2}}$ kN

Direct shear per bolt $= \frac{200}{\sqrt{2} \times 8} = 17.68$ kN

Direct Tension per bolt = 17.68 kN

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

$$k_b = \min\left\{ \frac{e}{3d_0}, \frac{p}{3d_0} - 0.25, \frac{f_{ub}}{f_u}, 1 \right\} = 0.49$$

$e \to 1.5d_0$, $p \to 2.5d$, $d_0 \to 16$, $f_{ub} \to 400$, $f_u \to 410$

~~$V_{db} = \min$~~ $V_{dsb} = \frac{f_{ub}}{\sqrt{3}\gamma_{mb}} 0.78 A_{sb}$ $\quad A_{sb} \to \pi/4 (16)^2$

$= 28.97$ KN

$$V_{dpb} = \frac{2.5 \times 0.49 \times 16 \times 10 \times 410}{1.25} = 64.28 \text{ KN}$$

$$V_{db} = \min\{28.97, 64.28\} = 28.97 \text{ KN}$$

$$T_{db} = \min\left\{ \frac{0.9 f_{ub}}{\gamma_{mb}} \cdot A_{nb}, \frac{f_y}{\gamma_{mo}} \times A_{sb} \right\}$$

$$= \min\left\{ \frac{0.9 \times 400}{1.25} \times 0.78 \times \frac{\pi}{4} \times 16^2, \frac{240}{1.1} \times \frac{\pi}{4}(16)^2 \right\}$$

$$= \min\{45.15, 43.85)$$

$$= 43.85 \text{ KN}$$

$$\left(\frac{V_{sb}}{V_{db}}\right)^2 + \left(\frac{T_b}{T_{db}}\right)^2 = \left(\frac{17.67}{28.97}\right)^2 + \left(\frac{17.67}{43.85}\right)^2$$

$$= 0.535 < 1 \quad \underline{\text{safe}}$$

The double angle sec^n connected back to back on opposite side of the T-bracket, in this case the bolt will be in double shear ⇒

$$V_{dsb} = \frac{28.97}{0.78} \times 1.78 = 66.11 \text{ KN}$$

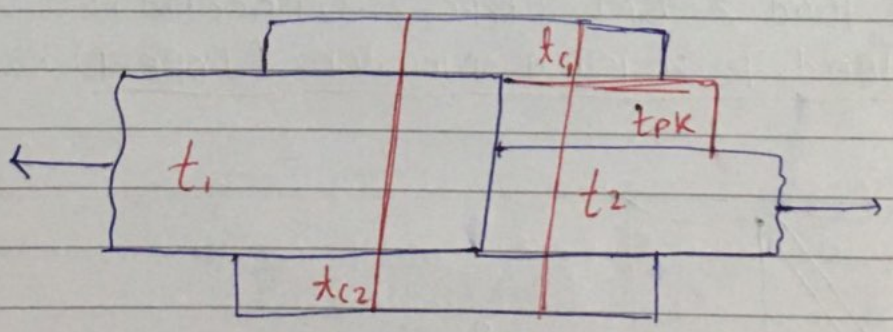
$$V_{dpb} = \frac{2.5\, k_b\, d\, t\, f_u}{\gamma_{mb}}$$

$$= \frac{2.5 \times 0.49 \times 16 \times 10 \times 410}{1.25} = 64.28 \text{ KN}$$

$$V_{db} = \min(66.11, 64.28) = 64.28 \text{ KN}$$

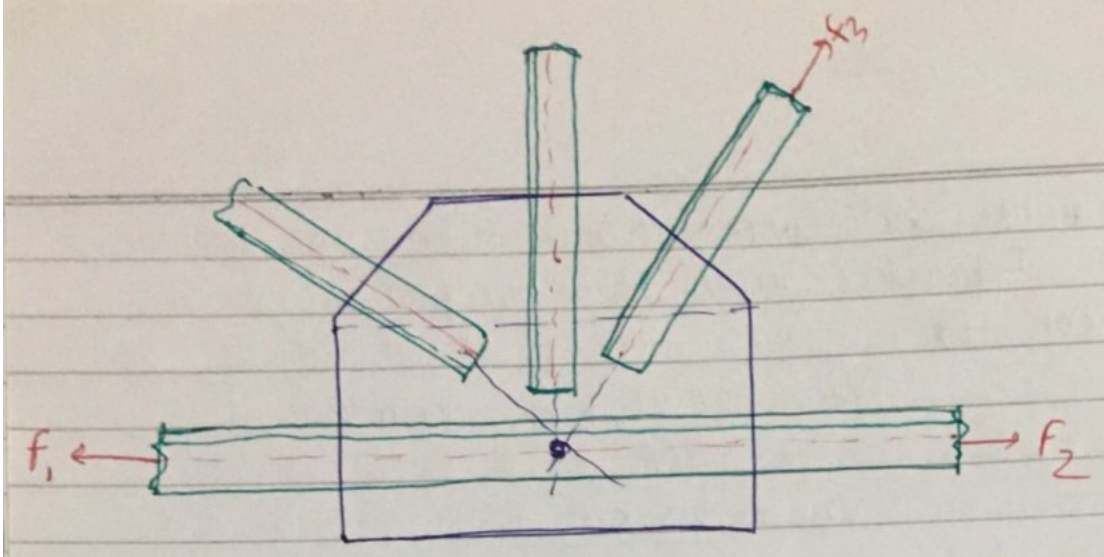
No. of bolts required $= \frac{200}{64.28} = 3.11$, provide → 4

NOTE In case of connection of two plates of Diff^n thicknesses using double cover butt joint we need to calculate the strength of joint as well as plate for both side and the smaller value shall be taken as the strength of the joint



② $V_{db} = \text{Min}\left\{ \frac{f_{ub}}{\sqrt{3}\,\gamma_{mb}} (n_n \cdot A_{nb} + n_s \cdot A_{sb}) \times \beta_{pk},\ \frac{2.5\, k_b\, d\, t\, f_u}{\gamma_{mb}} \right\}$ $\min\{t_2, t_{c1}+t_{c2}\}$

① $V_{db} = \text{Min}\left\{ \frac{f_{ub}}{\sqrt{3}\,\gamma_{mb}} (n_n \cdot A_{nb} + n_s \cdot A_{sb}),\ \frac{2.5\, k_b\, d\, t\, f_u}{\gamma_{mb}} \right\}$ $\min\{t_{c1}+t_{c2}, t_1\}$



$F_1 = F_2 \rightarrow$ NO Bolt

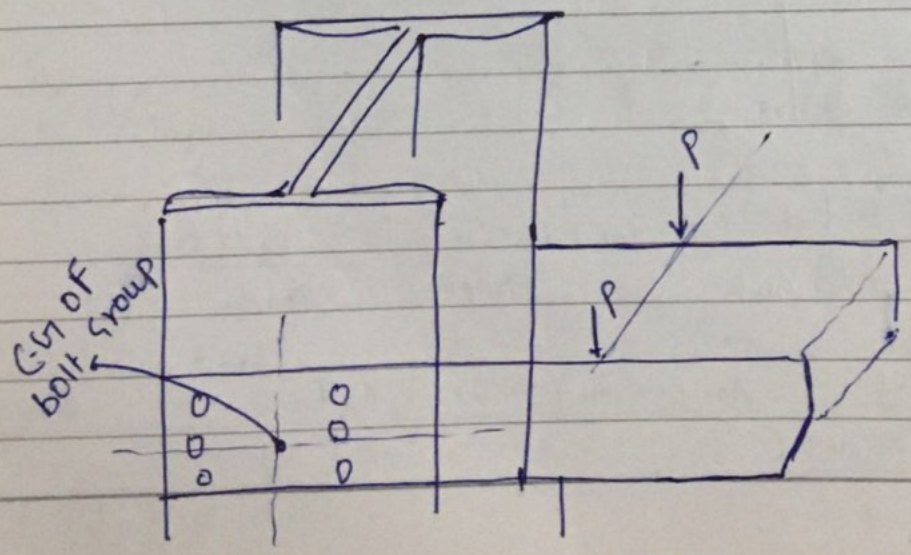
o/w $|F_1 - F_2|$ के लिए Bolt

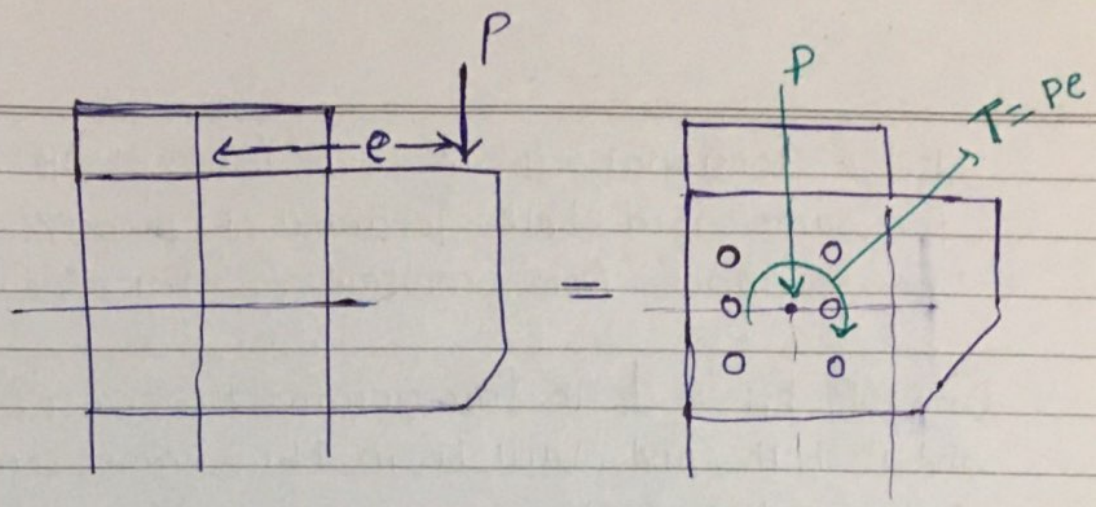
- For a contineous member, connection in the gusset plate shall be designed for a force of $(F_1 - F_2)$
- for discontineous member, connection shall be designed for the respective forces.

Eccentrically loaded connections :- Depending on How the bolts are placed in a connection, bolts can be subjected to ① direct load and twisting Moment (type-1 connection)

② Direct load & BM (type-2 connection)

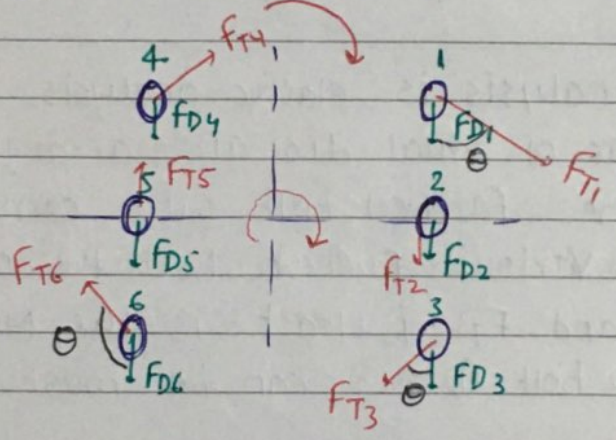
① Direct load + Twisting Moment (Type-1)





$$F_{Di} = \frac{P}{\sum_{i=1}^{n} A_i} \cdot A_i = \frac{P}{nA_i} \cdot A_i = \frac{P}{n}$$ (for same dia bolt)

$F_{Di} \rightarrow$ Direct force in i^{th} bolt



The line of action of F_{Di} is taken parallel to the line of applied force.

$$\tau = \frac{T \cdot r}{J}$$

$$F_{Ti} = \frac{T r_i}{J} \times A_i$$

$$F_{Ti} = \frac{T r_i}{\sum_{i=1}^{N} A_i r_i^2} \cdot A_i = \frac{T r_i}{\sum r_i^2}$$

for same dia bolts.

F_{Ti} = Torsional shear force in i^{th} bolt
for same Dia bolt Torsional SF $\propto Y_i$
ie Distance from centroid of bolt Group.

Dir^n of F_{Ti} is $\perp$ to line joining the CG of bolt Group and i^{th} bolt and will be in the same sense as that of torsional moment.
The resultant SF in i^{th} bolt

$$F_{ri} = \sqrt{F_{Di}^2 + F_{Ti}^2 + 2 F_{Di} F_{DTi} \cos\theta}$$

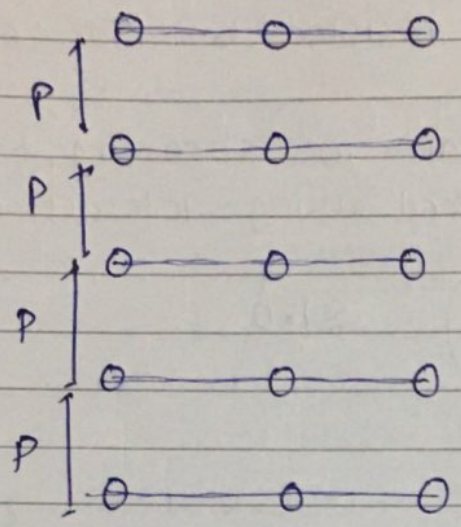
For safety of the bolt $F_{ri} < V_{db}$ (i.e bolt Value)

NOTE ⇒ • The above analysis is elastic analysis
• In case bolts are of equal dia and arrangement is symmetrical the farthest bolt will carry Max. resultant force or various farthest bolt the one which angle b/w F_{Di} and F_{Ti} is least is the most critical so in this case bolt ① & ③ can be most critical

• In general the bolt which is closest to the line of action of the force is the most critical.

• In plastic or strength analysis, it is assumed that eccentrically loaded bolt Group rotates about an instantaneous center of rotation
Elastic analysis is more Conservation.

• To Design the eccentric connection We've to assume no. and arrangement of bolts.



$$n = \sqrt{\frac{6M}{m\,p\,V_{db}}}$$

M → Twisting moment
$= P \cdot e$

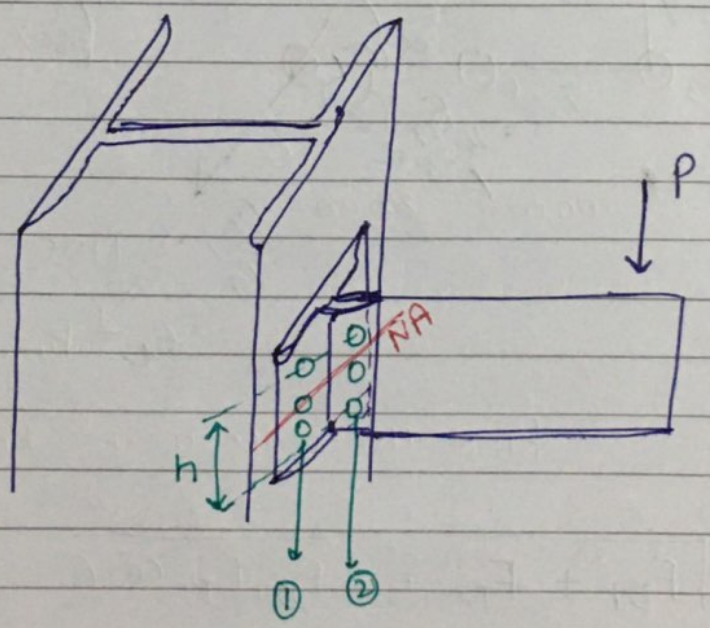
p → pitch

V_{db} → bolt value

n → no. of bolts per line

m → no. of lines of bolts.

② Direct load and BM



Bolts above NA → subjected to shearing and bearing, & Tension

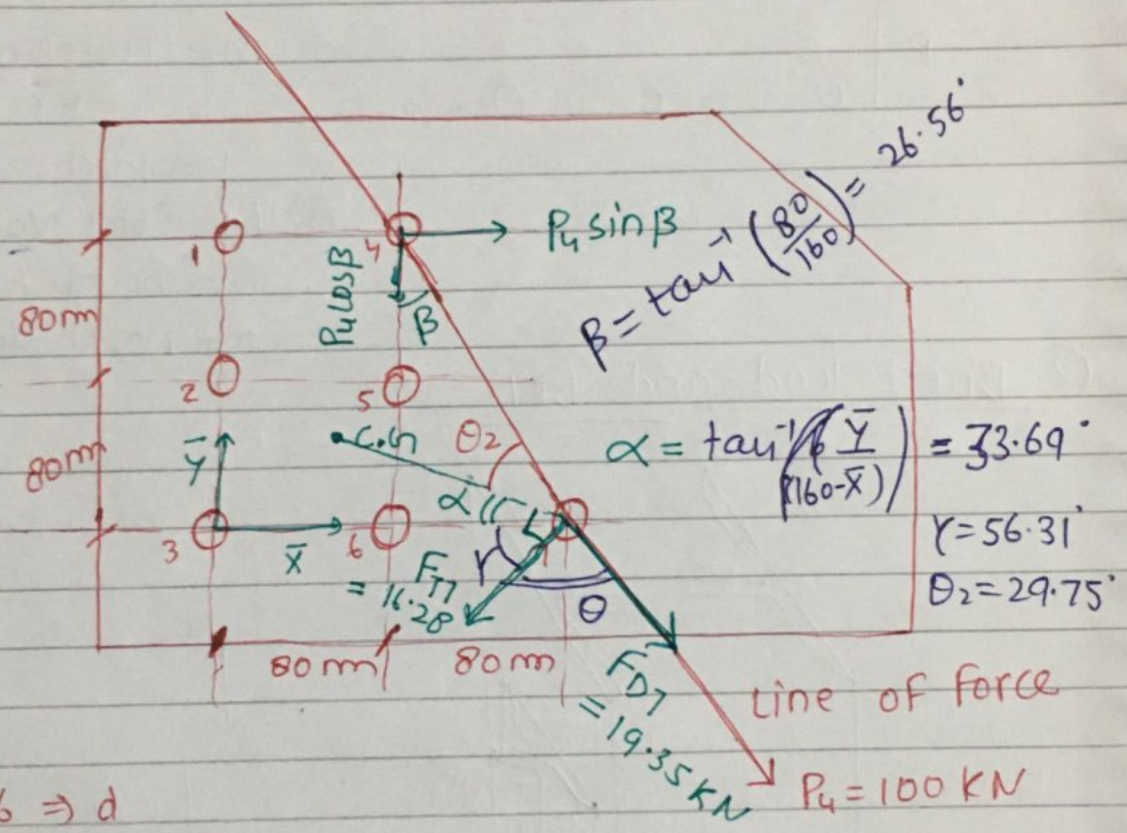
Bolts below NA → Bearing & shearing, No comp. stress

Above the top bolt, bracket will loose contant and hence h will to upto center line of top bolt.

- compressive stresses are resisted by flange of I-sec^n below the NA
- safety of the bolts that are in shearing, bearing and tension can be checked using interaction eq^n i.e

$$\left(\frac{V_{sb}}{V_{db}}\right)^2 + \left(\frac{T_b}{T_{db}}\right)^2 \leq 1.0$$

Que:



$$\text{Dia} = \begin{cases} 1 \to 6 \Rightarrow d \\ 7 \Rightarrow 1.2d \end{cases}$$

Find the force in 7th bolt.

$$F_{R7} = \sqrt{F_{D7}^2 + F_{T7}^2 + 2F_{D7}F_{T7}\cos\theta}$$

$$F_{D7} = \frac{P_4 \cdot A_7}{\sum_{i=1}^{7} A_i} = \frac{P_4(1.2d)^2}{6d^2 + (1.2d)^2} = \frac{P_4 \times 1.44}{7.44}$$

$$= 19.35\,kN$$

$$f_{T7} = \frac{T \cdot r_7}{J} \cdot A_7$$

C.G OF bolt Group

$$\bar{X} = \frac{3A \times 0 + 3A \times 80 + 1.44A \times 160}{7.44A} = 63.22 \text{ mm.}$$

$$\bar{Y} = \frac{3.44A \times 0 + 2A \times 80 + 2A \times 160}{7.44A} = 64.52 \text{ mm.}$$

$$T = P_u \cos\beta\,(80 - \bar{x}) + P_u \sin\beta\,(160 - \bar{x})$$

$$T = 100 \cos 26.56\,(80 - 63.22) + 100 \times \sin 26.56\,(160 - 64.52)$$

$$T = 5.77 \text{ KNm.}$$

$$F_{T7} = \frac{T \cdot r_7}{J} A_7$$

$$J = \sum_{i=1}^{7} A_i r_i^2$$

$$r_1 = \sqrt{(\bar{x})^2 + (160 - \bar{y})^2} = 114.59 \text{ mm}$$

$$r_2 = \sqrt{\bar{x}^2 + (80 - \bar{y})^2} = 65.08 \text{ mm}$$

$$r_3 = \sqrt{\bar{x}^2 + \bar{y}^2} = 90.33 \text{ mm}$$

$$r_4 = \sqrt{(80 - \bar{x})^2 + (160 - \bar{y})^2} = 96.94 \text{ mm}$$

$$r_5 = \sqrt{(80 - \bar{x})^2 + (80 - \bar{y})^2} = 22.83 \text{ mm.}$$

$$r_6 = \sqrt{(80 - \bar{x})^2 + \bar{y}^2} = 66.67 \text{ mm}$$

$$r_7 = \sqrt{(160 - \bar{x})^2 + \bar{y}^2} = 116.31 \text{ mm}$$

$$F_{T7} = \frac{\overset{5.77\times10^3}{T} \cdot \overset{116.31}{r_7} \; \overset{1.44A}{A_7}}{A\left(114.51^2 + 65.08^2 + 90.33^2 + 96.94^2 + 22.83^2 + 66.67^2 + 116.31^2 \times 1.44\right)}$$

$$F_{T7} = 16.28 \text{ KN}$$

$$\theta = 60.25^\circ$$

$$F_{R7} = \sqrt{F_{D7}^2 + F_{T7}^2 + 2F_{D7}F_{T7}\cos\theta}$$

$$F_{R7} = 30.85 \text{ KN}$$

④ WELDED CONNECTIONS

- Generally in structural work we use electrical arc welding
- Due to sparking, temp. rises to about 3300°– 5000°c

The welding rod is coated that deposits slag and prevent contact of atmospheric agent with the molten weld metal. This prevent the changes of corrosion and weld defects.

+ve

* In case of welding complicated shapes can be welded and alration can be easily done.
  Also DL of structure will be also less* than that of bolted structure. No cover plate required and less expensive.
  The rigidity of welded connecting will be higher than that of bolted connection (Bearing bolt)

-ve

* • Requires highly skilled labour, ~~exp~~ inspaction is difficult and expensive and fillet weld is prone to fatigue failure

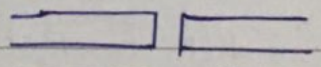
However in general welding is better than bolting.

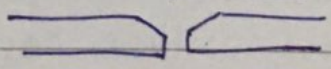
Various types of weld.

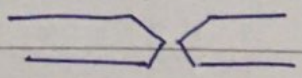
① Butt weld
② Fillet weld
③ plug weld
④ slot weld

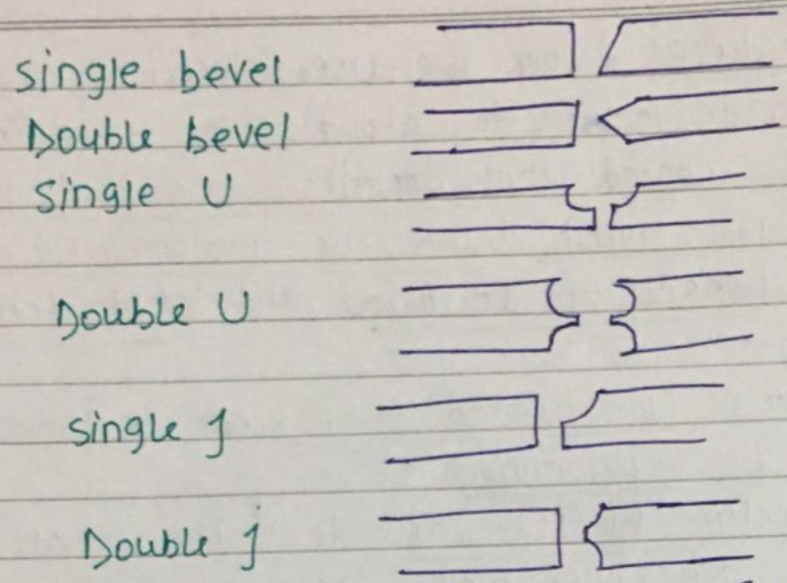
① Butt weld used to connect structural member that are alligned in same plan.

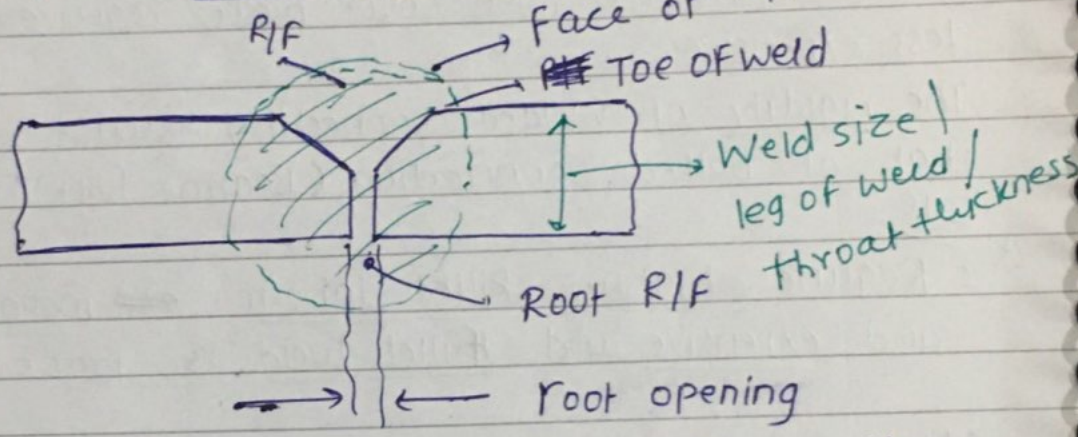
Diff^n types of Butt weld are:–

square butt weld 

Single V 

Double V 





- square butt weld is provided for sec^n less than 8 mm thick

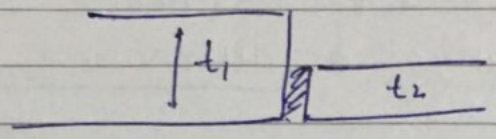
- single U, V, J for 12-40mm thick plate
  Double U, V, J for plate thickness >40 mm

- Butt weld is more suitable * for alternating stresses providing complete penetration is ensured

- Due to stress conc^n R/F is not suitable for alternating stress and in this case weld surface is made flush.
  However R/F makes butt weld stronger in static load.

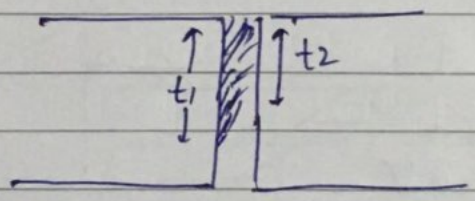
Weld code → IS 816
for Metal Arc Welding.

In any case the R/F shouldn't exceed 3 mm
if the weld metal is deposited in complete thickness, butt weld is called complete penetration butt weld o/w partial penetration butt weld.
In case of complete penetration butt weld, strength of connection is taken equal to that of the strength of the parent plate (thinner plate for Diff^n thicknesses) *



$t = \text{Min}(t_1, t_2)$

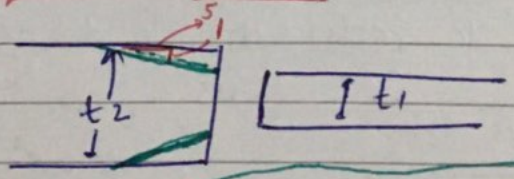
For incomplete penetration, effective throat thickness is taken min. thickness of weld commen to the parts being joined.



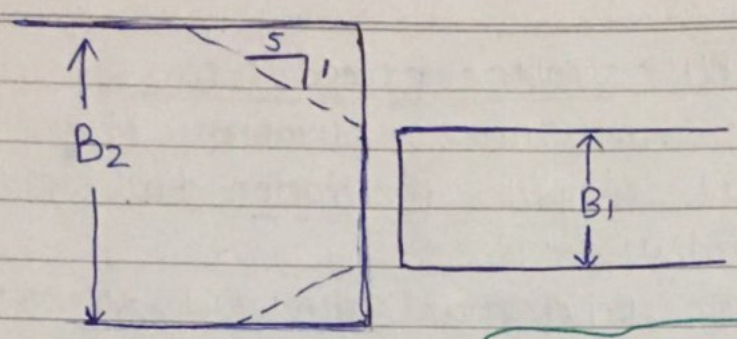
$t = \text{Min.}\{t_1, t_2\}$

In case the full penetration of butt weld is not achieved, an eff. throat thickness $\frac{7}{8}$ of the thickness of thinner member shall be provided. However for calculating the strength, eff. throat thickness is assumed to be 5/8 * of thickness of thinner member.

Joining plates with Different thickness and width in in case of butt weld



if $t_2 - t_1 > \frac{t_1}{4}$ or 3 mm, whichever is Greater, we provide tapering in the thicker part to match the thinner part and tapering should not be Greater than * $\boxed{1 \text{ in } 5}$

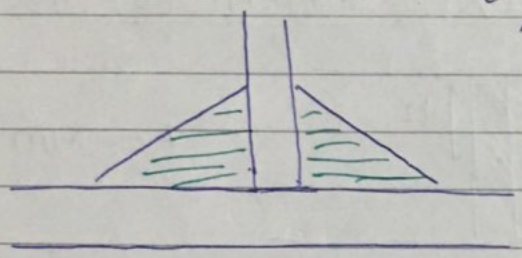


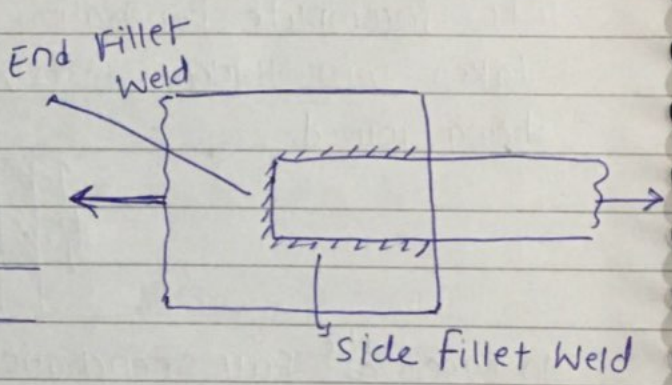
1 in 5

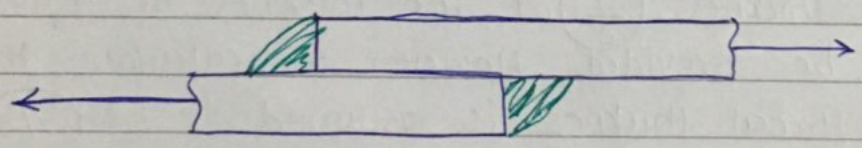
if $B_2 - B_1 > \frac{B_1}{4}$ then provide tappering.

- The above is generally done for structure to take care of Dynamic / repeating or alternating stresses.

Fillet Weld provided when two members to be jointed are in diff^n plane

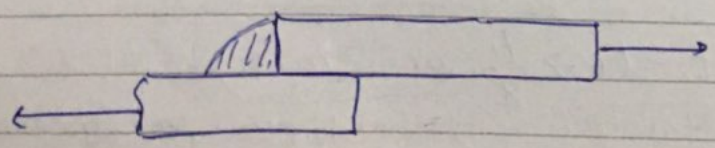


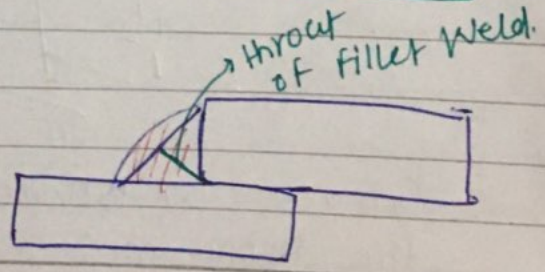




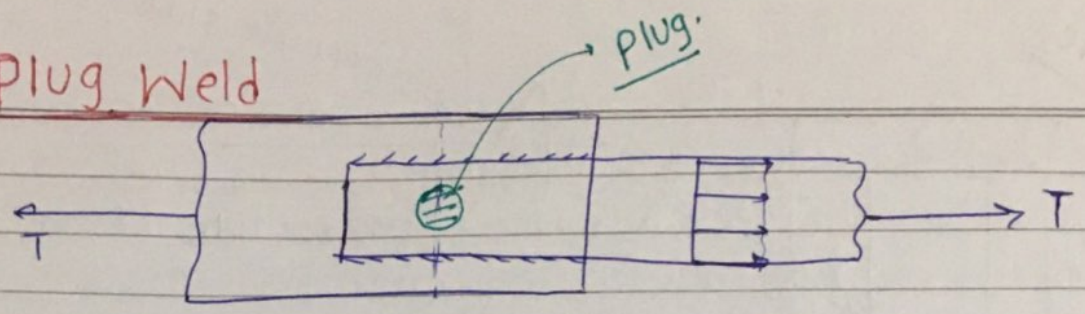
End Fillet Weld is generally stronger than side fillet Weld However For design purpose there is no diff^n b/w end and side Fillet Weld. *

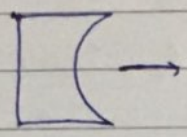
Fillet Weld is assummed to resist the load by shearing action on it's throat.



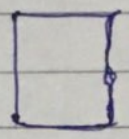


Plug Weld





→ Stress diagram without plug weld

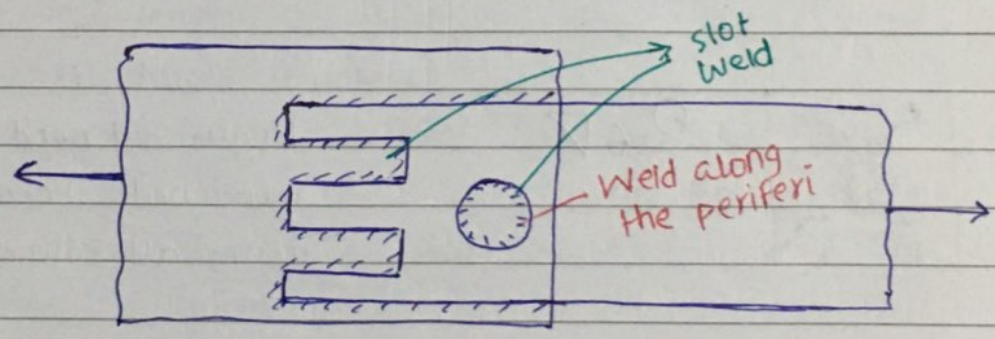


Stress dia with plug Weld
(More uniform)

*

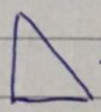
plug Weld is provided to make the stress uniform, fill up holes in connection and prevent buckling of overlapping parts.
As per IS code, plug Weld is not considered in strength calculations.

Slot Weld

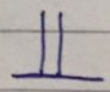


- In case of slot welding, and welding is done along the periferi of slot, it is considered in strength calculation
- Slot Weld is provided when the largest size of weld provided in the limited overlap length is not sufficient to resist the load.

Weld symbol



→ Fillet Weld

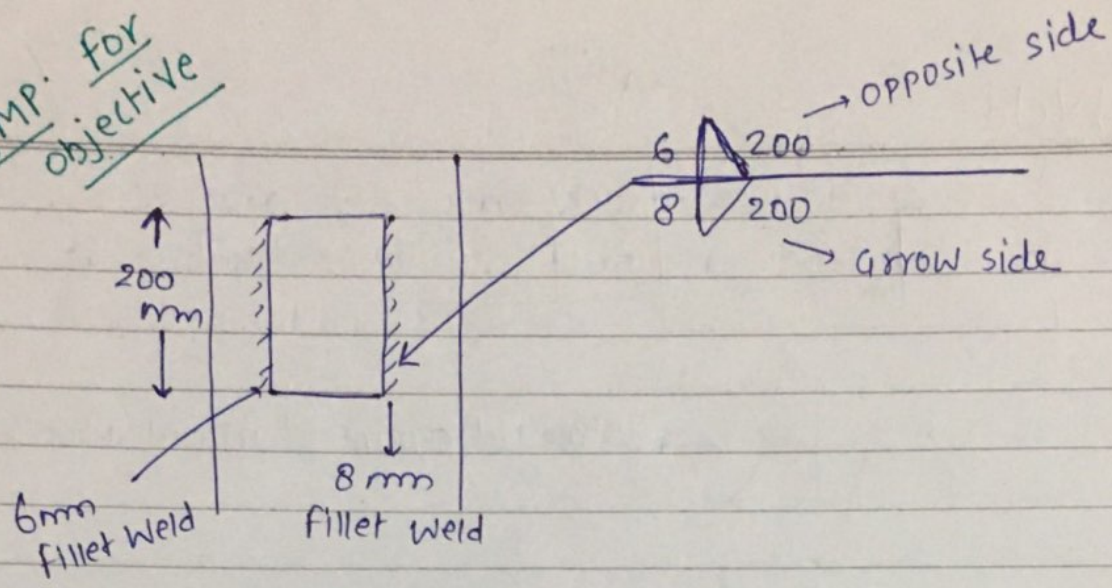


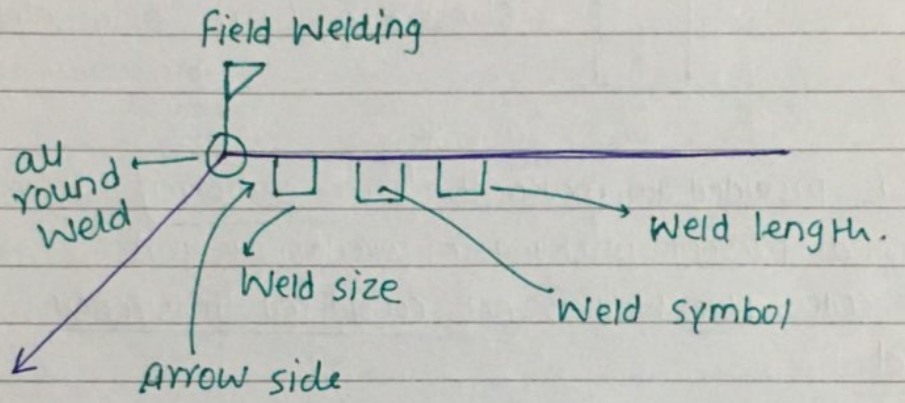
→ Square butt Weld

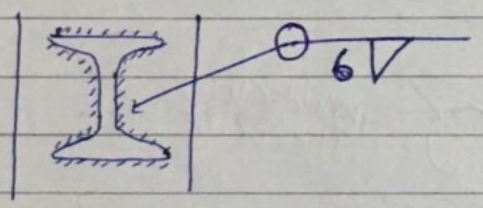
V → single V Y → single U

X → Double U

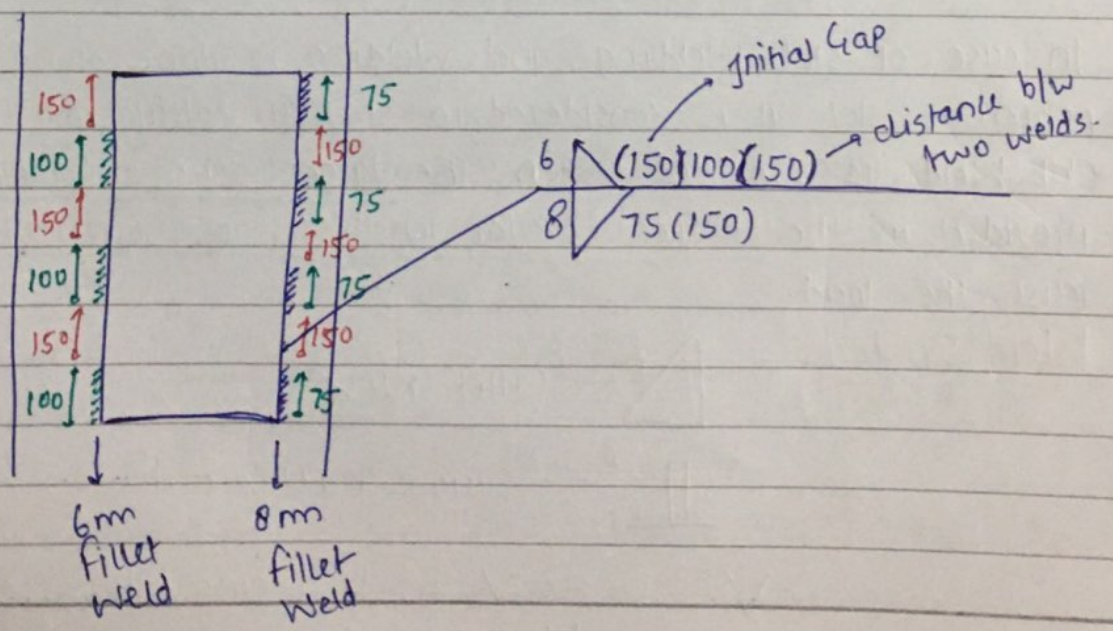
Imp. for objective







length not need to be mentioned, because it is all rounded welding.

