Ch-1

D.19.12.2019

INTRODUCTION

Advantages of steel section over reinforced section or concrete member :-

*Higher strength ⇒

→ It becomes higher ($\frac{\text{strength}}{\text{weight}}$) ratio compared to concrete structure.

$$\text{Higher}\left(\frac{S}{W}\right)_{S.S} \geq \left(\frac{S}{W}\right)_{C.C}$$

→ More economical (due to higher ($\frac{\text{strength}}{\text{weight}}$) ratio

→ Ductility of material measured by percentag of elongation

when percentage of Elongation > 15% ⇒ Ductile (ex- steel, Al

When 5% of percentage of Elongation ≤ 15% ⇒ Intermedi ex-(Mg)

when percentage of Elongation < 5% ⇒ Brittle mate (ex- cast iron, c

→ Easily repairable and maintenable.

→ High durability.

→ 100% scrap value.

→ overall cost of structure is less.

Disadvantages

→ corrosion problem (due to corrosion thickness decreases)

→ Less fire resistance (so it reduces f_y and f_u valu

<u>Type of Steel</u>

(1) standard steel
(2) fusion welding steel
(3) High tensile steel.

(1) <u>standard steel</u>

→ used for static loads. Ex.– Fe 410 S

→ carbon = 0·23 – 0·25 %

→ (P.E) percentage Elongation upto/after fracture = 23%

→ Welding if $t \leq 20$ mm (t = thickness of steel member)

(2) <u>Fusion welding steel</u>

→ It is used for dynamic and impact loads [Ex.– Fe 410 WA
Fe 410 WB

→ carbon = 0·20 – 0·25 %

→ percentage Elongation upto/after fracture = 23%

→ welding if $t > 20$ mm (t = thickness of steel member)

(3) <u>High tensile steel</u>

→ It is used for high tensile force. Ex → Fe 550, Fe 570

→ Carbon = 0.27 %

→ percentage elongation up to/after fracture = 20%.

properties of structural steel

(1) Physical properties:-

specific gravity = S = 7.85

young's modulus = E = 2×10^5

co-efficient of thermal expansion = $\alpha = 12 \times 10^{-6}/^\circ C$

poisson's ratio = $\mu = 0.25$ (IS : 800-1984)

$\mu = 0.30$ (IS : 800 - 2007)

$\Rightarrow E = 2C(1+\mu)$ ⇒ C = shear Modulus

$\Rightarrow E = 3K(1-2\mu)$ ⇒ K = Bulk modulus

(2) Mechanical properties

Steel	fy value			fu
	t<20mm	t=20-40mm	>40mm	
standard and fusion welding steel	250	240	230	410
High tensile steel		350		570
	←		→	

2nd half

Standard rolled steel section

(i) Rolled steel tube section (or) hollow section

(ii) Rolled steel bars [square bars and round bars]

(a) ISSQ = Indian standard square section (designated by side)

(b) ISRO = Indian standard round section (designated by dia.)

(iii) Rolled steel flats (when t ≥ 5 mm) ex: 100ISF10

(iv) Rolled steel plats (when t ≥ 5mm) ex: ISPL(100 x 12)

(v) Rolled steel sheets (when t < 5mm)

(vi) Rolled steel strips (ISST)

Ex. ISSQ 100 ⇒ side of square = 100mm
ISRO 120 ⇒ Dia = 120mm

(vii) ~~I-section~~

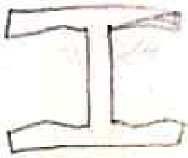
(vii) Rolled steel I - section

→ Designated by overall depth and weight. Ex: ISLB500 @ 750N/m

Overall length

weight

Types of Rolled I-section



(1) ISJB (Indian Standard Junior Beam)
(2) ISLB (Indian Standard light Beam)
(3) ISMB (Indian standard Medium weight Beam)
(4) ISWB (Indian standard Wide flanged Beam)
(5) ISHB (Indian standard Heavy Beam)

(viii) Rolled steel (C) channel section

→ Designated by overall depth and weight.

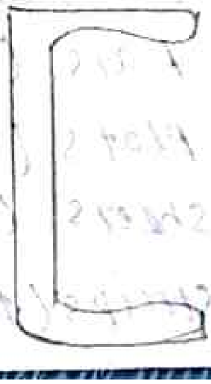
Ex - ISMC500 @ 750 $\frac{N}{mm}$

↓ Overall length

↓ weight

Types of Rolled steel (C) channel-section.

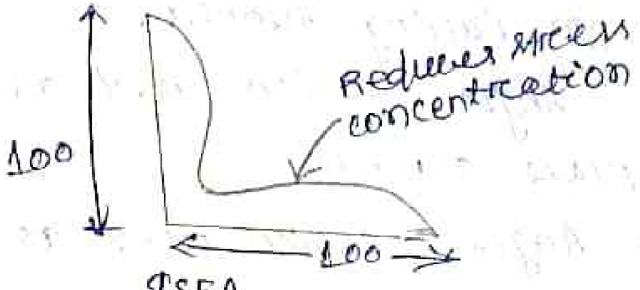
(a) ISJC - Indian standard junior channel section.
(b) ISLC = Indian standard light channel section.
(c) ISMC = Indian standard Medium weight with sloping flange.
(d) ISMCP = Indian standard medium weight with parallel flange.
(e) ISGC = Indian standard gate channel section.



(iv) Rolled steel Angle section

→ It is designated by its leg width and thickness

Ex - ISA $(\underbrace{100 \times 100}_{\text{legwidth}} \times \underset{\text{thickness}}{\downarrow 10})$



ISEA
(Indian standard Equal Angle)

(i) ISEA → Indian standard equal Angle

(ii) ISUA → Indian standard unequal Angle

(iii) ISBA → Indian standard Bulb Angle

NOTE

Angle section : Most suitable when

(i) for compression member (so used as a strut) ⇒ Equal angle section ($\because I_{zz} = I_{yy}$ (or) $C_{zz} = C_{yy}$ and r_{min} is more as compared to unequal angle section)

(ii) for tension member (so used as a tie) ⇒ Unequal angle section with longer leg is connected to gusset plate.
($I_{zz} > I_{yy}$ (or) $C_{zz} > C_{yy}$)

(iii) ISBA has more stiffness, more rigidity but it has more weight, thats why not used in structure. But particularly used in ship building construction

→ Flat section is best section for tension member

use of Angle section

→ In case of principal rafter in roof truss in

→ In lacing member.

→ Angle section used as stiffner member in plate girder.

→ Angle section used as cleat member in foundation.

D. 06.01.2020

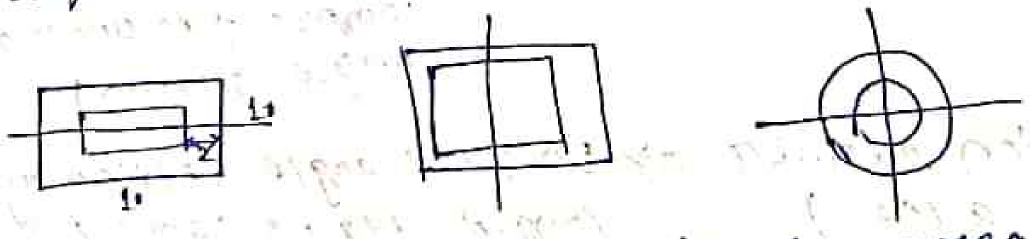
(x) Rolled steel tube / Hollow section :-

→ Hollow sections are designated by their outside dimension and thickness.

→ Hollow sections are best section for tension, compression & torsion.

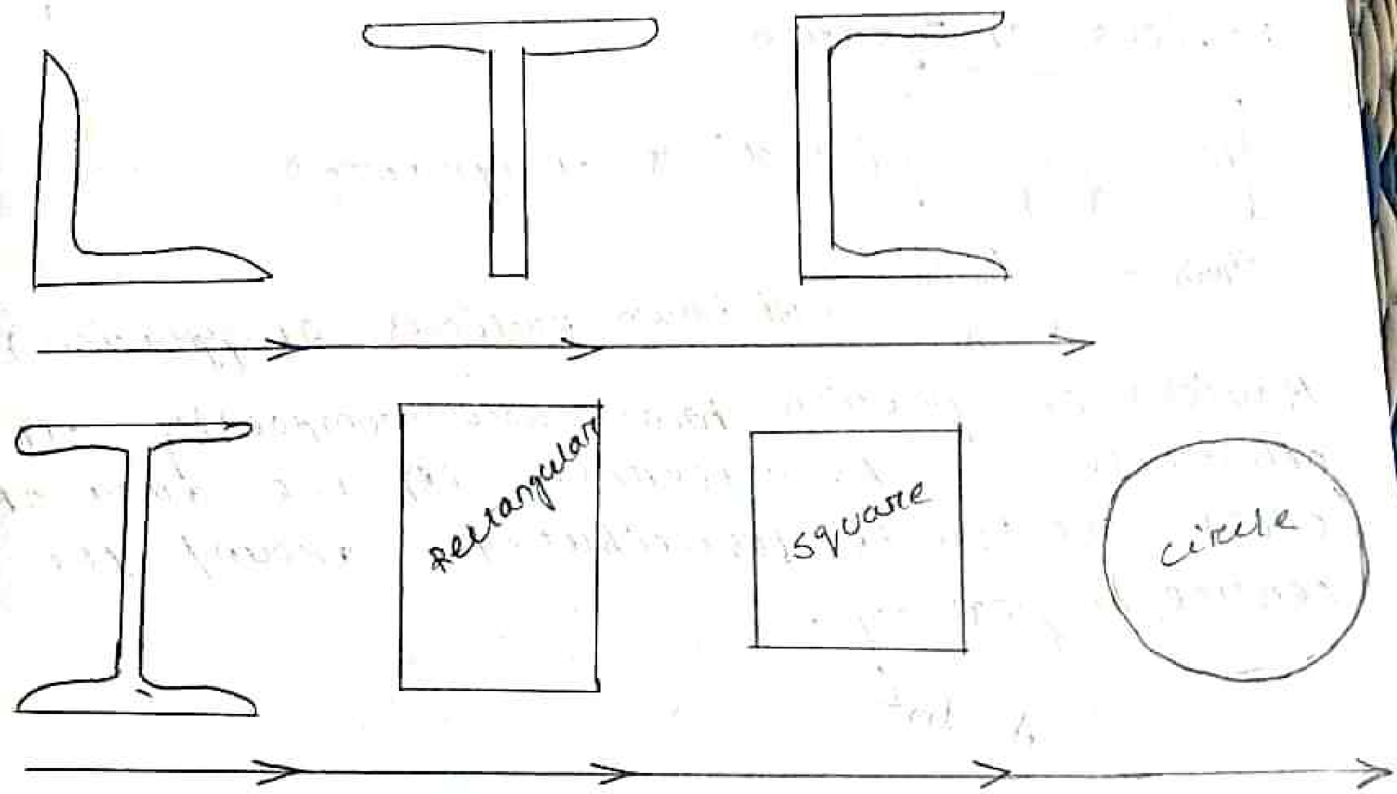
→ Box (closed type) type section is desirable for when torsion is dominant.