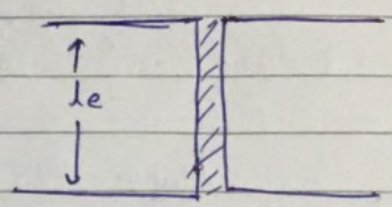


Design of butt Weld

As per IS code butt Weld shall be treated as parent metal with a thickness equal to throat thickness and the stress shall'nt exceed that permitted in the parent metal.

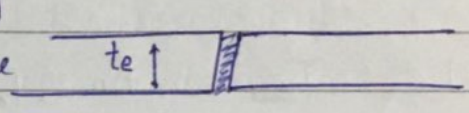
A) The design strength of butt Weld in Tension and comp. is governed by yielding *

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



PLAN

$F_y \rightarrow$ Min. of yield strength of Weld (f_{yw}) and yield strength of plate (f_y)



$l_e \rightarrow$ eff. length of weld

$t_e \rightarrow$ eff. throat thickness.

$\gamma_{mw} \rightarrow$ partial safety factor, 1.25 for shop Weld
1.5 for Field Weld

B) The design strength of butt Weld in shear is also governed by yielding * and.

$$V_{dw} = \frac{f_y}{\sqrt{3}\,\gamma_{mw}} l_e t_e$$

Que. two plates of 16 & 14 mm are to be joined by a groove Weld the joint is subjected to a factored tensile force of 430 KN, considering eff. length of Weld = 175 mm check safety of joint

A) For a singe V groove weld

B) for a Double V groove weld.

Take Fe410

NOTE In case of single V groove weld, incomplete penetration is considered and t_e shall be taken as $\frac{5}{8} \times \min(t_1, t_2)$

i) $$T_{dw} = \frac{f_y}{\gamma_{mw}} l_e t_e$$

$$t_e = \frac{5}{8} \times \min(t_1, t_2) = \frac{5}{8}(14) = 8.75 \text{ mm}$$

$$T_{dw} = \frac{250}{1.5 \text{ (assume field weld)}} \times 175 \times 8.75 = 255.20 \text{ KN} < 430 \text{ KN}$$

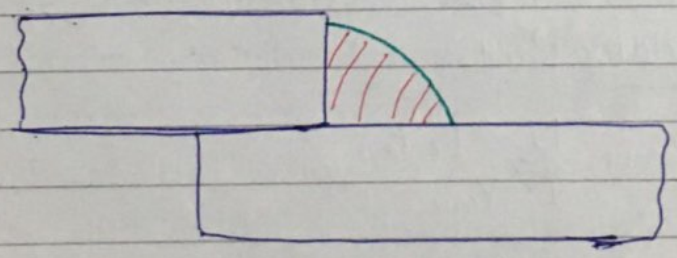
NOT safe

ii) $$T_{dw} = \frac{f_y}{\gamma_{mw}} \times l_e t_e$$

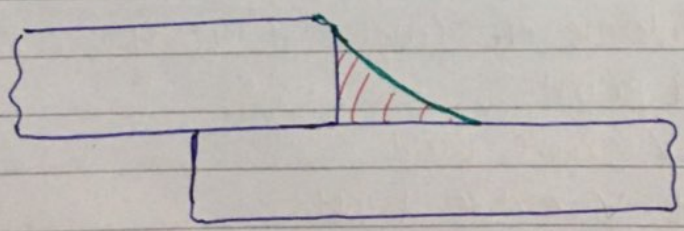
$$= \frac{250}{1.5} \times 175 \times \min(14, 16) = 408.33 \text{ KN} < 430 \text{ kN}$$

NOT safe

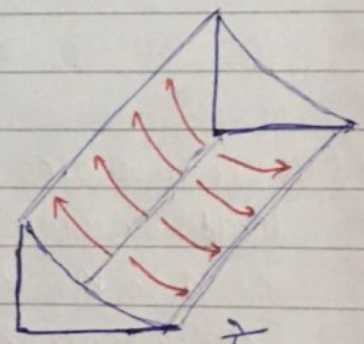
Design of Fillet Weld



Convex fillet weld



(Concave Fillet Weld)



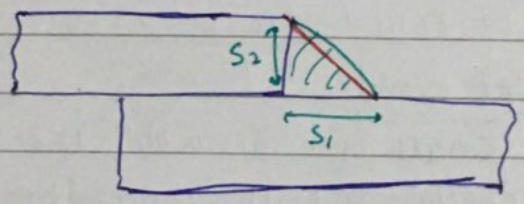
crack के chances है।

- We can provide concave & convex fillet weld
- Convex weld is stronger than concave weld due to higher throat thickness and penetration.
- Concave weld are most suitable under alternating stresses because it offers smooth path for the flow of stress
- In concave fillet weld there is a chance of crack on cooling hence it is generally made in two passes.

✓ The first slightly convex and other is build up to form a concave fillet weld *

IS code specifications

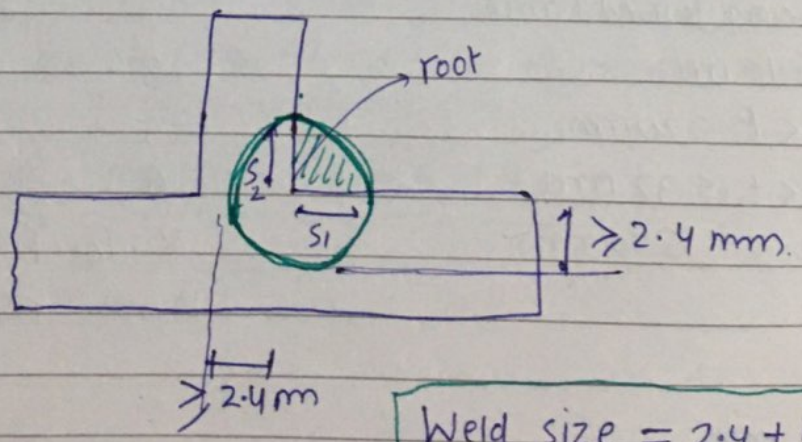
① **size of weld** The largest Right angle tringle that can be inscribed in weld gives size of weld. For normal fillet weld size is taken as min. weld leg size.



$S_1, S_2 \rightarrow$ size of weld / leg of weld

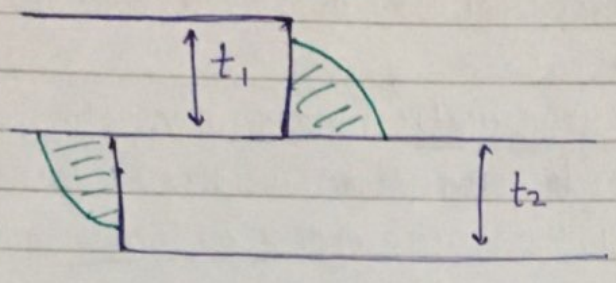
$$\text{size} = \min(S_1, S_2)$$

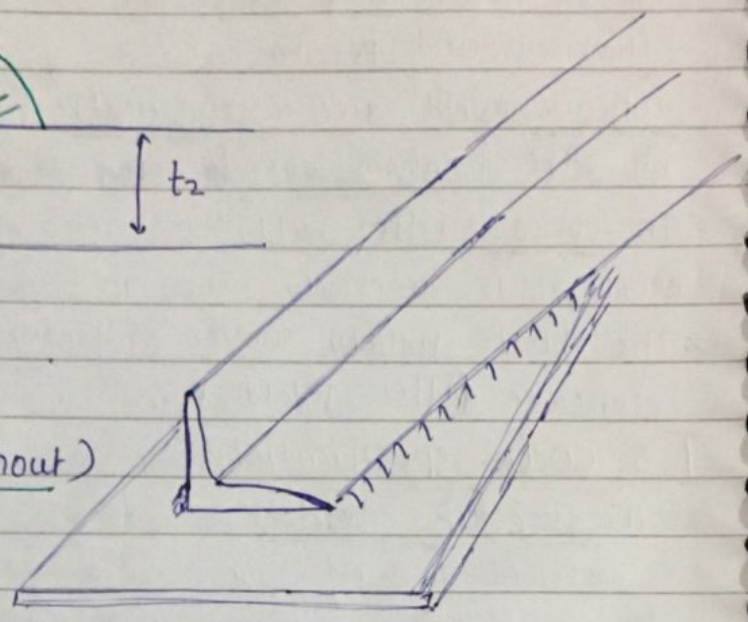
For deep penetration weld where depth of penetration beyond the root run is min. 2.4 mm, size of weld is taken as 2.4 mm + min. leg size



$$\boxed{\text{Weld size} = 2.4 + \min(S_1, S_2)}$$

(A) Max. size of Fillet Weld





For square edge *

$\boxed{\text{Max size} = t - 1.5\ mm}$

(to avoide chances of washout)

For round edge,

$\text{Max. size} = \frac{3}{4} t$

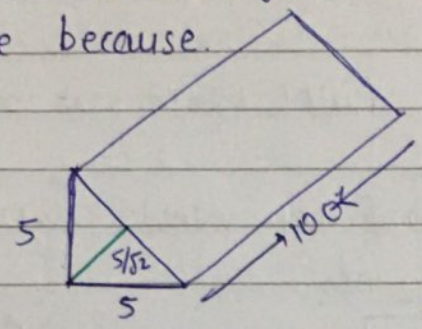
$t \rightarrow$ nominal thickness of round edge

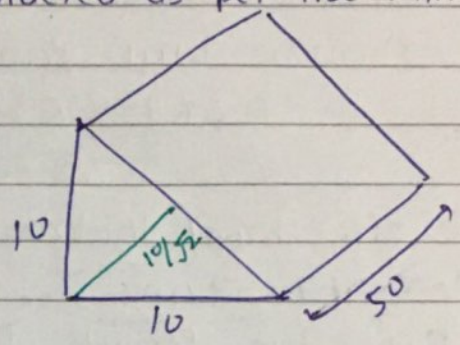
(B) Min. size of Fillet Weld

if we provide very small size of weld, the thicker plate will not be heated properly, and there can be inadequate joint b/w weld metal ~~plate~~ and plate. Also cooling rate of weld metal will be higher compared to that of plate. This can lead to brittle failure of weld or distortion in the joint.

thickness of $\boxed{\text{thicker}}$ plate being joined (mm)	Min. size of weld (mm)
* $t \leq 10\ mm$	3
$10 < t \leq 20\ mm$	5
$20 < t \leq 32\ mm$	6
$32 < t \leq 50\ mm$	8 for first run and 10 mm minimum (total)

* When thicker part is more than 50 mm, special precautions like preheating shall be done
- The Min. size however should not be more than thickness of thinner plate being joined.
- The Weld size is generally considered as per the Min. Weld size because.





$$\text{Area} = \frac{5}{\sqrt{2}} \times 100$$

$$\text{Volume} = \frac{1}{2} \times 5 \times 5 \times 100$$

$$= 1250$$

$$\text{Area} = \frac{10}{\sqrt{2}} \times 50$$

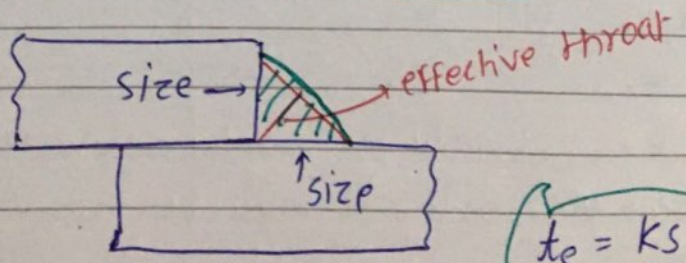
$$\text{Volume} = \frac{1}{2} \times 10 \times 10 \times 50$$

$$= 2500$$

- larger size Weld requires more than one run of weld hence the cost increases because of chipping and cleaning, Which is required for proper bond of successive runs.
- smaller size Weld is cheepar than a larger one for same strength since less Vol^m ~~is~~ of Welding is req.

However connection length will increase with small Weld size so in EXAM consider b/w min. and max. Weld size

② Effective throat thickness (t_e)



$$t_e = KS$$

S → size of Weld

*

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

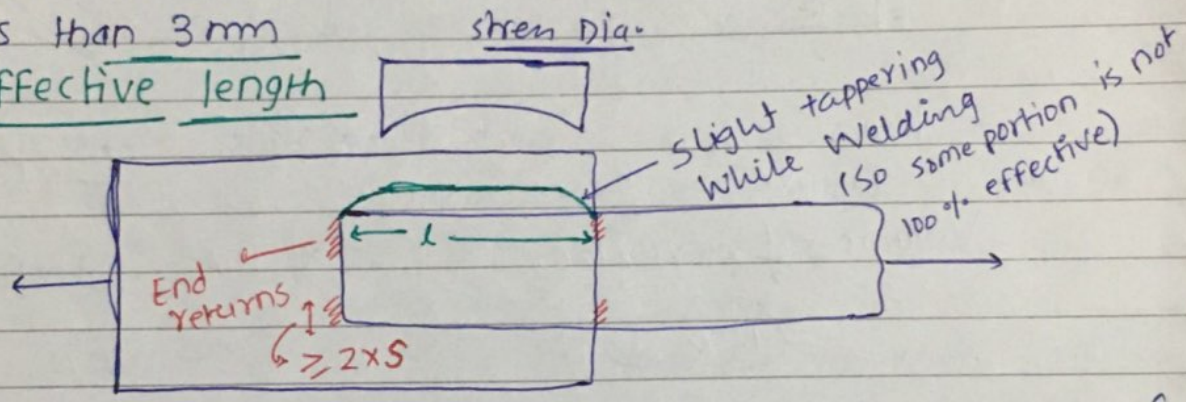
$\theta \rightarrow$ Angle b/w fusion faces.

throat thickness is the Min. thickness on which shearing will occure

if $\theta < 60°$, $\theta > 120°$, fillet weld not recommended!

The eff. throat thickness for a fillet weld shall not be less than 3 mm

③ Effective length

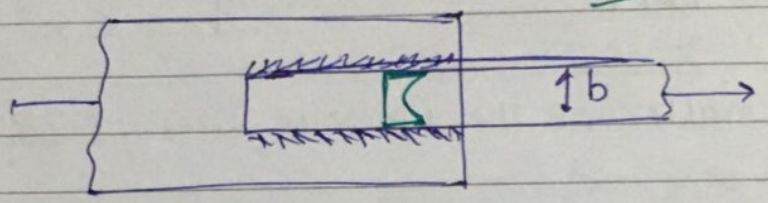


- eff. length is the length of fillet weld which is of specified size and req. throat thickness
- eff. length of fillet weld = overall length − 2S
- eff. length ≮ 4S
- eff. length of butt weld shall be taken as the length of the contineous full size weld but ≮ 4S
- In the structural Drawing the length mentioned is always eff. length.
- * Min length req. for a fillet weld to be 100% effective is 4S, if Min length < 4S then eff. size is taken as $\frac{1}{4}$ × eff. length
- End returns are provided to ~~relaxe~~ relieve the weld from high stress conc^n near the ends. It must be provided for tension

and parts carrying bending loads for welded joints subjected to eccentricity, stress reversal or impact load. End returns shall be taken as $\geq$ 2 x size of weld. *

- When end of an element is connected only by fillet weld parallel to the dir^n of stress than $b \ngtr l$

$$b \ngtr 16t$$



$t \rightarrow$ thickness of thinner plate being joined

$b \rightarrow$ width of smaller plate

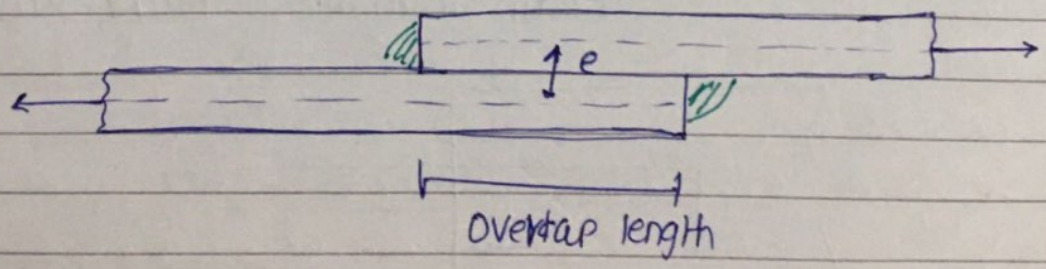
if the length of side filled weld exceeds 150 te, strength of weld is reduced by a factor β

$$* \quad \beta = 1.2 - \frac{0.2 l_j}{150 t_e} \leq 1$$

$l_j \rightarrow$ length of welded joint in dir^n of load transfer

④ Effective Area $\quad A_e = l_e t_e$

⑤ overlap length



* overlap shall be < max. (4t, 400 mm) → This will ensure that induced eccentricity is ignored and BM will not be considered.

⑥ Design strength of Fillet Weld

$$P_{dw} = \frac{F_u}{\sqrt{3}\,\gamma_{mw}} l_e t_e \quad *$$

$$t_e = K \cdot S$$

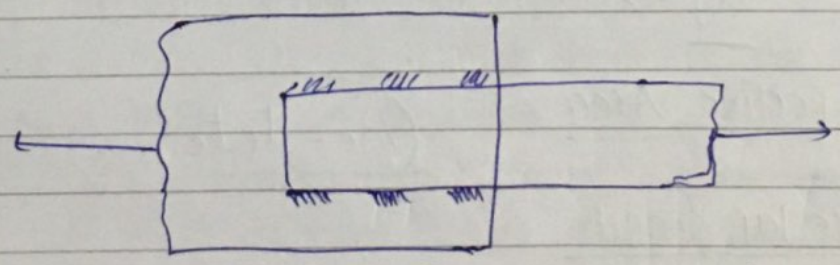
F_u → smaller of the ultimate stress of Weld or parent metal

Intermittent Fillet ~~bott~~ Weld

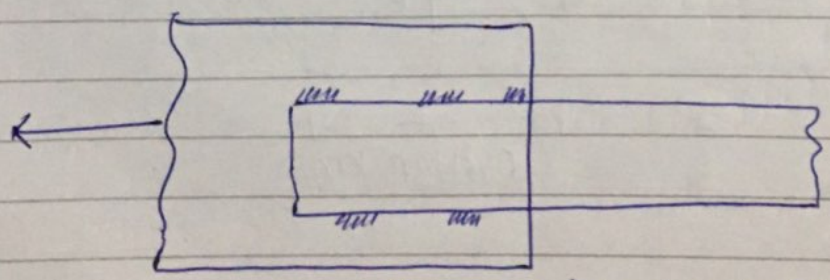
it may be provided if the length of smallest size weld required is < contineous length of joint

clear spacing b/w eff. length in case of intermitten fillet Weld shall be ≯ min. (12t, 200 mm), for member in Compressing and ≯ (16t, 300 mm) for member in Tension

t → thicknem of thinner part being joined.



Chain intermittent Filled Weld.

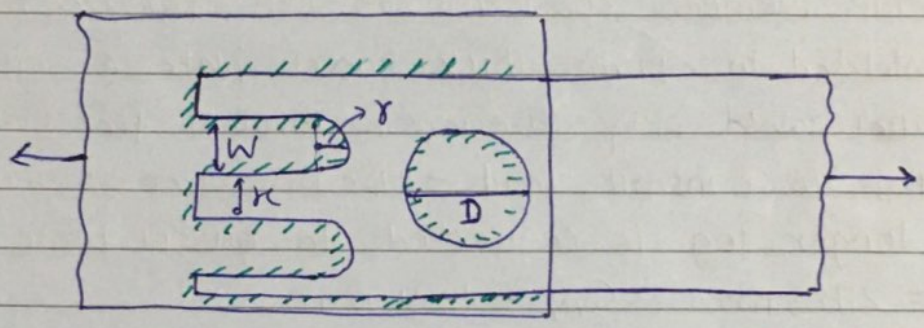


Staggered intermittent fillet Weld.

* staggered intermittent fillet weld are structurally better than chain intermittent fillet weld as it reduces distortion due to the balancing nature of weld chances of buckling will be less.

Design of plug and slot weld

if the overlap of plate is restricted and the applied force P can't be resisted given by providing continuous length of weld with max. size of weld we adopt slot welding. *

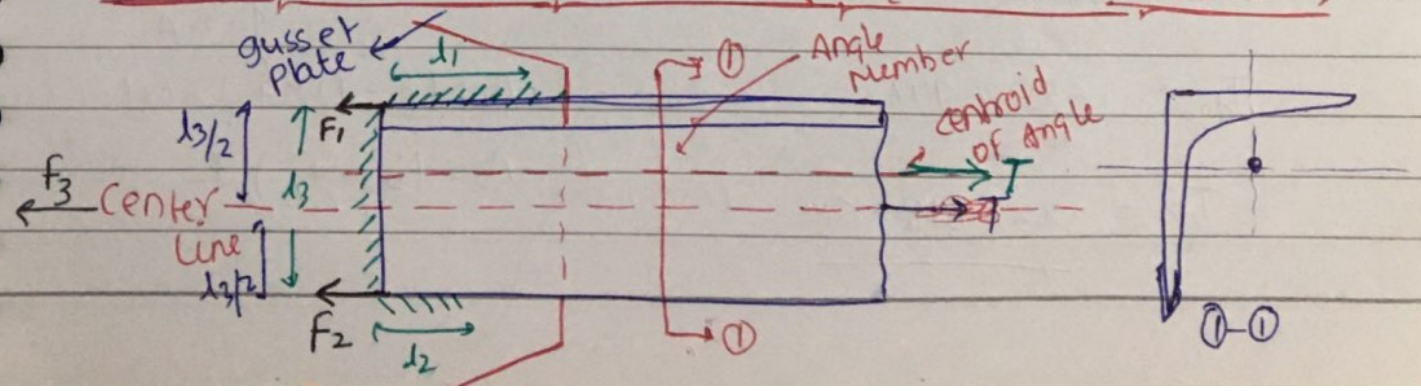


$r \nless \max(1.5t, 12\text{ mm})$
D or $W \nless \max(3t, 25\text{ mm})$
$x \nless \max(2t, 25\text{ mm})$ ***

corners @ enclosed end or slot should be rounded with a radius $\nless \max(1.5t, 12\text{ mm})$ *

* plug weld shan't be considered for strength calculations.

Fillet Weld for Truss Member (Unsymmetrical Member)



$$F_1 = \frac{f_u}{\sqrt{3}\,\gamma_{mw}} l_1 t_e \quad , \quad F_2 = \frac{f_u}{\sqrt{3}\,\gamma_{mw}} l_2 t_e \quad , \quad F_3 = \frac{f_u}{\sqrt{3}\,\gamma_{mw}} l_3 t_e$$

$$T = F_1 + F_2 + F_3$$

$$F_1 x - F_2 (l_3 - x) - F_3 \left(\frac{l_3}{2} - x\right) = 0$$

While designing the connection weld should be provided @ joint in such a way that resultant of forces resisted through weld passes through center of gravity of member.

Que A Tie member consisting of ISA 80x50x8 mm (Fe410) is welded to 12 mm thick gusset plate @ site using site fillet weld only, Design a moment free welded connection to transmit load = Design strength of member if the longer leg is connected to gusset plate.

$C_{zz} = 27.3$ mm. $C_{yy} = 12.4$ mm.

ISA 80x50x8 mm.

Gusset thickness = 12 mm. Fe410

$C_{zz} = 27.3$ mm. $C_{yy} = 12.4$ mm.

The design force that can be applied in member is -

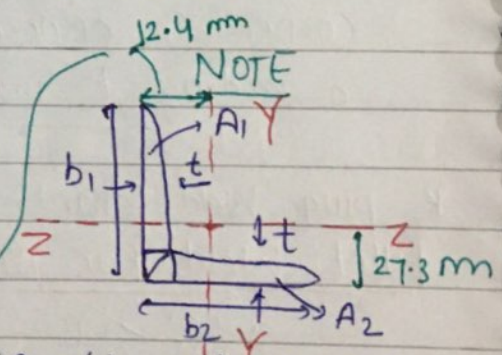
$$T_{dg} = \frac{f_y}{\gamma_{m0}} \times A_g = \frac{250}{1.1} \times A_g$$

$A = A_1 + A_2$

$A = 976\ mm^2$

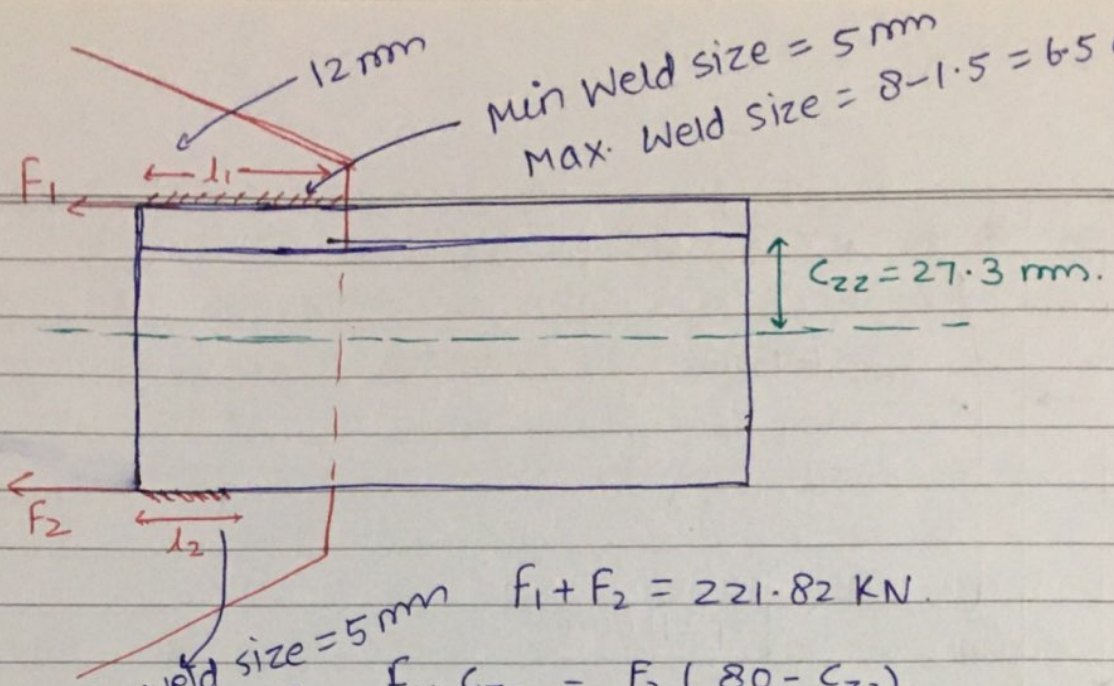
$$T_{dg} = \frac{250}{1.1} \times 976$$

$T_{dg} = 221.818$ KN



$A_1 = (b_1 - t)t + \frac{1}{2} \times t \times t$

$A_2 = (b_2 - t/2)\,t$



$F_1 + F_2 = 221.82$ KN.

$F_1 \cdot C_{zz} = F_2 (80 - C_{zz})$

$F_1 \cdot 27.3 = F_2 \times 52.7$

$F_1 = \frac{F_2 \times 52.7}{27.3} \rightarrow F_1 = 1.93 F_2$

$F_1 = 146.12$ KN. $F_2 = 75.696$ KN.

$F_1 = \frac{f_u}{\sqrt{3}\,\gamma_{mw}} \cdot l_1 t_e$

consider Weld size = 6 mm

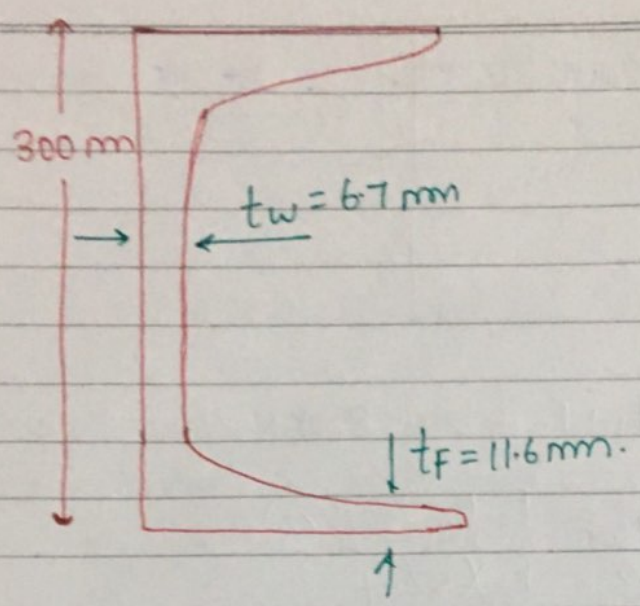
$t_e = KS \rightarrow 0.7 \times 6 = 4.2$

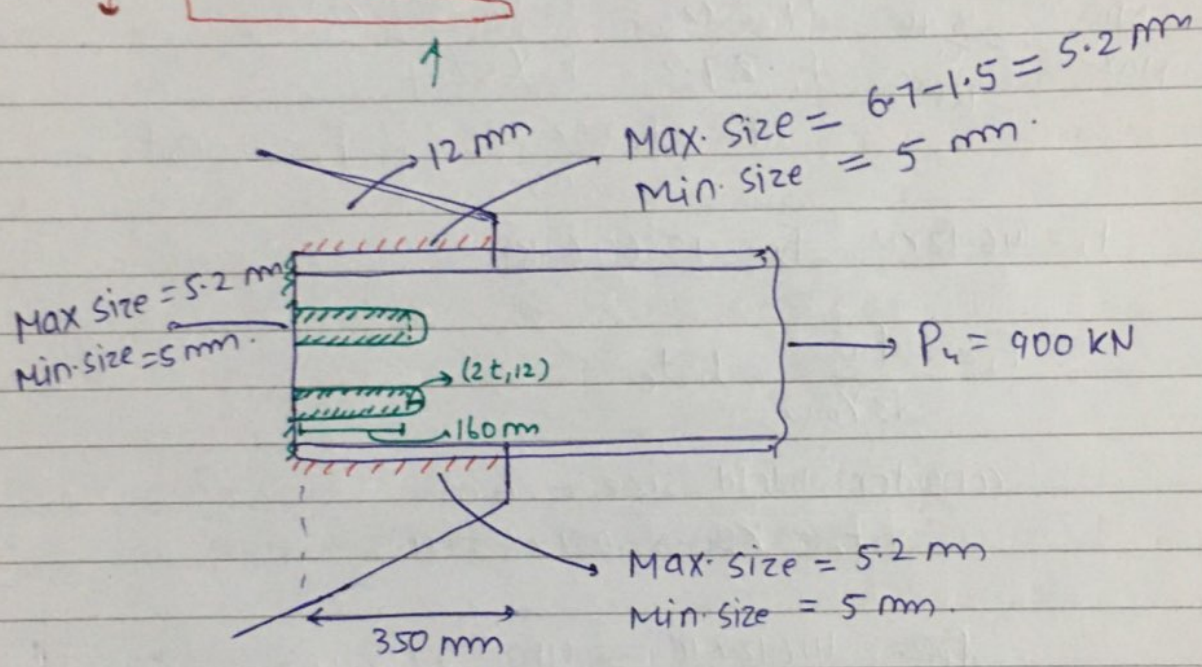
$146.12 \times 10^3 = \frac{410}{\sqrt{3} \times 1.5} \times l_1 \times 4.2$

$l_1 = 220.46$ mm.

$F_2 = \frac{f_u}{\sqrt{3}\,\gamma_{mw}} \times l_2 t_e$ $\quad l_2 = 114.21$ mm.

Que. An ISIC 300 (Fe410) is to carry factored tensile load of 900 kN the channel sec^n is to be welded @ site to a gusset plate of 12 mm thick. Design a fillet weld if overlap is limited to 350 mm. properties of ISIC 300 -





consider weld size of 5 mm.

$$t_e = ks = 0.7 \times 5 = 3.5 \text{ mm}.$$

$$P_u = \frac{f_u}{\sqrt{3}\,\gamma_{mw}} l_e t_e$$

$$900 \times 10^3 = \frac{410}{\sqrt{3} \times 1.5} l_e \times 3.5$$

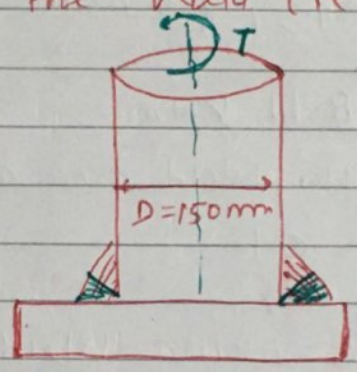
$$l_e = 1629.45 \text{ mm}.$$

As we've available weld length = 350+350+300 = 1000 mm
so we need to provide extra 629.45 mm.
so we can provide slot welding

let us provide 2 no. of slots with length $\frac{629.45}{4}$

$= 157.36\,mm \cong 160\,mm$

Que. A circular shaft of Dia 150 mm ~~of~~ is welded to a Rigid plate by means of 6mm Alround fillet weld Calculate the ultimate twisting moment that can be resisted by the Weld (fe410), shop Welding.



$$\tau = \frac{T \cdot r}{J}$$

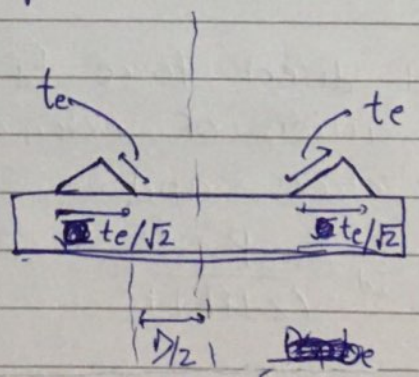
$$T = \frac{\tau J}{r}$$

size = 6mm
$\gamma_{mw} = 1.25$
Fe410

$$\tau = \frac{f_u}{\sqrt{3}\,\gamma_{mw}} = \frac{410}{\sqrt{3} \times 1.25}$$

$\tau = 189.37$ mpa

$$T = \frac{189.37\,J}{r}$$



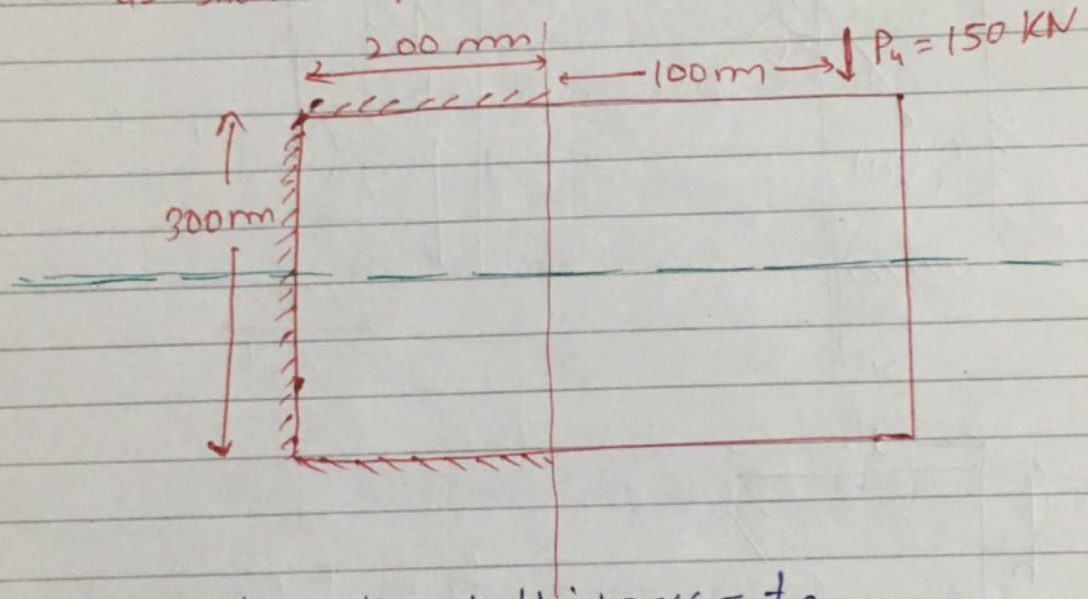
$$I_{yy} = (b \cdot t_e)(\bar{x})^2 + \left[\frac{t_e \cdot l^3}{12} + (l/2 - \bar{x})^2 \times t_e l\right] \times 2$$

$$F_r = \sqrt{F_D^2 + F_T^2 + 2F_DF_T\cos\theta}$$

For safety of connection $F_r \leq$ permissible stress in weld

$$F_r \leq \frac{f_u}{\sqrt{3}\,\gamma_{mw}}$$

Que. Find the size of weld req. on Horizontal & Verticle leg to resist a factored load of 150 KN as shown - Fe410, Shop Welding



consider throat thickness $= t_e$

$$\bar{x} = \frac{200 \times t_e \times \frac{200}{2} + 300 \times t_e \times 0}{200 t_e \times 2 + 300 \times t_e} = 57.14 \text{ mm}$$

$$f_D = \frac{P_u \rightarrow 150 \times 10^3 \text{ N}}{2 \times 200 \times t_e + 300 t_e}$$

$$f_D = \frac{214.28}{t_e} \text{ MPa}$$

Torsional shear stress $\tau = \frac{T \cdot r}{J}$.

$T = P_4 \times (200 + 100 - \bar{x}) \times 10^{-3}$

$T = 150 \times (300 - 57.14) \times 10^{-3}$

$T = 36.429$ kN.m

$$r = \sqrt{\left(\frac{300}{2}\right)^2 + (200 - 57.14)^2}$$

$r = 207.14$ mm.

$J = I_{xx} + I_{yy}$

$$I_{xx} = 200 \times t_e \left(\frac{300}{2}\right)^2 \times 2 + \frac{t_e (300)^3}{12}$$

$$I_{yy} = 300 \times t_e (\bar{x})^2 + 2 \times \left[\frac{t_e \times (200)^3}{12} - (200 - \bar{x})^2 \times t_e \times 200\right]$$

$J = I_{xx} + I_{yy} = 1.4297 \times 10^7 t_e$

$$F_T = \frac{Tr}{J} = \frac{36.429 \times 10^6 \times 207.14}{1.4297 \times 10^7 t_e} = \frac{527.79}{t_e} \text{ MPa.}$$

$$\Rightarrow f_r = \sqrt{f_d^2 + f_T^2 + 2 F_D F_T \cos\theta}$$

$$\cos\theta = \frac{200 - \bar{x}}{r} = 0.689$$

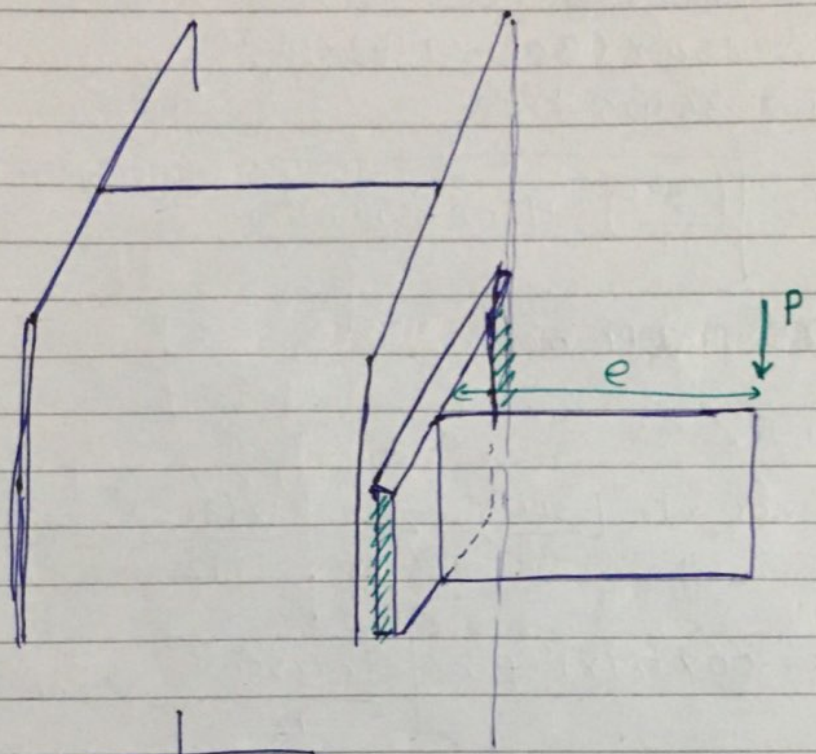
$$f_r = \frac{693.15}{t_e} \text{ MPa} \leq \frac{f_y}{\sqrt{3}\,\gamma_{mw}} = \frac{410}{\sqrt{3} \times 1.25}$$

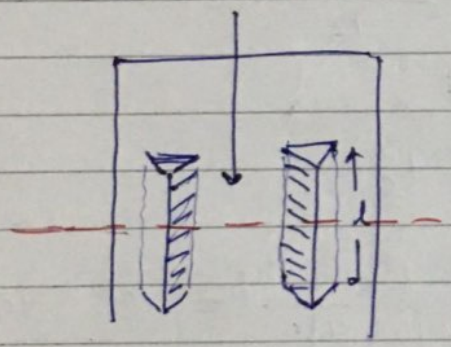
$t_e \geq 3.66$ mm

$ks \geq 3.66$ mm

$s \geq 5.23$ mm

Case-(ii) When fillet weld subjected to Direct load + BM





Direct shear $(f_s) = \dfrac{P}{2 l t_e}$

Bending stress $(f_b) = \dfrac{M \cdot y}{I}$

$= \dfrac{Pe}{\dfrac{2 \times t_e l^3}{12}} \left(\dfrac{l}{2}\right)$

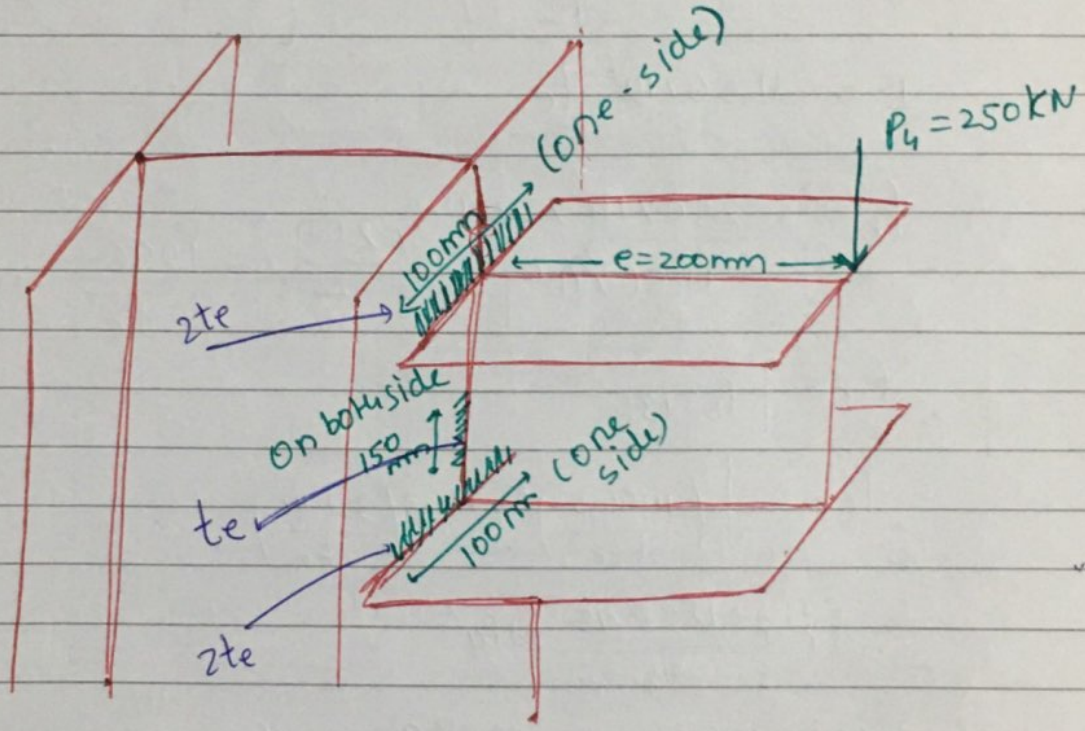
Resultant stress $= \sqrt{f_b^2 + 3 f_s^2}$

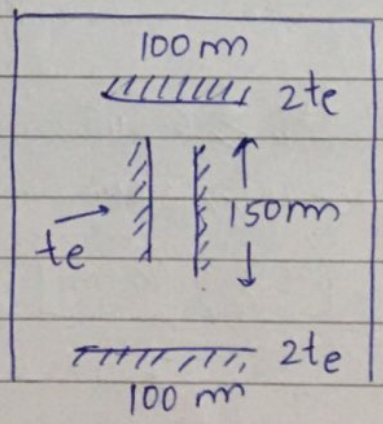
and for safety:

$$\sqrt{f_b^2 + 3 f_s^2} \leq \frac{f_u}{\sqrt{3}\,\gamma_{mw}}$$

NOTE check for combination of stresses need not be considered for fillet weld and sum of Normal stress and shear stress $\nless \dfrac{f_u}{\sqrt{3}\,\gamma_{mw}}$

Que Stup of ISHB200 is welded to the column flange of ISHB 400 using fillet welds, The weld connecting the flange has size double of the weld connecting the web of ISHB 200 with the column flange. length of weld connecting the flange is 100 mm and that connecting the web is 150 mm. The stup carries a factored load of 250 kN @ eccentricity of 200 mm. Determine the size of weld for safety of the connection Assuming shop welding, Fe410





stress due to direct load = $f_s = \dfrac{P_4 \rightarrow 250\times10^3}{(2\times100\times2t_e + 2\times150\times t_e)}$

$$f_s = \frac{357.14}{t_e} \text{ MPa.}$$

Stress due to BM $f_b = \dfrac{M}{I}\cdot Y$

$$I = 2\left[t_e\times\frac{(150)^3}{12}\right] + 2\times\left[100\times2t_e\times\left(\frac{200}{2}\right)^2\right]$$

$$I = 4.56\times10^6\, t_e$$

$$f_b = \frac{250\times10^3\times200}{4.56\times10^6}\times\frac{200}{2} = \frac{1096.5}{t_e} \text{ MPa.}$$

$$f_r = \sqrt{f_b^2 + 3f_s^2}$$

$$f_r = \sqrt{\left(\frac{1096.5}{t_e}\right)^2 + 3\times\left(\frac{357.14}{t_e}\right)^2}$$

$$f_r = \frac{1258.95}{t_e} \text{ MPa.}$$

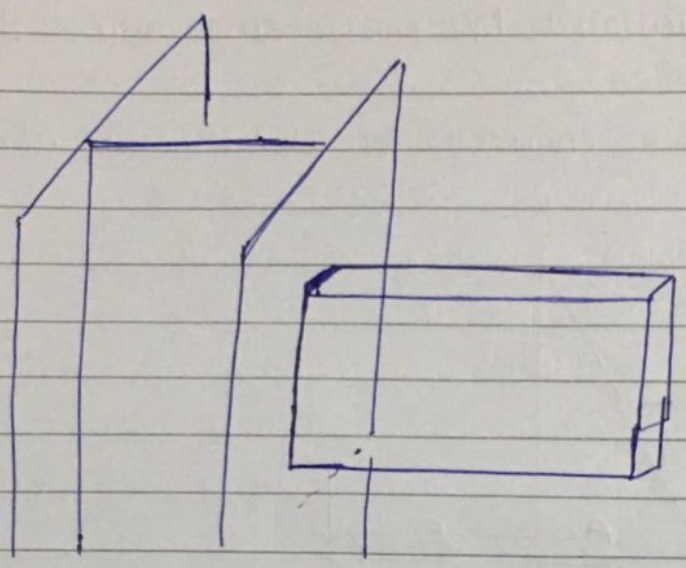
for safety $\dfrac{1258.95}{t_e} \leq \dfrac{f_y \rightarrow 410}{\sqrt{3}\,\gamma_{mw} \rightarrow 1.25}$

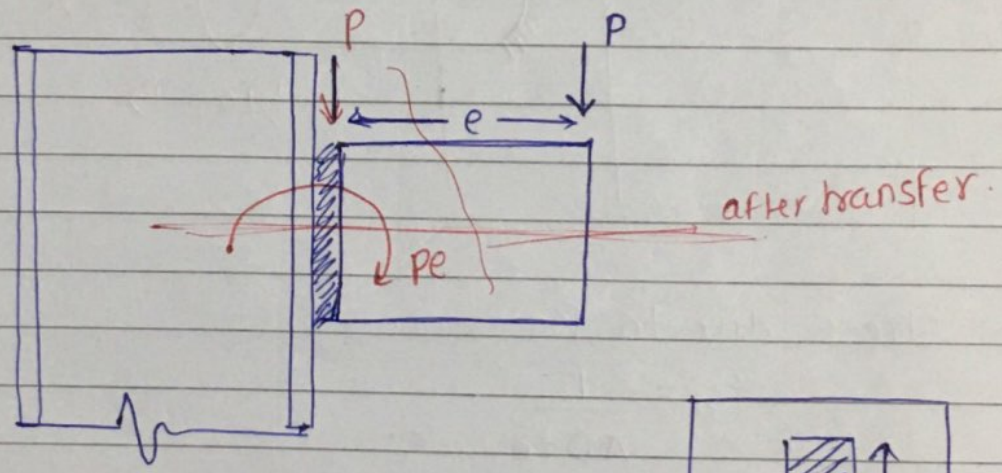
$$t_e \geq 6.648 \text{ mm}$$

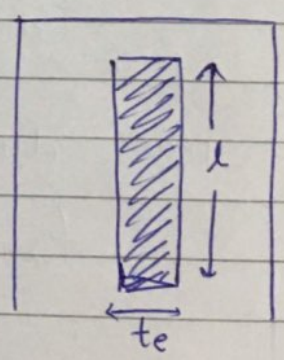
$$0.7S \geq 6.648$$

$$\boxed{S \geq 9.49 \text{ mm}}$$

case(iii) Butt weld subjected to Direct load & BM







stress due to direct force, $f_s = \frac{P_u}{l t_e}$

stress due to bending moment $(f_b) = \frac{M}{I} \cdot y$

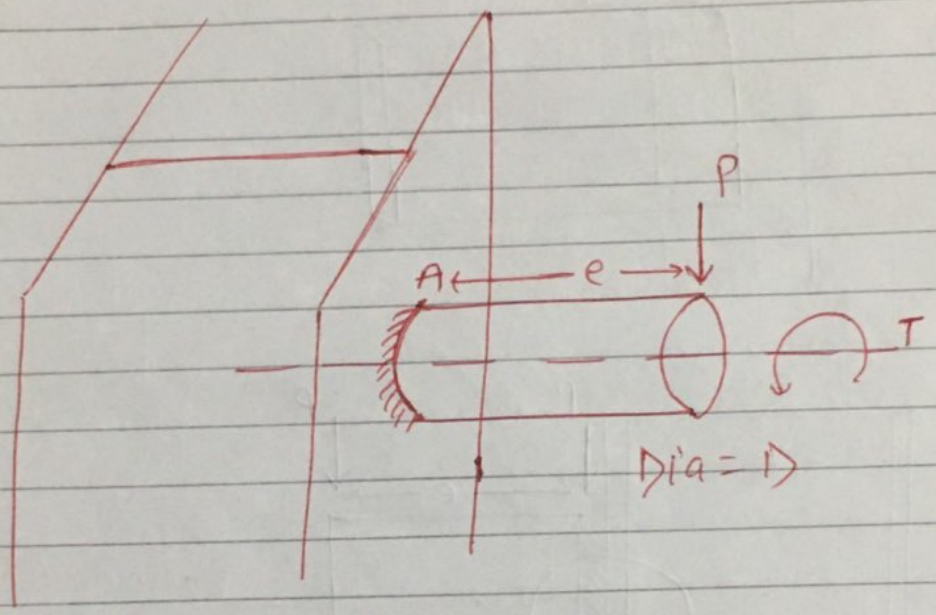
$$f_b = \frac{P_u \cdot e}{\frac{t_e \cdot l^3}{12}} (l/2)$$

$$f_r = \sqrt{f_b^2 + 3 f_s^2}$$

For safety $f_r \leq \frac{f_y}{\gamma_{mo}}$ However γ_{mw} in design.

For design of bracket connection subjected to bending and shear, the weld depth can be assumed slightly higher than that calculated by considering stress due to bending only.
Finally safety shall be ensured for the combination of stress.

Que:



Stress due to 'P'

$$f_{s_1} = \frac{P}{\pi D t_e}$$

stress due to torsion

$$f_{s_2} = \frac{T \cdot D/2}{\pi D t_e (D/2)^2}$$

$$f_s = \sqrt{f_{s_1}^2 + f_{s_2}^2}$$

$$f_b = \frac{M}{I} \cdot y = \frac{Pe \cdot D/2}{\frac{1}{2}\left[\pi D t_e (D/2)^2\right]} \quad \{ I = J/2$$

TENSION MEMBERS

Member subjected to Axial tension is called tension member or tie bar.

Tension member may fail in following modes.

i) Gross section yielding
ii) Net section Rupture
iii) Block shear failure

Design strength of Tension member

① For Gross-section yielding

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

$A_g \rightarrow$ gross sec^n Area
$\gamma_{m0} \rightarrow 1.1$ (yielding)

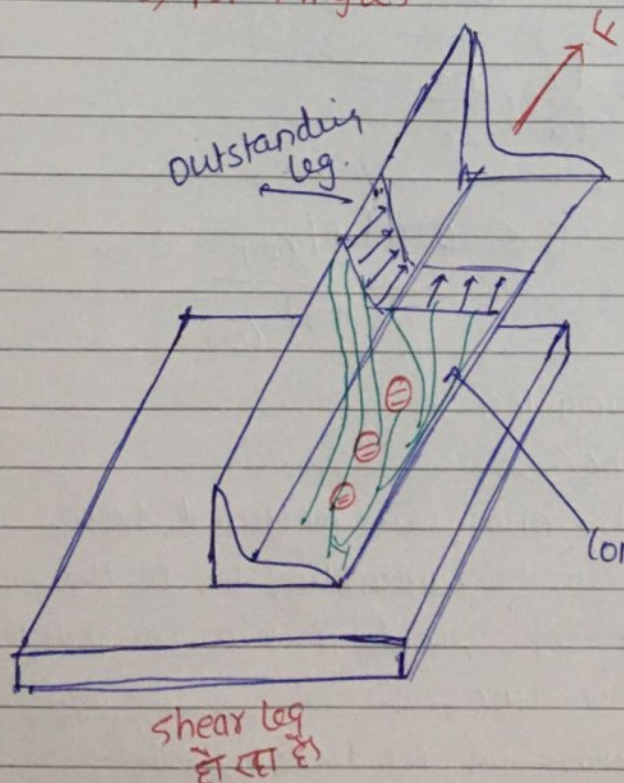
② Net-section Rupture

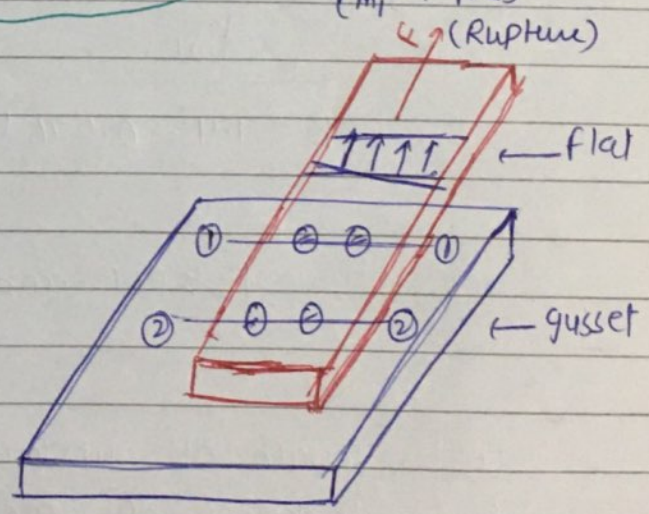
A) for plate and flats

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

$A_n \rightarrow$ net sec^n Area
$\gamma_{m1} \rightarrow 1.25$

B) for Angles





In case of Angles, We've connected leg and outstanding leg. Force is transfered from gusset to the connected leg and this force gets transfered as tensile stress over the entire c/s.

At the joint if the length of connection is small connected leg will have higher stress than the outstanding leg. Thus stress in outstanding leg, lags behind the stress in the connected leg. And this phenomenon is called shear leg. Due to shear leg effectiveness of the outstanding leg will be less and hence in case of angles, it is better to adopt connection through longer leg. We prefer unequal angles in case of tension members.

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

$$\beta = 1.4 - 0.076 \frac{w}{t} \frac{f_y}{f_u} \cdot \frac{b_s}{l_c}$$

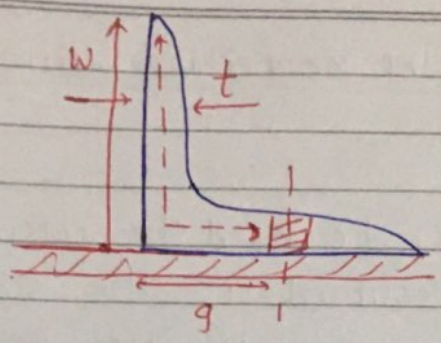
$$\beta \geq 0.7 \text{ and } \beta \leq \frac{0.9 f_u / \gamma_{m1}}{f_y / \gamma_{m0}}$$

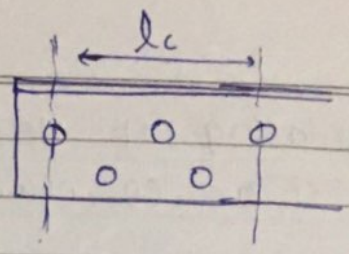
$w \rightarrow$ length of outstanding leg

$t \rightarrow$ thickness of member

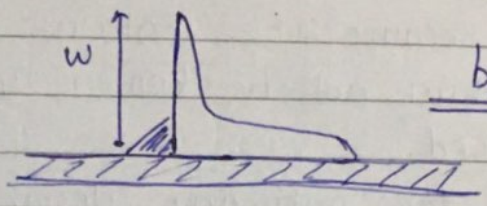
$b_s \rightarrow$ shear leg width and it is measured from the farthest point on the outstanding leg to the nearest line of bolt or weld in the connected leg along the center line

$l_c \rightarrow$ length of connection i.e c-c distance b/w extreme bolts in dir^n of force



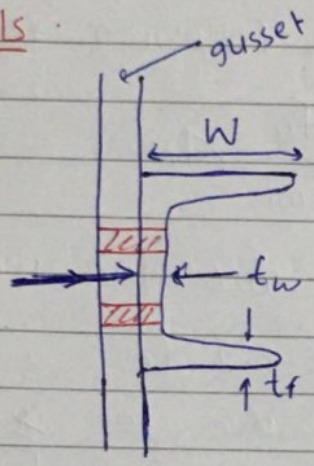


$b_s = w + g - t$

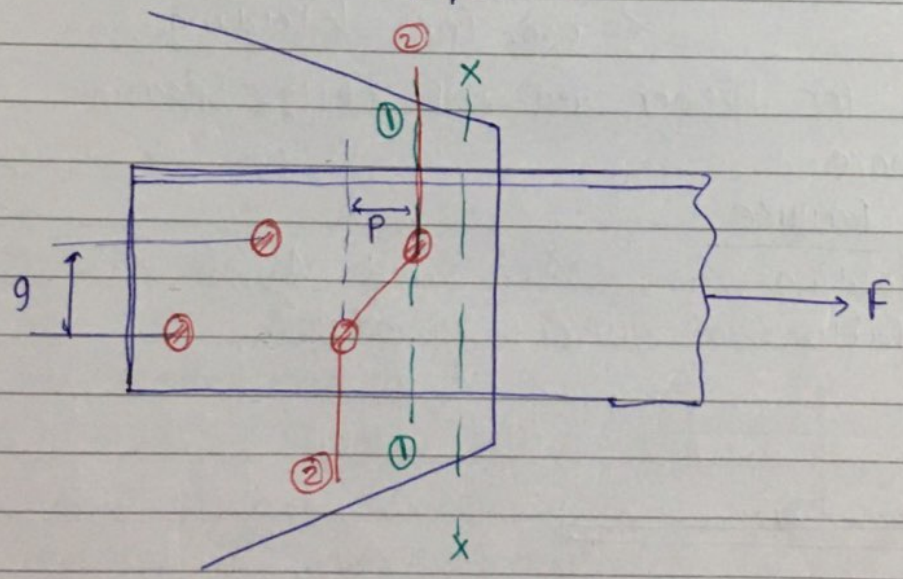


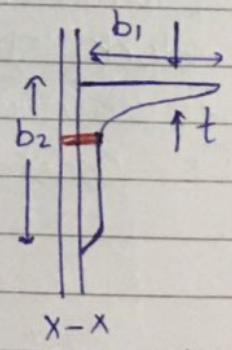
$b_s = w$

For channels.



$t = t_f$





$A_{go} = (b_1 - \frac{t}{2}) \times t$

$A_{nc}(1\text{-}1) = (b_2 - \frac{t}{2})\, t - d_o t$

$A_{nc}(2\text{-}2) = (b_2 - \frac{t}{2})\, t - 2 d_o t + \frac{P^2}{4g} t$

$A_{nc} = \min(A_{nc_1}, A_{nc_2})$

~~$A_{go} = tb$~~

NOTE Increasing the length of connection will reduce the shear leg effect.

* The formula for T_{dn} can't be used to calculate the net area of the sec^n required in the start of design because it contains variables like b_s, W, t l_c etc. which will not be known unless the section is finalised.

Therefore for primary design the Rupture strength can be calculated by using following formula.

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

A_n = Net Area of the Angle sec^n

$\alpha = 0.6$ for 1 or 2 bolts.

$\alpha = 0.7$ for 3 bolts.

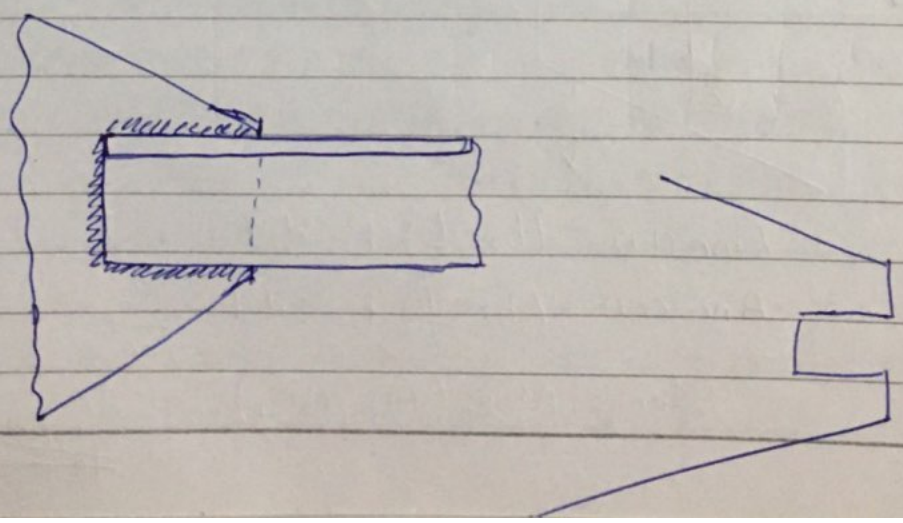
$\alpha = 0.8$ for ≥ 4 bolts.

$\alpha = 0.8$ for Welding.

α is higher for higher no. of bolts because l_c will be more.

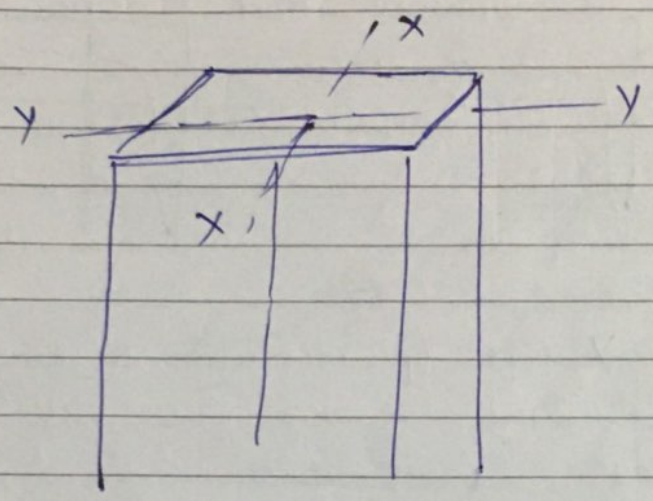
③ Block shear failure

In case of Welding $A_{tn} = A_{tg}$ and $A_{vn} = A_{vg}$ i.e consider failure of gusset plate



slenderness ratio

$$\lambda_{max.} = \frac{l_{eff}}{r_{min}} \quad , \quad r_{min} = \sqrt{\frac{I_{min}}{A}}$$

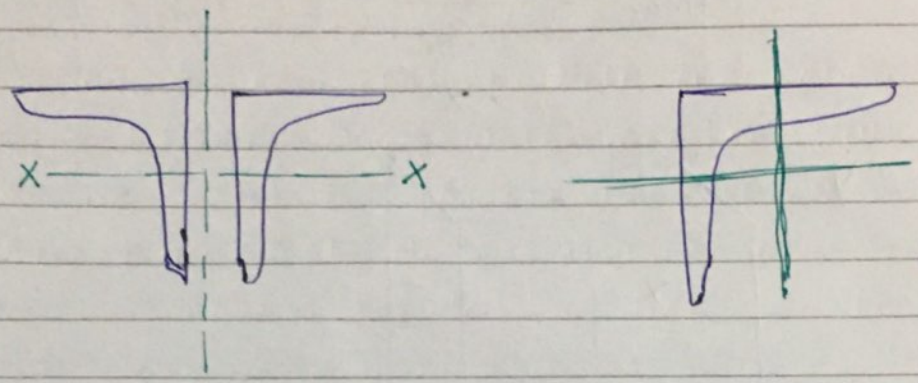


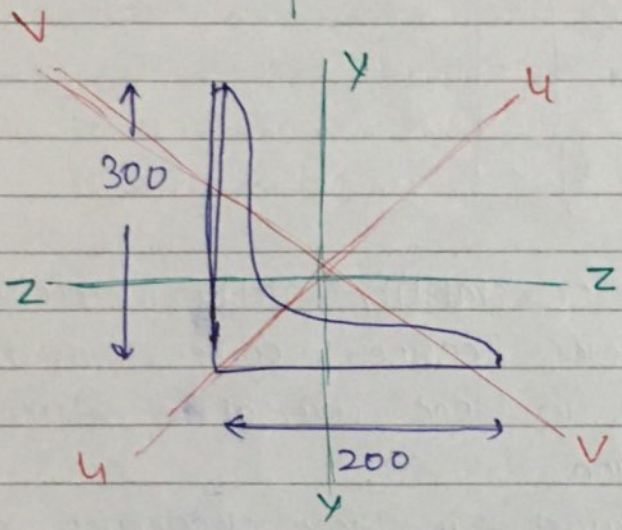
There is no buckling or stability problem in case of tension member However tension member may be subjected to compression in load reversal or during transportation or erection
hence to safeguard against this Max. slenderness ratio is restricted.

Description	Max^m slenderness
① A Tension member in which reversal of direct stress due to other than wind / EQ	180
② members normally acting as a tie but in which reversal of loading is due to Wind / EQ (like tie Member in a truss)	350

Description max: λ

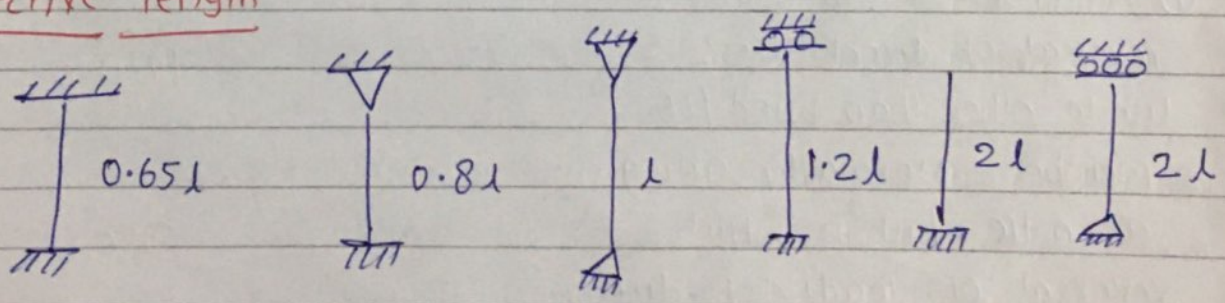
③ Members always under tension except pretension member 400



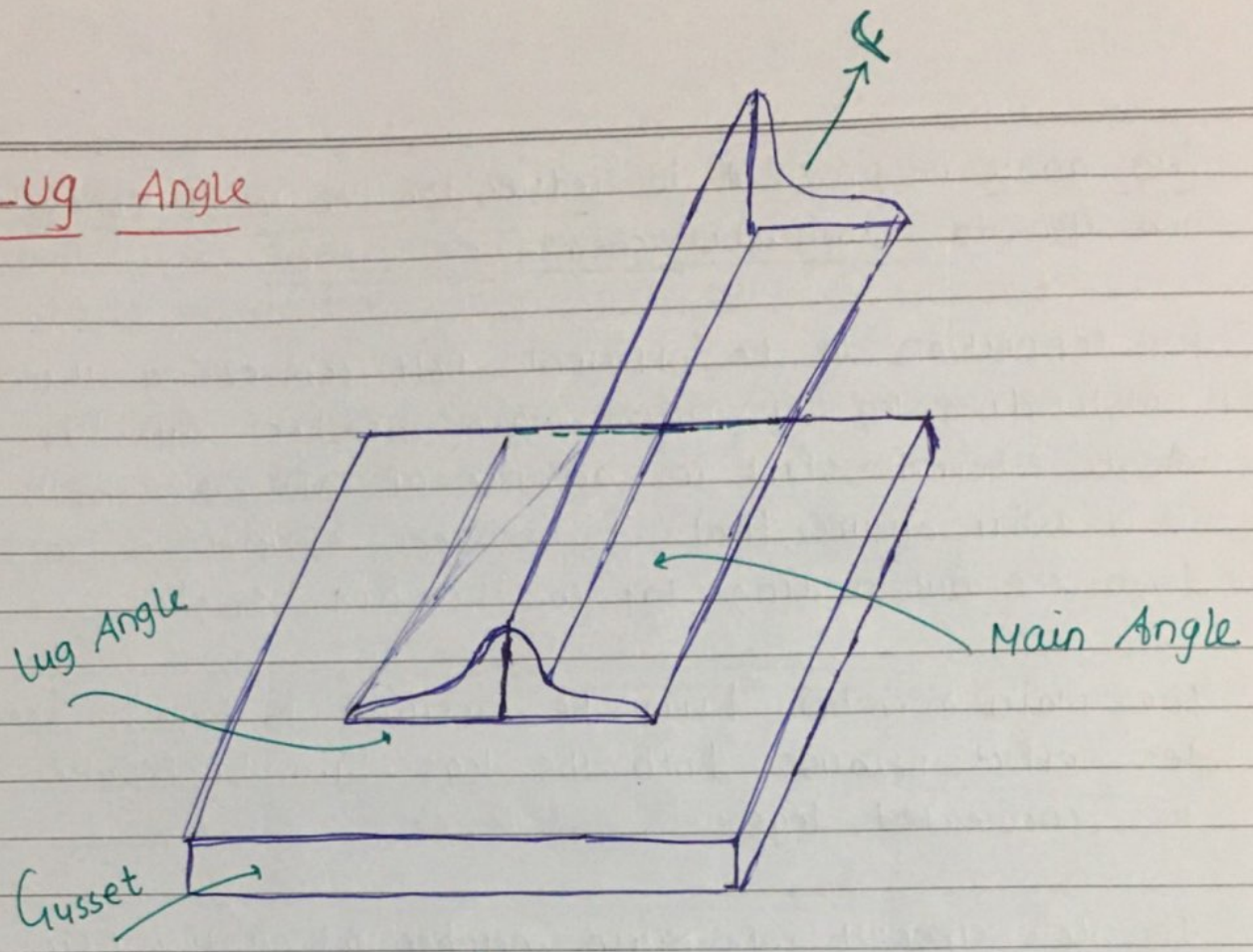


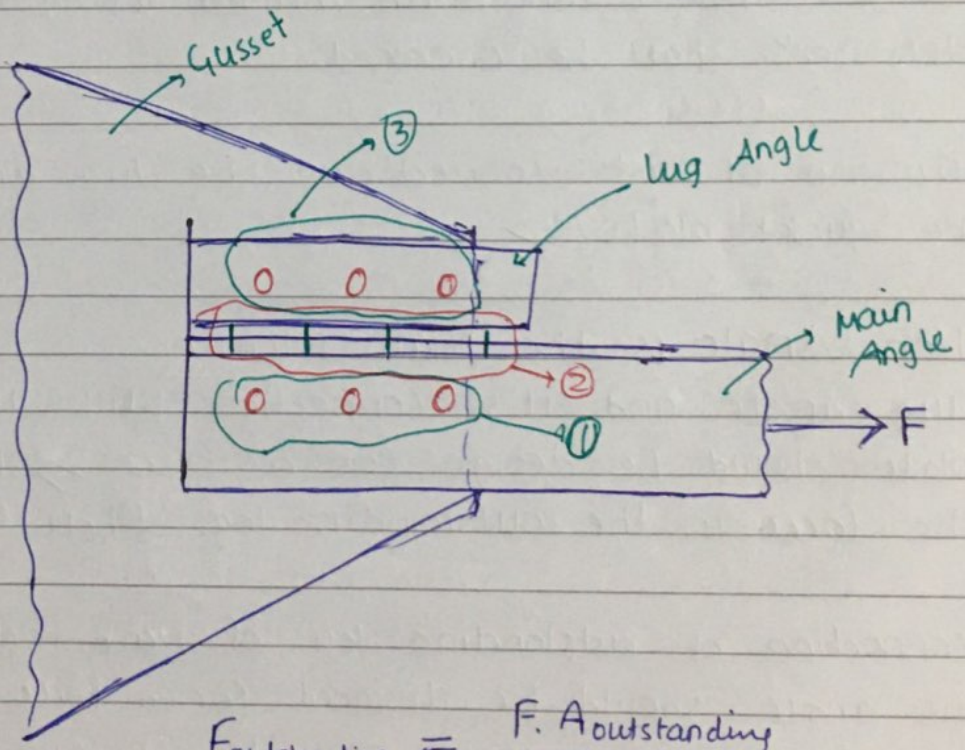
if sec^n is having atleast one axis of symmetry then that axis and an axis ⊥ to it passing through the sec^n can be minor Axis.

Effective length



Lug Angle





$$F_{outstanding} = \frac{F.A_{outstanding}}{A_{total}}$$

Bolts ① → Main member + Gusset

Bolts ② → outstanding leg of Main member + outstanding leg of lug angle

Bolts ③ → lug angle + Gusset

lug angle is provided to reduce the length of connection and also to eliminate shear leg effect

for connection to be efficient. bolt connecting the outstanding leg of ~~memb~~ main member with lug Angle should start in advance of all other bolts. This will ensure that force gets efficiently transfered from the outstanding lag to the lug Angle.

The Main member will be assume to have no shear leg effect because both the legs will be treated as connected legs.

Tensile strength of Main member along the Min. Net sec^n shall be checked.

Min. no of bolts connecting the lug angle with the gusset plate ≥ 2

When Angle is the Main Member

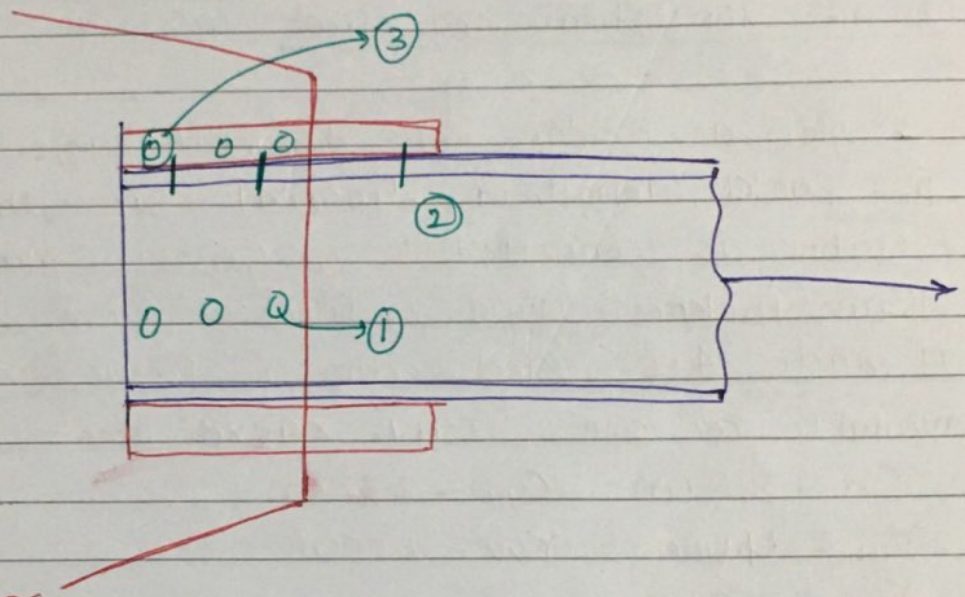
① lug angle and it's connection with the Gusset plate should be design for a force $\geq$ 1.2 times the force in the outstanding leg. (bolt Group-3)

② connection of outstanding leg of Main member with lug angle should be designed for a force $\geq$ 1.4 times the force in the outstanding leg of Main member (Bolt Group-2)

When channel is the main member

lug angles and their connections with Gusset plate should be designed for a force $\geq 1.1 \times F_{outstanding}$

Connection of outstanding legs of main member with the lug angle should be design for a force $\geq 1.2 \times F_{outstanding}$



Design steps

① $A_{net} = \dfrac{T_u}{\dfrac{\alpha f_u}{\gamma_{m1}}}$ Calculate Net Area req.

② increase A_n by 20–40% to obtain Gross Area.

③ Consider a section and check for gross yielding and slenderness ratio $T_u \leq \dfrac{A_g f_y}{\gamma_{m0}}$ $\rightarrow A_g \geq \dfrac{T_u}{f_y/\gamma_{m0}}$

$\lambda_{max} = \dfrac{l_{eff}}{r_{min}} \not> \lambda_{recommended}$

④ design connection and check for net section rupture and block shear failure

NOTE 2 - lines of bolts can be provided on 1 leg of angle only when the connected leg length is ≥ 125 mm

- 2 lines of bolts can be provided on the web of a channel only when depth of channel ≥ 150 mm.