→ Ideal steel member is to with stand axial compressive force i.e, circular hollow section.



for same cross sectional area or weight a circular hollow section gives higher moment of inertia (MI) and polar moment of inertia and radious of gyration about any diameter axis as compared to solid circular section. Hence circular hollow section is more efficient and economical to resist axial compression, strongest section as a compression member is equal solid circular section.

Most suitable section as a compression member equal to hollow circular section.



(compressive strength) & (torsional strength) in increasing order as a strongest secti

As a beam

→ Most ~~strongest~~ & suitable section is 'I' section.

→ Most ~~strongest~~ strongest 'I' section as a beam is ISWB

→ Most suitable 'I' section as a beam is ISMB.

As a compression member

- → Strongest 'I' section as a compression member is ISWB.
- → Most suitable 'I' section as a compression member ISHB

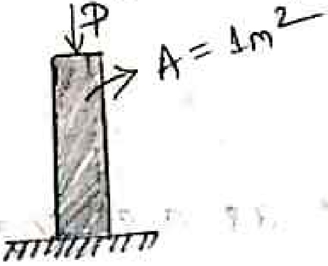
D. 07.01.2020

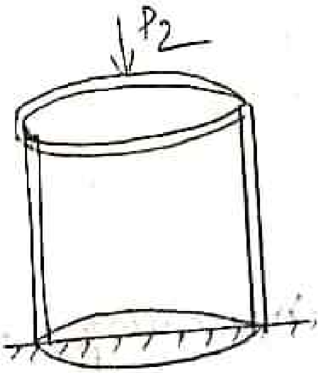
Radious of gyration

$$\boxed{r = \sqrt{\frac{I}{A}}}$$ (Radious of gyration)

$$r_{min} = \sqrt{\frac{I_{min}}{A}}$$ (Minimum radious of gyration)

Radious of gyration means how compactly or otherwise the material is the area of cross section is distributed around the centre of gravity.





$P_2 > P_1$

$r_{min} \uparrow \Rightarrow$ material scattered $\uparrow \Rightarrow$ Buckling $\downarrow$ is aim for engineer

So, always more radious of gyration for any structural member

Slenderness ratio :-

→ It is used for long column.

→ It is denoted by λ

$$\boxed{\lambda = \frac{l}{r_{min}}}$$

where,

l = effective length

r_{min} = minimum radious of gyration.

$\lambda \uparrow \Rightarrow r_{min} \downarrow \Rightarrow$ stability decreases $\Rightarrow$ Buckling of column $\uparrow$

Euler's critical compressive load carrying capacity

$$P_{cr} = \frac{\pi^2 EI}{l^2}$$

$P_{cr} \downarrow$ crippling load

$$= \frac{\pi^2 EI}{(Kl)^2} \quad [\because l = Kl]$$

(Kl) ↑ depend on end condition

$$= \frac{\pi^2 EAr^2}{l^2} \quad \left[\because I = Ar^2,\ r = \sqrt{\frac{I}{A}}\right]$$

$$= \frac{\pi^2 EA}{\left(\frac{l}{r}\right)^2}$$

$$= \frac{\pi^2 EA}{\lambda^2} \quad \left[\because \frac{l}{r} = \lambda\right]$$

Load and Load combinations :-

limit state design (L.S.D) ⇒ partially safety factor (P.S.F)
(IS : 800 - 2007)

(P.S.F) → γ_f (load factor), γ_m (Resistance factor)

Load factor (γ_f)

$$\gamma_f = \frac{\text{Design load}}{\text{characteristics load}}$$

$\Rightarrow$ Design load $= \gamma_f \times$ characteristics load

Resistance factor (γ_m)

$$\gamma_m = \frac{\text{characteristics strength}}{\text{Design } \cancel{\text{load}} \text{ strength}}$$

① load factor (γ_f) value

② load combination	γ_f value
DL + LL →	1.5
DL + WL →	1.5
DL + (WL/EL) + LL →	1.2 (for Roof trusses)

(2) Resistance factor (γ_m) value

→ Resistance factor against yield stress (fy)

⇒ fy ⇒ γ_{m0} = 1.1 (Also against buckling)

→ Resistance factor against ultimate stress (fu)

fu ⇒ γ_{m1} = 1.25

D.08.01.2020

→ Resistance factor for connection (γ_{m1}) is γ_{mf} br)

γ_{mb} = 1.25 [Expect weld connection]

→ Resistance factor for weld connection

↳ γ_{mw} = 1.25 [shop fabrication]

→ γ_{mw} = 1.5 [field fabrication]

Connection is three types

(1) Reveted / Bolted connection

(2) Welded connection

(3) Hanger connection

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Limit state Design

Limit state of collapse

(1) limit state of strength
⇒ (1) yielding
(2) Buckling
(3) flexure
(4) shear

(2) stability
⇒ (1) Against sliding
(2) Against overturning

limit state of serviceability

→ Deformation
→ Deflection
→ vibration
→ corrosion
→ fibre
→ fatigue checks

NOTE

The requirement to be satisfied for safety of structural connection in structural member and design action should be less than equals to design strength.

$$\boxed{\text{Design action} \leq \text{Design strength}}$$

⇓ ~~Design~~ Due to material own strength

Design action is due to design value of internal forces (shear force, bending moment, twisting moment and torsion) occurs due to external load.

$$\text{Factored (or) Design load} = \overset{1.5}{\gamma_F} \times \begin{pmatrix} \text{permissible, safe} \\ \text{characteristics} \\ \text{working} \end{pmatrix} \text{load}$$

$$\text{Design strength} = \frac{\text{characteristics strength}}{\gamma_m}$$

Ch-02 STRUCTURAL STEEL FASTENERS AND CONNECTIONS

BOLTED CONNECTION :-

Types of Bolt

(1) Black or unfinished or Bearing type or slip type
or ordinary bolt

→ These are used for light structure subjected to static load.

→ It allow slip of plate and load transfer through the bearing action.

→ It is made up of low carbon steel i.e, mild steel

Ex:- ~~M5~~ M5 to M36

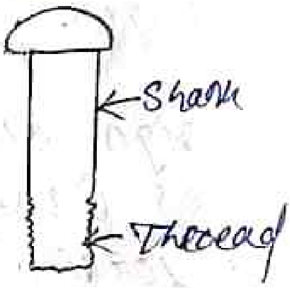
But most common M16, M20, M24, M30

(Number indicates shank diameter of bolt)

$\overset{(S)}{\text{Shank}}$ $\overset{(A)}{\text{area}}$ of $\overset{(b)}{\text{bolt}}$ $= A_{sb}$

$\overset{(n)}{\text{Thread}}$/$\overset{(B)(A)}{\text{Net area}}$ of $\overset{(b)}{\text{bolt}}$ $= A_{nb}$

$= \boxed{0.78 A_{sb}}$



Grade of bolt varies from 4.6 to 8.8

4.6 grade ~~of~~ bolt meant → $f_{ub} = 400$ MPa (or) N/mm^2

→ $f_{yb} = 400 \times 0.6 = 240$ MPa

where, f_{yb} = yield stress of bolt

f_{ub} = Ultimate stress of bolt

Ques Grade of bolt = 5.6 -

Grade of

$f_{ub} = 5 \times 100 = 500$ Mpa

$f_{yb} = 500 \times 0.6 = 300$ Mpa

Ques Grade of bolt = 7.2

~~Grade~~ $f_{ub} = 7 \times 100 = 700$ MPa

$f_{yb} = 700 \times 0.2 = 140$ MPa

Ques

M20

Shank Area$^{(Asb)}$ = $\frac{\pi}{4} \times 20^2$

$= 314$ mm^2

(Thread area) $A_{nb} = 0.78 \times A_{sb}$

$= 0.78 \times 314$

$= 245$ mm^2

(2) HSFG (High Strength Friction Grip bolt)

→ These bolts are fixed by developing maximum tension in bolt such that plates develop maximum compression between them.