In case of angles as tension member, prefer to connect longer leg in case of unequal angle to minimise the shear leg effect

Qu. Design a bridge truss diagonal subjected to $T_u = 300$ KN, length of diagonal = 3m, the tension member is connected to a Gusset plate of 16 mm thickness using 2 - lines of 20mm dia bolt of grade 4.6, steel section is Fe 410 grade and available sec^n are - ISA 125 x 95 x 8, Area = 17 cm^2

$C_{xx} = 3.8$ cm, $C_{yy} = 2.32$ cm

$r_{uu} = 4.41$ cm, $r_{vv} = 2.05$ cm

$r_{xx} = 3.97$ cm, $r_{yy} = 2.81$ cm

ISA 150 x 115 x 10, Area = 25.7 cm^2 $C_{xx} = 4.57$ cm

$C_{yy} = 2.84$ cm, $r_{uu} = 5.31$ cm, $r_{vv} = 2.48$ cm

$r_{xx} = 4.76$ cm, $r_{yy} = 3.41$ cm

Considering more than 4 no. of bolts take $\alpha = 0.8$

$$\text{Net Area} = \frac{T_u}{\alpha f_u / \gamma_{m1}} = \frac{300 \times 10^3}{\frac{0.8 \times 410}{1.25}} = 1143.29 \text{ mm}^2$$

Consider $A_g = A_n \times 1.2$

$A_g = 1371.95$ mm^2

ISA 125 x 95 x 8, Area = 1700 mm^2

consider ISA 125 x 95 x 8

$$A_{g,req.} = \frac{T_u}{f_y/\gamma_{m0}} = \frac{300\times10^3}{250/1.1} = 1320\ mm^2 < A_{g,\ provided.}$$

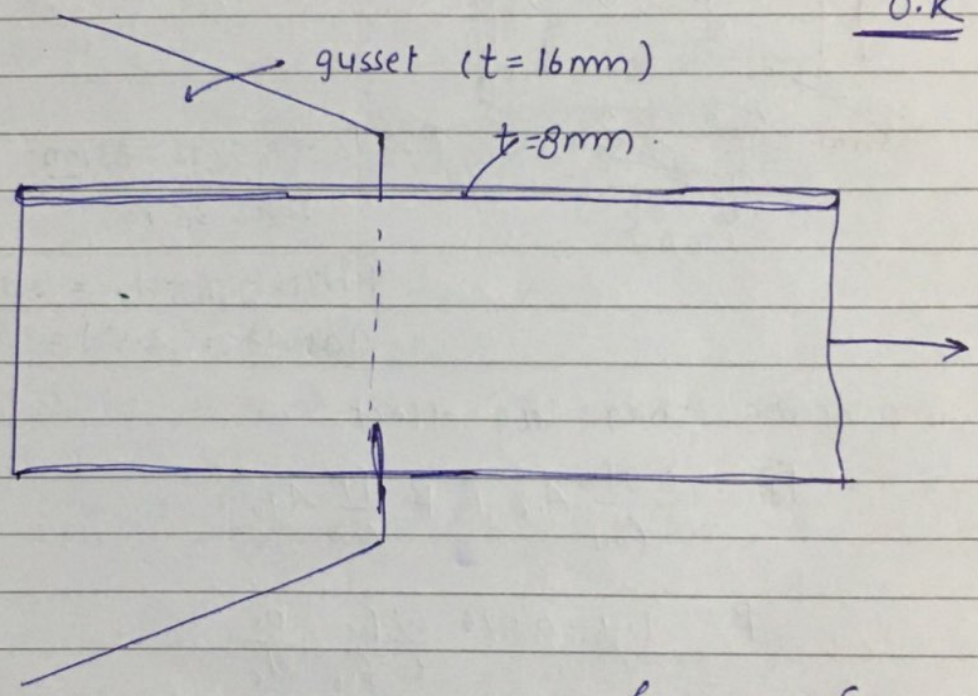
O.K

$$\lambda_{max} = \frac{l_{eff}}{r_{min}} = \frac{Kl}{r_{min}} = \frac{1\times3\times10^3\ mm}{r_{min}} =$$

in case of single angle member take $l_{eff} = l$
(No recommandation given in IS800:2007)

$$\lambda_{max.} = \frac{1\times3\times10^3}{2.05\times10} = 146.34 \not> 180$$

O.K



Bolt is in single shear

$$\left\{ k_b = Min\left\{ \frac{e}{3d_0}, \frac{p}{3d_0} - 0.25, \frac{f_{ub}}{f_u}, 1 \right\} \right.$$

$$V_{dsb} = 45.27\ kN$$

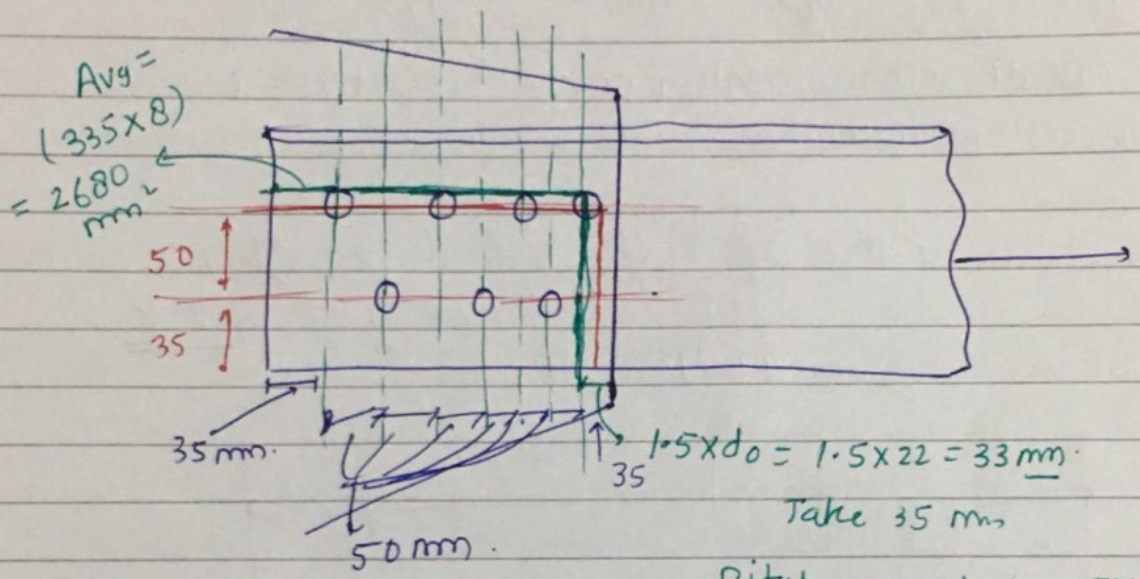
$$V_{dpb} = \frac{2.5\,k_b\,d\,t\,f_u}{\gamma_{mb}} = \frac{2.5\times0.5\times20\times8\times410}{1.25}$$

$$= 65.60\ kN.$$

$$V_{db} = 45.27\ kN$$

No. of bolts req. $= \frac{T_u}{V_{db}} = \frac{300}{45.27} = 6.62$

provide 7 no. of bolts.



$1.5 \times d_0 = 1.5 \times 22 = 33$ mm.

Take 35 mm

Pitch $= 2.5d = 2.5 \times 20 = 50$ mm.

gauge $= 2.5d = 50$ mm.

Considering shear lag effect

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

$$\beta = 1.4 - 0.076 \frac{w}{t} \frac{f_y}{f_u} \cdot \frac{b_s}{l_c}$$

$$\beta \geq 0.7 \quad , \quad \beta \leq \left(\frac{0.9 f_u / \gamma_{m1}}{(f_y / \gamma_{m0})} \right)$$

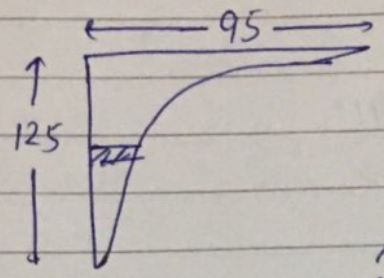
$b_s = w + g - t$
$= 95 + 40 - 8$
$= 127$ mm

$$\beta = 1.4 - 0.076 \times \frac{95}{8} \times \frac{250}{410} \times \frac{127}{300 \text{ mm.}}$$ 50x6

$$\beta = 1.167 \geq 0.7$$

$$\beta \leq \left(\frac{0.9 \times 410 / 1.25}{250 / 1.1} \right) = 1.298$$

$\beta = 1.167$



$A_{go} = (95 - 8/2) \times 8 = 728\ mm^2$

A_{nc} (1-1) $= (125 - 8/2) \times 8 - d_o t$

$= (125 - 4) \times 8 - 22 \times 8$

$= 792\ mm^2$

A_{nc} (2-2) $= \left(125 - \frac{8}{2}\right) \times 8 - 2 d_o t + \frac{p^2}{4g} t$

$= 121 \times 8 - 2 \times 22 \times 8 + \frac{50^2}{4 \times 50} \times 8$

$= 716\ mm^2$

$\Rightarrow A_{nc} = \text{Min}(762, 716)$

$A_{nc} = 716\ mm^2$

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

$T_{dn} = \frac{0.9 \times 410}{1.25} \times 716 + \frac{1.167 \times 250}{1.1} \times 728$

$= 404\ kN > 300\ kN$ <u>O.K</u>

Block shear failure

$A_{vg} = 8 \times (35 + 50 \times 6) = 2680\ mm^2$

$A_{tg} = 8 \times (35 + 50 - \frac{22}{2}) = 592\ mm^2$

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

$$T_{db1} = \frac{250 \times 2680}{\sqrt{3} \times 1.1} + \frac{0.9 \times 410}{1.25} \times 592$$

$$T_{db1} = 526.41 \text{ KN}$$

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

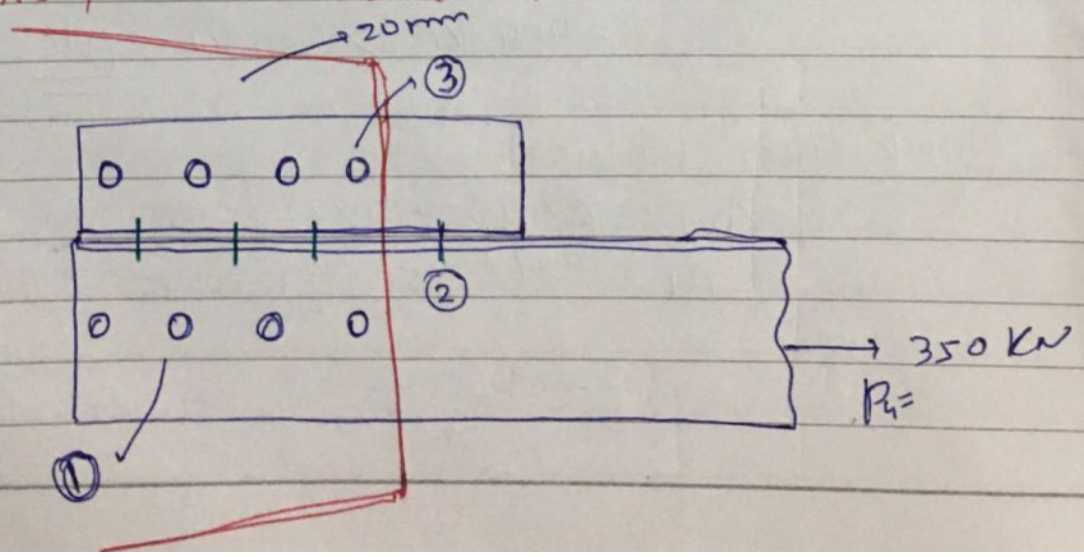
$\cancel{A_{vn} =}$

$$A_{vn} = (35 + 6 \times 50 - 3.5 \times 22) \times 8$$
$$= 2064 \text{ mm}^2$$

$$A_{tg} = 85 \times 8 = 680 \text{ mm}^2$$

$$T_{db2} = 506.32 \text{ KN} > 300 \text{ KN}.$$

Que. An ISA 100×75×10 is used as a tensio member with longer leg connected to 20 mm thick gusset plate the member carries a factored load of 350 KN connection has been done with lug angle, the bolt is M20 of grade 4.6 and steel Fe410. Design the connection if the available secⁿ for lug angle are 100×75×6, 75×75×8 and 100×75×8



Member shan't be designed as already given in the Que.

$$f_{outstanding} = \frac{F}{A_{total}} \times A_{outstanding}$$

$$= \frac{350}{(100+75-10)\times 10} \times (75-10/2)\times 10 = 148.4\ kN$$

M20 bolt of grade 4.6

$V_{dsb} = 45.27\ kN$

$$V_{dpb} = \frac{2.5\, K_b\, d\, t\, f_u}{\gamma_{mb}} = \frac{2.5\times 0.5\times 20\times 6\times 410}{1.25} = 49.20\ kN$$

$V_{db} = 45.27\ kN$

Considering 6mm thickness of lug angle also the bearing strength of bolt = 49.20 kN

$\Rightarrow$ bolt Value = 45.27 kN

no. of bolts req. to connect both the outstanding legs $= \frac{1.4\times 148.4}{45.27} = 4.58$

$\cong$ 5 bolts

no. of bolts req. to connect Gussest plate with lug angle $= \frac{1.2\times 148.4}{45.27} = 3.39$

$\cong$ 4 bolts.

lug angle shall be designed for a force of $1.2\times 148.4 = 178.08\ kN$

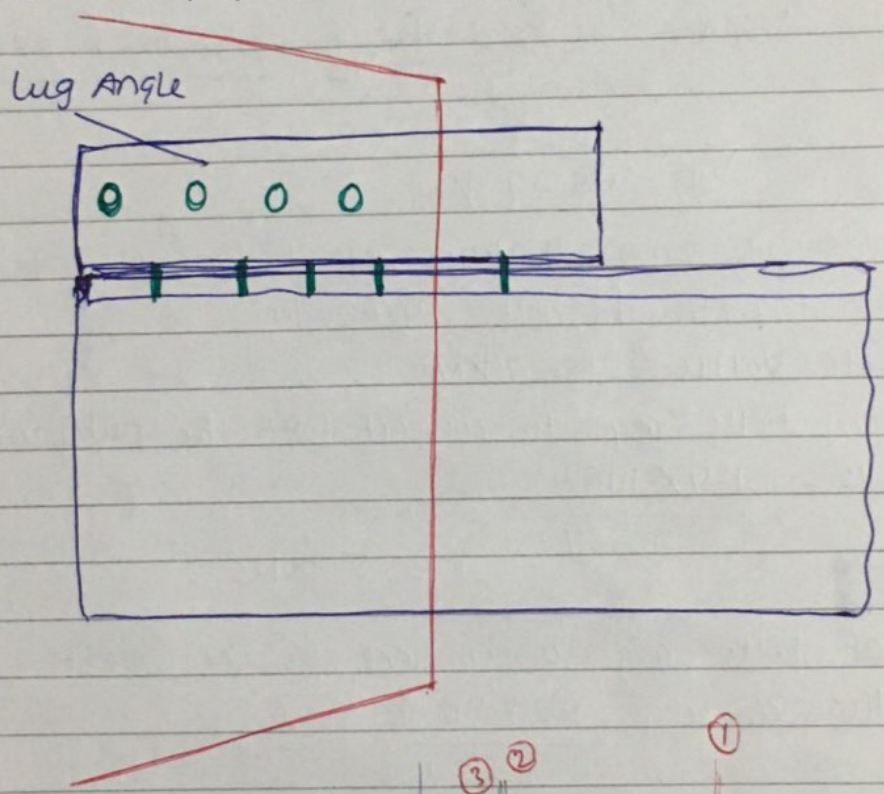
Net area of lug angle req. $= \frac{178.08}{0.9\, f_u/\gamma_{m1}} = 603\ mm^2$

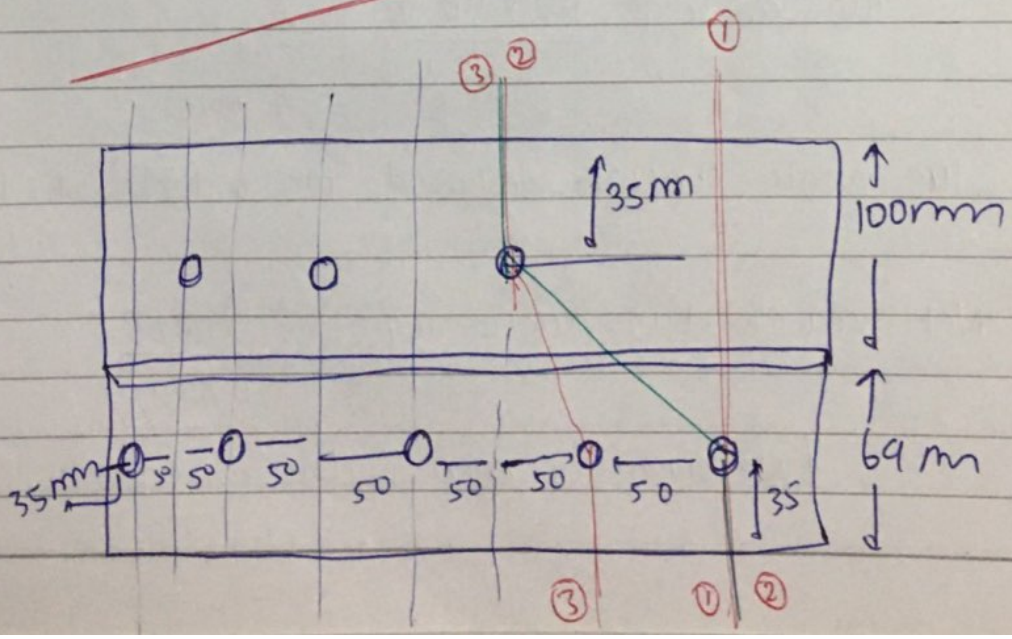
(No shear lag effect in lug angle)

Consider Gross Area of lug angle as = 1.2x603
= 723.6 mm²

100x75x6
A = (100+75-6)x6 = 1014 mm²
75x75x8
A = (75+75-8)x8 = 1136 mm²
Consider angle section 100x75x6 mm.

Net secⁿ rupture for lug angle :-





$$A_{nc(1-1)} = (169-22)\times 6 = 882\ mm^2$$

$$A_{nc(2-2)} = \left(169 - 2\times 22 + \frac{p^2}{4g}\right)\times t$$

$$= \left(169 - 44 + \frac{100^2}{4\times 99}\right)\times 6 = 901.55\ mm^2$$

$$A_{nc(3-3)} = \left(169 - 2\times 22 + \frac{p^2}{4g}\right)\times t$$

$$= \left(169 - 44 + \frac{50^2}{4\times 99}\right)\times 6 = 787.86\ mm^2$$

COMPRESSION MEMBER

A structural member which is straight and subjected to axial compressive force only with no initial imperfection is called as a ~~con~~ ideal compression member.

Type of compression Member

1) column for stanchion
2) Strut → used in roof ~~strut~~ trusses and bracing system
3) boom of a crane

Classification of c/s

A plate element of a c/s may buckle locally due to compressive stress. This local buckling can be avoided before the limit state is achieved by limiting width to thickness ratio of each element of c/s subjected to axial force, Bm or shear

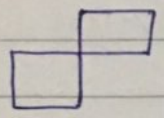
When plastic analysis is used, sec^n should be capable of forming plastic Hinge without local buckling

If Elastic analysis is used, Member should be capable of developing ~~the~~ yield stress before local buckling

As per IS 800 : 2007, Based on local buckling the c/s available are :-

i) plastic sec^n (class-1)
ii) Compact sec^n (class-2)
iii) semi-compact sec^n (class-3)
iv) slender sec^n

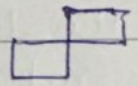
① **plastic section** sec^n which develops full plastic moment capacity and has significant rotational capacity req^d for failure of structure by formation of plastic hinge i.e before collapse mechanism local buckling doesn't occure ($\theta_2 > 6\theta_1$)



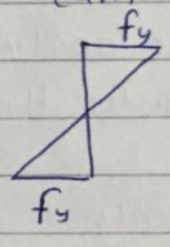
$\theta_2 \rightarrow$ lower limit of rotation for plastic sec^n

$\theta_1 \rightarrow$ Rotation @ onset of plasticity.

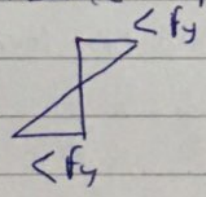
② **compact section** The c/s which can develop plastic hinge but has inadequate plastic hinge rotation capacity for formation of plastic mechanism due to local buckling ($\theta_2 < 6\theta_1$)



③ **semi-compact section** section which can develop yield stress @ extreme fibre and can't develop plastic hinge due to local buckling



④ **slender sec^n** sec^n in which local buckling occurs even before extreme fibre reaching the stress of fy.

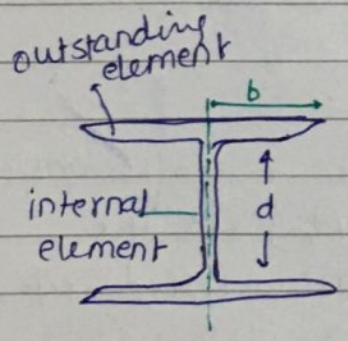


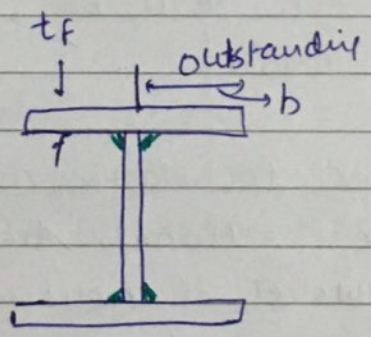
As per IS 800-2007

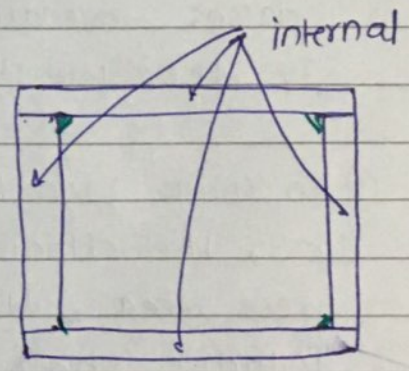
$$\varepsilon = \sqrt{\frac{250}{f_y}}$$

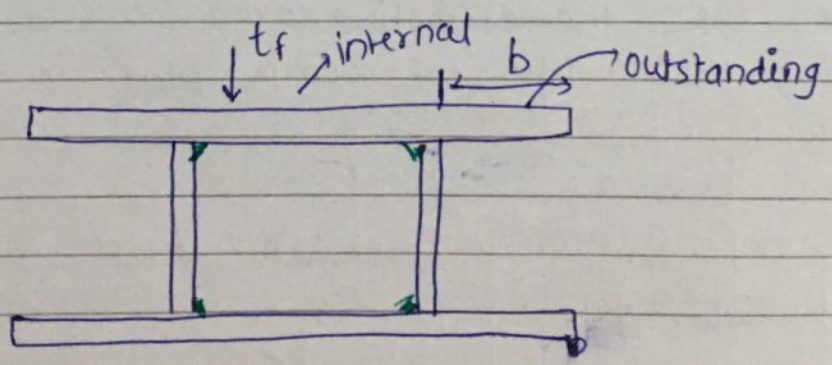
As per IS 800:2007

		Ratio	Plastic	compact	semi-compact
outstanding element of comp. flange	Rolled sec^n	b/t_f	9.4ε	10.5ε	15.7ε
	Welded sec^n	b/t_f	8.4ε	9.4ε	13.6ε
internal element of comp. flange	comp. due to Bending	b/t_f	29.3ε	33.5ε	42ε
	Axial comp.	b/t_f	Not Applicable.		42ε
web of I-sec^n	When N.A @ Mid depth	d/t_w	84ε	105ε	126ε
	Axial comp.	d/t_w	Not Applicable		42ε

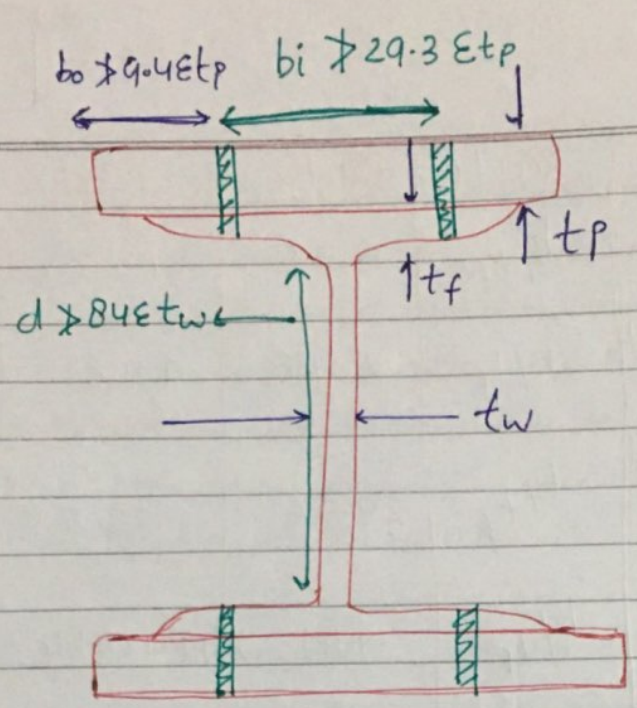








Qu.



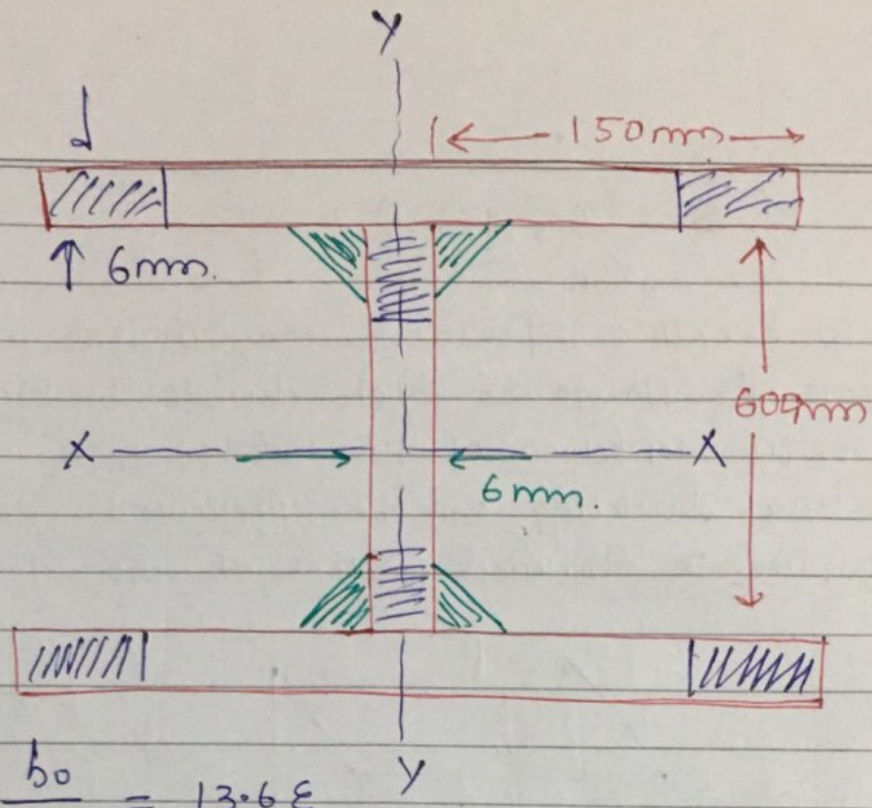
$\varepsilon = \sqrt{\frac{250}{f_y}}$

for the above secⁿ to be plastic secⁿ mark the max. permissible width of internal and outstanding elements.

NOTE ① When diffⁿ elements of a cls for under diffⁿ classes, overall secⁿ shall be classified as governed by the lowest class.

② In comp. member, for plastic, compact & semi-compact secⁿ, the effective sectional Area (A_e) is the gross Area. However if holes are not filled up, deduction shall be made for this.
for slender secⁿ, the effective area is calculated by deducting the width of comp. plate in excess of semi-compact secⁿ

Qu. Find Ae for the following Axial comp. member
take fy = 250 mpa



$$\frac{b_o}{t_f} = 13.6\varepsilon$$

$$\varepsilon = \sqrt{250/250} = 1$$

$$b_o = 13.6\,\varepsilon\, t_f = 13.6 \times 1 \times 6 = 81.6\ mm.$$

$$\frac{d}{t_w} = 42\varepsilon$$

$$d = 42\varepsilon t_w = 252\ mm.$$

$$A_e = 4 \times 81.6 \times 6 + 2 \times 6 \times 6 + 252 \times 6 = 3542.4\ mm^2$$

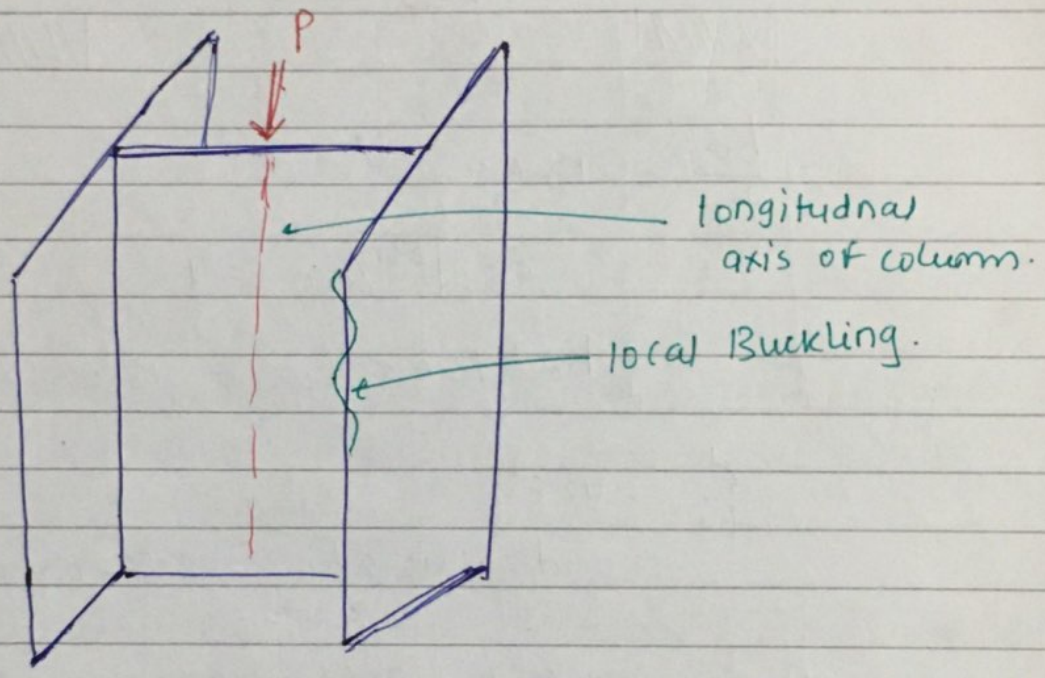
Area shall be neglected from such a loc that we get a min inertia.

possible failure mode in comp. member

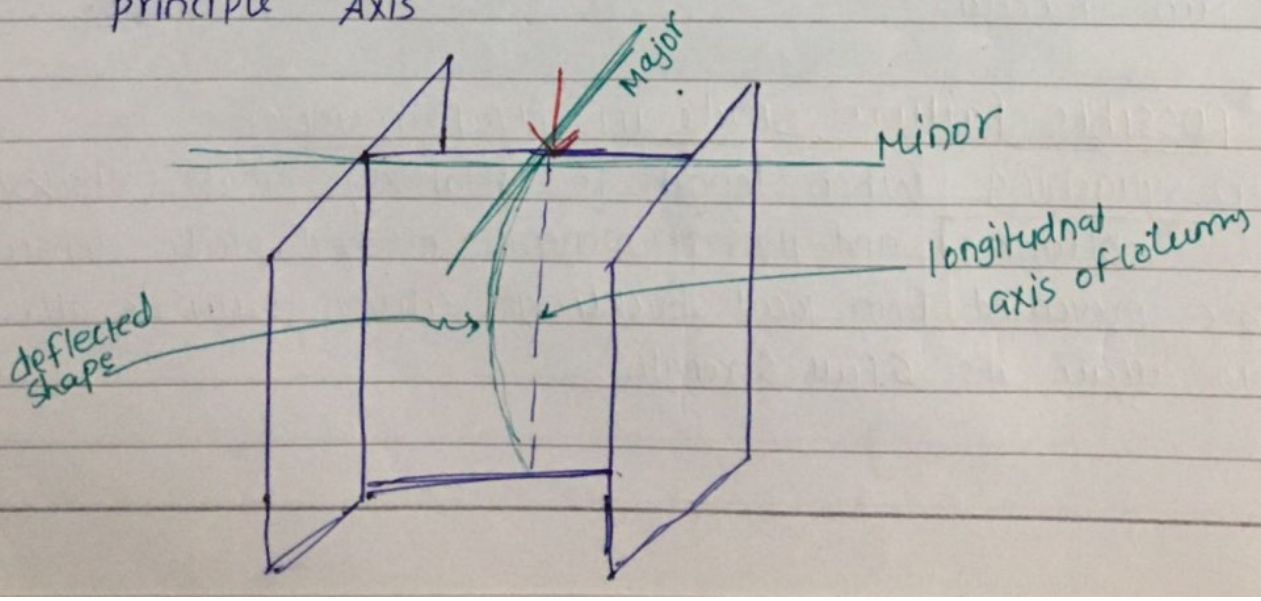
(i) squashing When length is relatively small (stocky columns) and it's component ~~plated~~ plate elements are prevented from local buckling. Column will be able to attain it's full strength.

$$P_d = \frac{f_y}{\gamma_{m0}} \times A_e$$

ii) local buckling failure may occure in the plate elements like flange or web due to buckling with no overall deflection of the member ⊥ to the applied load, local buckling can be prevented by limiting the width to thickness ratio of the plate element.

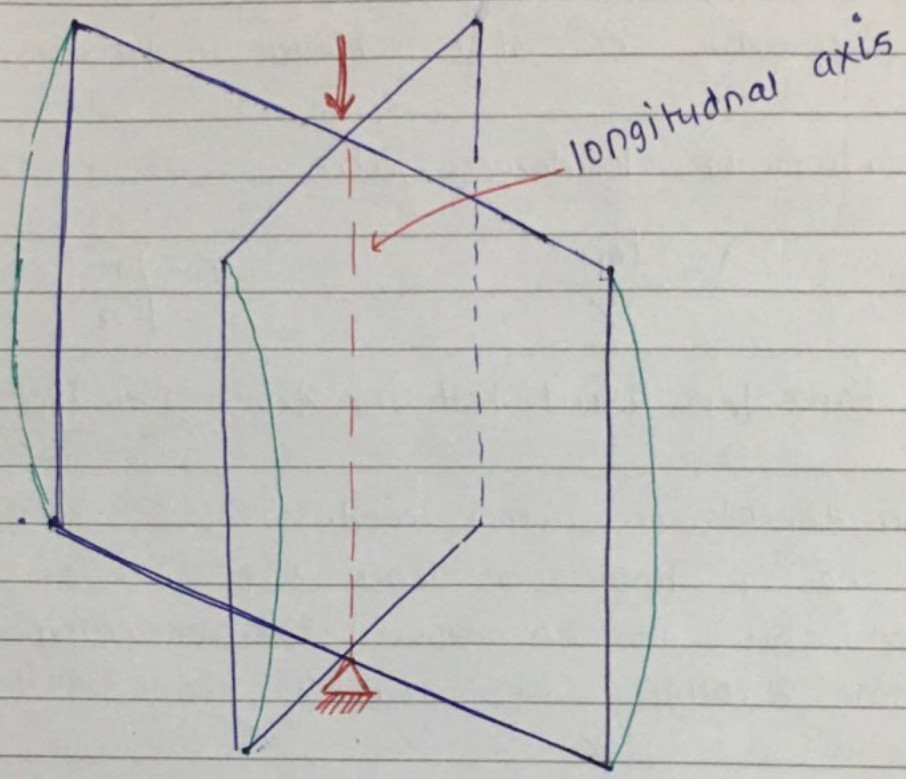


iii) overall flexural buckling. In this case, failure of member occurs by excessive deflection about weaker principle Axis



As λ value ↑, load carrying capacity ↓

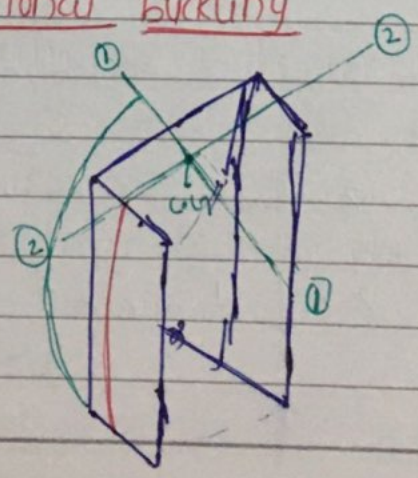
iv) Torsional buckling

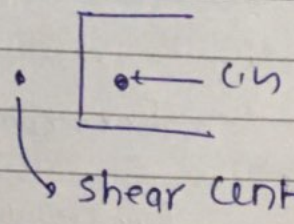


- Torsional buckling failure occurs by twisting about shear center i.e twisting about longitudnal axis

- Torsional buckling is a possible mode of failure for point symmetric sec^n and it can only properly occure when the shear center and centroid of c/s coincides.

v) Flexural Torsional buckling





- it is caused by a combination of flexural and torsional buckling. the member bends as well as twists, it occurs only in unsymmetrical secⁿ and singly symmetrical secⁿ

- Doubly symmetric secⁿ ain't subjected to flexural torsional buckling.

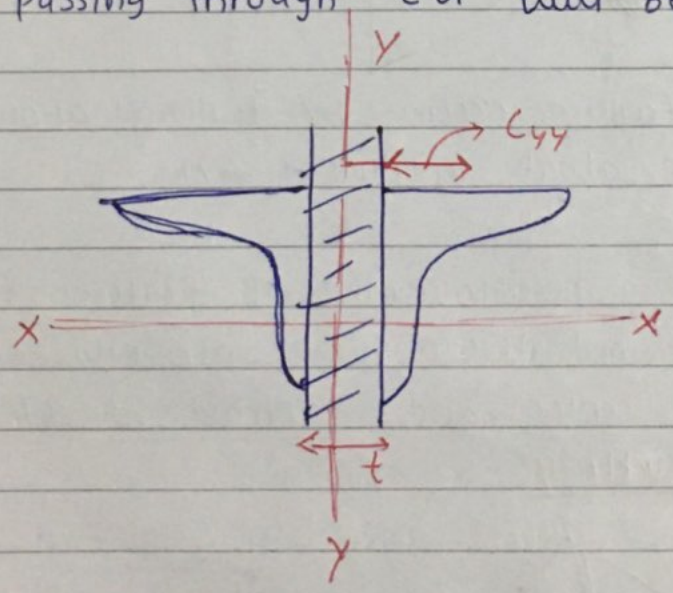
Determination of slenderness ratio

$$\lambda = \frac{l_{eff}}{r} \qquad r = \sqrt{\frac{I}{A}}$$

for same l_{eff} about both the Axis $\lambda_{max} = \frac{l_{eff}}{r_{min}}$

- l_{eff} depends on support cond.

- if the secⁿ is having at least one axis of symmetry then that axis or an axis ⊥ to symmetrical axis and passing through C.G will be minor principle axis



$$I_{xx,\,comb} = 2\, I_{xx,\,one}$$

$$I_{yy,\,comb} = 2 \times \left[I_{yy,\,one} + A_{one} \left(C_{yy} + \frac{t}{2} \right)^2 \right]$$

$\lambda \not> \lambda_{max}$ recommended by IS code.

Type	λ_{max}
i) comp. member carrying load due to DL + LL	180
ii) comp. carrying load due to WL / EQ only	250
iii) Member normally acting as tie in a roof truss or bracing system but subjected to possible reversal due to action of wind / EQ	350

Column strength

$$P_{cr} = \frac{\pi^2 E I}{l_{eff}^2}$$

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

Commen hot rolled sec^n and built up steel member used to carry axial comp. usally fail by flexural buckling or flexural torsional buckling

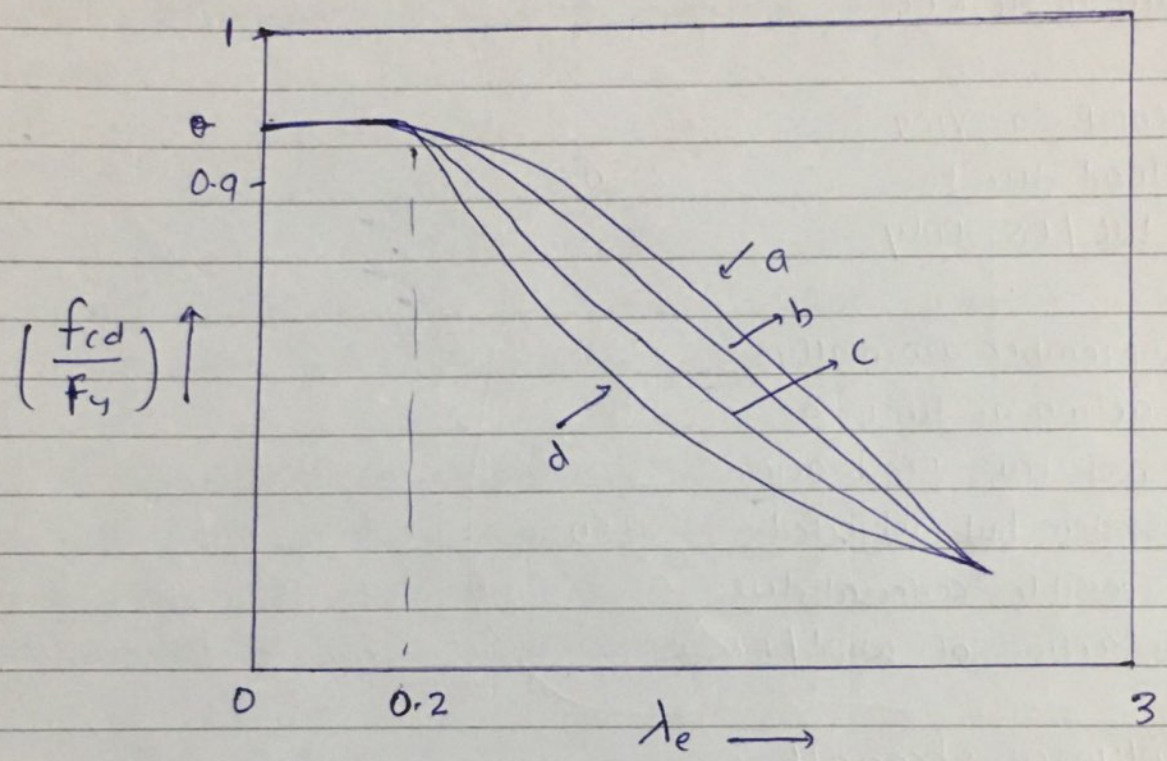
The buckling strength of this member is affected by

i) residual stress

ii) initial bow

iii) accidential eccentricity

To account for these factors large number of tests were carried out and column buckling curve a, b, c, d were established. These a, b, c, d are called buckling class.



This curve is based on perry - robertson formula.

f_{cd} of axially loaded column is given by –

$$P_d = f_{cd} A_e$$

f_{cd} is design comp. stress.

class - a → safe with least imperfection *

class - b → safe with med^m imperfection ✓

class - c → safe with lot of imperfection

class - d → safe with Max^m imperfection

Calculation of f_{cd} for Flexural Buckling

- Axially loaded comp. member

$$f_{cd} = \frac{f_y/\gamma_{m0}}{\phi + \sqrt{\phi^2 - \lambda_e}} = \chi \frac{f_y}{\gamma_{m0}} \leq \frac{f_y}{\gamma_{m0}}$$

$\chi \rightarrow$ strength reduction factor

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

$\lambda_e \rightarrow$ non-dimensional effective slenderness ratio

$$\lambda_e = \sqrt{\frac{f_y}{f_{cc}}} = \sqrt{\frac{f_y \lambda^2}{\pi^2 E}} = \sqrt{\frac{f_y}{\pi^2 E}} \cdot \lambda$$

$f_{cc} \rightarrow$ Euler's buckling stress

$\alpha \rightarrow$ imperfection factor based on buckling class

NOTE: if $\lambda_e < 0.2$, then the column can achieve full plastic resistance i.e buckling eff. can be neglected.

Buckling class as per IS 800-2007.

Rolled Angle, channel sec^n, T sec^n, solid sec^n (Plat, Flats etc)

$\Downarrow$

Buckling class - C

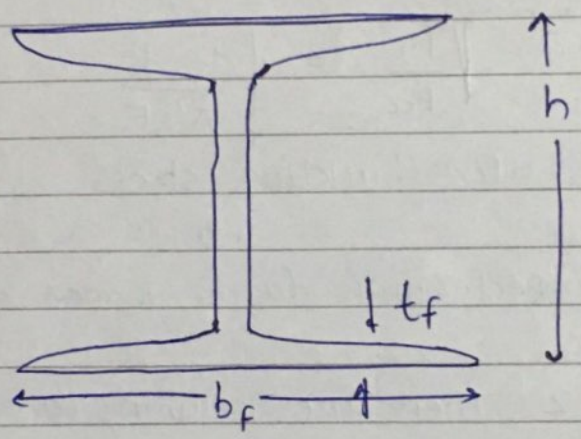
Built-up member → Buckling class - C

Hollow sec^n (Hot rolled) → A

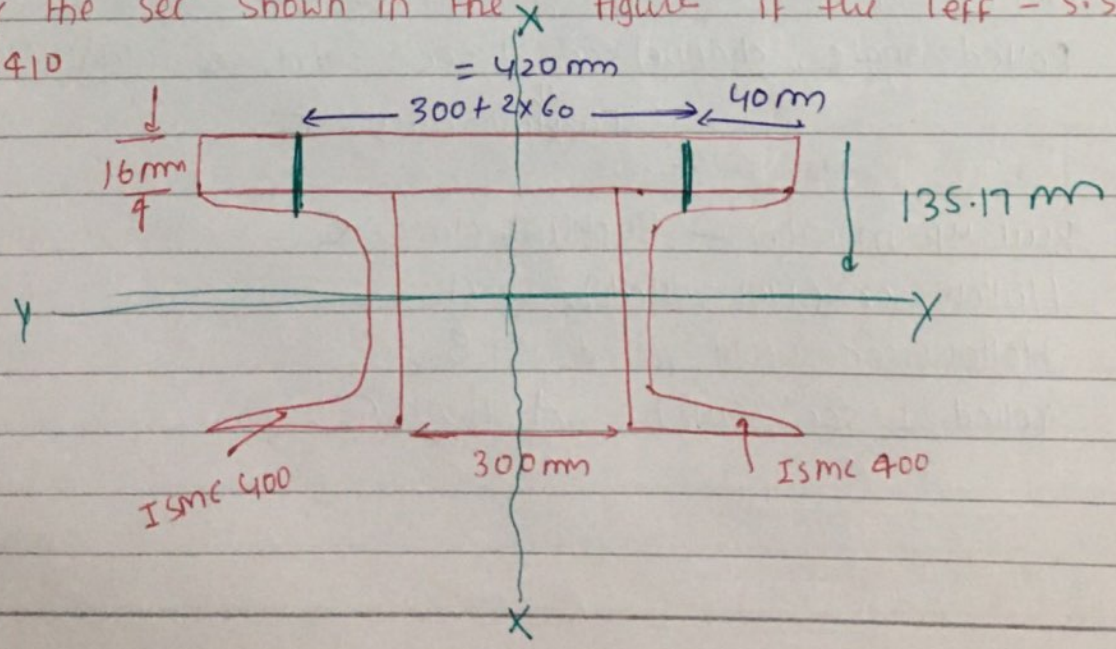
Hollow sec^n (cold rolled) → B

Rolled - I sec^n can be A, B or C

limit	Buckling about	Buckling class.
$\frac{h}{b_f} > 1.2$ and $t_f \le 40mm$	Z-Z	a
	Y-Y	b
$\frac{h}{b_f} > 1.2$ and $40 < t_f < 100$ mm.	Z-Z	b
	Y-Y	c



Que. Calculate the design comp. load that can be carried by the sec^n shown in the figure if the $l_{eff} = 5.5$ m Fe410



ISMC 400

$d = 400\ mm$ $A = 6293\ mm^2$ $b_f = 100\ mm$, $t_f = 15.3\ mm$

$t_w = 8.6\ mm$ $I_{zz} = 15.083 \times 10^7\ mm^4$, $I_{yy} = 0.5048 \times 10^7\ mm^4$

$g = 60\ mm$ $C_{yy} = 24.2\ mm$

Design comp. load $P_d = f_{cd} A_e$

$b_i = 420\ mm$

$b_o = 40\ mm$

$b_o = 100\ mm$

$$\frac{b_i}{t_f} \leq 42\varepsilon \Rightarrow b_i \leq 42 \times 1 \times 16$$

$$b_i \leq 672\ mm$$

$$\underline{\varepsilon = 1}$$

$$\frac{b_o}{t_f} \leq 15.7\varepsilon$$

$$b_o \leq 15.7 \times 1 \times 15.3 = 240.21\ mm > b_f\ (100\ mm)$$

We can consider all the area of c/s as the sec^n can be classified as semi-compact or better than that.

Total eff. Area :-

$$A_e = 2 \times 6293 + 16 \times 500 = 20586\ mm^2$$

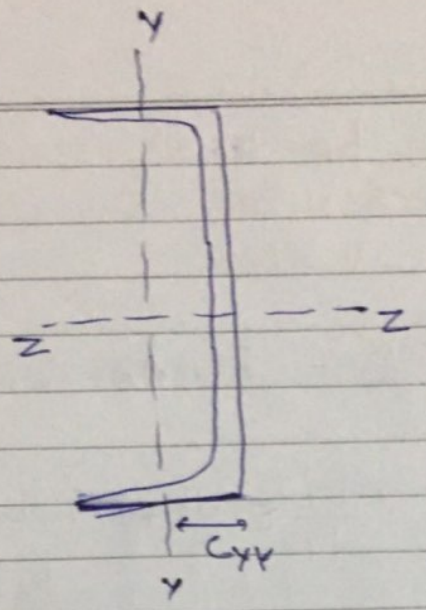
$$f_{cd} = \frac{f_y / \gamma_{mo}}{\phi + \sqrt{\phi^2 - \lambda_e^2}}$$

$$\phi = 0.5[1 + \alpha(\lambda_e - 0.2) + \lambda_e^2]$$

$$\lambda_e = \sqrt{\frac{f_y}{\pi^2 E}} \cdot \lambda \quad , \quad \alpha = 0.49$$

$$\lambda_{max} = \frac{l_{eff}}{r_{min}}$$

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



$$\bar{Y} = \frac{16\times500\times\frac{16}{2} + 2\times6293\times\left(16+\frac{400}{2}\right)}{16\times500 + 2\times6293}$$

$\bar{Y} = 135.17$ mm.

$$I_{xx} = \frac{16\times500^3}{12} + 2\times\left[\underbrace{I_{yy}}_{0.5048\times10^7} + \underbrace{A_{one}}_{6293}\left(\underbrace{C_{yy}}_{24.2} + \frac{300}{2}\right)^2\right]$$

$I_{xx} = 55.869\times10^7$ mm^4

$$I_{yy} = \frac{500\times16^3}{12} + 500\times16\times\left(135.17-\frac{16}{2}\right)^2$$

$$+ 2\left[\underbrace{I_{zz.one}}_{15.083} + \underbrace{A_{one}}_{6293}\left(216-135.17\right)^2\right]$$

$I_{yy} = 51.35\times10^7$ mm^4

$I_{min} = 51.35\times10^7$

$$\lambda_{max} = \frac{l_{eff}}{r_{min}} = \frac{5.5\times10^3}{\sqrt{\frac{51.35\times10^7}{20586}}} = 34.82$$

$$\lambda_e = \sqrt{\frac{f_y}{\pi^2 E}} \cdot \lambda = 0.392$$

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

$$\phi = 0.5\left(1 + 0.49(0.392 - 0.2) + 0.392^2\right)$$

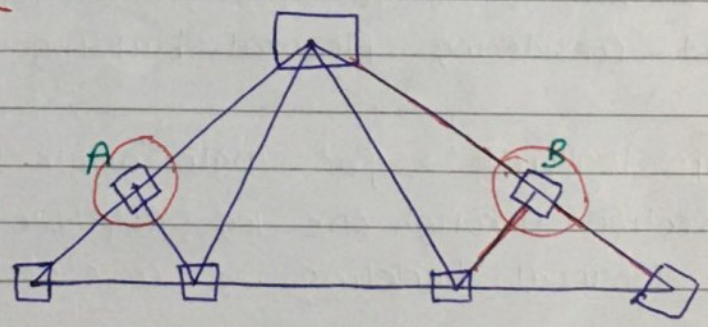
$$\phi = 0.62$$

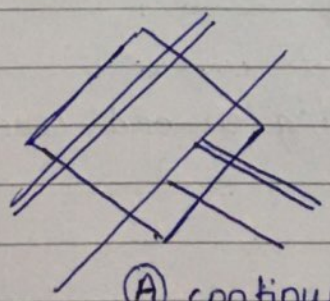
$$f_{cd} = \frac{f_y/\gamma_{m0}}{\phi + \sqrt{\phi^2 - \lambda_e^2}} = \frac{250/1.1}{0.62 + \sqrt{0.62^2 - 0.392^2}}$$

$$f_{cd} = 206.54 \text{ mpa}$$

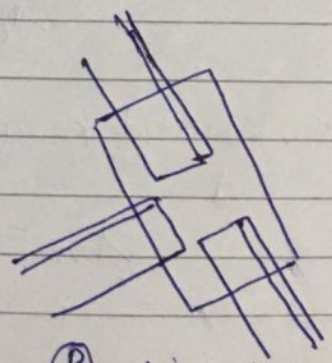
$$P_d = f_{cd} \times A_e = 206.54 \times 20586 = 4251.83 \text{ kN}$$

Angle struts





(A) continuous angle strut



(B) Discontinuous angle strut

Angle struts → continuous
Angle struts → Discontinuous

Angle struts → single Angle
Angle struts → Double Angle → same side of Gusset
Double Angle → opposite side of Gusset

Single Angle Strut comp in single angle strut may be transfered either concentrically or eccentrically by connecting only one leg to Gusset plate

Concentric loading → in case of concentric loading (Gusset on both legs) for single and f_{cd} shall be calculated considering flexural buckling using buckling class C

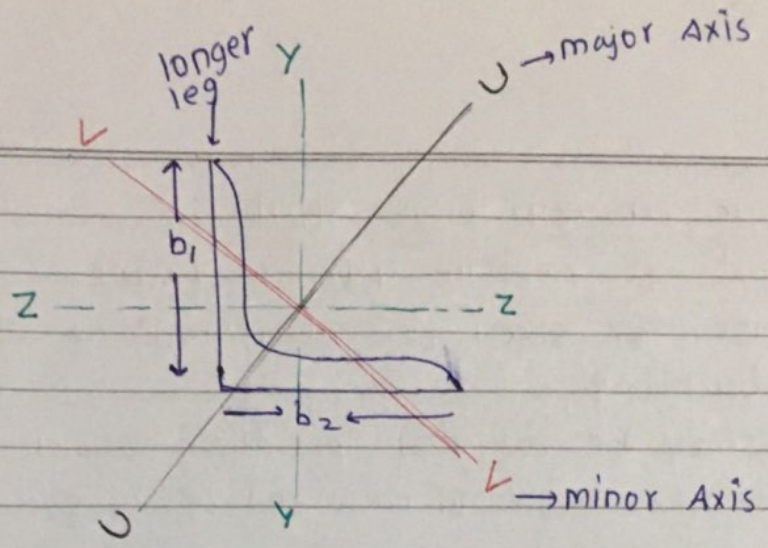
Eccentric loading → for single angle loaded eccentrically i.e connection through one leg only we need to consider flexural torsional buckling. In case of flexural torsional buckling

$$\lambda_e = \sqrt{K_1 + K_2 \lambda_{vv}^2 + K_3 \lambda_\phi^2}$$

K_1, K_2 and K_3 are constant depending on end cond.

$$\lambda_{vv} = \frac{l/r_{vv}}{\varepsilon \sqrt{\frac{\pi^2 E}{250}}}$$

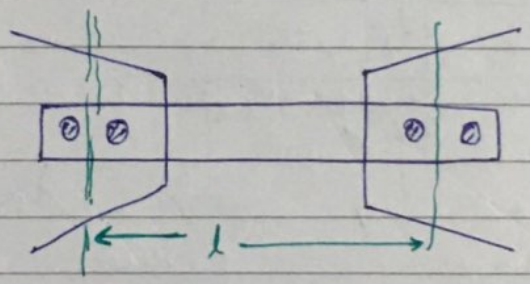
$$\lambda_\phi = \frac{\frac{b_1+b_2}{2t}}{\varepsilon \sqrt{\frac{\pi^2 E}{250}}}$$



$$\lambda_{vv} = \frac{\frac{l}{r_{vv}}}{\sqrt{\frac{\pi^2 E}{f_y}}} \qquad \lambda_\phi = \frac{\left(\frac{b_1+b_2}{2t}\right)}{\sqrt{\frac{\pi^2 E}{f_y}}}$$

$l \rightarrow$ c/c distance of supporting member and take

$l_{eff} = l$



Values of K_1, K_2 and K_3

No. of bolts at each end connection	Gusset/connecting member Fixity/Rigidity	K_1	K_2	K_3
≥ 2	Fixed	0.2	0.35	20
	Hinged	0.7	0.6	5
1	Fixed	0.75	0.35	20
	Hinge.	1.25	0.5	60

In case of Welded connection K_1, K_2 and K_3 shall be consider for Fixed cond. with > 2 bolt

Double Angle discontinuous strut

A) connected on opposite side of Gusset

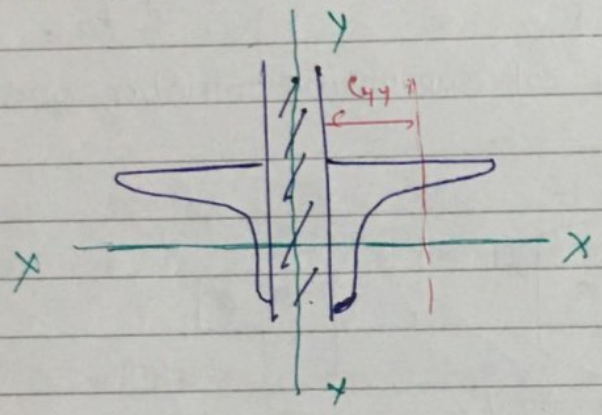
In this case fcd shall be calculated by using flexural buckling.

connection shall be by not less than 2 bolts or rivets in line along the angles @ each end or by equivalent welding.

l_{eff} shall be taken as follows:-

- for buckling in plane of truss → $0.7 - 0.85\, l$
- Buckling out of plane of truss → l

l → c/c distance b/w connection.



$$\lambda_e = \sqrt{\frac{f_y}{\pi^2 E}}\ \lambda$$

λ → $\max(\lambda_{xx}, \lambda_{yy})$

$$\lambda = \frac{l}{r}$$

$$\lambda_{xx} = \frac{l_{eff,xx}}{r_{xx}} = \frac{(0.70 - 0.85)\, l}{r_{xx}}$$

$$\lambda_{yy} = \frac{l_{eff,yy}}{r_{yy}} = \frac{l}{r_{yy}}$$

$$r_{xx,comb} = \sqrt{\frac{I_{xx,comb}}{A_{comb}}} = \sqrt{\frac{2 I_{xx,one}}{2 A_{one}}} = r_{xx}$$

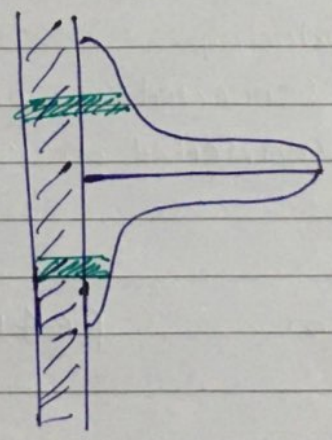
$$r_{yy,comb.} = \sqrt{\frac{I_{yy,comb}}{A_{,comb}}} = \sqrt{\frac{2\left[I_{yy,one} + A_{one}\left(C_{yy} + \frac{t}{2}\right)^2\right]}{2\,A_{one}}}$$

$$= \sqrt{r_{yy}^2 + \left(C_{yy} + t/2\right)^2}$$

B) connected on the same side of Gusset

in this case fcd shall be calculated considering Flexural torsional buckling. and treat the angle as single angle eccentrically loaded.

To calculate the total capacity, calculate capacity of one angle and multiply this by 2



$$\lambda_e = \sqrt{K_1 + K_2\lambda_{vv}^2 + K_3\lambda_\phi^2}$$

$$\lambda_{vv} = \frac{l/r_{vv}}{\sqrt{\frac{\pi^2 E}{250}}} \quad , \quad \lambda_\phi = \frac{\frac{b_1+b_2}{2t}}{\sqrt{\frac{\pi^2 E}{250}}}$$

Continuous Double Angle

It shall be designed as axially loaded member considering flexural buckling

eff. length is taken as $(0.7-1)\,l$ in the plane of truss and l out of plane

Que: calculate the strength of a discontinuous strut of $l = 3.5\,m$, the strut consist of 2 unequal angles $100\times75\times8$, use 10 mm thick gusset plate Fe 410, properties of Angle sec^n –

$A = 13.4\ cm^2$, $C_{xx} = 3.16\ cm$, $C_{yy} = 1.8\ cm$

$r_{xx} = 3.14\ cm$, $r_{yy} = 2.1\ cm$

$r_{uu} = 3.48\ cm$ $r_{vv} = 1.59\ cm$

Calculate considering

A) two Angles connected on opposite side of gusset

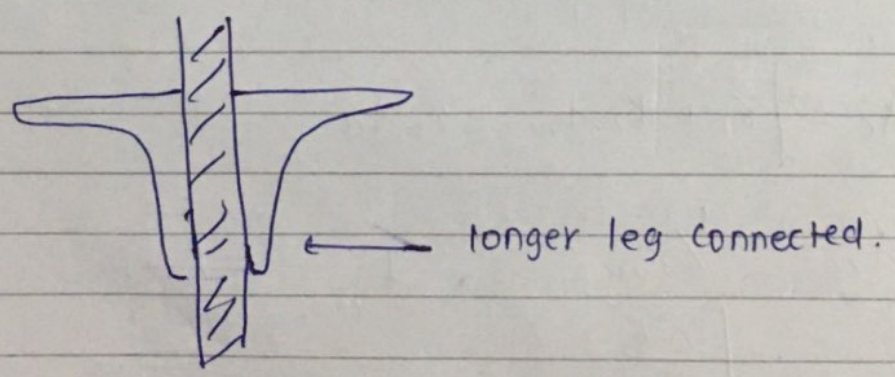
b) two Angles connected on same side of gusset

$l = 3.5\ m$

$100\times75\times8$

gusset $= 10\ mm$ Fe410

a)



consider Flexural buckling.

$$f_{cd} = \frac{\frac{f_y}{\gamma_{mo}}}{\phi + \sqrt{\phi^2 - \lambda_e^2}}$$

$$\lambda_e = \frac{f_y}{\sqrt{\frac{\pi^2 E}{250}}} \cdot \lambda_{max}$$

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

$$\lambda_{xx} = \frac{l_{eff,xx}}{r_{xx,comb}} = \frac{0.85 l}{r_{xx,one}} = \frac{0.85 \times 3.5 \times 10^2}{3.14}$$

in the plane of truss

$$= 94.74$$

$$\lambda_{yy} = \frac{l_{eff,yy}}{r_{yy,comb}} = \frac{3.5 \times 10^2}{\sqrt{2.18^2 + (1.87 + \frac{1}{2})^2}}$$

out of plane

$$= 108.69$$

$$\lambda_e = \sqrt{\frac{f_y}{\pi^2 E}} \cdot \lambda_{max}$$

$$\lambda_e = \sqrt{\frac{250}{3.14^2 \times 2 \times 10^5}} \times 108.69$$

$$\lambda_e = 1.223$$

$$\phi = 0.5\left(1 + 0.49\,(1.223 - 0.2) + 1.223^2\right]$$

$$\phi = 1.498$$

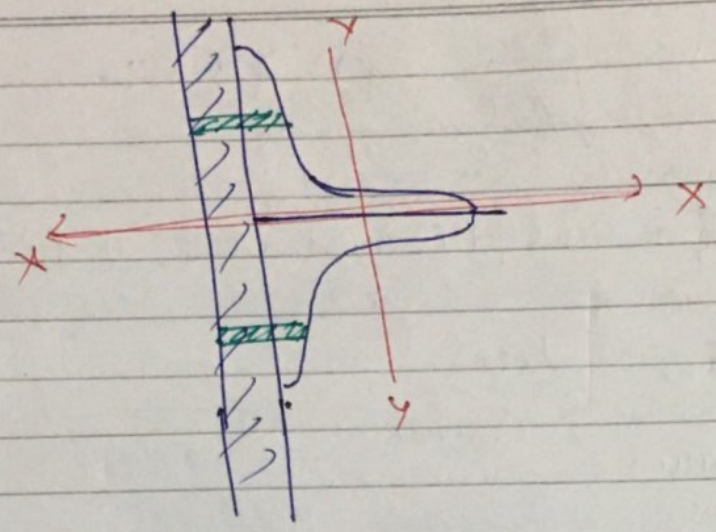
$$f_{cd} = \frac{250/1.1}{1.498 + \sqrt{1.498^2 - 1.223^2}}$$

$$f_{cd} = 96.18 \text{ MPa}$$

$$\text{Design force} = f_{cd} \times A_e = 96.18 \times 2 \times 13.4 \times 10^2$$

$$= 257.76 \text{ kN}$$

b)



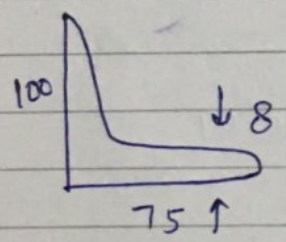
When the Angles are connected on same side we need to consider flexural torsional buckling

Alternative 1

considering single Angle @ a time

capacity = 2× capacity of single angle

$$\lambda_e = \sqrt{K_1 + K_2 \lambda_{vv}^2 + K_3 \lambda_\phi^2}$$



$$\lambda_{vv} = \frac{\frac{3.5 \times 10^3}{15.9}}{\sqrt{\frac{\pi^2 E}{f_y}}} = 2.478$$

$$\lambda_\phi = \frac{\frac{b_1 + b_2}{2t}}{\sqrt{\frac{\pi^2 E}{f_y}}} = 0.123$$

Assuming fixed cond. and more than 2 bolts

$K_1 = 0.2$, $K_2 = 0.35$, $K_3 = 20$

$$\lambda_e = \sqrt{K_1 + K_2 \lambda_{ev}^2 + K_3 \lambda_\phi^2}$$

$$\lambda_e = \sqrt{0.2 + 0.35 \times 2.47^2 + 90 \times 0.123^2}$$

$$\lambda_e = 1.624$$

$$\phi = 0.5\left[1 + \overset{0.49}{\alpha}(\lambda_e - 0.2) + \lambda_e^2\right]$$

$$\phi = 2.167$$

$$f_{cd} = \frac{f_y/\gamma_{mo}}{\phi + \sqrt{\phi^2 - \lambda_e^2}} = \frac{250/1.1}{2.167 + \sqrt{2.167^2 - 1.624}}$$

$$= 63.10 \text{ mpa}$$

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

$$= 63.10 \times 2 \times 13.4 \times 10^2$$

$$= 169.10 \text{ kN}$$

Alternative-2

considering both the angles as single unit width flexural torsional buckling

$$r_{xx,comb.} = \sqrt{\frac{I_{comb,xx}}{A_{comb.}}} = \sqrt{\frac{2 \times \left[I_{xx,one} + A_{one} \times C_{xx}^2\right]}{2 A_{one}}}$$

$$r_{xx,comb} = \sqrt{r_{xx,one}^2 + C_{xx}^2}$$

$$= \sqrt{3.14^2 + 3.1^2} = 4.4124 \text{ cm}$$

$$r_{yy,comb.} = \sqrt{\frac{2 I_{yyone}}{2 A_{one}}} = r_{yy,one} = 2.18 \text{ cm}.$$

$r_{min} = 2.18$ cm.

$$\lambda_{vv} = \frac{l/r_{min}}{\sqrt{\frac{\pi^2 E}{250}}} = \frac{\frac{3.5\times10^3}{21.8}}{\sqrt{\frac{3.14^2\times2\times10^5}{250}}} = 1.080$$

$\lambda_\phi = 0.123$

$$\lambda_e = \sqrt{K_1 + K_2\lambda_{vv}^2 + K_3\lambda_\phi^2}$$

$$\lambda_e = \sqrt{0.2 + 0.35\times1.80^2 + 20\times0.123^2}$$

$\lambda_e = 1.28$

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

$$\phi = 0.5\left(1 + 0.49(1.28 - 0.2) + 1.28^2\right)$$

$\phi = 1.58$

$$f_{cd} = \frac{f_y/\gamma_{mo}}{\phi + \sqrt{\phi^2 - \lambda_e^2}} = 90.66 \text{ mpa.}$$

$P_d = f_{cd}\times A$

$= 90.66\times2\times13.4\times10^2$

$= 243.02$ KN.

Design of comp. Members

Design of comp. member is a trial and error procedure because to find out gross Area we require f_{cd}, However f_{cd} itself is dependent on c/s. so we can't have a direct sol^n and we need to adopt trial & Error –

Step-1 calculate factored load

Step-2 Assume f_{cd} b/w 0.4 – 0.6 f_y or we can Conside λ Using the following table based on experience

Type	λ
Single Angle	100-150
Double Angle	80-120
single channel	90-110
Double channel	40-80
single I	80-150
Double II	30-60

Step-3 Assume sec^n based on Gross Area calculated and find out λ of the assumed sec^n

λ < permissible value as per IS code

Step-4 calculate f_{cd} for considered sec^n and find out load carrying capacity

if load carrying capacity > factored load

$\Rightarrow$ sec^n is safe

or else consider a larger sec^n or a diff^n arrangement

The load carrying capacity shall not be generally more than 5% of the factored load.

* comp member में shear lag तो होगा But उससे पहले ही member Buckle हो जायेगा।

Que Design a single angle strut to carry a factored load of 70 kN, length of strut is 3m b/w the intersection. it is connected to 12 mm thick gusset plate using M20 bolts. of property class 4.6 Fe410 → steel.

For flexural buckling.

λ	f_{cd} (MPa)
70	152
80	136
90	121
100	107
110	94.6
120	83.7
130	74.3
140	66.2
150	59.2

sec^n available are.

ISA 80×50×8
$r_{vv} = 10.6$ mm
$A = 978$ mm^2

ISA 90×90×6
$r_{vv} = 17.5$ mm
$A = 1050$ mm^2

$P_u = 70$ kN , $l = 3$ m. gusset = 12 mm.

Either we can consider a perticular λ for single angle strut and based on the f_{cd}, the angle sec^n can be selected. Finally we need to ensure that the selected angle satisfies λ and strength criteria.
Or in exam cond. we can consider the smallest angle sec^n and check the adequacy if not satisfied then select the next available sec^n.

consider $\lambda = 120$ hence $f_{cd} = 83.7$ mpa.

gross Area req^n $\frac{P_u}{f_{cd}} = \frac{70000}{83.7} = 836$ mm^2

Single Angle Tension member में unequal
comp. member में equal prefer करते है. λ_{vv} कम आयेगा

let us consider ISA 80×50×8 with $A_g = 978\ mm^2$

$$\lambda_{max} = \frac{l_{eff}}{r_{min}} = \frac{3\times10^3}{10.6} = 283.01 > 180$$

hence not safe

consider ISA 90×90×6 with $A_g = 1050\ mm^2$

$$\lambda_{max} = \frac{l_{eff}}{r_{min}} = \frac{3\times10^3}{17.5} = 171.42 < 180$$

safe.

In case of comp. member given an option consider equal angle for single angle strut because for same c/s area equal angle will have higher r_{vv} and higher f_{cd} value

for single angle strut, we need to consider Flexural torsional buckling

$$\lambda_e = \sqrt{K_1 + K_2\lambda_{vv}^2 + K_3\lambda_\phi^2}$$

$K_1 = 0.2$, $K_2 = 0.35$, $K_3 = 20$

$$\lambda_{vv} = \frac{\frac{l}{r_{vv}}}{\sqrt{\frac{\pi^2 E}{f_y}}} = \frac{\frac{3\times1000}{17.5}}{\sqrt{\frac{3.14^2\times200000}{250}}} = 1.92$$

$$\lambda_\phi = \frac{(b_1+b_2)/2t}{\sqrt{\frac{\pi^2 E}{f_y}}} = \frac{\frac{(90+90)}{12}}{\sqrt{\frac{3.14^2\times2\times10^5}{250}}} = 0.168$$

$$\lambda_e = 1.43$$

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

$$\phi = 0.5\left(1 + 0.49(1.438 - 0.2) + 1.438^2\right)$$

$$\phi = 1.837$$

$$f_{cd} = \frac{f_y/\gamma_{m0}}{\phi + \sqrt{\phi^2 - \lambda_e^2}} = 76.26\ \text{mPa}$$

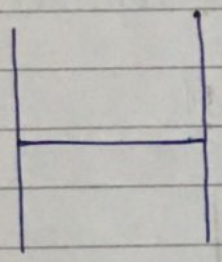
$$P_d = f_{cd} A_g$$

$$P_d = 76.26 \times 1050$$

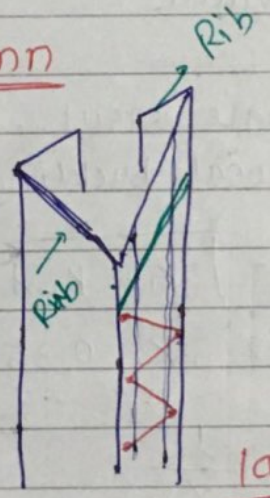
$$P_d = 80.07\ \text{KN} > 70\ \text{KN}$$

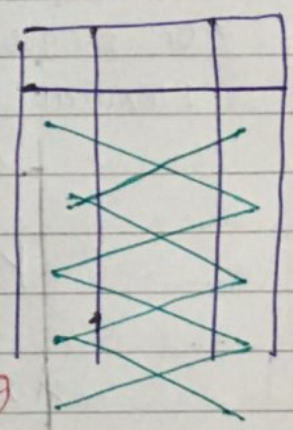
O.K

Built-up ~~Members~~ Column

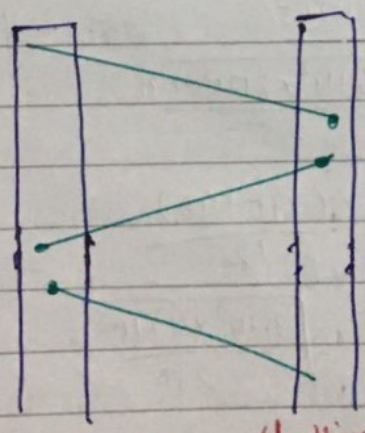


Rolled sec^n

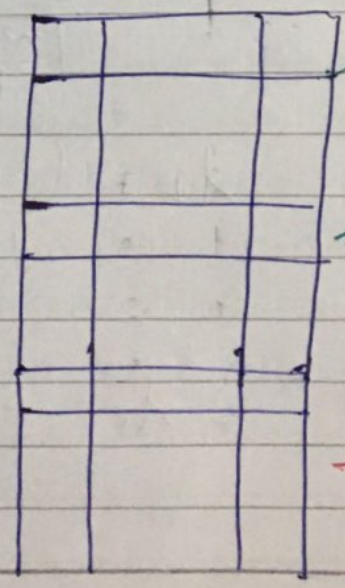




lacing



lacing (lattice colum)



Batten

When a large radius of gyration in z-diff^n dir^n or special shape is req^r we provide built up sec^n can be with lacing (lattice column) or batten preferable when comp. load is less and eff. length is more

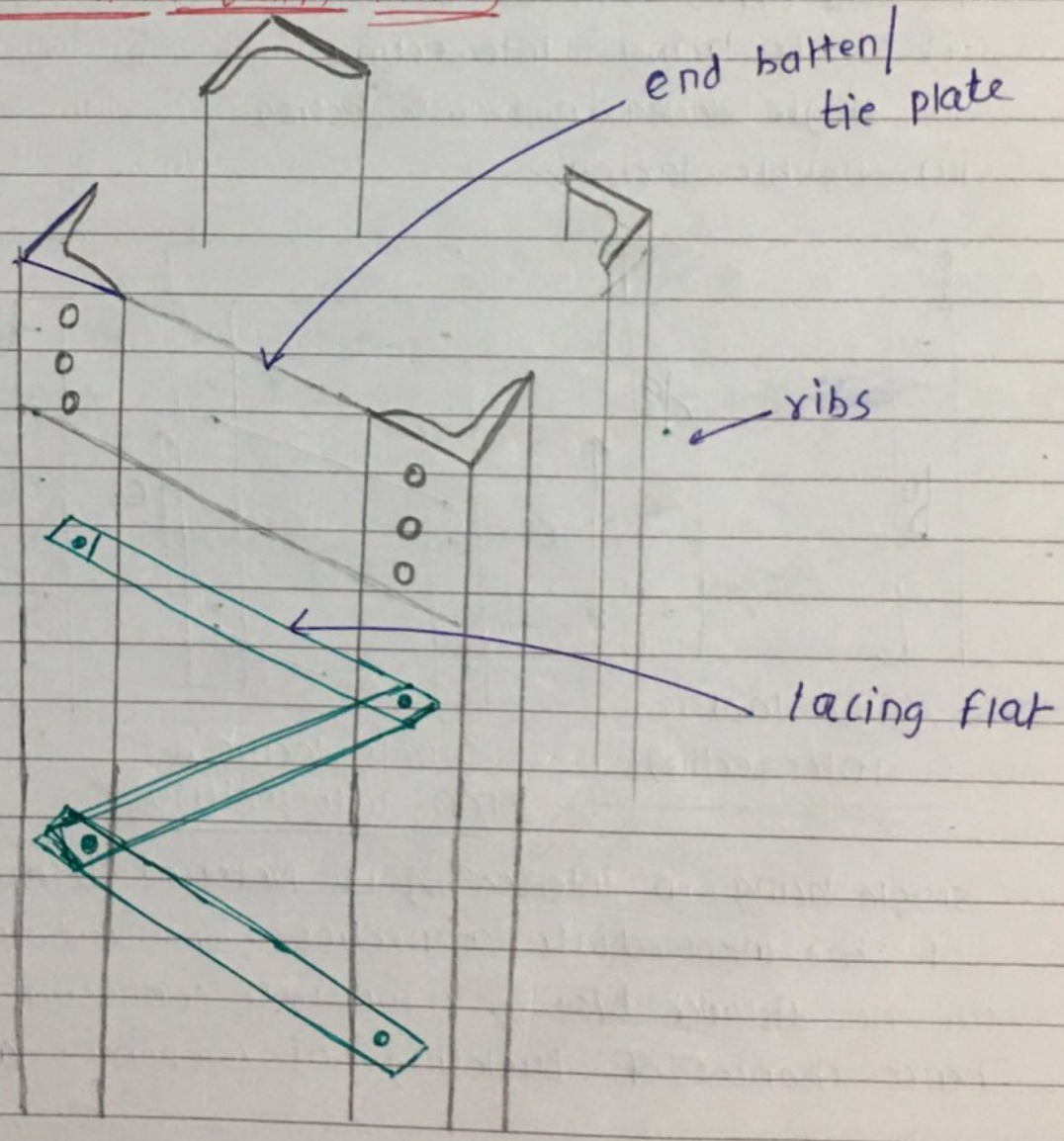
primary function of connecting system

i) hold the ribs in place and equalise stress

ii) points of intermediate support

iii) not considered as axial load carrying element

Built up column with lacing



- for designing the column with lacing, we need to first design the column and decide the spacing b/w the ribs i.e before lacing design column design has to be completed
- laced column is adopted in case of eccentrically loaded column and batten column is adopted in case of axially loaded column.

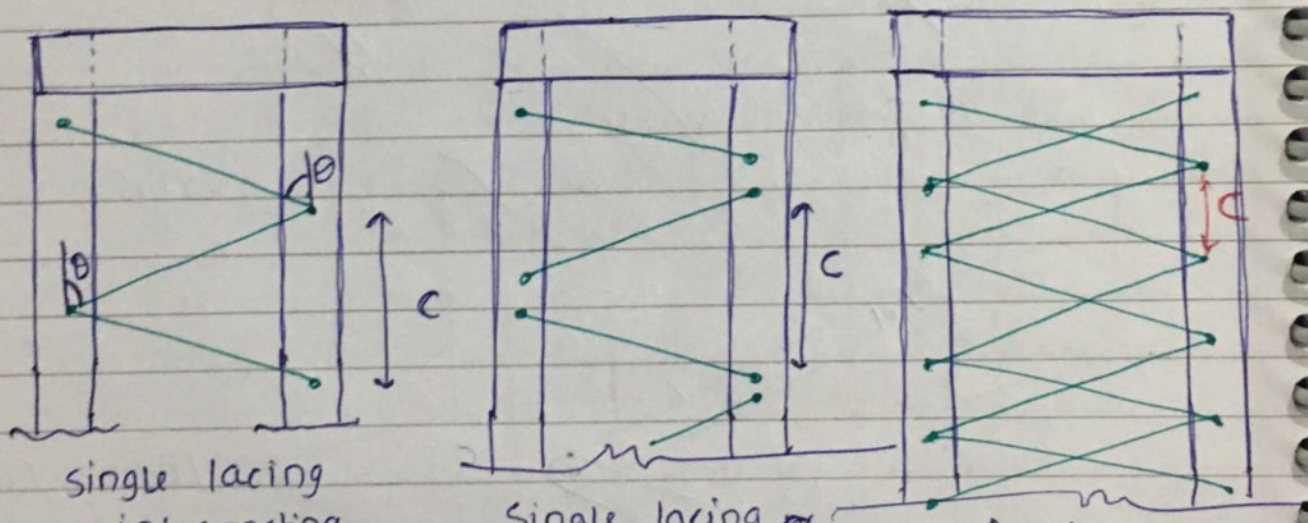
- Rigidity of battened column is less than laced column

Various types of lacing can be –

i) single lacing intersecting

ii) single lacing non-intersecting

iii) Double lacing.



single lacing intersecting

Single lacing non-intersecting

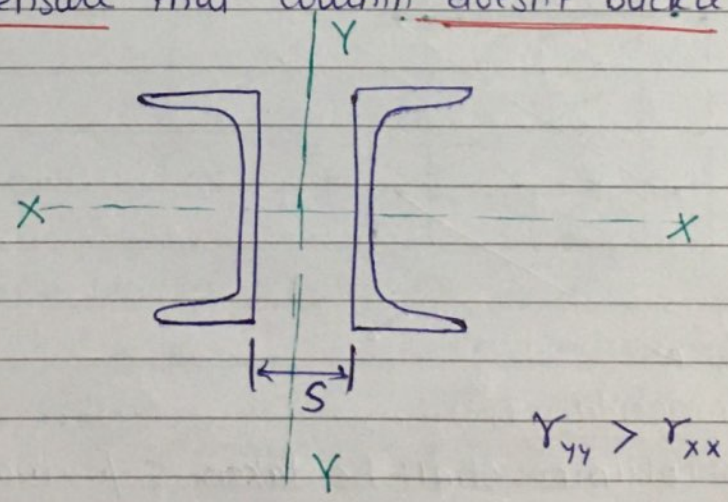
Double lacing

single lacing non-intersecting is preferred because failure of one intermediate connection won't lead to increase in the distance b/w intermediate connection significantly, hence chances of buckling of component column is less

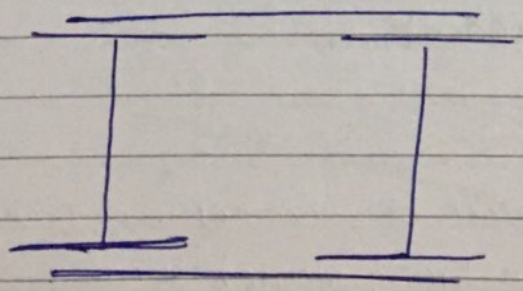
permitted inclination of lacing with the longitudnal axis of column is 40°–70°. * by increasing this angle we can reduce the value to C i.e height of component column even by increasing angle to 70°, if we are not able to prevent the chances of buckling of component column then we can provide double lacing.