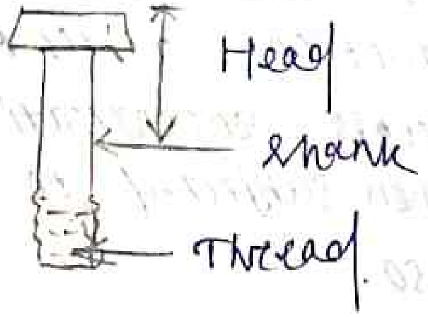
→ These bolt does not allow slip of bolt so load transfer takes place ~~dts~~ due to the friction between the plates.

→ These are used in structures which subjected to dynamic loads, vibration, impact loads & most suitable for structures fatigue loads & subjected to reversal of stresses.

Ex - 10.9S
 - 12.4S

NOTE

Bolt is designated by MdL



MdxL

where, M = metric size

d = shank dia.

L = shank length

~~T_{nd2}~~

$$T_{db_2} = \frac{0.9 A_n f_{ub}}{\gamma_{m1}}$$

$$= \frac{0.9\times448\times400}{1.25} = 129024\ N$$

= ~~12.90 KN~~

⇒ $T_{db_2} = 129$ KN

So, smaller value is taken

$\boxed{T_{db1} = 68.51\ KN}$ $\boxed{V_{dsb} = 45.08\ KN}$ $\boxed{V_{dpb} = 130.56\ KN}$

∴ Design strength of bolt = V_{db} = smaller of V_{dsb}, V_{dpb}, T_{db}

⇒ $\boxed{V_{db} = V_{dsb} = 45.08\ kN}$

no. of bolts required to resist factor load 562.5KN

$$\Rightarrow n = \frac{P_d}{V_{db}}$$

$$= \frac{562.5}{45.08}$$

= 12.47 nos

≃ 13 nos

NOTE

Cover plate thickness

for single cover butt joint thickness of cover plate = $1.25\, t_{min}$

for double cover butt joint thickness of cover plate = $\frac{5}{8} t_{min}$

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D. 09. 01. 2020

Types of bolted joint (or) connection :-

There are 3 types of connection :-

(1) Shear type connection ⇒ (1) Lap (2) Butt

× (2) Hanger type ⇒ Tension type connection

× (3) Combined shear and ~~bending~~ tension type ⇒ Bracket connection

(1) Shear type connection

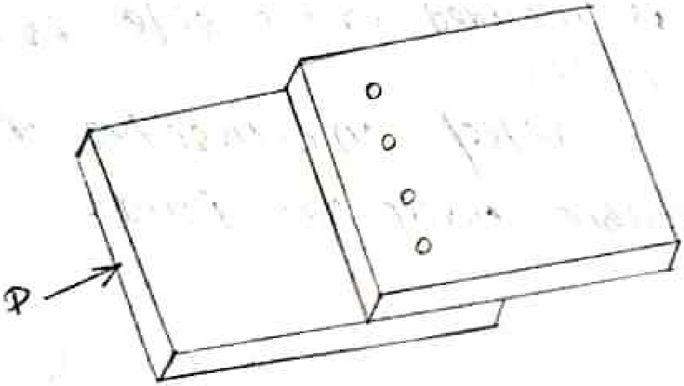
It is of two types

(1) lap connection

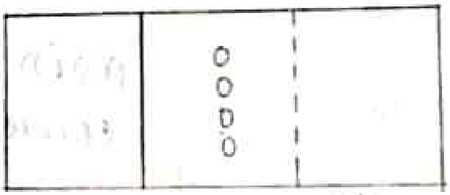
(2) Butt connection

(1) Lap joint

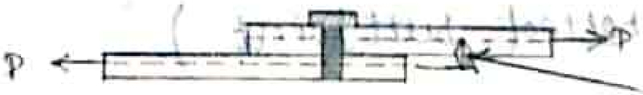
⇒ The two members to be connected are over lapped & connected together. Such a joint is called as lap joint.



(lap joint)



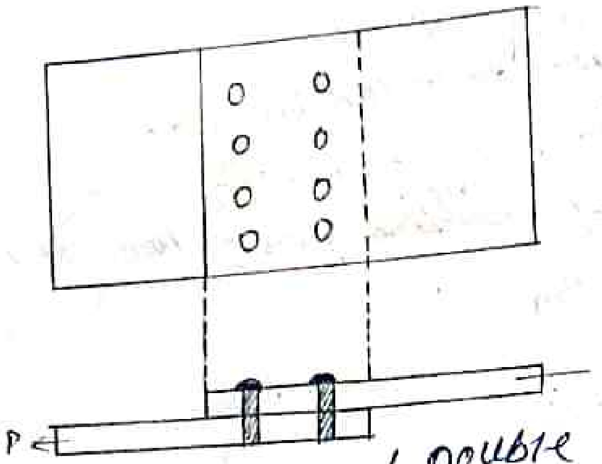
Plan



(elevation)

e = eccentricity = cause moment; so tension failure may occur

(single bolted lap joints)



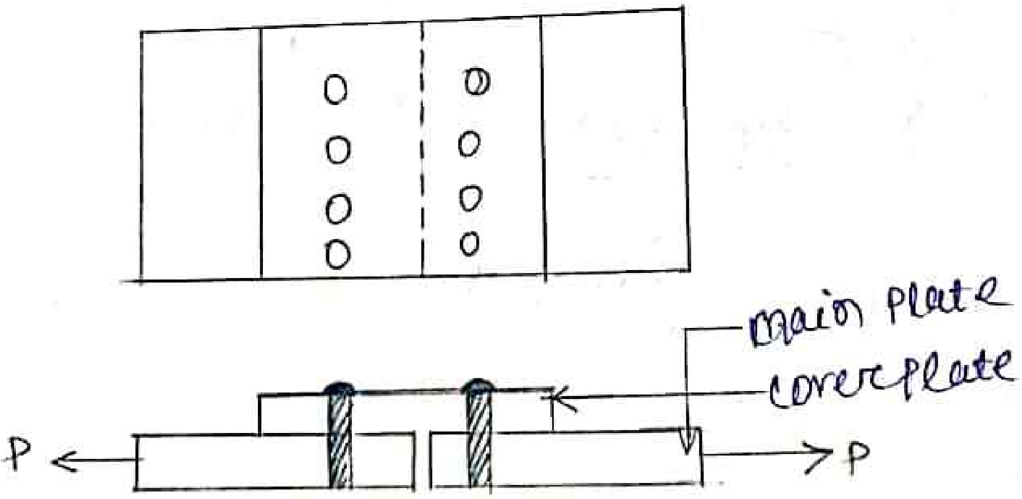
(Double bolted lap joint)

Butt joint

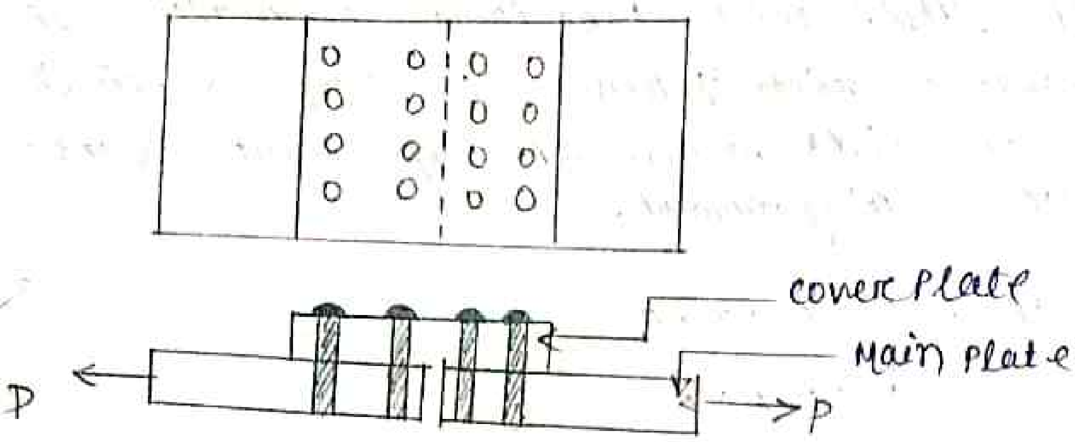
→ The two members ~~are~~ to be connected are ~~plays~~ placed into end. Additional plate provided on either one or both sides, called covers plate, are placed and are connected to the main plate ~~are~~ as shown in the figure bellow.

→ If the coverplate is provided on one side is called single cover butt joint.

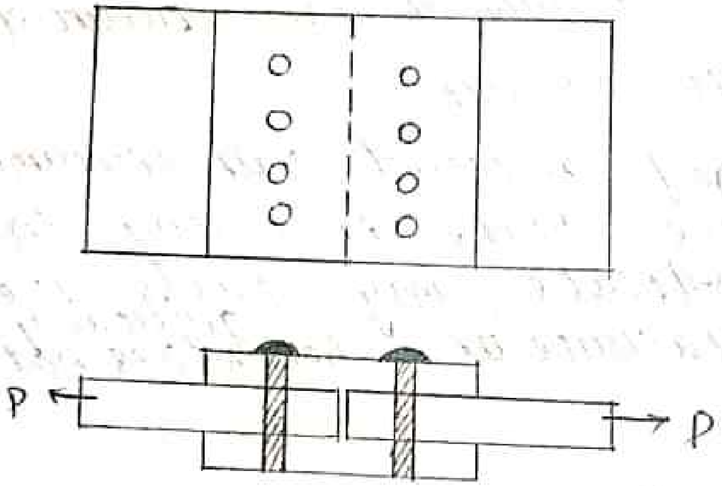
→ If the cover plate provided on both sides of the main plate is called double cover butt joint.