At the top & bottom of laced column we provide battens called as end battens / tie plate
End batten ensures the balancing of unbalanced force in lacing, make the ends rigid and avoid distortion of end c/s

Generally r_{yy} should be slightly greater than r_{xx} for two channel or two I-sec^n. This can be achieved by increasing the spacing ~~that~~ ~~re~~ beyond that req^r for $r_{xx} = r_{yy}$
This will ensure that column doesn't buckle in plane of lacing.



$r_{yy} > r_{xx}$

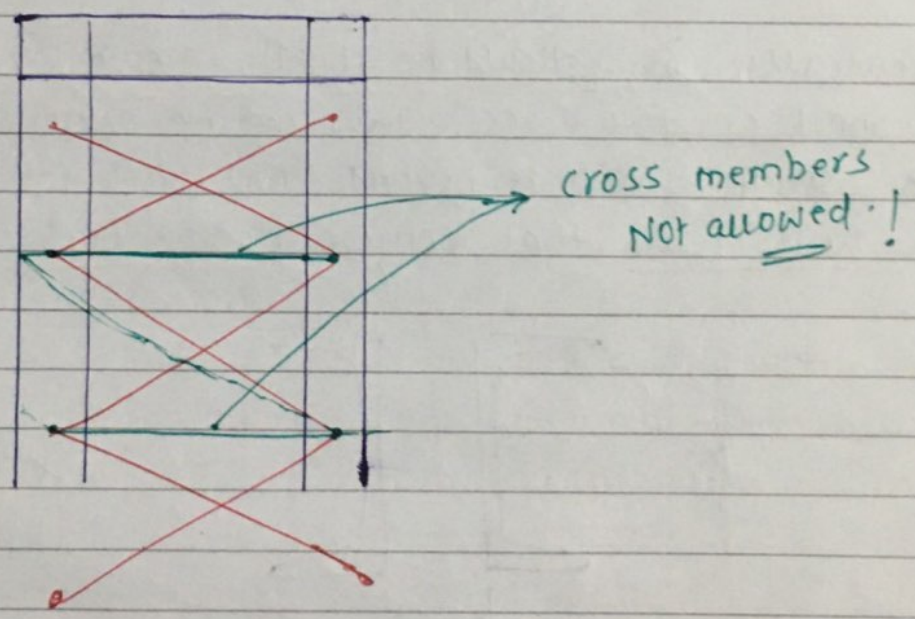


(2-I sec^n)

- lacing should not be varied along the length of column i.e c/s and angle of inclination shan't be varied along the length

- lacing on opposite faces should be mirror image and lacing on adjacent faces should be staggered. To reduce the height of component column.

- cross members are not desirable in case of double lacing column i.e lacing shan't be combined with batten o/w lacing will start ~~the~~ sharing axial load for which it has not been designed.



Design recommendations

① λ for laced column shall be taken 5% more than λ ~~for~~ calculated using l_{eff} and r_{min}

$\Rightarrow \quad \lambda = 1.05\lambda_0$

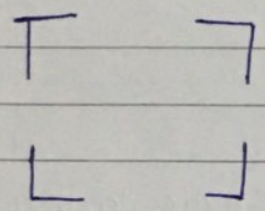
② The spacing b/w ribs in case of channel or I-sec^n shall ensure $r_{yy} > r_{xx}$ ($\perp$ to lacing plane)

③ Angle of inclination of lacing with longitudnal axis of column shall be b/w 40°–70° (for design purpose consider 45°)

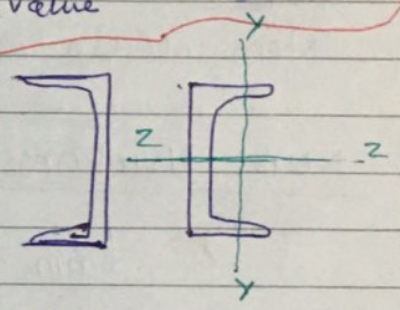
④ The max. spacing b/w intermediate connection should be such that λ of component column i.e

$$\frac{c}{r_{min,one}} \not> 50$$

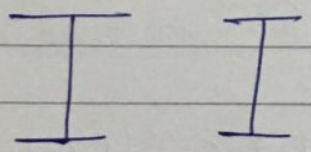
$$\not> 0.7\,\lambda_{whole\ value} = 0.7 \times 1.05\,\lambda_o$$



$r_{min,one} = r_{vv}$



$r_{min,one} = r_{YY}$



$r_{min,one} = r_{YY}$

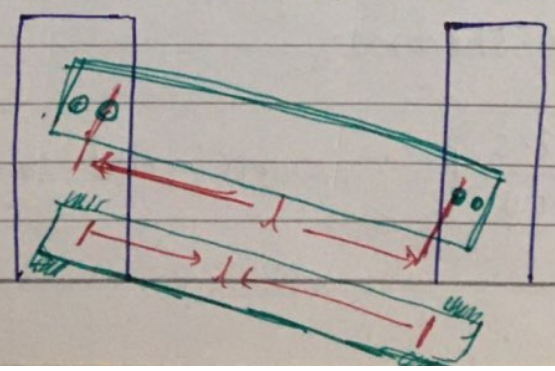
5) Effecting length of lacing bar

$l_{eff} = l$ (for single lacing)

$= 0.7\,l$ (for double lacing)

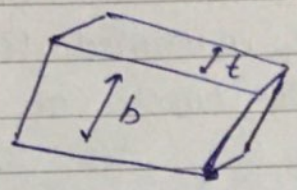
$= 0.7\,l$ for welded lacing

$l \rightarrow$ Distance b/w inner bolts & welds.



⑥ Max. λ

For single lacing $\frac{l_{eff}}{r_{min}} \not> 145$



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

⑦ Min. width of lacing flat

min width of lacing flat shall be 3 times of the Nominal dia of bolt.

⑧ Min. thickness of lacing bar

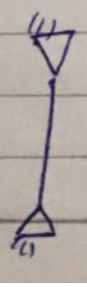
$$* \quad t_{min} = \frac{l_{eff}}{40} \rightarrow \text{for single lacing}$$

$$= \frac{l_{eff}}{60} \rightarrow \text{for Double lacing}$$

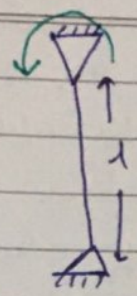
⑨ Design force for lacing

lacing should be designed for T & C due to SF in column There can be SF in column due to bending, lateral loading or accidental eccentricity or incidental eccentricity.

Due to incidental or * unavoidable eccentricity transverse SF is taken as $\boxed{2.5\%}$ of factored axial load. This shear is devided equally in all parallel planes resisting the shear.

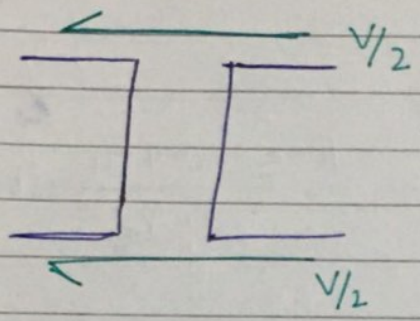


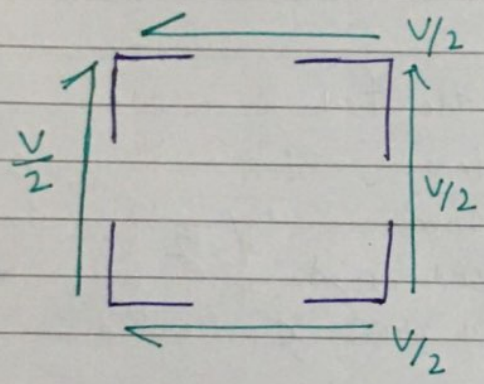
$$SF = V = \frac{2.5}{100} P_u \quad \text{, when axial load only}$$

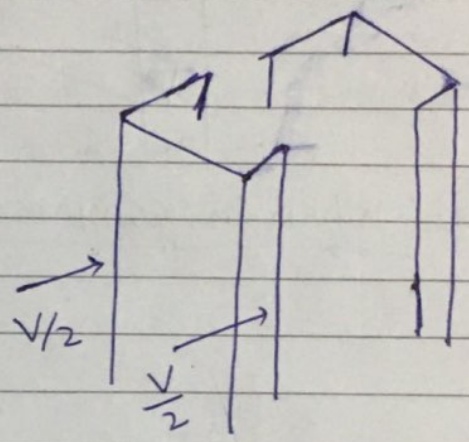


$SF = V = \frac{M_4}{l}$, when moment only.

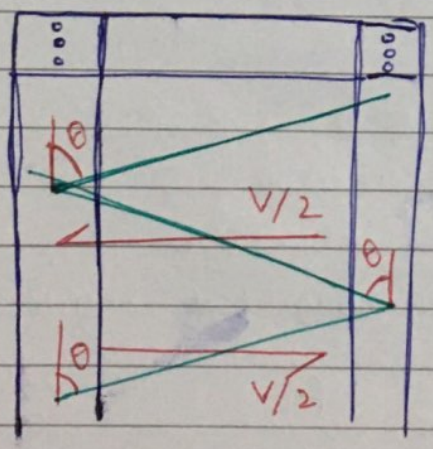
$SF = V = \frac{2.5P_4}{100} + \frac{M_4}{l}$; when axial force and moment both.

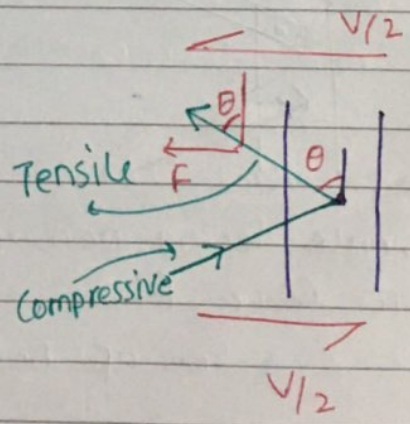






(10) Tension & comp. in lacing due to shear.





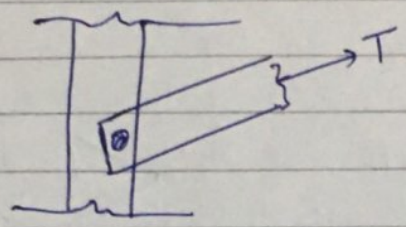
$F\sin\theta = V/2$

$F = \frac{V}{2\sin\theta}$

if @ a joint 2-lacing bars are meeting, one of them will be in tension and other will be in comp.

(11) Check for safety for lacing Bar

A) When in Tension



$$F \leq \frac{0.9 f_u}{\gamma_{m1}} \cdot (b - d_o) t$$

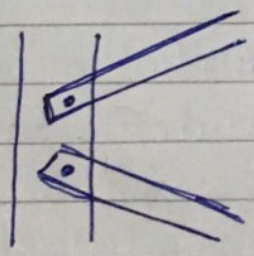
B) When in Compression

$$F \leq A_e \cdot f_{cd}$$

$$\lambda = \frac{l_{eff}}{r_{min}} = \frac{l_{eff}}{t/\sqrt{12}}$$

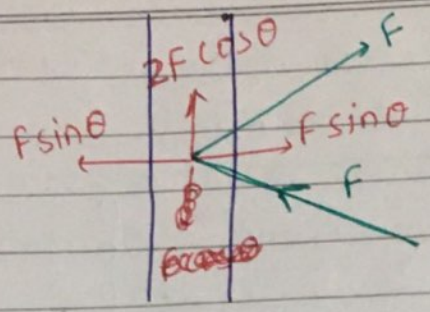
Calculate f_{cd} based on λ considering buckling class C (solid sec^n)

(12) Design for connection



single lacing non-intersecting, bolt shall be designed for force F

$$\text{no. of bolts} = \frac{F}{V_{db}}$$



Net force $= 2F\cos\theta$

$= 2\,\frac{V}{2\sin\theta}\cos\theta$

$= V\cot\theta$

no. of bolts $= \frac{V\cot\theta}{V_{db}}$

The bolts will be in single shear for the resultant force and for the individual force also.

$$V\cot\theta > \frac{V}{2\sin\theta}$$

$$\frac{V\cos\theta}{\sin\theta} > \frac{V}{2\sin\theta}$$

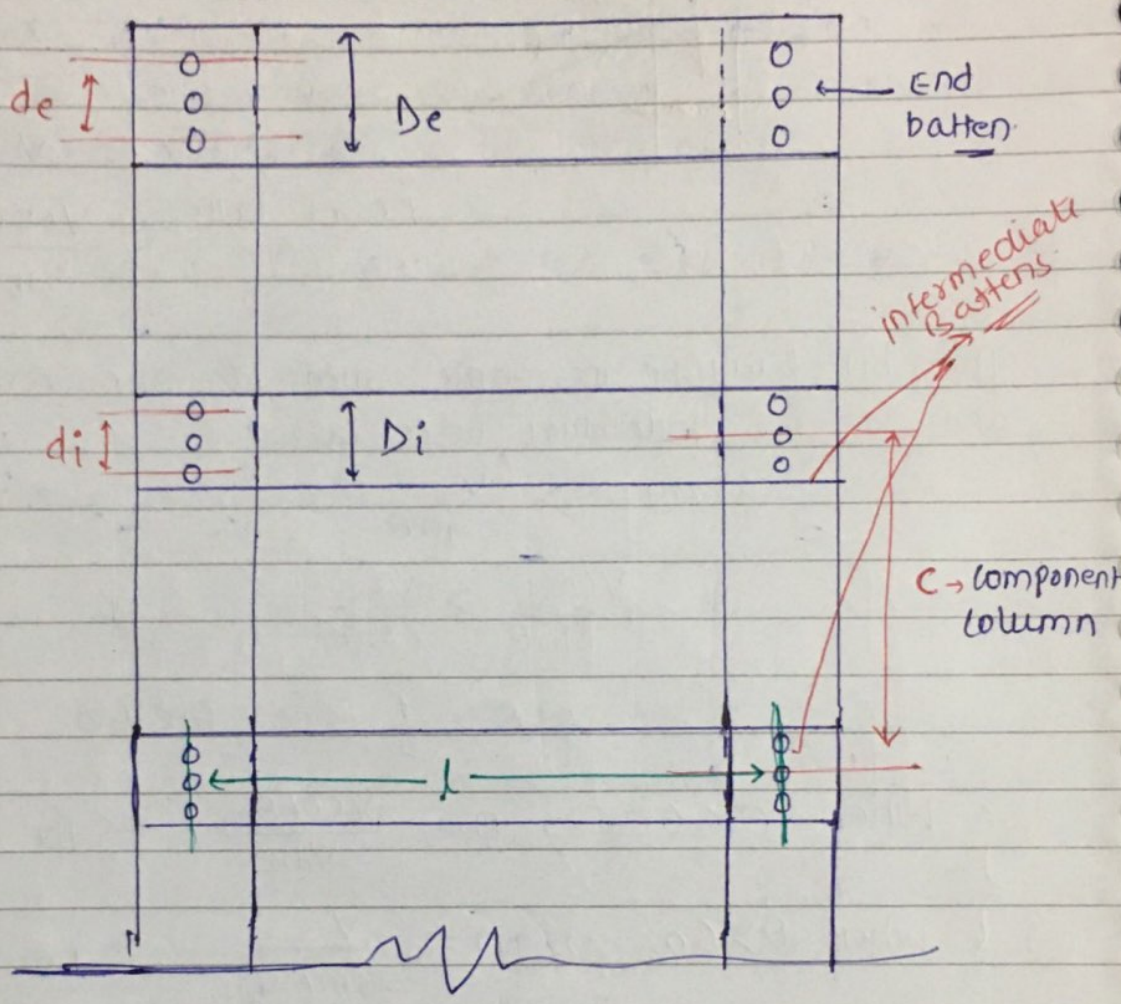
$$\cos\theta > \frac{1}{2} \Rightarrow \theta < 60^\circ$$

When $\theta < 60^\circ \Rightarrow n = \frac{V\cot\theta}{V_{db}}$ {for resultant}

When $\theta > 60^\circ \Rightarrow n = \frac{V}{2\sin\theta\, V_{db}}$ {for single force}

At the top & bottom of laced column, we need to provide end batten and recommendation will be covered in Batten column.

Battened column



Design recommendation

① no. of batten shall be such that column is devided into not less than 3 parts longitudnally i.e min. no. of intermediate battens should be 2

② λ of batten column

λ shall be taken as $1.1\lambda_o$

③ check for local buckling of component column

$$\frac{C}{r_{min,one}} \ngtr 50$$

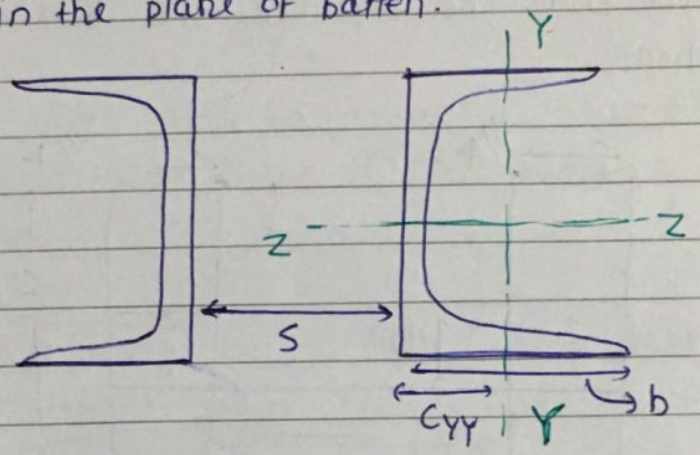
$$\ngtr 0.7\lambda_{whole\ column}$$

④ Width of batten.

A) For intermediate Batten

The intermediate batten shall have an effective depth of not less than $\frac{3}{4} \times$ perpendicular distance b/w the centroid of main member.

but in no case it shall be less than $2 \times$ width of one member in the plane of batten.



$d_i \nless 2b$

$d_i \nless \frac{3}{4}(S + 2C_{YY})$ $\qquad D_i = d_i + 2e$

B) For end batten.

$d_e \nless 2b$

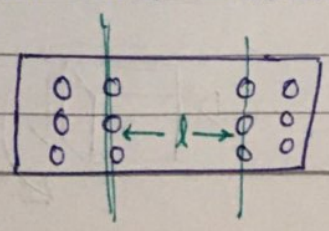
$d_e \nless S + 2C_{YY}$

$D_e = d_e + 2e$ $\qquad e \rightarrow$ edge distance.

⑤ Thickness of batten

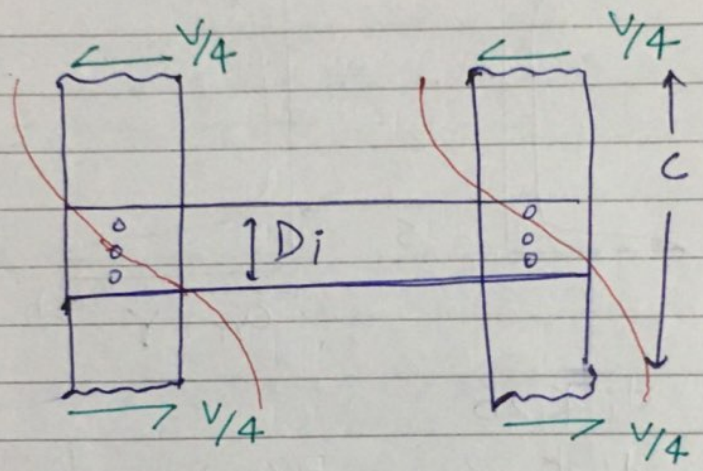
Shall be $< \frac{l}{50}$

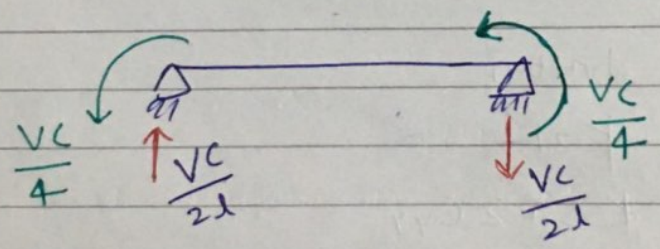
$l \rightarrow$ Distance b/w inner most connecting line of bolt or weld.

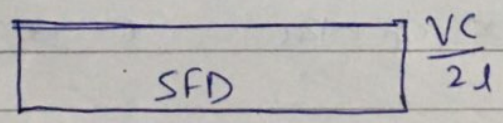


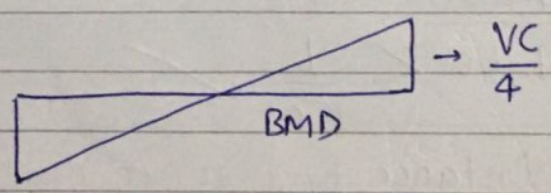
⑥ Shear due to accidential eccentricity shall be taken as $\boxed{2.5\%}$ of the axial loading and this shear shall be devided equally b/w parallel planes resisting the shear if there is lateral loading or moment acting on the column, shear will be calculated due to these also.

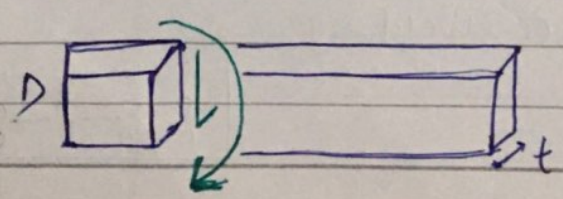
⑦ Batten plate shall be designed for BM and SF due to transverse shear.











The Batten plate shall be designed for $BM = \frac{Vc}{4}$ and $SF = \frac{Vc}{2l}$

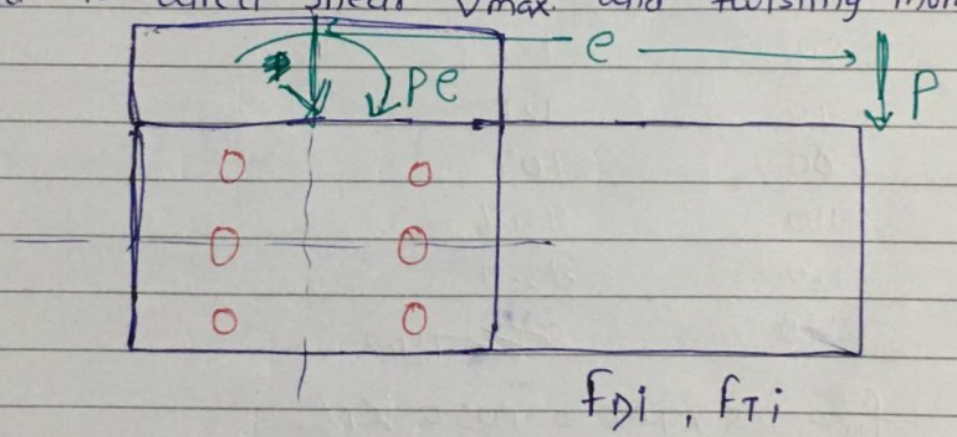
$$M_{max} = \frac{f_y}{\gamma_{m0}} \times \frac{tD^2}{6}$$

$$V_{max} = \frac{f_y}{\sqrt{3}\gamma_{m0}} tD$$

The above check shall be done for intermediate Batten and if intermediate Batten is safe then end batten will be safe.

Design for connection

The connection shall be designed as eccentric connection subjected to direct shear V_{max} and twisting moment M_{max}



$$F_{Ri} = \sqrt{F_{Di}^2 + F_{Ti}^2 + 2F_{Di}F_{Ti}\cos\theta}$$

For safety $F_{Ri} < V_{db}$ (Bolt value)

Qu. Design a Built up column to carry an axial load of 700 kN use 2 channel back to back $l_{eff} = 10$ m, provide single lacing with bolted connection. adopt grade of steel Fe 410, M20 Bolt, 4.6

Available channel sec^n are -

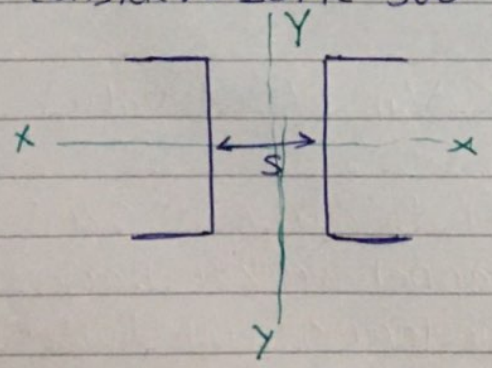
ISMC 300	ISMC 350
$A = 4564 mm^2$	$A = 5366 mm^2$
$r_{xx} = 118.1 mm$	$r_{xx} = 136.6 mm$
$r_{yy} = 26.18 mm$	$r_{yy} = 28.3 mm$
$C_{yy} = 23.6 mm$	$C_{yy} = 24.4 mm$
$b = 90 mm$	$b = 100 mm$
$g = 50 mm$	$g = 60 mm$

$f_y = 250 MPa$, Buckling class → C

λ	f_{cd} (MPa)
50	183
60	168
70	152
80	136
90	121
100	107
110	94.6
120	83.7
130	~~59.2~~ 74.2

$f_u = 1.5 \times 700 = 1050 kN$

consider 2-ISMC 300 back-back



$$\lambda_{max} = \frac{l_{eff}}{r_{min}}$$

$r_{yy} > r_{xx}$

$$= \frac{l_{eff}}{r_{xx,com}}$$

$$r_{xx\ com} = \sqrt{\frac{2\ I_{xx\ one}}{2A_{one}}} = r_{xx\ one}$$

$$\lambda_{max} = \frac{l_{eff}}{r_{xx,one}} = \frac{10\times10^3}{118.1} = 84.67 < 180$$

O.K

for a laced column, $\lambda_{max} = 1.05\lambda_o$

$= 1.05 \times 84.67$

$= 88.91$

$$f_{cd} = 136 + \frac{(121-136)}{(90-80)}(88.91-80)$$

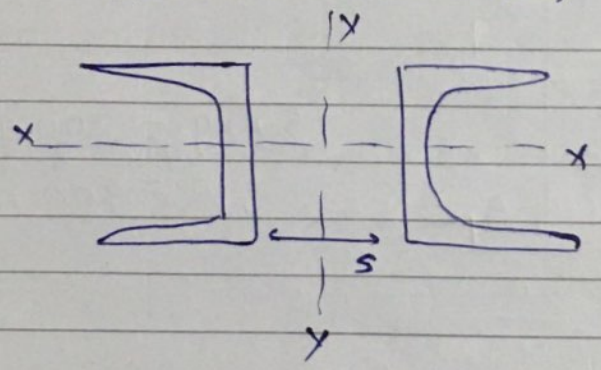
$f_{cd} = 122.63$ MPa

Axial load carrying capacity $= 122.63 \times 2 \times 4564$

$= 1119.4$ KN > 1050 KN

O.K

Spacing b/w the channels. (S)



In order to make $r_{yy} > r_{xx}$ i.e no buckling in lacing plane we need to ensure -

$$I_{yy\,comb} > I_{xx,\,comb}$$

$$\frac{= 2\times\left[I_{yy,one} + A_{one}\left(C_{yy} + \frac{S}{2}\right)^2\right]}{2\,A_{one}} = \frac{2\times I_{xx\,one}}{2\,A_{one}}$$

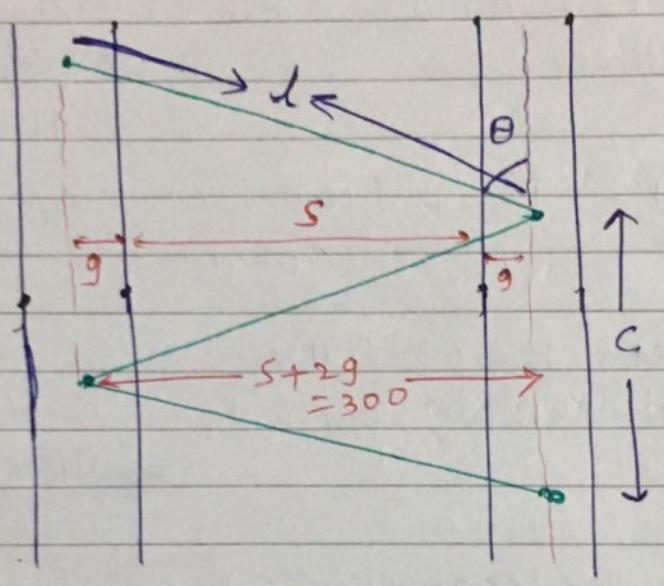
$$= r_{yy\,one}^2 + \left(C_{yy} + \frac{S}{2}\right)^2 = r_{xx,one}^2$$

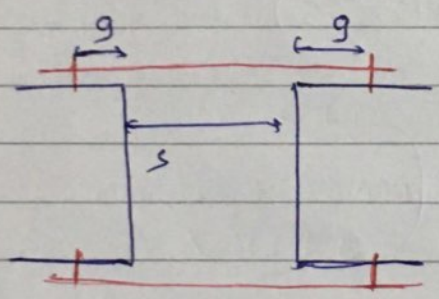
$$= 26.18^2 + \left(23.6 + \frac{S}{2}\right)^2 = 118.1$$

$S = 183.12$ mm

let us provide spacing b/w channels $S = 200\,mm$

lacing Design





$S + 2g = 200 + 2 \times 50$
$= 300\ mm.$

consider $\theta = 45°$

$$l = \frac{300}{1/\sqrt{2}} = 424.26\ mm.$$

thickness of lacing flat $= \frac{l_{eff}}{40} = \frac{424.26}{40} = 10.6\,mm$

consider thickness $= 12\,mm$

width of lacing flat $= 3d$
$= 3 \times 20 = 60\,mm.$

$b = 60\,mm,\ t = 12\,mm$

Check for buckling of component column

$C = 2 \times (S + 2g) = 600$ mm.

$$\lambda_{component,column} = \frac{c}{r_{min,one}} = \frac{600}{26.18} = 22.91$$

$\not> 50$

$\not> 0.7 \lambda_m$

$\not> 0.7 \times 88.91$

$\not> 62.23$

OK.

Check for buckling of lacing flat

$$\lambda_{max} = \frac{l}{r_{min}} = \frac{l}{t/\sqrt{12}} = \frac{424.26}{12/\sqrt{12}} = 122.47$$

$\not> 145$

O.K

Shear force $= \frac{2.5}{100} \times P_u = \frac{2.5}{100} \times 1050 = 26.25$ kN.

comp./tensile force on lacing flat

$$f = \frac{V}{2 \sin\theta} = \frac{26.25}{2 \sin 45^\circ} = 18.56 \text{ kN}.$$

Check in comp.

f_{cd} in lacing flat for $\lambda = 122.47$

$$f_{cd} = 83.7 + \left(\frac{74.2 - 83.7}{130 - 120}\right) \times (122.47 - 120)$$

$f_{cd} = 81.37$ MPa.

force carrying capacity of lacing flat $= f_{cd} A_e$

$= 81.37 \times 12 \times 60$

$= 58.58$ kN $>$ 18.56 kN

Check in tension

Net sec^n Rupture strength $= \frac{0.9 f_u}{\gamma_{m1}} (B - d_o) t$

$= \frac{0.9 \times 410}{1.25} (60 - 22) \times 12$

$= 134.6 \text{ kN} > 18.56 \text{ kN}$

O.K

Cross sec^n yielding $= \frac{f_y}{\gamma_{m0}} \times B \times t$

$= \frac{250}{1.1} \times 60 \times 12 = 163.63 > 18.56$ kN

OK.

Design of connection

for M20 bolt, $V_{db} = 45.27$ kN

$\theta = 45^\circ < 60^\circ$

connection shall be design for a force of

$V \cot\theta = 26.25 \times \cot 45 = 26.25$ kN

No. of bolts $= \frac{26.25}{45.27} \cong 1$ bolt.

Que Design the column given in previous Que. considering batten plate.

λ for batten column will be $\rightarrow 1.1 \lambda_o$

$= 1.1 \times \frac{l_{eff}}{r_{xx}}$

$= 1.1 \times \frac{10 \times 10^3}{118.1} = 93.14$

$f_{cd} = 121 + \frac{(107 - 121)}{100 - 90} \times (93.14 - 90)$

$f_{cd} = 116.6$ mpa

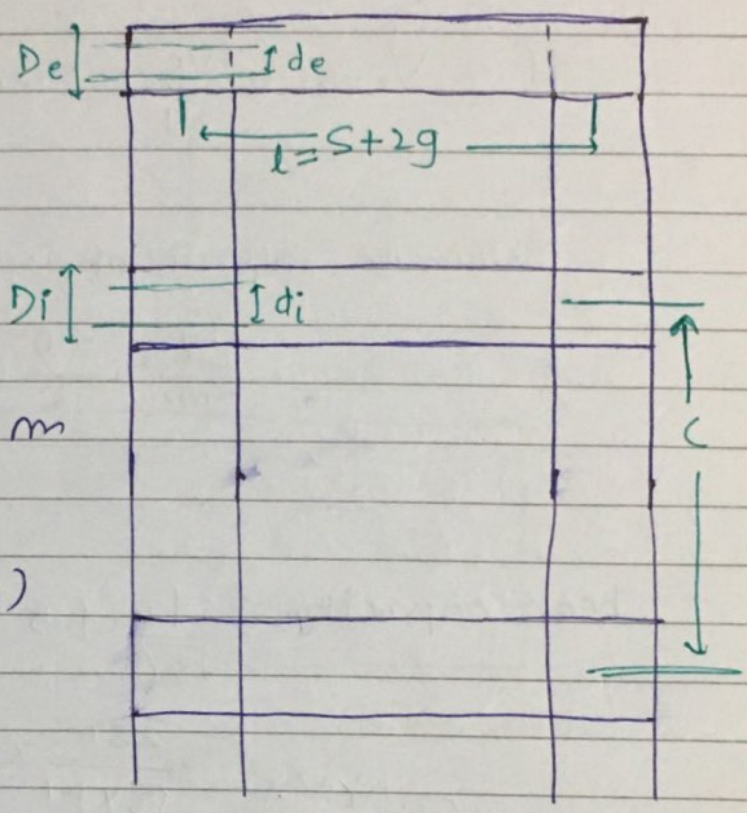
$P_d = f_{cd} \times A_e = 116.6 \times 2 \times 4564 = 1064.32$ kN.

> 1050 kN

O.K

consider $S = 200$ mm.

Design of Batten



$l = S + 2g$

$= 300$ mm

thickness $= \frac{300}{50}$

$= 6$ mm.

$d_i \nless 2b = 2 \times 90 = 180$ mm

$\nless \frac{3}{4}(S + 2C_{yy})$

$= \frac{3}{4}(200 + 2 \times 23.6)$

$= 185.4$ mm

$D_i = d_i + 2e$

$= 186 + 2 \times 1.5 \times 22 = 252$ mm

$d_e \nless 2b = 180$

$\nless S + 2C_{yy} = 247.2$ $= 247.2$ mm

$D_e = 247.2 + 2 \times 1.5 \times 22 = 313.2$ mm

$D_e = 315$ mm.

Check for buckling of component column

$\frac{C}{r_{min,one}} \ngtr 50$

$\ngtr 0.7 \times \lambda_{whole} = 0.7 \times 93.14 = 65.2$

$C \ngtr 50 \times r_{min,one} = 50 \times 26.18$

$= 1309$ mm

take $c = 1200$ mm.

BM, $M_{max} = \frac{VC}{4} = \frac{26.25 \times 1.2}{4} = 7.87$ kNm

SF, $V_{max} = \frac{VC}{2l} = \frac{26.25 \times 1.2}{2 \times 0.3} = 52.5$ kN

moment capacity of intermediate Batten

$$= \frac{f_y}{\gamma_{m0}} \times \frac{tD^2}{6} = \frac{250}{1.1} \times \frac{6 \times 252^2}{6}$$

$$= 14.43 \text{ kNm} > 7.87 \text{ kNm}$$

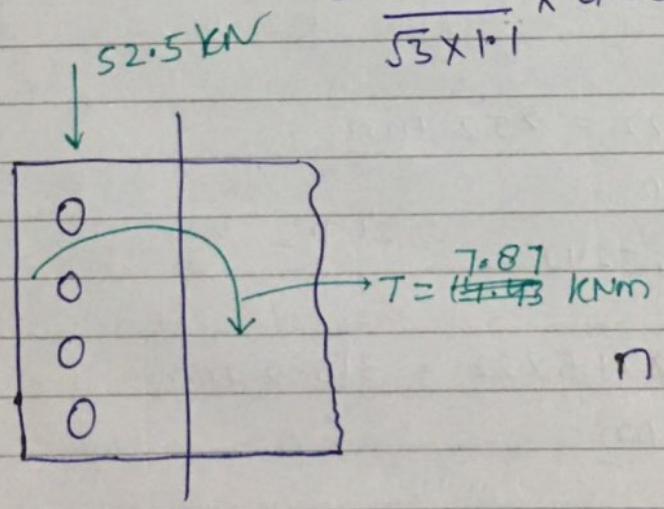
O.K.

Shear capacity $= \frac{f_y}{\sqrt{3}\gamma_{m0}} t.D$

$$= \frac{250}{\sqrt{3} \times 1.1} \times 6 \times 252 = 198.63 \text{ kN}$$

> 52.5 kN.

O.K.



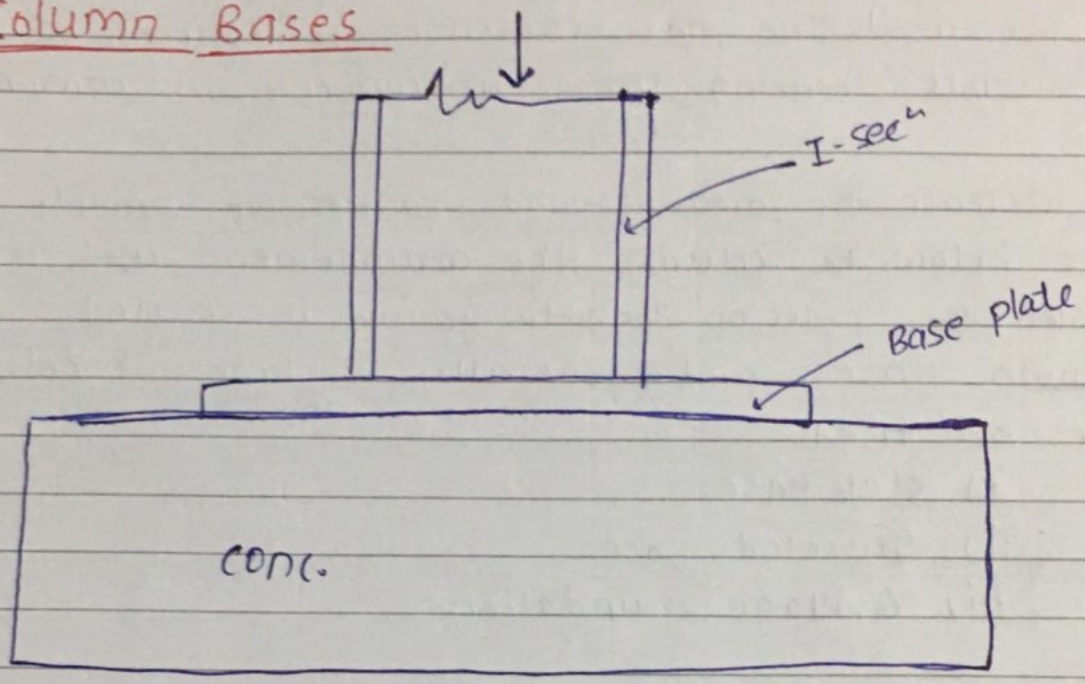
$$n = \sqrt{\frac{6M}{mPV_{ab}}}$$

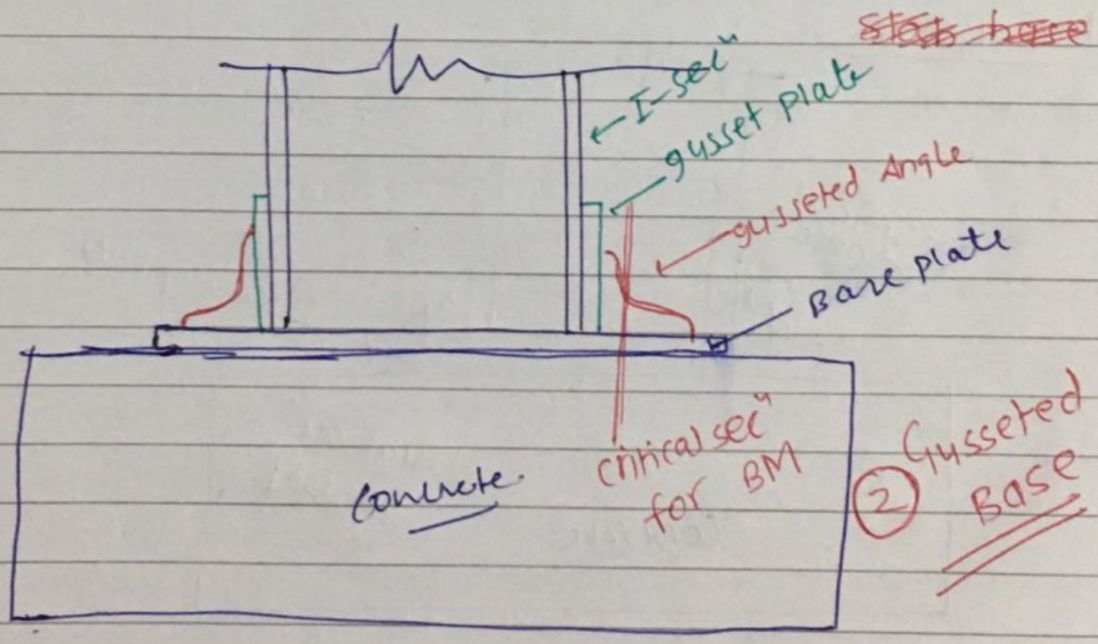
$f_{Di} = \frac{V}{n}$

$f_{Ti} = \frac{T r_i}{J} . A_i$

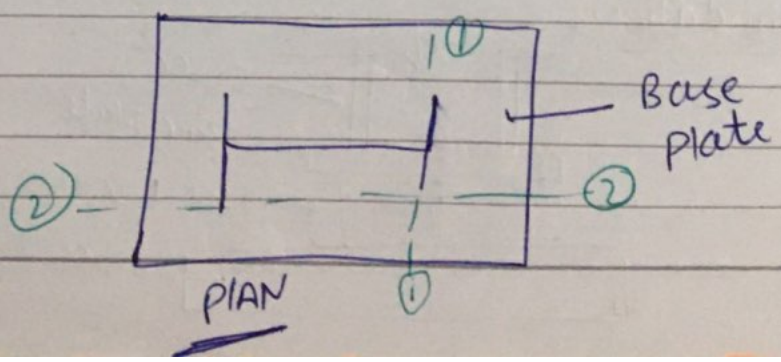
connection can be designed as eccentric connection for a single line of bolt.

Column Bases





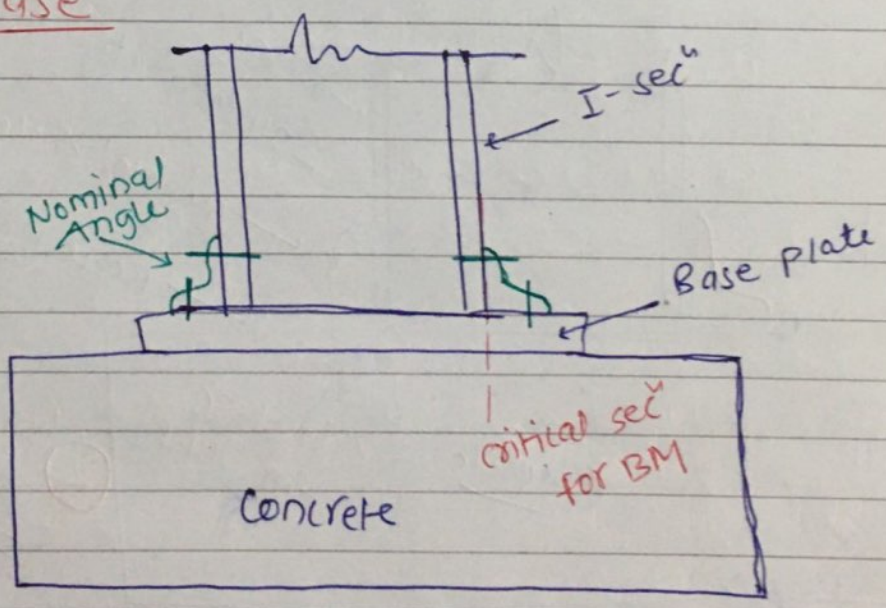
② Gusseted Base



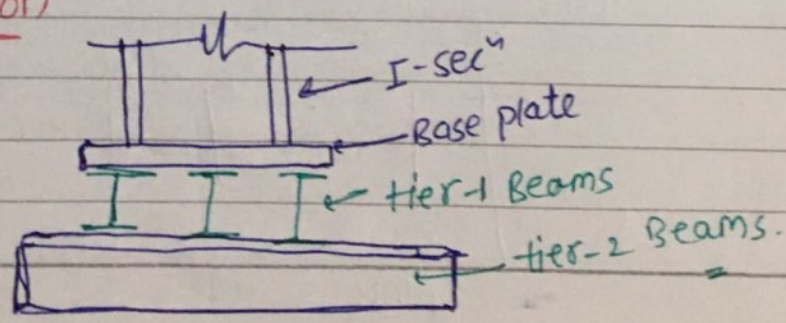
If steel column is directly placed on a concrete pedestal, concrete will crush, to safeguard against this, the area of load transfer needs to be increased. The max. pressure on concrete < the permissible bearing stress in concrete in compression.

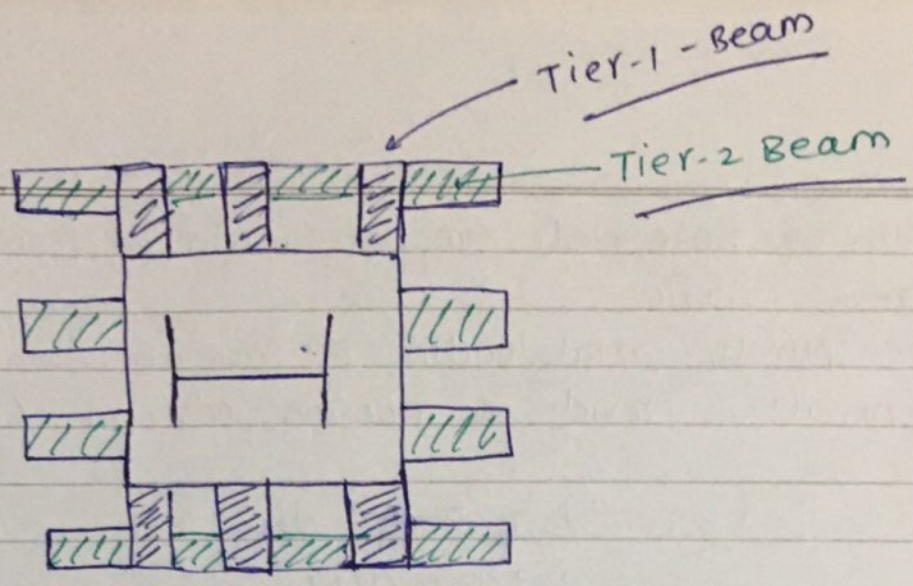
- To increase the area of load transfer, we provide base plate below the column. the arrangement used to transfer the column load to ground is called column Base and generally 3 types of column Bases are used.
  - i) slab base
  - ii) Gusseted base
  - iii) Grillage foundation.

① Slab Base



③ Grillage Foundation





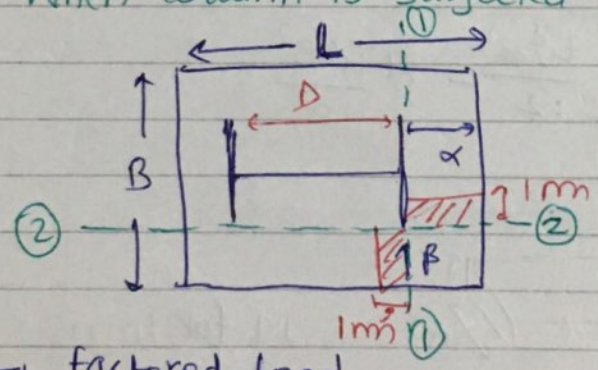
*

→ Slab base is generally used for axial loading
Gusseted base is used for Axial loading + moment
Grillage foundation is used when supporting soil is very weak and load is to be transfered @ a shallow depth
it is also used for temporary construction.

NOTE The beams in Grillage foundation shall be designed for BM + SF + web crippling.

Slab Base

Case (i) When column is subjected to pure Axial loading.



Base pressure $= \frac{P_u}{BL} = w$

$P_u \rightarrow$ factored load

Generally for Axial loading Max. comp. stress in Concrete can be taken as $0.45 f_{ck}$
We can consider length Based on depth of column and nominal angle size,

Width of Base plate can be calculated from permissible Stress criteria

- for purely axial loading in case of slab base we generally provide a noming angle of 60x60x8 mm

$$L_{min} = D + 2\times60 + 2\times5$$
$$= D + 130 \text{ mm}.$$

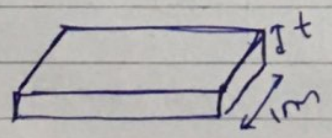
$$\frac{P_u}{LB} < 0.45 f_{ck}$$

The slab base will bend about sec^n ①-① & sec^n ②-②.
Taking into account the poisson's effect
the design moment $M = M_{11} - \mu M_{22}$

$$M_{11} = w \cdot \alpha \cdot 1 \cdot \frac{\alpha}{2} = \frac{w\alpha^2}{2}$$

$$M_{22} = w\beta \cdot 1 \cdot \frac{\beta}{2} = \frac{w\beta^2}{2}$$

$$\Rightarrow M = \frac{w\alpha^2}{2} - \mu\frac{w\beta^2}{2}$$



M for 1mm strip =
$$= 1.2\frac{f_y}{\gamma_{m0}} \times \frac{1\times t^2}{6}$$

$$\Rightarrow \frac{1.2 f_y}{\gamma_{m0}} \times \frac{t^2}{6} > \frac{w\alpha^2}{2} - \mu\frac{w\beta^2}{2}$$

$$t \geq \sqrt{\frac{2.5w(\alpha^2 - \mu\beta^2)}{f_y/\gamma_{m0}}}$$

~~$\mu = 0.3$ for elastic zone.~~

The above expression is Valid for uniform Base pressure

As per IS code

$$t \geq \sqrt{\frac{2.5\, w\, (\alpha^2 - 0.3\beta^2)}{(f_y/\gamma_{m0})}}$$

also $t \geq t_f$

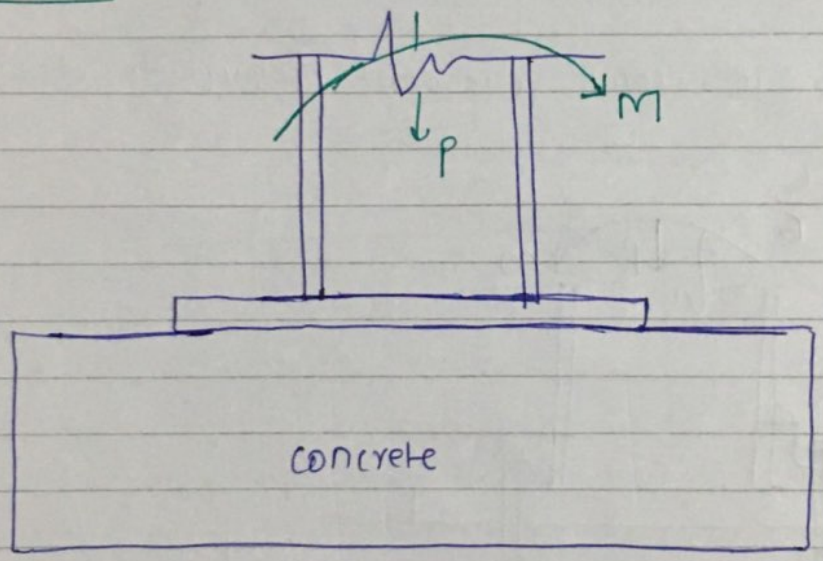
where α = larger overhang

β = shorter overhang.

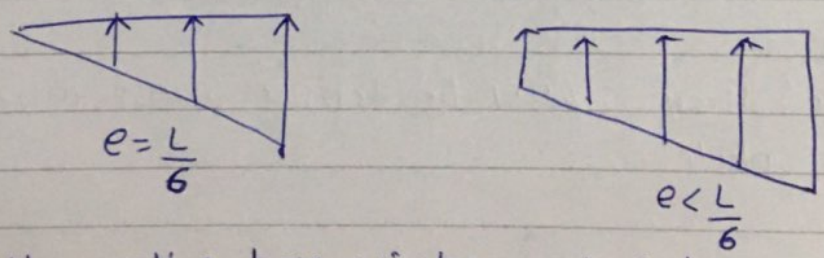
When base pressure is non-uniform, calculate M_{11} and M_{22}

$$M = M_{11} - \mu M_{22}$$

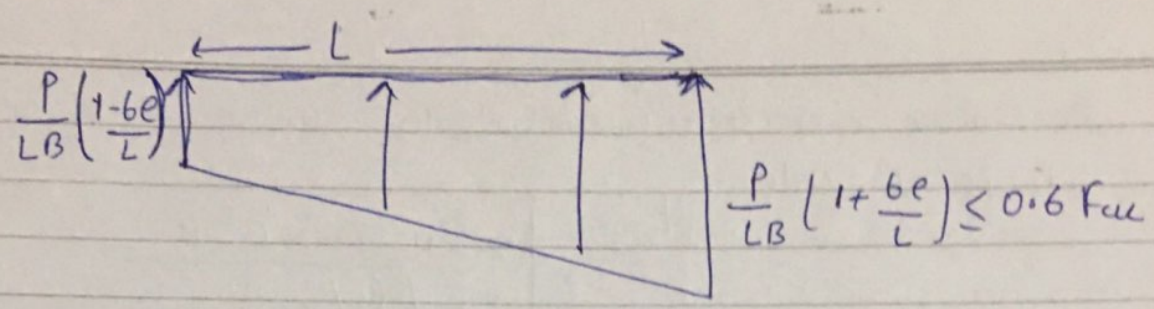
Case II When Axial load + moment in the column



a) When $e \leq \frac{L}{6}$



the entire base will be subjected to comp. stress.



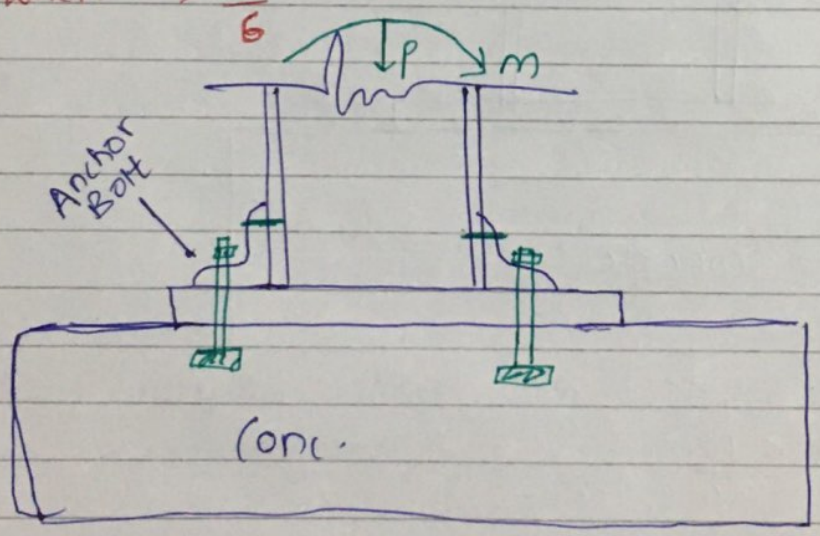
for safety in bearing $\frac{P}{LB}\left(1+\frac{e}{L}\right) \leq$

from above eq^n we can calculate B
consider $L \geq 6e$

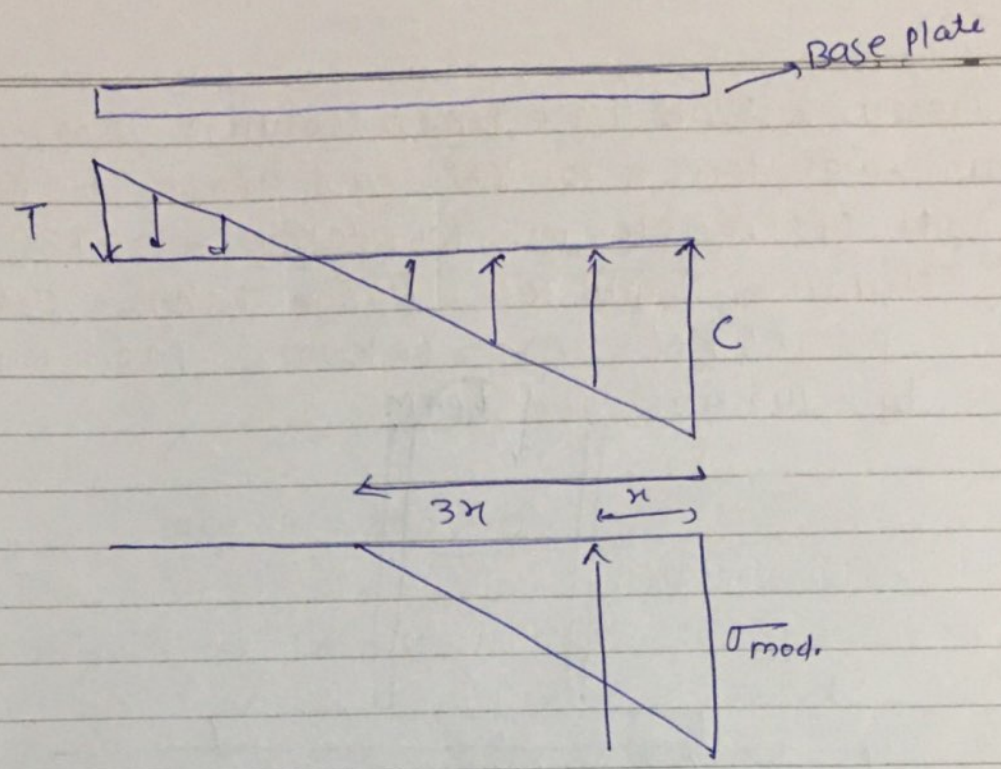
As per IS code when Base pressure is varying linearly, max. permissible Base pressure $= 0.6 f_{ck}$

As the Base is in comp we don't require to design Anchor bolt.

b) When $e > \frac{L}{6}$

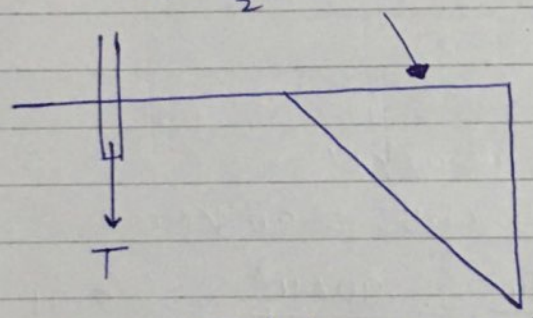


in this case there will be tensile stress in some part of base plate



$$\Rightarrow \frac{1}{2} \cdot \sigma_{mod.} \, 3x \cdot B = P$$

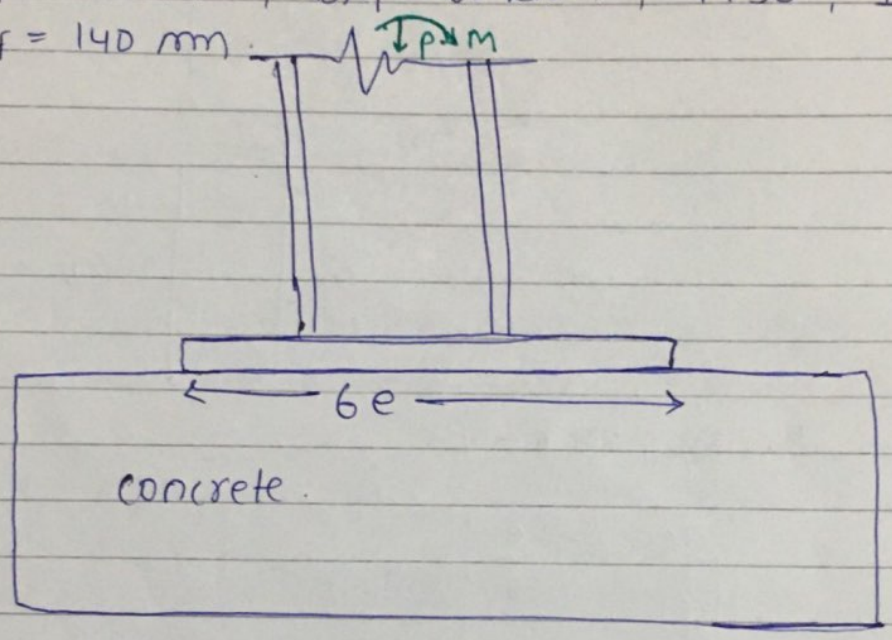
$$x = \frac{L}{2} - e$$



In this case we need to also consider the tensile force of the anchor bolt and hence total force on the base will be force of the modified base pressure diagram − Tensile force on the bolt. However the x value of the base pressure diagram will change due to the tensile force.
The exact base pressure diagram can be obtained by Trial & error procedure.

Que: Design a slab base for a column carrying an Axial force = 700 KN and BM = 60 KN.m
Grade of concrete in foundation → M30
column → ISMB 300, flange width = 140 mm

P = 700 KN, BM = 60 KNm, M30, ISMB 300
b_f = 140 mm.

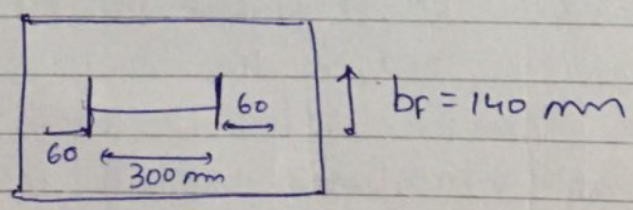


$P_u = 1.5 \times 700 = 1050$ KN

$M_u = 60$ KNm. $\times 1.5 = 90$ KNm

$$e = \frac{M_u}{P_u} = \frac{90 \times 10^3}{1050} = 85.71 \text{ mm.}$$

length of base plate $L \geq 6e \geq 6 \times 85.71$
≥ 514.28 mm.



$L_{min} = 300 + 2 \times 60 + 2 \times 5 = 430$ mm

Assume $L = \underline{600 \text{ mm}}$.

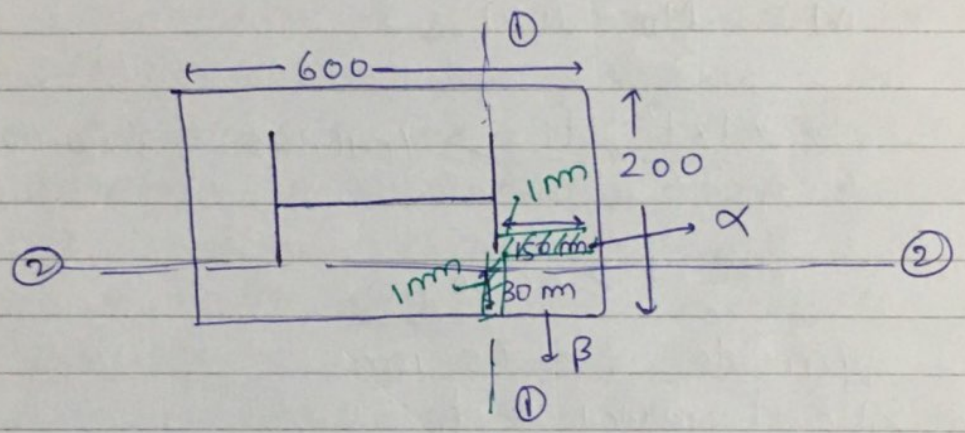
$$\Rightarrow \frac{P_u}{LB}\left(1+\frac{6e}{L}\right) \leq 0.6\, f_{ck}$$

$$\frac{1050\times10^3}{600\times B}\left(1+\frac{6\times85.71}{600}\right) \leq 0.6\times30$$

$$B \geq 180.5 \text{ mm}$$

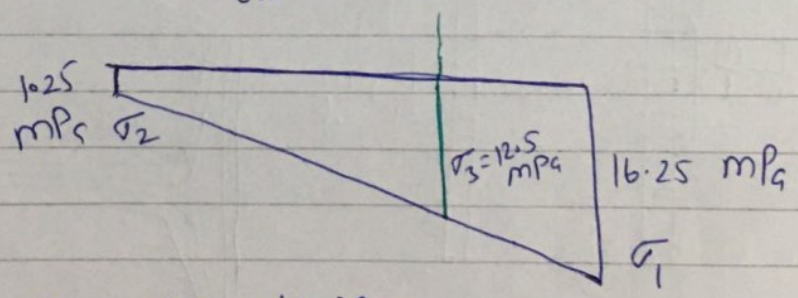
Take $B = 200$ mm.

Dimensions of base plate $L\times B = 600\times200$ mm.



$$\sigma_{max.} = \frac{P_u}{LB}\left(1+\frac{6e}{L}\right) = 16.25 \text{ mPa}$$

$$\sigma_{min} = \frac{P_u}{LB}\left(1-\frac{6e}{L}\right) = 1.25 \text{ mPa}$$

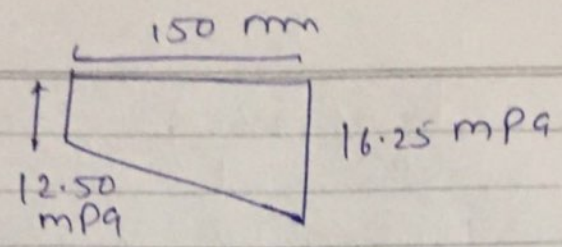


$$M = M_{11} - \mu M_{22}$$

$$\sigma_3 = 1.25 + \frac{(16.25-1.25)}{600}\times 450$$

$$\sigma_3 = 12.5 \text{ mPa}$$

$M_{11} =$



$$M_{11} = 0.5\,(16.25 + 12.50) \times 150 \times \left(\frac{150}{3}\left(\frac{2\times16.25 + 12.50}{16.25 + 12.50}\right)\right)$$

$$M_{11} = 0.16875 \text{ KNm.}$$

$$M_{22} = 12.5 \times 30 \times \frac{30}{2} = 0.005625 \text{ KNm.}$$

$$M = M_{11} - \mu M_{22}$$

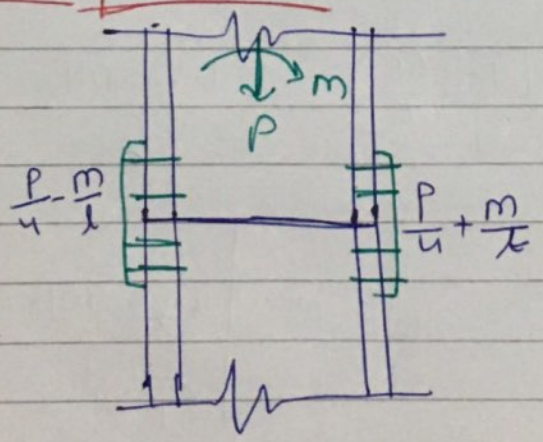
$$1.2 \times \frac{1 \times t^2}{6} \times \frac{f_y}{\gamma_{mo}} \geq (0.16875 - 0.3 \times 0.005625) \times 10^6$$

$$\Rightarrow t \geq 60.60 \text{ mm.}$$

provide $t = 65$ mm.

Also t provided $> t_f$

Column splices



① We need to connect 2- Column due to limited available length
② change in c/s due to decrease in load

The ideal location of column splice is @ a height of
* * $\frac{h}{4}$ from Top or bottom of column.
h → floor height

Case-(i) When the higher story column flange rest completly on the lower story column flange, Then we consider 2-cases —
A) column ends are milled (smooth)/ machine finish
B) column ends are not milled.

When column ends are milled, all the direct load will be assumed to be transfered to lower story column flange through Direct bearing. and in this case splice plates are designed simply to keep the 2-Column in position.

However in general we design splice plates for 50% of the applied load. i.e $\frac{P}{4}$ for each splice plate (Not a codal criteria)

For a column with P & M
design splice for → $\frac{M}{2}$, when milled (as per code)

Design splice for → $\frac{P}{4} + \frac{M}{2}$, $\frac{P}{4} - \frac{M}{2}$ } when milled (general practice)

Case(ii) When column ends are not milled we need to design the splice for entire axial load i.e for each splice load will be $P/2$

In case of column carrying moment, slice plate may be subjected to tensile force, when $\frac{P}{2} - \frac{M}{I} < 0$

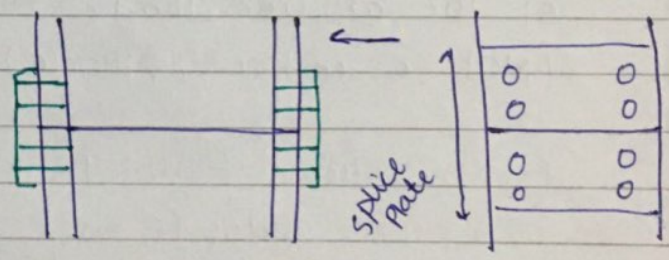
(Not milled)

In this case splice plate shall be designed as a Tension slice

Once the design force is calculated, width of splice plate shall be taken as the width of column flang and for calculation of thickness we can assume-

$$f_{cd} = \frac{f_y}{\gamma_{m0}}$$

because due to very small length and intermediate bolts, buckling of splice plate won't be considered.



$$\frac{\text{Design force in the splice}}{f_y/\gamma_{m0}} = b_f \times t$$

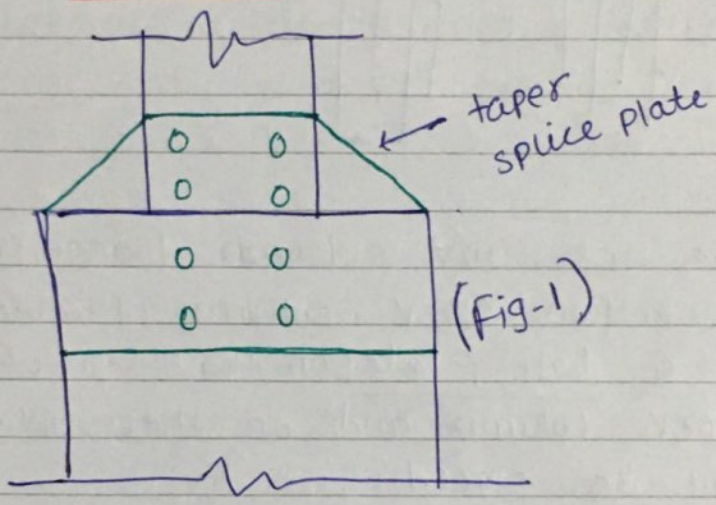
$t \rightarrow$ thickness of splice plate

- Connection shall be designed for the obtained force considering bolt in single shear.

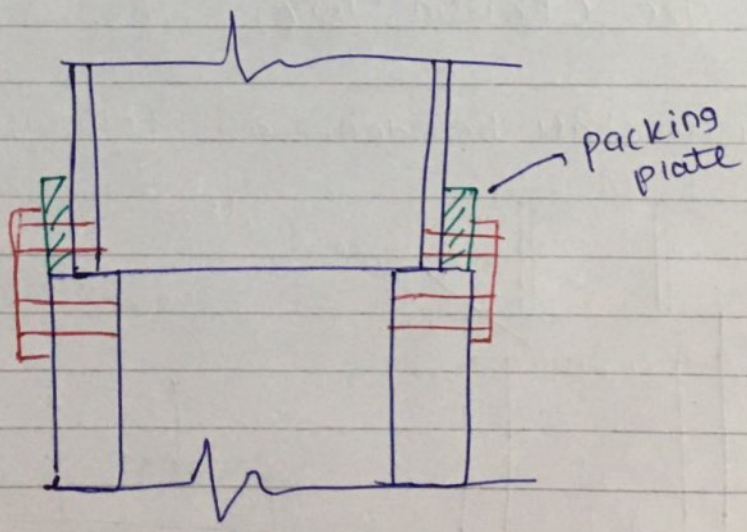
- if the higher story column flange < ~~bottom~~ lower story column flange, width of the splice plate = width of bottom story column flange.

However we need to taper the splice plate to match the width of the higher story column flange (fig. 1)

In case of diff^n column size, if full bearing is available on the bottom story column then we may need a packing plate for splicing (fig. 2)



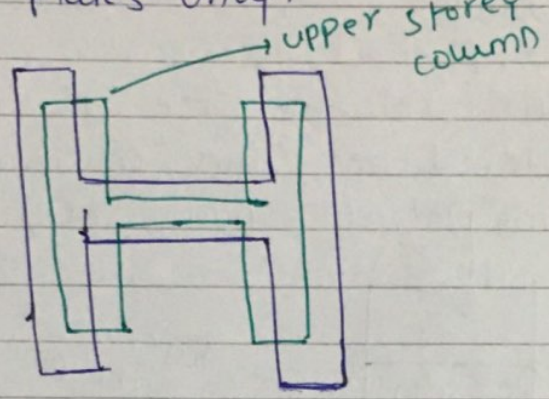
(Fig-1)



(fig. 2)

Case-(ii)

In case the upper story column bears partially on the lower story column, then we need to design each spliced plate for a force of P/2 i.e all loads are assumed to be transfered through splice plates only.

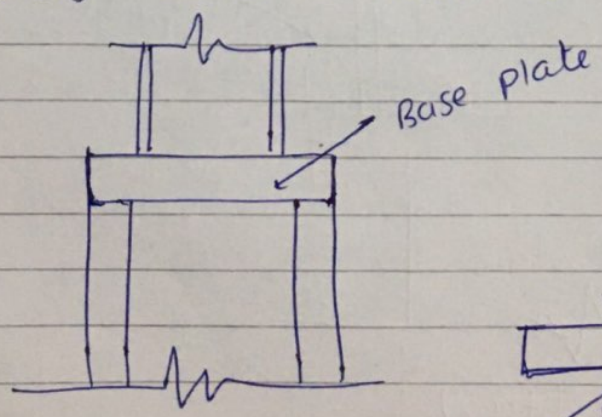


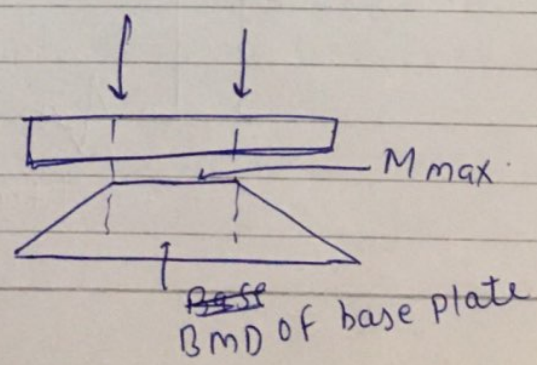
Case-(iii)

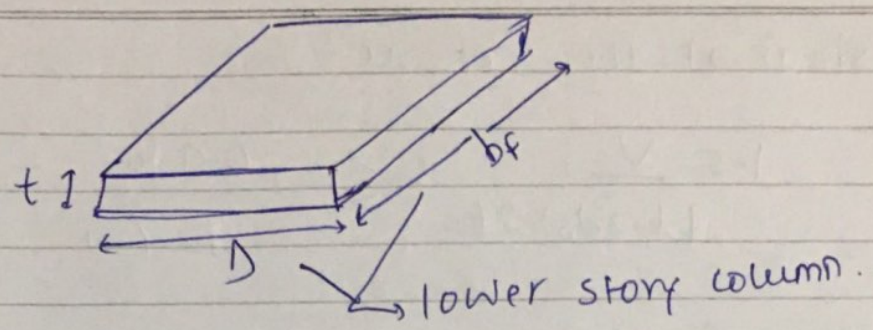
When the top story column flange is not resting on bottom story column flange we need to provide a base plate on the top of the bottom story column and on these plates the column shall be placed.

length + width of base plate shall be same as column size (Bottom story)

The Design shall be done as follows :-





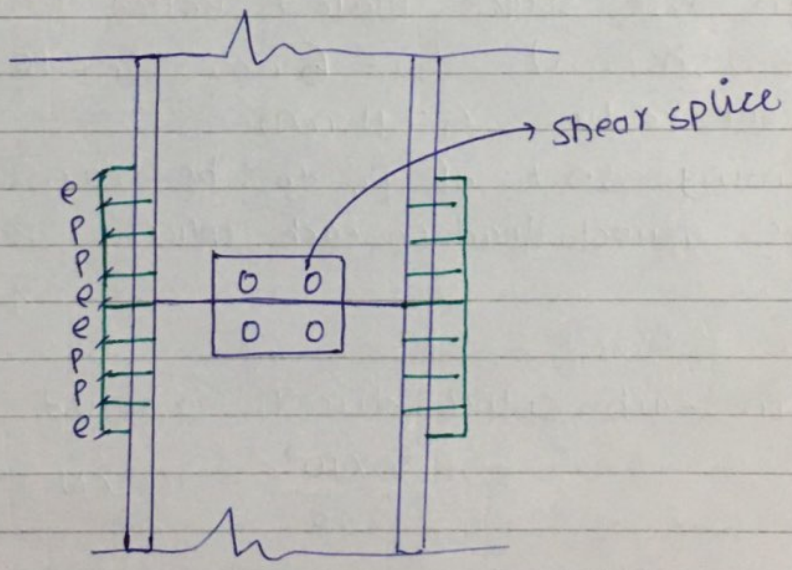


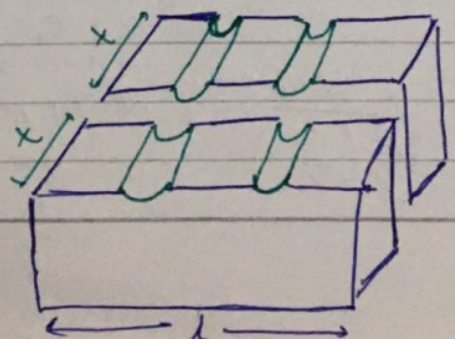
$$M_{max} \leq \frac{f_y}{\gamma_{mo}} \frac{b_f t^2}{6}$$

NOTE

It is generally assumed that axial load is transfered by flange only and the lateral load / shear is taken care by web.

If the column is also experiencing shear we need to provide shear splice in the web / web splice.





Shear at the net sec^n

$$1.5 \frac{V}{(l-2d_o)\,2t} \leq \frac{0.9 f_u}{\sqrt{3}\,\gamma_{m1}}$$

Shear @ the gross sec^n

$$\frac{1.5\,V}{2t \cdot l} \leq \frac{f_y}{\sqrt{3}\,\gamma_{m0}}$$

1.5 factor is to consider the max. shear stress

Que. A column sec^n ISMB 300 is to be sliced consider column ends to be machined for complete bearing, grade Fe 410, $P_u = 450$ kN $M_u = 30$ kNm, $V_u = 90$ kN

Design the splice plate & Bolted connection ($b_f = 250$ mm) $t_w = 6$ mm, $t_f = 10.6$ mm. root radius $r_1 = 11$ mm.