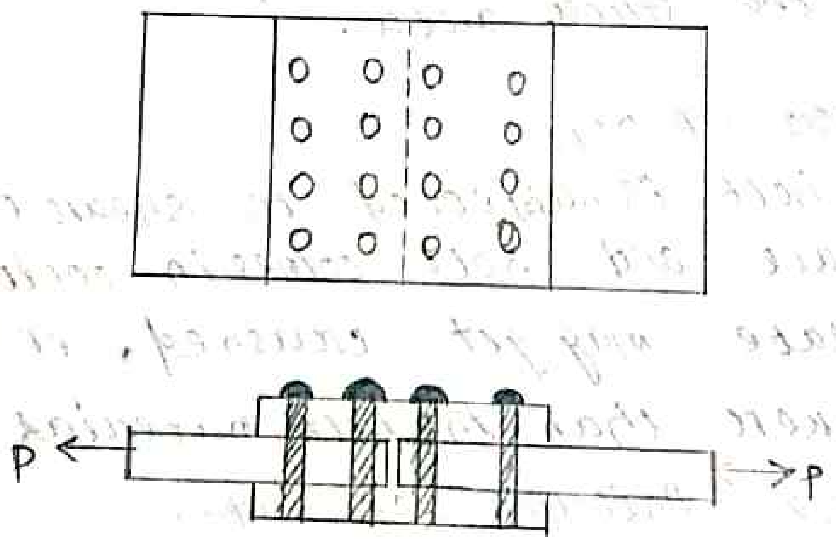
(single cover Single bolted butt joint)



(single cover double bolted butt joint)



(Double cover single bolted butt joint)



(Double cover double bolted butt joint)

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NOTE

In case of double cover bolt joint eccentricity of forces doesnot exist & hence bending is eliminated where as it exists in case of lap joint. Thats why cause for bending moment.

Failure of Bolted joint

(1) Shear failure of Bolt

→ Shear stresses are develop when the plates slip due to applied forces. When the maximum factor shear force in the bolt exceed the nominal shear capacity of bolt then shear failure of bolts occured

(2) Bearing failure of Bolt

→ The bolt is crushed arround half circumference. It occurs when the plate is strong in bearing and heaviest stressed plate may press the bolt shank. Bearing failure of bolt generally does not occurs in practice

(3) Tension failure of Bolt

→ It occurs when the bolt is subjected to tension and may fail at the stress area.

(4) Bearing failure of Plate

→ When an ordinary bolt is subjected to shear forces this slip takes place and bolt comes in contact with the plates. The plate may get crushed, if the plate material is weaker than the bolt material.

(5) Tension or Tearing failure of plates

→ It occurs when the bolts are stronger than plates during tension.

(6) Block Shear failure

⇒ It occur when the block of material within the bolted area breaks are away from the remainder area. The possibility of this failure increases when high strength bolts are used. This type of failure occurs with shear on one planes and tension on perpendicular plane leading to fall of hatched portion of the plate.

NOTE

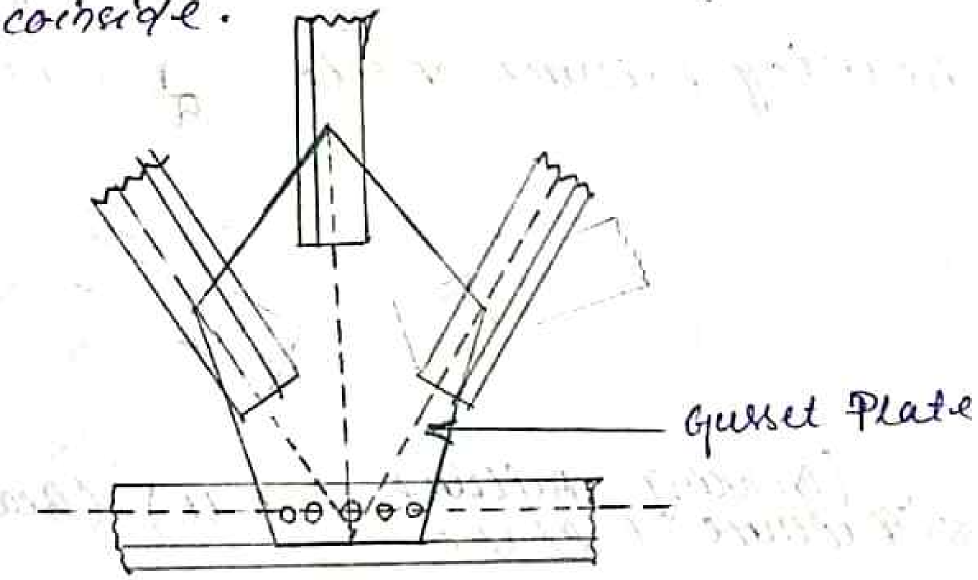
Imp

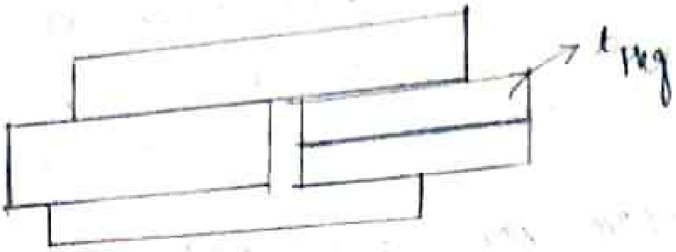
Gusset Plate

→ A gusset plate is a plate provided to make a connections at the place where more than one member is to be jointed.

Ex:⇒ Joint of truss, gross girder etc

→ The lines of action of truss member meeting at joints are assume to coincide.





Factored (or) Design Bearing strength of bolt (V_{dpb})

P → for bearing

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

where,

V_{npb} = Nominal bearing capacity of bolt

$$\boxed{V_{npb} = 2.5 K_b d t f_u'}$$

$$\boxed{V_{dpb} = \frac{2.5 K_b d t f_u'}{\gamma_{mb}}}$$

where,

K_b = Bearing factor

K_b = Smaller of (i) $\frac{e}{3d_0}$ (ii) $\frac{p}{3d_0} - 0.25$ (iii) $\frac{f_{ub}}{f_u}$ (iv) 1.0

~~d_0 = di~~ d = diameter of bolt

e = edge distance

p = pitch, t = thickness of main ~~ba~~ plate in case of lap joint

d_0 = diameter of bolt hole

t = smaller of (i) thickness of main plate (ii) sum of thickness of cover plate in case of butt joint

f_{ub} = ultimate sheare stress of plate

t = aggregate thickness of connected plate which experiencing bearing stress in same direction.

f_u' = Smaller of f_{ub} & f_u

D.13.01.2020

Spacing Of Bolt holes

→ pitch, edge ~~e~~distance, end distance define the spacing of bolt holes in a joint

Pitch (p)

→ It is distance betweens the centres of two consecutive bolts measured along a row of bolts. A row generally refers to a line of bolts placed parallel to the forces in a member.

→ For wide Plates piten may also bedefined as centre to centre (c/c) distance of bolts measured along the length of member or the connection

→ when bolts are placed staggered the pitch will be ~~in~~ referred to as staggered pitch

Gauge (g)

→ It is distance between adjacent bolt lines or the c/c distance between two consicutive bolt measured along the width of the member or connection.

→ Gauge linesare also called as bolt line.

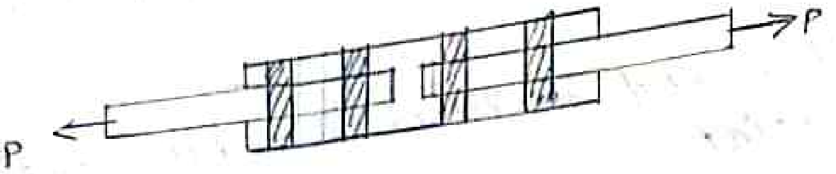
Edge distance

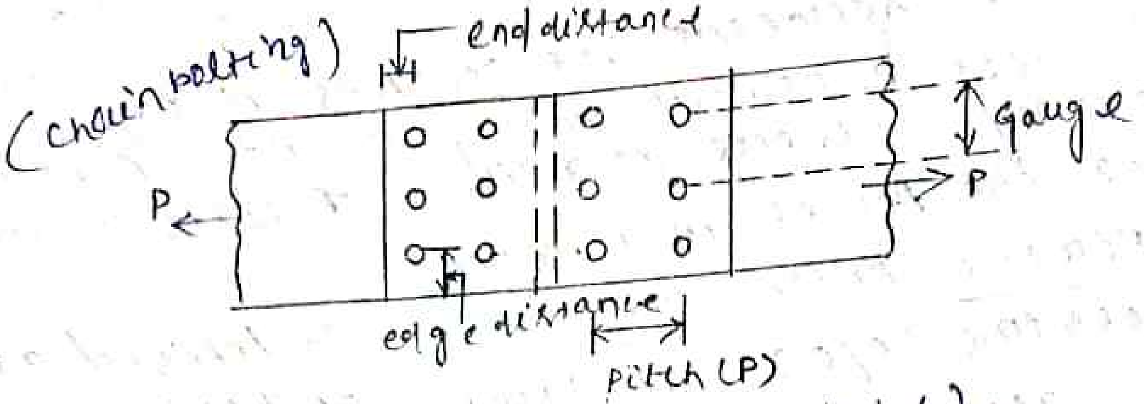
→ This is distance at right angle to the direction of forces or stresses from the centre of bolt hole to the adjacent edge of the member. ~~whereas~~

End distance

→ This is the distance parallel to the direction of stresses or forces from the centre of bolthole to the end of the element is called the end distance.

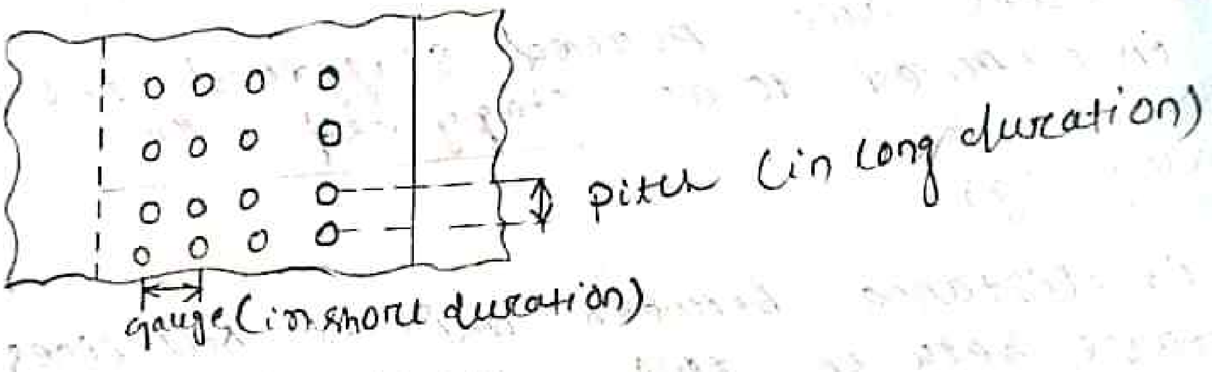
(1)

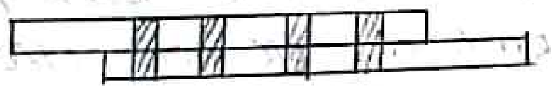




(Double cover butt joint)

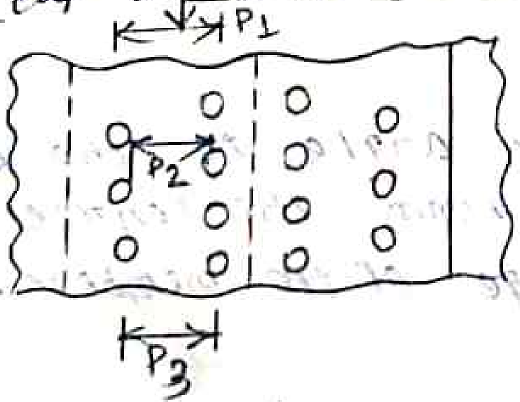
(2)





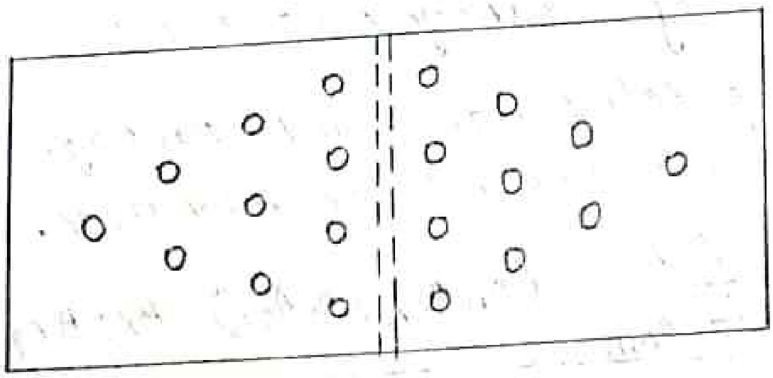
(lap joint with wide plates)

(3)



(staggered bolting)

(4)



Double cover butt joint
(Diamond bolting)

(5) Pack or stitch bolting

⇒ Packing or stitching bolts are used to make the sections act in unison & to prevent buckling in compression members, when two or more section are in contact.

⇒ They are not consider in strength calculation.

D. 14.01.2020

Factored Design shear strength of bolt (V_{dsb})

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

where; V_{nsb} = Nominal shear strength of bolt

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

$$\Rightarrow \boxed{V_{dsb} = \frac{f_{ub}}{\sqrt{3}\,\gamma_{mb}} (n_n A_{nb} + n_s A_{sb}}$$

where f_{ub} = Ultimate tensile stress

n_n = no. of shear plane with threads intercepting the shear plane.

n_s = no. of shear plane with shank (without threads) intercepting shear plane.

(1.25)
γ_{mb} = partially safety factor for bolt material

Again for long joint, grip and package, the above shear strength value will be reduced.

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

where,

β_{lj} = A reduction factor to allow the overloading of the end bolts that occur in long joint or connection.

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

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

A_{nb} = Threaded area or net tensile stress area of bolt

A_{sb} = shank area or Nominal shank area

$$\boxed{A_{nb} = 0.78\, A_{sb}}$$

(i) Reduction factor for long joint (β_{lj})

→ when $l_j > 15d$ in the direction of load, then the shear capacity is reduced.

$$\boxed{\beta_{lj} = 1.075 - \frac{l_j}{200d}} \text{ for } 0.75 \leq \beta_{lj} \leq 1.0$$

where l_j = length of joint & is taken as the distance between first & last row of bolt in a joint measured in the direction of the load transfer.

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

When a grip length of a bolt increases, the bolt is subjected to a greater bending moment due to shear forces acting on its shank. Therefore total thickness (t) is greater than $5d$, then the shear capacity of bolt is reduced.

$\boxed{t > 5d}$ then the shear capacity of bolt is reduced

where,
t = total thickness of connected plate

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

Note
$$\left(\begin{array}{l} \beta_{lg} \ngtr \beta_{lj} \\ \beta_{lg} \ngtr 8d \end{array} \right)$$

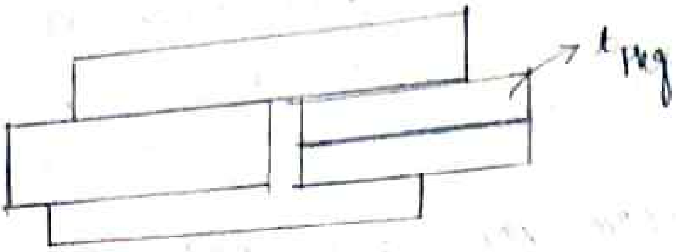
(iii) **Reduction factor for packing plate**

→ When the packing plate are more than 6mm in thickness, the shank of the bolt of is subjected to bending which affect the nominal shear capacity of the bolt by a reduction factor of β_{pkg}

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

when $t_{pkg} > 6mm$

where, t_{pkg} = thickness of packi plate



Factored (or) Design Bearing strength of bolt (V_{dpb})

P → for bearing

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

where,

V_{npb} = Nominal bearing capacity of bolt

$$\boxed{V_{npb} = 2.5 K_b d t f_u'}$$

$$\boxed{V_{dpb} = \frac{2.5 K_b d t f_u'}{\gamma_{mb}}}$$

where,

K_b = Bearing factor

K_b = Smaller of (i) $\frac{e}{3d_0}$ (ii) $\frac{p}{3d_0} - 0.25$ (iii) $\frac{f_{ub}}{f_u}$ (iv) 1.0

~~d_0 = di~~ d = diameter of bolt

e = edge distance

p = pitch , t = thickness of main ~~ba~~ plate in case of lap joint

d_0 = diameter of bolt holes

t = smaller of (i) thickness of main plate (ii) sum of thickness of cover plate in case of butt joint

f_{ub} = ultimate sheare stress of Plate

t = Aggregate thickness of connected plate which experiencing bearing stress in same direction.

f_u' - Smaller of f_{ub} & f_u

Factored (or) Design tensile strength of bolt (T_{db})

$$T_{db} = \frac{T_{nb}}{\gamma_m} = \frac{\text{Nominal (characteristics) tensile strength of bolt}}{\text{Partially safety factor}}$$

⇒ Applicable when tensile load is applied on bolt.

⇒ when A bolt is subjected to factored or design tensile load (T), it make chance to fail by

(1) Gross section yielding failure

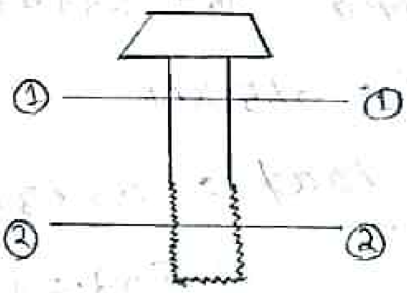
(2) Net section rupture / fracture (or) brittle fracture failure.