Assuming 50% of P_u to be transfered through splices, Direct load on each splice $= \frac{P_u}{4} = \frac{450}{4} = 112.5$ kN

load on each splice due to applied moment

$$= \frac{30 \times 10^3}{300} = 100 \text{ kN}$$

so force on splices $= 112.5 + 100 = 212.5$ kN (c)

$112.5 - 100 = 12.5$ kN (c)

thickness of splice plate $\frac{212.5 \times 10^3}{b_f \left(\frac{f_y}{\gamma_{m0}}\right)} = 3.74$ mm

consider (provide) thickness of splice plate = 6 mm.

Assume M20, 4.6

$V_{dsp} = 45.27$ KN

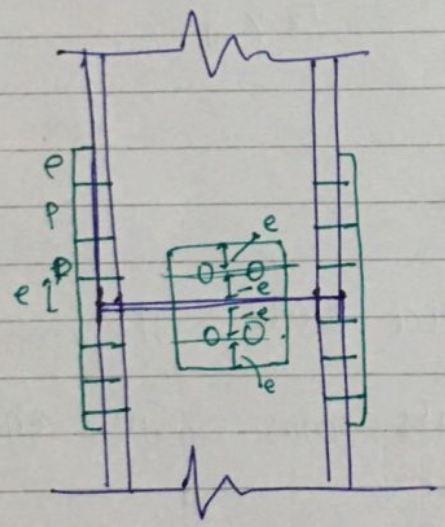
$$V_{dpb} = \frac{2.5 \times K_b \times d \times t \times f_u}{\gamma_{mb}} = \frac{2.5 \times 0.5 \times 20 \times 6 \times 410}{1.25}$$

$V_{dpb} = 49.2$ KN

$V_{db} = 45.27$ KN

$$\text{No. of bolts} = \frac{212.5}{45.27} = 4.69$$

6 bolts.



length of splice plate $= 2 \times (2e + 2p)$

$= 2 \times (2 \times 1.5 d_o + 2 \times 2.5 d)$

$= 332$ mm

provide $l = 332$ mm

~~Shear splice~~

Shear splice design.

$$V_{dsb} = \frac{45.27}{0.78} \times 1.78 = 103.3 \text{ KN}$$

bolts are in double shear

$$V_{dpp} = \frac{2.5\, K_b \times d \cdot t\, f_u}{\gamma_{mb}} = \frac{2.5 \times 0.5 \times 20 \times 6 \times 410}{1.25}$$

$$= 49.2 \text{ kN}$$

$V_{ab} = 49.2$ kN.

$\Rightarrow$ No. of bolds req. $= \frac{90}{49.2} = 1.83$

provide 2 bolts.

Net sec^n

$$\frac{1.5\, V_u}{(l_s - 2d_o)\, 2t} \leq \frac{0.9\, f_u}{\sqrt{3}\, \gamma_{m1}}$$

$d_o \rightarrow 22$, $t \rightarrow 6$

$\Rightarrow l_s \geq 110$ mm

provide width of splice plate = 110 mm.

To accomodate 2 bolts min. width req. is $p+2e$

$= 116$ mm.

hence provide $\rightarrow$ 120 mm

length of splice plate $= 4e$

$= 132$ mm

Provide = 140 mm.

BEAMS

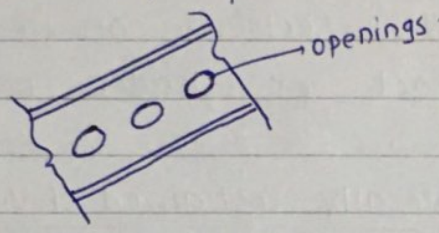
Joists closly spaced beams to support roof

Girder A large beam supporting a no. of Joists.

purlin Beams provided over truss to support roof.

Girt Horizontal beams spanning the wall column of industrial buildings to support wall coverage.

Castellated Beams Beams with opening in b/w, Higher MOI low Axial load capacity.

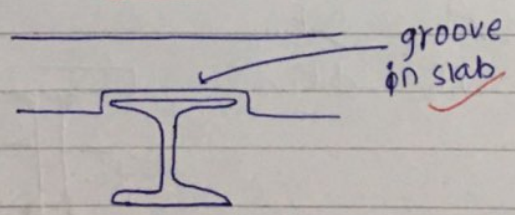


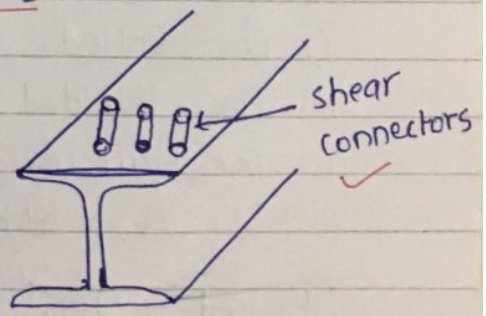
Beam can have lateral buckling of comp. flange on account of bending comp.

Lateral buckling of comp. flange causes instability in beam also leads to torsional moment. This buckling is also called as lateral torsional buckling

To avoid lateral torsional buckling we need to support comp. flange of beam and it can be done by providing cross Beams, making the comp. flange heavier or by inserting the comp. flange into concrete slab also we can provide shear connectors in the comp. flange.

This type of beam which are laterally stable can be called as laterally restrained beams.



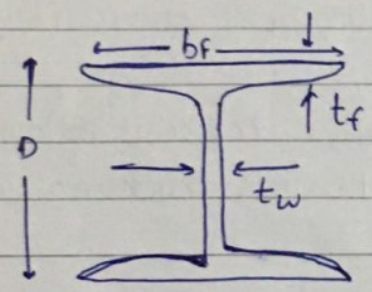


As per IS code full lateral restrain to comp. flange may be assumed to exist. if the frictional or other positive restrain (sagging moment) of a floor connection to the comp. flange of the member is capable of resisting lateral force $\geq$ 2.5% of the Max. force in the comp. flange of the beam.

Classification of section

sections can be plastic, compact, semi-compact or slender (as discussed in previous chapter)

Bending strength of laterally restrained beams



Design shear capacity $= \frac{f_y}{\sqrt{3}\gamma_{m_0}} \cdot A_v$

$A_v \rightarrow$ shear Area

Before finding out the Bending strength, we need to investigate weather the sec^n is under High shear case or low shear case.

Design strength of Web in shear

shear is always assumed to resisted by web only

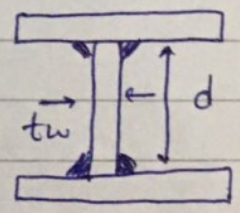
Design shear capacity $V_d = \frac{f_y}{\sqrt{3}\gamma_{m_0}} \cdot A_v$

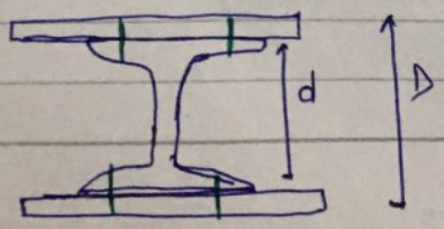
i) for Rolled sec^n $A_v = D t_w$

ii) for Welded sec^n $A_v = d t_w$

iii) for Built up sec^n

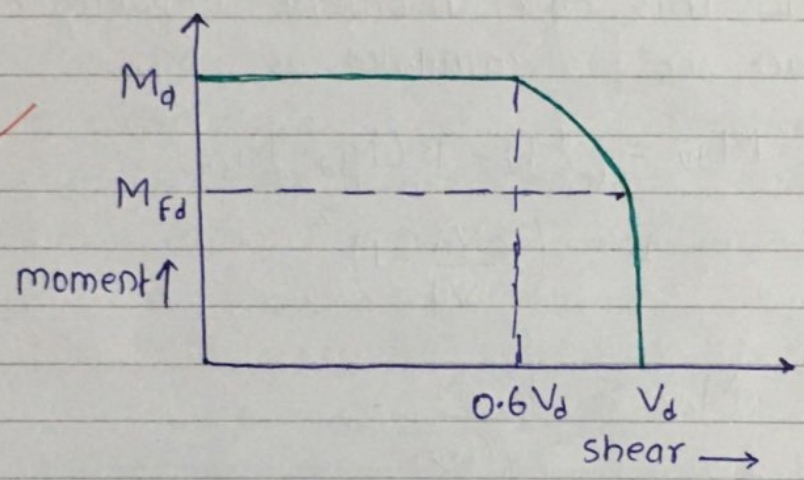
$A_v = d t_w$





- For safety in shear, Max. factored SF i.e $V_{u,max} \leq V_d$
- If Max. applied factored SF in Beam is $< 0.6\,V_d$ then it is called low shear case. and if $V_{u,max} \geq 0.6\,V_d$ then high shear case.

Moment capacity of sec^n is independent of shear for low shear case, However under high shear case moment capacity of sec^n gets reduced.* (web is not considered)
The interaction diagram b/w moment capacity and shear is shown below.



$M_d \rightarrow$ Design moment capacity of sec^n (F+W)
$M_{Fd} \rightarrow$ moment capacity of flange only
$V_d \rightarrow$ Design shear capacity

Moment capacity of sec^n

Case-(i) low shear case $\{V_{u,max} < 0.6\,V_d\}$

$$M_d = \beta \frac{f_y}{\gamma_{m0}} \cdot Z_p$$

$\beta = 1$ for plastic & compact sec^n, Because these sec^n can go upto M_p

$\beta = \frac{Z_e}{Z_p}$; for semi-compact sec^n, Because this sec^n can develop M_y only

Also $M_d \leq 1.2 \frac{f_y}{\gamma_{mo}} Z_e$ ⟶ for S.S Beams — ①

$M_d \leq 1.5 \frac{f_y}{\gamma_{mo}} \cdot Z_e$ ⟶ for Cantilever Beams — ②

① & ② checks are provided to prevent the unset of plasticity under working load.

Case-(ii) High shear case $\{ V_{u,max} \geq 0.6 V_d \}$

In this case moment capacity of secⁿ will be less than full moment capacity because the complete Web can't be taken to be eff. in resisting the moment. in this case moment capacity of plastic and compact secⁿ is calculated as M_{dv}

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

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

$$M_{dv} \leq \frac{1.2 f_y}{\gamma_{mo}} \cdot Z_e$$

$M_d \rightarrow$ plastic design moment of secⁿ disregarding high shear effect i.e for low shear case

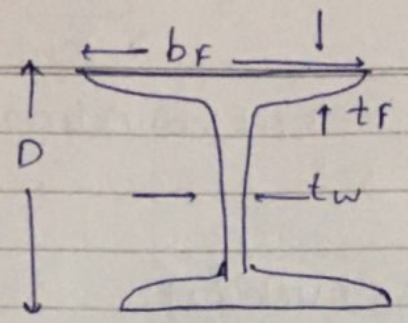
$M_d = \frac{f_y}{\gamma_{mo}} Z_p$ for plastic and compact secⁿ

M_{fd} = plastic design strength of Area of the c/s excluding the shear Area.

$V_u \rightarrow$ applied SF

$V_d \rightarrow$ shear strength (Design) as governed by Web Yielding or Web buckling.

* Shear में $\sqrt{3}$ नहीं



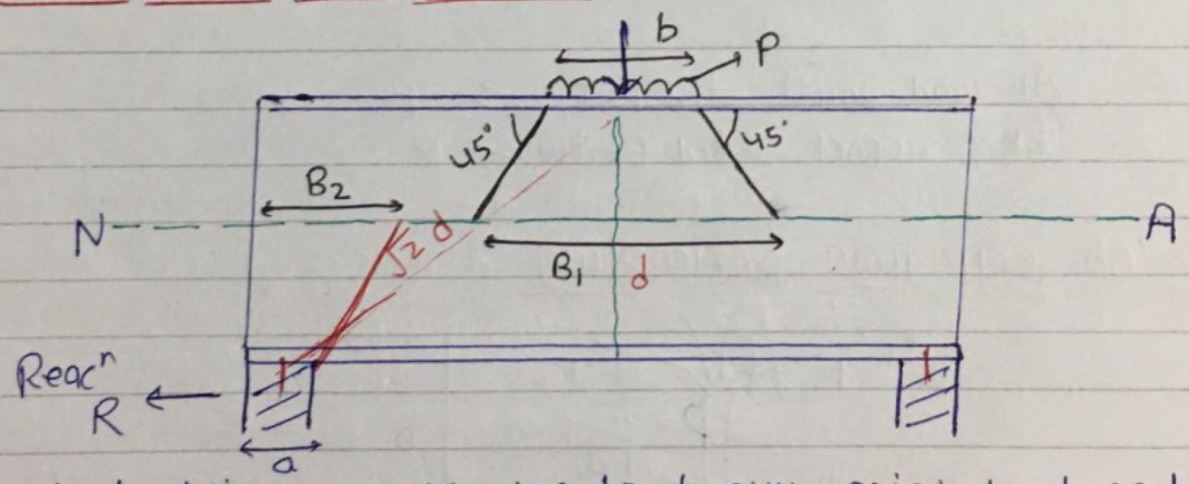
$$M_{fd} = M_d - \frac{f_y}{\gamma_{mo}} \underbrace{\frac{t_w D^2}{4}}_{Z_p}$$

NOTE For semi-compact sec^n even in case of high shear, M_d will be taken as $\frac{f_y}{\gamma_{mo}} \cdot Z_e$

Check for deflection

- Calculation of max. deflection can be done using service load, because deflection is a serviceability criteria
- The max. permissible limit for deflection is given in IS code, however under general cond. We'll assume max. permissible deflection to be $\frac{\text{span}}{300}$ *

Check for web buckling



Web buckling occurs due to heavy point load and diagonal comp. in the web.

A) For low shear case

* (i) When $\frac{d_1}{t_w} \leq 67\varepsilon$ *, no need to check for web buckling.

$$\frac{\sqrt{2}\,d}{2} = l_{eff} = \frac{d}{\sqrt{2}}$$

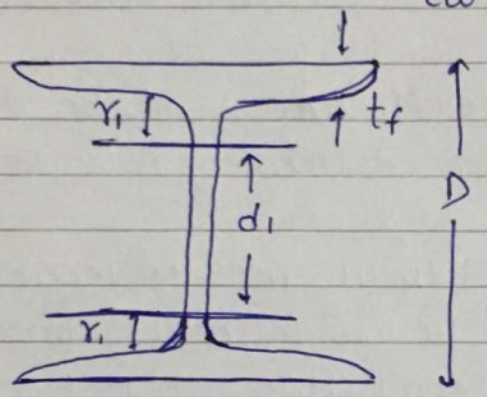
$$\varepsilon = \sqrt{\frac{250}{f_y}}$$

(ii) $\frac{d_1}{t_w} > 67\varepsilon$, check for Web buckling.

B) For High shear case

Always check for Web buckling.

To check for Web buckling web shall be treated as a column of $\lambda = 2.5\frac{d_1}{t_w}$



$\Rightarrow d_1 = D - 2(t_f + r_1)$

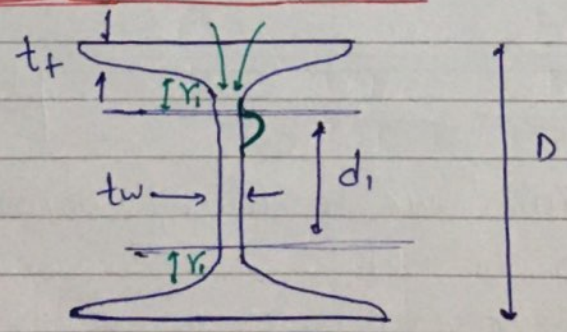
$r_1 \rightarrow$ root radius.

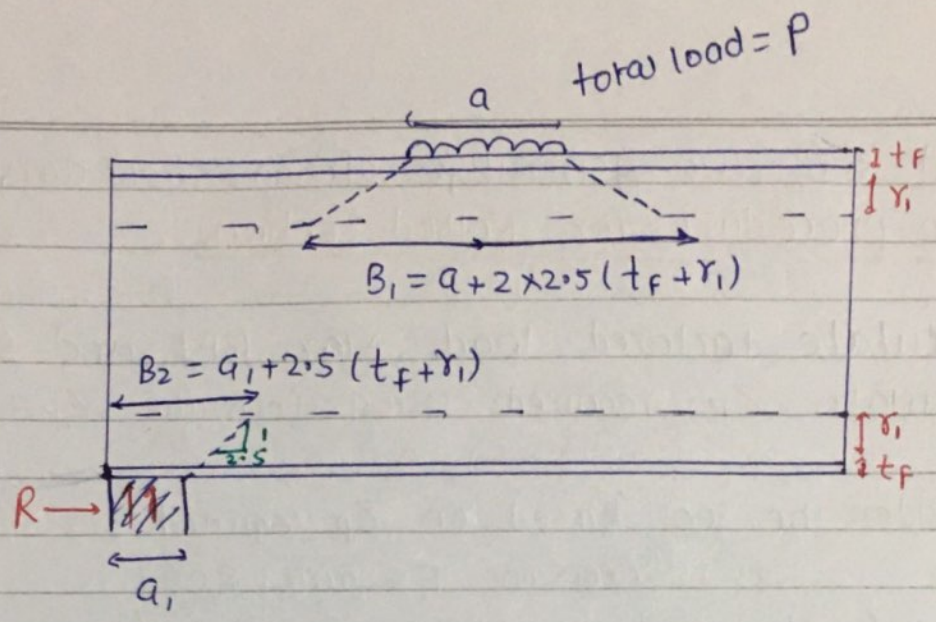
f_{cd} shall be calculated based on flexural buckling using the λ obtained. and considering buckling class C

At load point $f_{cd}\,\beta_1\,t_w \geq P$

At support $f_{cd}\,\beta_2\,t_w \geq R$

Web crippling / crimpling





At load point $\frac{P}{B_1 t_w} \leq \frac{f_y}{\gamma_{m0}}$

At support $\frac{R}{B_2 t_w} \leq \frac{f_y}{\gamma_{m0}}$

- Beams may be subjected to web crippling due to heavy point load
- To check for web crippling we need to ensure that the bearing stress at the root of fillet $< \frac{f_r}{\gamma_{m0}}$

NOTE Generally if the ~~web~~ Beam is safe in web crippling then it will be safe in web buckling.

~~Check~~ Steps for Beam Design

1) Shear
2) Moment
3) Deflection
4) Buckling
5) Crippling
6) local Buckling of W and F (Like plastic, compact sec etc)

4, 5, 6 → are called secondary checks
1, 2, 3 → are called primary checks.

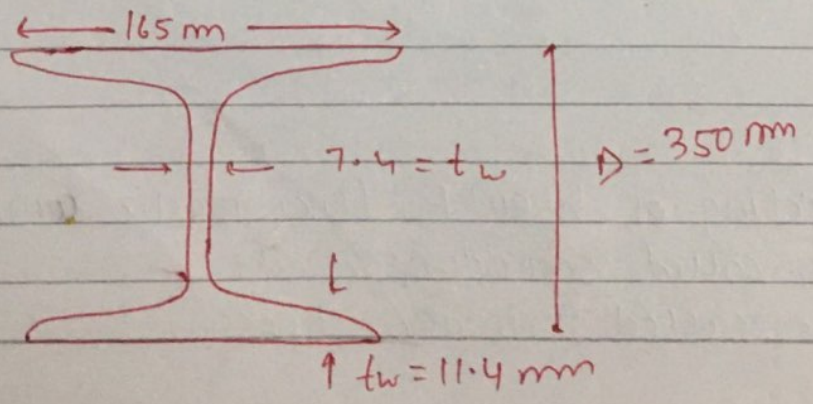
Rolled secⁿ are generally safe in secondary failure.

Design procedure for Rolled section

1. calculate factored load, Max BM and SF
2. calculate Z_p required using formula $Z_p = \frac{M_u}{f_y/\gamma_{mo}}$
3. consider the secⁿ based on Z_p required.
   Given in IS 800 : 2007
4. classify the secⁿ as plastic, compact / semi-compact
5. Calculate Design shear strength (V_d)
6. ensure $V_{u,max} \leq V_d$
   check for low / high shear case
7. Find out moment capacity of secⁿ based on classification of secⁿ & low / High shear case
8. check for Deflection.
9. check for Web buckling in case of $\frac{d}{t_w} > 67\varepsilon$ or high shear case.
10. check for Web crippling.

Que. Determine the Bending strength of ISLB 350 considering the beam to be laterally supported. Assume low shear case and take Fe 410

properties :- $D = 350$ mm $b_f = 165$ mm.
$t_f = 11.4$ mm $t_w = 7.4$ mm
$Z_e = 751.9\ cm^3$ $Z_p = 851.11\ cm^3$



Fe 410 , $f_y = 250$ mpa . $\varepsilon = \sqrt{\frac{250}{f_y}} = 1$

$$M_d = \frac{\beta f_y}{\gamma_{m0}} \cdot Z_p$$

$$\frac{b_f}{2} = \frac{165}{2} = 82.5 \text{ mm.}$$

$$\frac{b}{t_f} = \frac{82.5}{11.4} = 7.23 < 9.4\,\varepsilon \quad (\varepsilon \to 1)$$

Section is plastic secⁿ

$$M_d = \beta \frac{f_y}{\gamma_{m0}} \cdot Z_p = 1 \times \frac{250}{1.1} \times 851.11 \times 10^3$$

$$M_d = 193.43 \text{ kNm}$$

However, $M_d \not> \frac{1.2 f_y}{\gamma_{m0}} \times Z_e = \frac{1.2 \times 250}{1.1} \times 751.9 \times 10^3$

$$M_d = 193.43 \text{ kN}$$

Que. A simple supported beam has eff. span of 4m. it carries a UDL of 10 kN/m including self wt. Design appropriate secⁿ using Fe 410, available secⁿ are as follows.

ISLB 200	ISMB 150	ISMB 200
$D = 200$ mm	$D = 150$ mm	$D = 200$ m
$b_f = 100$ m	$b_f = 80$ m	$b_f = 100$ m
$t_f = 7.3$ mm	$t_f = 7.6$ m	$t_f = 7.8$ m
$t_w = 5.4$ mm	$t_w = 4.8$ m	$t_w = 5.7$ mm
$Z_e = 169.7$ cm³	$Z_e = 96.9$ cm³	$Z_e = 223.5$ cm³
$Z_p = 184.34$ cm³	$Z_p = 110.48$ cm³	$Z_p = 253.86$ cm³
$r_1 = 9.5$ mm	$r_1 = 9$ m	$r_1 = 11$ mm
Shape factor = 1.086	1.140	1.135

factored load $= 1.5 \times 10 = 15$ kN/m

$$BM_{max.} = \frac{wl^2}{8} = \frac{15 \times 4^2}{8} = 30 \text{ kNm}$$

$$V_{u.max} = \frac{wl}{2} = \frac{15 \times 4}{2} = 30 \text{ kN}$$

$$Z_{p\,req.} = \frac{M_u}{f_y/\gamma_{m0}} = \frac{30 \times 10^6}{250/1.1} = 132 \text{ cm}^3$$

Consider ISLB 200

$$\frac{b_f}{2t_f} = \frac{100}{2 \times 7.3} = 6.8 < 9.4\varepsilon$$

$$\frac{d_1}{t_w} = \frac{D - 2t_f - 2r_1}{t_w} = \frac{200 - 2(7.3 + 9.5)}{5.4}$$

$$= 30.8 < 84\varepsilon$$

The sec^n shall be considered as a plastic sec^n
Consider sec^n is safe in bending.

$$V_d = \frac{f_y}{\sqrt{3}\gamma_m} \cdot A_v = \frac{250}{\sqrt{3} \times 1.1} \cdot 200 \times 5.4 = 141.71 \text{ kN}$$

$> V_{u,max}$

Safe in shear

$$0.6 V_d = 85.05 \text{ kN}$$

$$V_{u.max} < 0.6 V_d$$

~~Saf~~ low shear case

Check for Deflection

$$\delta_{max} = \frac{5}{384} \frac{wl^4}{EI} = \frac{5}{384} \times \frac{10 \times (4 \times 10^3)^3}{2 \times 10^5 \times 169.7 \times 10^3} \times \frac{200}{2}$$

$$= 9.821 \text{ mm.}$$

NOTE → d_1 → Height of web having thickness t_w

$$\frac{l}{300} = \frac{4\times10^3}{300} = 13.33 \text{ mm} < 9.82 \text{ mm}$$

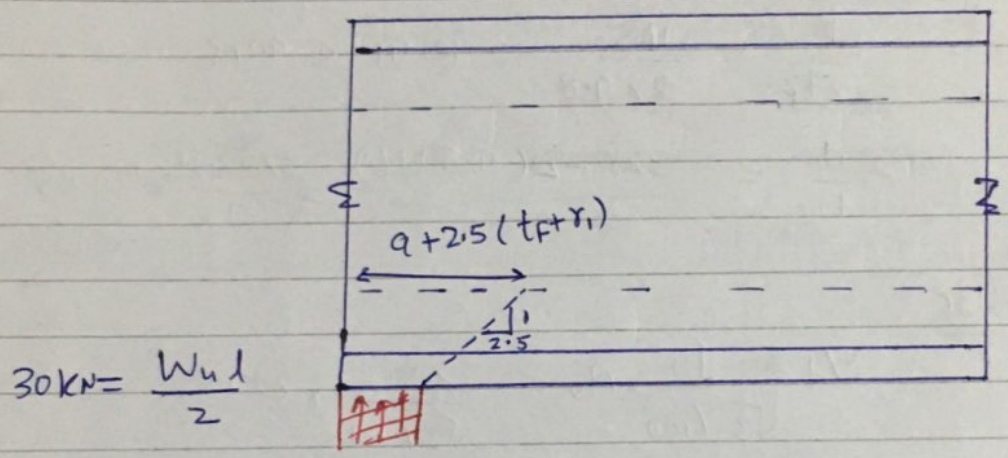
O.K

Check for web buckling

$$\frac{d_1}{t_w} = \frac{166.4}{5.4} = 30.81 < 67\varepsilon$$

O.K

Check for web crippling



assume support width = 200 mm. (a)

$$= \frac{30\times10^3}{[a+2.5(t_f+r_1)]\times t_w} = 22.96 \text{ MPa} \not> \frac{f_y}{\gamma_{m0}}$$

O.K

Que: Design a laterally supported beam of $l_{eff} = 6$ m. for following Data, ~~set~~ Fe 410, $M_u = 150$ kNm $V_u = 210$ kN Also check for web buckling and no need to check for crippling & deflection.

Available sec^n

~~ISLB~~ ISLB 325

$D = 325$ mm. $b_f = 165$ mm.

$t_f = 9.8$, $t_w = 7$ mm

$r_1 = 16$ mm, $Z_e = 607.7$ cm^3

$Z_p = 687.76$ cm^3

ISLB 350

$D = 350$ mm $b_f = 165$ mm

$t_f = 11.4$ mm $t_w = 7.4$ m

$r_1 = 16$ mm, $Z_e = 751.9$ cm^3

$Z_p = 851.1$ cm^3

λ	100	110	120
f_{cd}	107	94.6	83.7

$$Z_{p,req.} = \frac{150 \times 10^6}{(250/1.1)} = \frac{M_u}{f_y/\gamma_{m0}}$$

$$= 660 \text{ cm}^3$$

Assuming above sec^n to be plastic or compact consider ISLB 325

$$\frac{b_f}{2t_f} = \frac{165}{2 \times 9.8} = 8.41 < 9.4\varepsilon$$

$$\frac{d_1}{t_w} = \frac{325 - 2(9.8+16)}{7} = \frac{273.4}{7} = 39.05 < 84\varepsilon.$$

Henc· plastic

$$V_d = \frac{f_y}{\sqrt{3}\,\gamma_{m0}} \times A_v = \frac{250}{\sqrt{3} \times 1.1} \times 2275$$

$$= 298.5 \text{ KN}$$

$$V_{u\,max} > 0.6\, V_d$$

$$210 > 0.6 \times 298.5$$

$$210 > 179.1 \text{ KN}$$

High shear case.

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

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

$$\beta = \left(\frac{2 \times 210}{298.5} - 1\right)^2 = 0.1656$$

$$M_d = \frac{f_y}{\gamma_{m0}} \times Z_p = \frac{250}{1.1} \times 687.76 = 156.3 \text{ kN}$$

$$M_{fd} = M_d - \frac{f_y}{\gamma_{m0}} \cdot \frac{t_w d^2}{4}$$

$$M_{fd} = 156.3 - \frac{250}{1.1} \times \frac{7 \times 325^2}{4}$$

$$M_{fd} = 114.29 \text{ kNm}$$

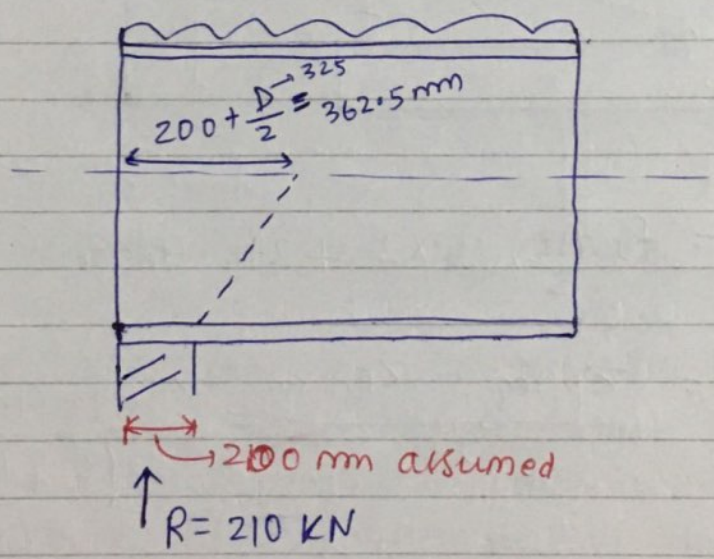
$$M_{dv} = 149.37 \text{ kN} \cong 150 \text{ kN}$$

O.K

Check for Buckling

$$\lambda = 2.5 \frac{d_1}{t_w} = 97.625$$

Consider $f_{cd} = 107$ MPa.

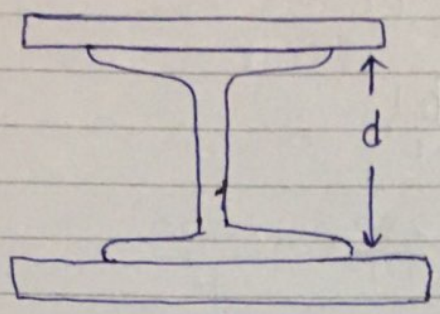


capacity $= 362.5 \times t_w \times 107$

$= 271.5 \text{ kN} > 210 \text{ kN}$

Safe in web buckling

Design procedure for built up Beams (Plated Beams)



plated Beams.

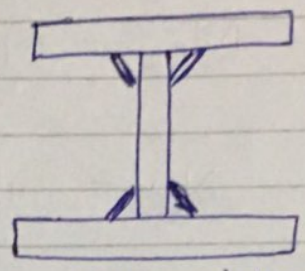
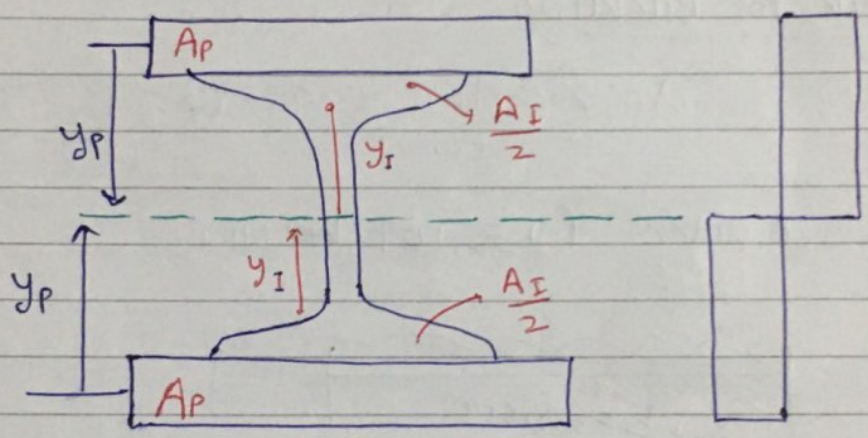


plate girder

$$A_p = \frac{Z_{p,req} - Z_{p,I\,sec^n}}{d}$$



$$\cancel{f_y} \cdot Z_p = \cancel{f_y}\,\frac{A_I}{2} \times y_I \times 2 + 2\cancel{f_y} \cdot A_p \times y_p$$

$$A_p = \frac{Z_{p,req} - Z_{p,I\,sec^n}}{2 \times y_p} = \frac{Z_{p,req} - Z_{p,I\,sec^n}}{2 \times \left(\frac{d}{2} + \frac{t_p}{2}\right)}$$

$$A_p = \frac{Z_{p,req} - Z_{p,I\,sec^n}}{d}$$

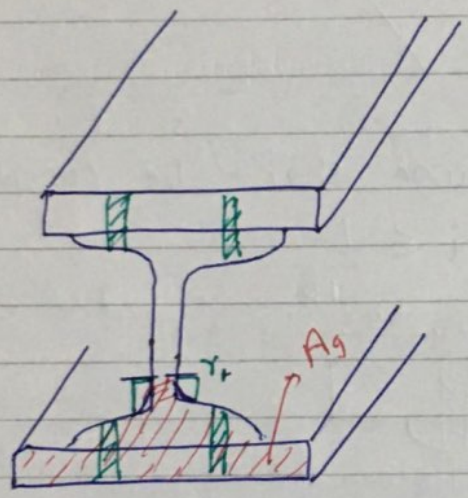
② consider the plate dim^n and classify the sec^n as plastic, compact or semi-compact

③ check for safety in shear & classify low/High shear case

④ Find out the moment capacity based on sec^n classification and ensure $m_u < M_{dv}$

⑤ check for Deflection, Web buckling, Web crippling.

Effect of Bolt holes



$$\frac{0.9 f_u}{\gamma_{m1}} A_n \geq \frac{f_y}{\gamma_{m0}} A_g$$

Effect of Bolt hole in Tension flange need not be considered in Z_p if $\frac{0.9 f_u}{\gamma_{m1}} A_n \geq \frac{f_y}{\gamma_{m0}} A_g$

i.e gross sec^n yielding governs the strength

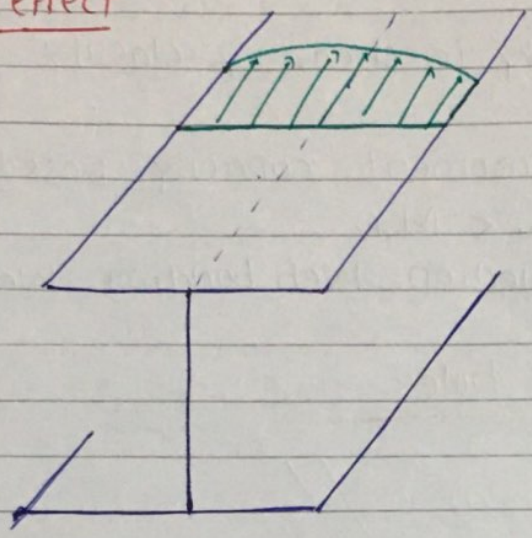
$A_n \rightarrow$ net Area

$A_g \rightarrow$ gross Area of Tension flange upto root of fillet

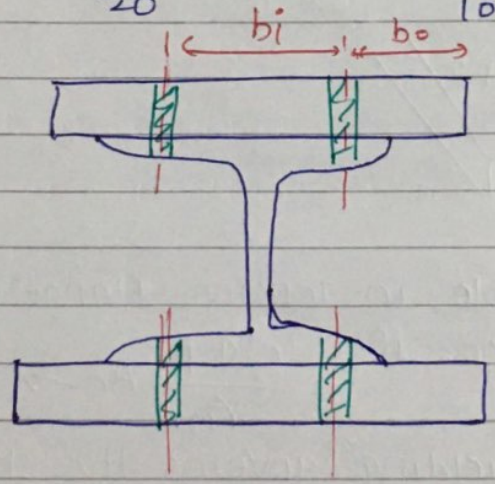
if the above criteria is not satisfied, the eff. flange Area in Tension (A_{EF}) will be taken from the following eq^n –

$$\frac{0.9 f_u}{\gamma_{m1}} A_n = \frac{f_y}{\gamma_{m0}} A_{eF}$$

Shear lag effect



shear lag effect of flange shan't be considered when $b_o \leq \frac{l_o}{20}$, $b_i \leq \frac{l_o}{10}$



$l_o \rightarrow$ Distance b/w points of OBM.

Que Design a steel Beam secⁿ having $l_{eff} = 6.65$ m. it is supported @ ends with end bearing of 150 mm. factored DL & LL are 63 kN/m. Total depth of Beam is restricted to 375 mm. Assume Beam to Beam laterally supported, secⁿ available is ISWB 350

ISWB 350

$D = 350\,mm$ $b_f = 200\,mm$.

$t_f = 11.4\,mm$ $t_w = 8\,mm$.

$r_1 = 12\,mm$, $Z_p = 995.49\,cm^3$, $Z_e = 887\,cm^3$

$W_u = 63\,kN/m$, $l = 6.65\,m$

$$M_u = \frac{W_u l^2}{8} = \frac{63 \times 6.65^2}{8} = 348.25\,kNm$$

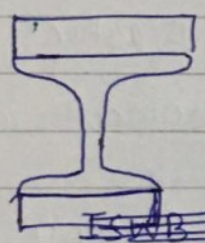
$$V_u = \frac{W_u l}{2} = \frac{63 \times 6.65}{2} = 209.475\,kN.$$

$$Z_{p.req} = \frac{M_u}{f_y/\gamma_{m_0}} = \frac{348.25 \times 10^3}{250/1.1} = 1532.3\,cm^3$$

$Z_{p.available} = 995.4$

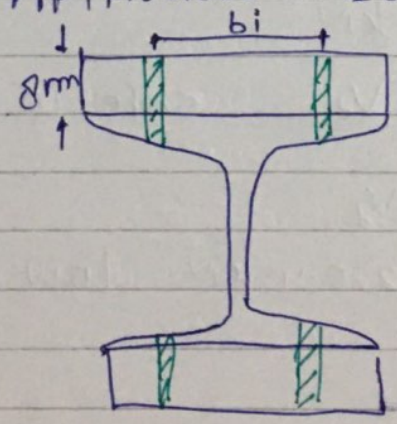
$$\text{Area of plate req} = \frac{Z_{p\,req} - Z_{p\,available}}{d}$$

$$= \frac{1532.3 - 995.4}{350 \times 10^{-1}} = 15.34\,cm^2$$



~~ISWB 350~~ consider plate of width 200mm and thickness 8 mm

$A_{p,provided} = 200 \times 8 = 1600\,mm^2 > 1534\,mm^2$



$$b_i \cong t_w + 2r_1 + 2\frac{d_o}{2}$$

$$= 8 + 2\times12 + 2\times\frac{22}{2}$$

$$b_i = 54 \text{ mm}.$$

$$b_o = \frac{200-54}{2} = 73 \text{ mm}.$$

$$t_f = 11.4 \text{ mm}.$$

$$\frac{b_i}{t_p} = 6.75 < 29.3\,\varepsilon \quad \Rightarrow \text{plastic}$$

$$\frac{b_o}{t_p} = \frac{73}{8} = 9.125 < 9.4\,\varepsilon \Rightarrow \text{plastic}$$

$$\frac{d_1}{t_w} = \frac{350-2(11.4+12)}{8} = 37.9 < 84\,\varepsilon.$$

plastic

section is plastic

hence adequate from bending moment capacity

$$V_d = \frac{f_y}{\sqrt{3}\gamma_{mo}} A_v = \frac{250}{\sqrt{3}\times1.1}\times350\times8 = 367.4 \text{ kN}$$

$$V_u = 209.47 \text{ kN}$$

$V_{u,max} < V_d$ safe in shear.

$$V_u < 0.6V_d$$

$209.47 < 220.4$ kN low shear case.

Check for bolt holes Area

Consider M20, 4.6, 2 no. of bolts c/s.

$A_{net} = (B - 2d_o)t$ $\quad t = t_p + t_F$

$= (200 - 2 \times 44) \times 19.4$ $\quad = 8 + 11.4$

$A_{net} = 3026.4$

$A_g = B \cdot t$

$= 200 \times (8 + 11.4)$

$A_g = 3880$

$$\frac{0.9 f_u}{\gamma_{m1}} A_{net} > \frac{f_y}{\gamma_{m0}} \times A_g$$

$893.4 > 881.8$ kN.

As the strength is governed by gross sec^n yielding No Need to consider reduction in Zp due to bolt holes.

Check for shear lag

$$\frac{l_o}{20} = \frac{6.65 \times 10^3}{20} = 332.5 \text{ mm} > 73 \text{ m}.$$

$$\frac{l_o}{10} = 665 \text{ mm} > 54 \text{ m}.$$