(1) $$T_{db_1} = \frac{A_{sb} f_{yb}}{\gamma_{m0}}$$ (when bolt fails in yielding)

(2) $$T_{db_2} = \frac{0.9 A_{nb} f_{ub}}{\gamma_{m1}}$$ (when bolt fails in ultimate stress)

[A_n = Net rupture area] D. 16.01.2020

∴ T_{db} = least of T_{db_1} and T_{db_2}



If failure at ①-① ⇒ It is yielding failure

If failure at ②-② ⇒ It is net section rupture

⇒ Design strength of bolt (V_{db}) = Bolt value = least of V_{dsb}, V_{dpb} & T_{df} (Garg)

⇒ No. of bolts required to resist design (or) factored axial load (P_d) = $\boxed{n = \frac{P_d}{V_{db}}}$

NOTE

Assume steel of grade Fe 410 if grade of bolt not specified. Also assume that the threads of bolt fa in the shear planes if not specified. Assume e = 33 mm, P = 50 mm if not specified. pitch

Ques Design

Design a lap joint to connect two plates 300 mm wide 16 mm thick using 20mm diameter bolt of grade 4.6. The applied service load is 375 kN

Soln

Given data

lap joint ⇒ ~~It is~~ Bolt will be in single shear

width of plate = B = 300 mm
Assume, 410 grade plate ⇒ $f_u = 410$ MPa
thickness of plate = t = 16 mm

diameter of bolt = d = 20mm

hole dia of bolt = $d_h = d_0$

$= 20 + 2$

$= 22$

[$d_h = d + 2$ when d = 16 to 24 mm]

$A_{sb} = \frac{\pi}{4} \times 20^2 \Rightarrow A_{sb} = 314\ mm^2$

4.6 grade bolt

$A_{nb} = 0.78 \times A_{sb}$

$= 0.78 \times 314$

$A_{nb} = 244\ mm^2$

$f_{ub} = 400$ mpa

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

Applied service load = P = 375 KN

⇒ Design (or) factored load $= 1.5 \times 375$

$= 562.5\ kN = P_d$

$\gamma_{mb} = 1.25$

(1) Design shear strength of 1 bolt

$$V_{dsb} = \frac{f_{ub}}{\sqrt{3}\,\gamma_{mb}}\left(n_n A_{nb} + n_s A_{sb}\right)$$

[Assume shear plane intercept thread ⇒ $n_n = 1$, $n_s = 0$]

$$= \frac{400}{\sqrt{3} \times 1.25}\left(1 \times 244 + 0\right)$$

⇒ $V_{dsb} =$ ~~480~~ 45079.50 N

⇒ $V_{dsb} = 45.08$ KN

[$e_{min} = 1.7 d_h$
$= 1.7 \times 22$ — 40mm
$= 37.4\ mm \simeq 50mm$
$P_{min} = 2.5 d$
$= 2.5 \times 20 = 50mm$]

Scanned by CamScanner

(ii) Design bearing strength of bolt

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

k_b = smaller of (i) $\frac{e}{3d_0} = \frac{40}{3 \times 20} = 0.6$

(ii) $\frac{p}{3d_0} \overset{0.25}{\cancel{-0.25}} \quad \frac{50}{60} = 0.51$

(iii) $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.97$

(iv) 1.0

$$\Rightarrow \boxed{k_b = 0.51}$$

$$\Rightarrow V_{dpb} = \frac{2.5 \times 0.51 \times 20 \times 16 \times 400}{1.25}$$

$$\Rightarrow V_{dpb} = 130560 \text{ N}$$

$$\Rightarrow V_{dpb} = 130.56 \text{ KN}$$

$$T_{db_1} = \frac{A_{sb} f_{yb}}{\gamma_{m0}}$$

$$\Rightarrow T_{db1} = \frac{314 \times 240}{1.1}$$

$$\Rightarrow T_{db1} = 68509 \text{ N}$$

$$\Rightarrow T_{db_1} = 68.51 \text{ KN}$$

$$T_{db2} = \frac{0.9 A_n f_{ub}}{\gamma_{m1}}$$

$A_n = (p - n d_h)t$

$= (50 - 1 \times 22)\ 16$

$= 28 \times 16$

$= 448 \text{ mm}^2$

~~T_{bd}~~ $_2$

$$T_{db_2} = \frac{0.9 A_n f_{u}}{\gamma_{m1}}$$

$$= \frac{0.9\times 448 \times 400}{1.25} = 129024 \text{ N}$$

$=$ ~~12.90 KN~~

$\Rightarrow T_{db_2} = 129$ KN

So, smaller value is taken

$\boxed{T_{db1} = 68.51 \text{ KN}}$ $\boxed{V_{dsb} = 45.08 \text{KN}}$ $\boxed{V_{dpb} = 130.56 \text{KN}}$

∴ Design strength of bolt $= V_{db}$ = smaller of V_{dsb}, V_{dpb}, T_{db}

$$\Rightarrow \boxed{V_{db} = V_{dsb} = 45.08 \text{ KN}}$$

no. of bolts required to resist factor load 562.5 KN

$$\Rightarrow n = \frac{P_d}{V_{db}}$$

$$= \frac{562.5}{45.08}$$

$= 12.47$ nos

$\simeq 13$ nos

NOTE

Cover plate thickness

For single cover butt joint thickness of cover plate $= 1.25 t_{min}$

For double cover butt joint thickness of cover plate $= \frac{5}{8} t_{min}$

Analysis

Calculate the strength of the 20mm diameter bolt of grade 4.6 for the following cases. The main plates to be joined on 12mm thick.

(a) lap joint

(b) single cover butt joint, cover plate thickness = 10mm

(c) Double cover butt joint, when each of the cover plate being 8mm thick

D.17.01.2020

Given data

diameter of bolt (d) $= 20mm$

diameter of bolt holes (d_h) $= 20 + 2$

$= 22mm$

thickness (t) $=$ ~~10~~ 12 mm

shank area of bolt

$A_{sb} = \frac{\pi}{4} \times 20^2$

$= 314 \; mm^2$

thread area of bolt

$A_{nb} = 0.78 \times A_{sb}$

$= 0.78 \times 314$

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

4.6 grade

$f_{ub} = 4 \times 100 = 400 \, MPa$

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

Assume Fe 410 grade of steel $= f_u = 410 MPa$

$\gamma_{mb} = 1.25$

(i) lap joint

The bolt will be in single shear and bearing

design shear strength of bolt

$$N_{dsb} = \frac{f_{ub}}{\sqrt{3}\,\gamma_{mb}} (n_n A_{nb} + 3 A_{ns})$$

(Assume shear plane intercept at thread of bolt)
($n_n = 1$, $n_s = 0$)

$$V_{dsb} = \frac{f_{ub}}{\sqrt{3}\,\gamma_{mb}}(n_n A_{nb} + n_s A_{ns})$$

$$= \frac{400}{\sqrt{3} \times 1.25}(1 \times 245)$$

$$= 45264.26\,N$$

$$= 45.26\,KN$$

Design bearing strength of one bolt

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

Assume pitch(p) = 50mm, e = edge distance = 33

k_b = smaller of (i) $\frac{\overset{\rightarrow 33}{e}}{3d_0} = \frac{33}{3 \times 22} = 0.5$

(ii) $\frac{\overset{\rightarrow 50}{p}}{3d_0} - 0.25 = \frac{50}{3 \times 22} - 0.25 = 0.5$

(iii) $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.97$

(iv) 1.0

$\Rightarrow \boxed{K_b = 0.50}$

$$V_{dpb} = \frac{2.5 \times 0.50 \times 20 \times 12 \times 400}{1.25}$$

$$= 96000\,N$$

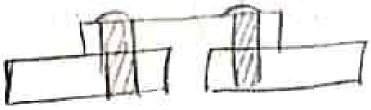
$$= 96\,kN$$

∴ The strength of bolt $= V_{db}$

$\Rightarrow$ smaller of V_{dsb} & V_{dpb}

$\Rightarrow \boxed{V_{dsb} = 45.26\,KN}$

(ii) single cover buttjoint when cover plate thickness=10mm



Bolt will be in single shear.

The shear strength of one bolt $= V_{dsb}$

$$\Rightarrow \frac{f_{ub}}{\sqrt{3}\gamma_{mb}}\left(n_n A_{nb} + n_s A_{sb}\right)$$

$$\Rightarrow 45.26 \text{ kN}$$

The bearing strength of one bolt $= V_{dpb} = \frac{2.5 k_b \times d \times t \times f_u}{\gamma_{mb}}$

t = thickness of main plate in case of lap joint

t = smaller of (i) thickness of main plate } in case of
(ii) sum of thickness of cover plate } buttjoint

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

$$= \frac{2.5 \times 0.5 \times 20 \times 10 \times 400}{1.25}$$

$$= 80000 \text{ N}$$

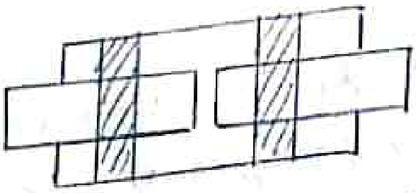
$$\Rightarrow \boxed{V_{dpb} = 80 \text{ kN}}$$

∴ Design strength of bolt in case of single cover butt joint $= V_{dpb}$

smaller of V_{dsb} & V_{dpb}

$$\Rightarrow \boxed{V_{dsb} = 45.26 \text{ kN}}$$

(iii) Double cover butt joint, when each of the cover plate being 8mm thick



The bolt will being double shear and bearing.

The shear strength of one bolt in double shear V_{dsb}

$$\Rightarrow V_{dsb} = 2\times\frac{f_{ub}}{\sqrt{3}\,\gamma_{mb}}\left(n_n A_{nb} + n_s A_{ns}\right) \quad [\because n_n = 2,\ n_s = 0]$$

$$\Rightarrow V_{dsb} = 2\times 45.26$$

$$\Rightarrow \boxed{V_{dsb} = 90.52\ KN}$$

Design bearing strength of one bolt $= V_{dpb} = \frac{2.5\times k_b \times d\times t\times f_u'}{\gamma_{mb}}$

$$V_{dpb} = \frac{2.5\times k_b\, d\, t\, f_u'}{\gamma_{mb}}$$

, t = smaller of (i) thickness of main plate = 12 mm
(ii) sum of cover plate thickness = 16 mm (8+8)
{t = 12}

$$= \frac{2.5\times 0.5\times 20\times 12\times 400}{1.25}$$

$$= 96000\ N$$

$$\Rightarrow \boxed{V_{dpb} = 96\ kN}$$

∴ strength of bolt in case of double cover butt joint

= smaller of V_{dsb} & V_{dpb}

$$\Rightarrow \boxed{V_{dsp} = 90.52\ KN}$$

Note

→ Design strength of bolt = V_{db} = smaller of V_{dsb} & V_{dpb}

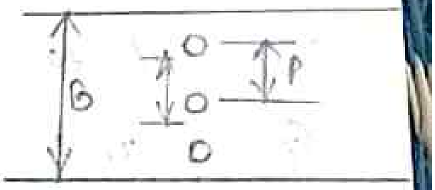
→ No. of bolt required to resist the factored load (P_d): $n = \frac{P_d}{V_{db}}$

Design bearing (or) Tensile strength of plate:- (T_{dp})

$$T_{dp} = \frac{T_{np}}{\gamma_m} \Rightarrow \boxed{T_{dp} = \frac{0.9 A_n f_u}{\gamma_{m1}}}$$

where, A_n = net rupture area of plate $= (B - n d_h) t$

$= (p - n d_h) t$



D. 18.01.2020

NOTE

→ Design strength of bolted connection (or) joint = least of V'_{dsb}, V'_{dpb} & T_{dp} (smaller)

V'_{dsb} = design shear strength of bolted joints

$= n \times V_{dsb}$ (n = total no. of bolts in lap joint)

V'_{dpb} = design bearing strength of bolted joint

$= n \times V_{dpb}$ (n = total no. of bolt in one side in case of butt joint

→ Efficiency of bolted connection (or) joint =

$$\Rightarrow \overset{eta}{\eta}_{(\%)} = \frac{\text{Design strength of joint}}{\text{Design strength of solid plate } (T_{dsp}}\times 100$$

where $T_{dsp} = \frac{A_g f_y}{\gamma_{m0}}$ ($A_g = B \times t$) → thickness of plate. → Gross Area

Efficiency of bolted connection or joint η (%)

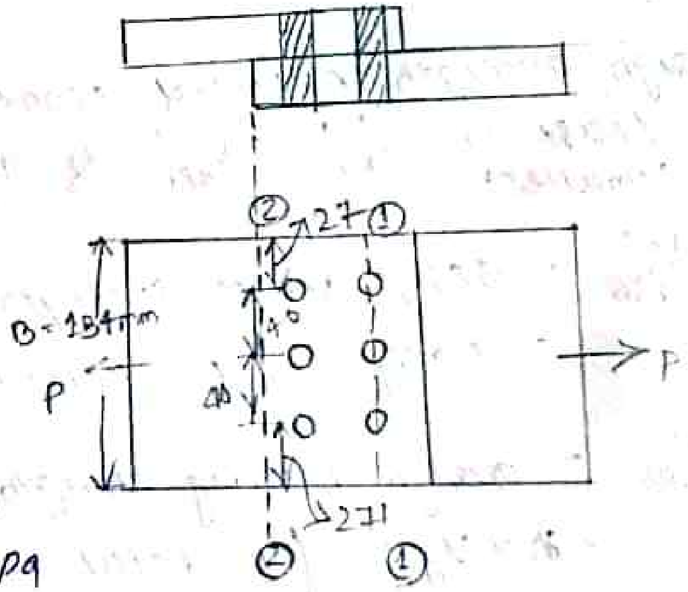
$$\eta(\%\text{age}) = \frac{\text{Design strength of joint per pitch}}{\text{Design strength of solid plate per pitch}} \times 100$$

$$\boxed{T_{dsb} = \frac{A_g f_y}{\gamma_{m0}} \quad (A_g = p \times t)}$$

Ques

Find the maximum force that can be transmitted through a double bolted chain lap joint consisting of 6 bolt in 2 rows. Given that M16 gr bolts in 2rows are grade 4.6 & plates of Fe410 are used. The thickness of plate are connected to be taken as 10 mm & 12 mm

Given data



No. of bolts = n = 6

M16 bolt ⇒ d = 16 mm
↓ ⇒ shank dia. of bolt
Metric

hole dia of bolt = 16 + 2
= 18 mm

Bolt of grade 4.6

$f_{ub} = 4 \times 100 = 400\text{MPa}$

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

Plate grade Fe 410 . f_u = 410 MPa

$t = 10$ mm

$A_{sb} = \frac{\pi}{4} \times 16^2 = 201 \text{mm}^2$

$A_{nb} = 0.78 \times A_{sb}$

$= 0.78 \times 201$

= ~~157.6~~ mm²

= 157 mm²

Maximum force that can be transmitted through this boltedjoint .

strength of joint = Smaller of V'_{dsb} , V'_{dpb} & T_{dp}

$V'_{dsb} = \eta \times V_{dsb}$

$V'_{dpb} = \eta \times V_{dpb}$

$T_{dp} = \frac{0.9 A_n f_u}{\gamma_{m_1}}$

lapjoint ⇒ Bolt will be in single shear

(i) Design~~ed~~ shear strength of 1 bolt

V' ~~$_{dsb} = \eta \times$~~ V_{dsb}

$V_{dsb} =$ ~~$\eta \times$~~ $\frac{f_{ub}}{\sqrt{3}\,\gamma_{mb}}(n_n A_{nb} + n_s A_{ns})$ $[n_n = 1 \ , \ n_s = 0]$

$=$ ~~$\eta \times$~~ $\frac{400}{\sqrt{3} \times 1.25}(1 \times 157 + 0)$

[shear plane intercept thread of the bolt

$= 29006.07$ N

$= 29.01$ kN

⇒ Design shear strength of lap joint $= \eta \times V_{dsb} =$

$= 6 \times V_{dsb}$

$= 6 \times 29.01$

$= 174.06$ kN

(2) Design bearing strength of 1 bolt $= V_{dsb}$

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

Minimum Pitch (P) = $2.5d = 2.5 \times 16 = 40$ mm

minimumend distance = $e = 1.5 d_h = 1.5 \times 18$

$= 27$ mm

k_b = smaller of (1) $\frac{e}{3d_0} = \frac{27}{3\times18} = 0.5$

(2) $\frac{p}{3d_0} - 0.25 = \frac{40}{3\times18} - 0.25 = 0.49$

(3) $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.97$

(4) 1.0

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

$$= \frac{2.5 \times 0.49 \overset{\times 18}{} \times 10 \times 400}{1.25}$$

$$= 62.7 \text{KN}$$

⇒ design bearing strength of joint = $n \times V_{dpb}$

$= 6 \times 62.7$

~~= 372 KN~~

$= 376.32$ KN

(3) Design tearing strength of plate

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

⇒ width of plate = B = $2\times40 + 2\times27$

$= ~~80~~ 134$ mm

At section ③-③

⇒ $A_n = (B - n d_h) t$

$= (134 - 3\times18) \times 10$

$= 80 \times 10 = 800 \text{ mm}^2$

$$T_{dp} = \frac{0.9 \times 800 \times 410}{1.25} = 236.16 \text{ KN}$$

∴ Design strength of ~~materiale~~ of joint

= Lesser of V'_{dsb} , V'_{dpb} & T_{df}

= 174 kN

∴ maximum force that can be transmitted = 174 kN

D.27.01.2020

➢ ~~Straight~~ state the assumption in design of bearing bolts

> The stress distribution on the plates betw^n the bolt hole is uniform.

> The friction betw^n the plate is negligible

> The shear in stress is uniformly distributed ~~&~~ over the cross-section of the bolt.

> The bolts in a group share the direct ~~bolt~~ load equally

> Bending stresses develop in the bolts is neglected.

What are the advantages of butt joint over lap joint

=> There is no eccentricity develop in butt joint so load carrying capacity is higher that lap joint because in lap joint some eccentricity occurs which may cause bending.

> Butt joint is permanently leak proof and strong

Shear strength of HSFG bolts

$$\boxed{V_{df} = \frac{V_{nsf}}{\gamma_{mf}} = \frac{\mu_f n_e K_h F_0}{\gamma_{mf}}}$$

where

μ_f = slip factor (or) friction co-efficient

n_e = number of interfaces offering frictional resistance to slip

K_h = Constant best of type of hole

F_0 = proof load => minimum bolts tension at installation

$\boxed{F_0 = f_0 A_{nb}}$ where, f_0 = proof stress

f_o = Proof stress = 0.7 fub

fub = ultimate tensile stress of bolt

γ_{mf} = 1.1 for slip resistance design at service load

= 1.25 for slip resistance design at ultimate load.

WELDED CONNECTION

Advantages of welded connection over bolted connection

> welded joint are economical because bolt materials are eliminated.

> welded structures are more rigid ~~contact~~ compare to bolted joint.

> As this joint is rigid joint, moment is less shear force is less, less deflection and less radious of gyration. So 100% efficiency can be achieved by using this joint

> lighter in weight so less cost.

> Alteration can be done with less expenses in case welding as compare to bolting.

> The process of welding is quicker in comparasion between welding....

> The process of welding is silent where as in case of riveting a lot of noise is produced.

Disadvantages

> More skilled person is required to make welded joint as compared to welded joint.

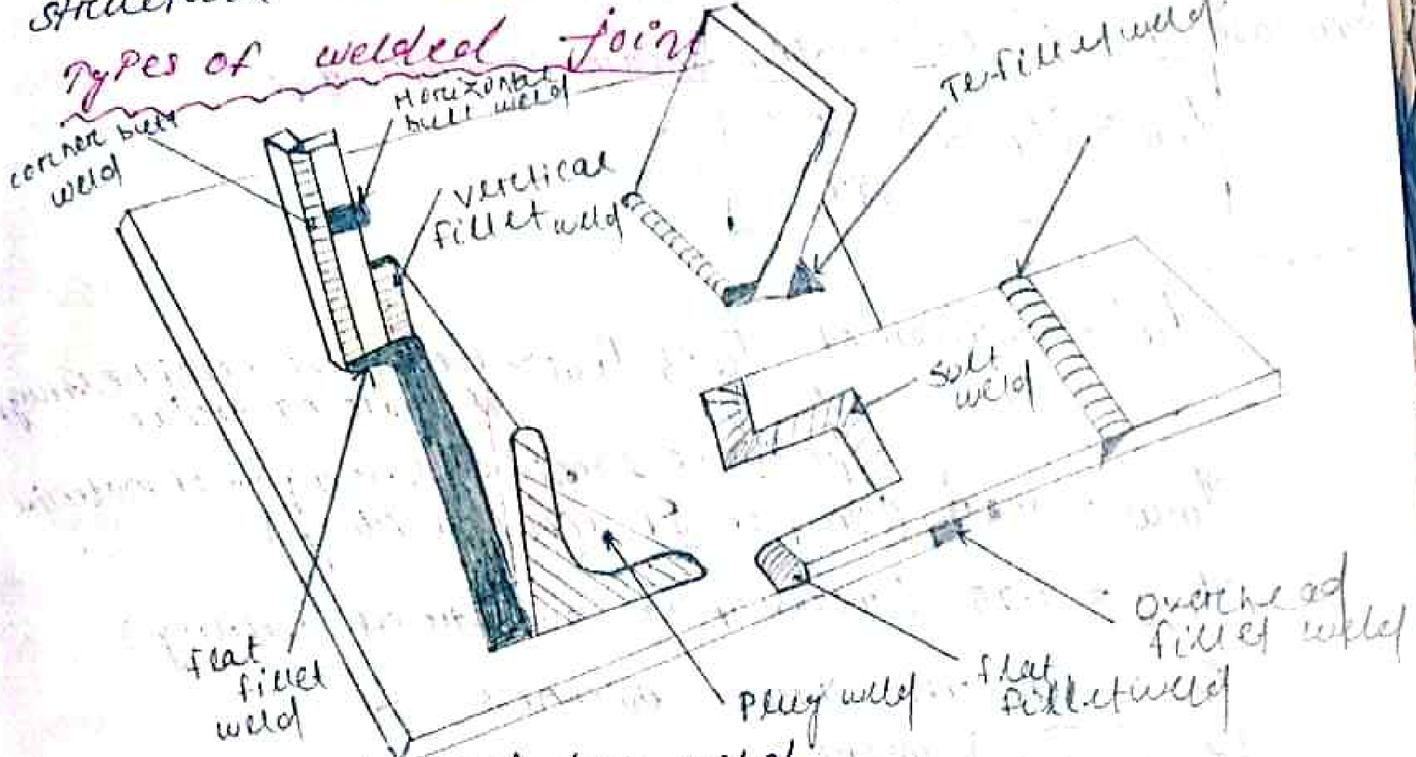
> The inspection of welded joint is difficult and expensive where as bolted joints can be inspected simply by tapping a joint with a hammer.

> Welded structures are brittle in nature so under dynamic load, welded connection is first fai then bolted connection.

~~→ fatigue is less of a welded struct~~

→ fatigue strength is less in case of welded structural as compare to bolted connection.

Types of welded joint



(i) fillet weld (or) lap weld

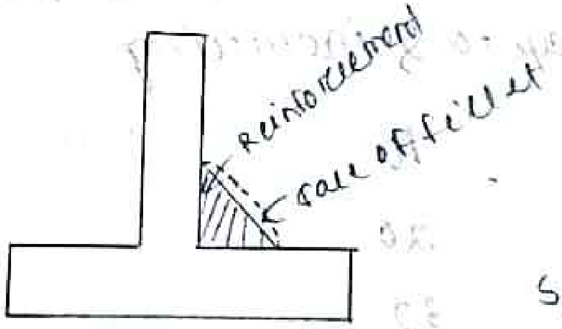
(ii) Groove weld (or) butt weld

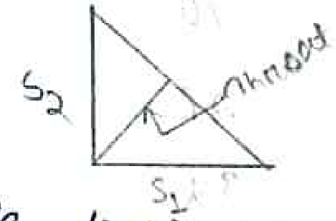
(iii) plug and slot weld

(i) fillet weld (or) lap weld

It is provided when fusion faces are perpendicular in two different plane.

Typical cross-section of fillet weld.





NOTE

(1) Standard cross-section of lap weld = Right angle triangle.

(2) Standard angle for fillet weld section = 45°

3) fillet weld symbol = ◣

4) fillet welds are effective in resisting the shear stresses.

(5) Throat of fillet weld is weakest section of fillet weld.

(6) Design strength of fillet weld

In case of fillet weld, shear is critical

$$\boxed{P_{dw} = (t_t\ l_w)\ \frac{f_u'}{\sqrt{3}\,\gamma_{mw}}}$$

where,

f_u' = smaller of f_u & f_{uw} → ultimate tensile strength of plate material

f_u ↓ ultimate tensile strength of material

$\gamma_{mw} = 1.5$ (in case of field welding)

$\gamma_{mw} = 1.25$ (workshop (or) industrial welding)

t_t = effective throat thickness

$\boxed{t_t = KS}$ where, S = size of weld

Angle between fusion faces(θ)	60-90	91-100	101-106	107-113	114-120
K	0.7	0.65	0.6	0.55	0.5

D. 28.02.2020

Minimum size of weld

	Thickness of the thicker plate	Smin (mm)
Above	up to & including	
0	10	3
10	20	5
20	32	6
32	50	8 (1st run) 10 (2nd run)

Maximum size of the weld (S_{max})

$S_{max} = \frac{3}{4} \times t_{min} = 0.75 t_{min}$ (for ~~square~~ rounded edges)

$S_{max} = t_{min} - 1.5$ {for ~~rounded~~ square edges)

t_{min} = minimum thickness of the ~~plate~~ thinner connected member

$t_t l_w$ = effective Area (A_e)

l_w = effective length of weld $= L_w - 2S$

L_w = overall length of weld

$2S$ = end return

End return

→ An excess length of the weld on the $\perp$ar case for a length of 2S is provided to avoid the initiation weld failure from the edge. This is known as end return.

→ This provision applies in the particularly fillet weld in tension.

Reduction factor for long joint

If $L_j > 150 t_t$ then $\beta_{lj} = 1.2 - \frac{0.2 L_j}{150 t_t}$

$$\boxed{\beta_{lj} \le 1}$$

L_j = length of side fillet weld parallel to the direction of a load in a member

$$\boxed{P_{dw} = (t_t \cdot l_w) \frac{f_u}{\sqrt{3}\,\gamma_{mw}} \times \beta_{lj}}$$

Specification for fillet weld

→ length of a parallel weld on each side should not be less than β (width of a plate)

$L_{parallel} \nless \beta$

→ effective length of weld should not be less than 4s

$l_w \nless 4s$ (or) 40 mm
(which ever is less)

→ $l_0 \nless 4t$ (or) 40 mm
(which ever is more)

l_0 = overlap length of weld
t = thickness of thinner connected member.

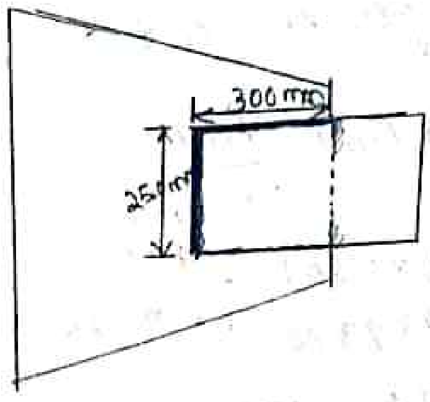
→ $\beta \nless 16t$

If $\beta > 16t$, slot & plug weld are provided

Que-1 Design a welded lap joint for two plates of size 120 mm x 8mm & 120mm x 12mm for maximum efficiency. Assume shop welding and fe 410 grade of steel.

Que-2 A tie member in a truss girder is 250mm x 14mm in size. It is welded to a 10mm thick gusset plate by a fillet weld. The over lap of the member is 300mm & weld size is 6mm. Determine the design strength of joint, if the welding is done as shown in the fig. what is the increase in that length of joint, if the welding is done all arround. Assume shop welding.

Qus-2



Given data

Fe 410 grade steel

$f_u = 410$ Mpa ; $f_y = 250$ Mpa

shop weld, $\gamma_{mw} = 1.25$

effective length of weld, $l_w = 2 \times 300 + 250 = 850$ mm

effective ~~the~~ throat thickness $= t_t = ks$
$= 0.7 \times s$

$S_{min} = 5$ mm

$S_{max} = \frac{3}{4} \times t_{min}$

$= \frac{3}{4} \times 10$

$= 7.5$

$\therefore$ size of weld $= 6$ mm

rounded edges

$t_t = ks$
$= 0.7 \times 6$
$= 4.2$

Solution

$$P_{dw} = (t_t l_w) \frac{f_u}{\sqrt{3}\,\gamma_{mw}}$$

$$= (4.2 \times 850) \times \frac{410}{\sqrt{3} \times 1.25}$$

$= 676059$ N

→ $\boxed{P_{dw} = 676 \text{ kN}}$

When welding is done all arround

effective length of weld = l_w = $2\times300+2\times250 = 1100$ mm

$$P_{dw} = (t_t\, l_w)\,\frac{f_u}{\sqrt{3}\,\gamma_{mw}}$$

$$= (4.2\times1100)\,\frac{410}{\sqrt{3}\times1.25}$$

$= 874893$ N

$= 874.$ KN

∴ Increase in strength of weld $= 874-676$

$= 198$ KN

Qs 1 D.29.01.2020

Design a welded lap joint for two plates of size 120mm × 8mm and 120mm × 12mm for maximum efficiency. Assume shop welding and Fe 410 grade of steel.

Given data

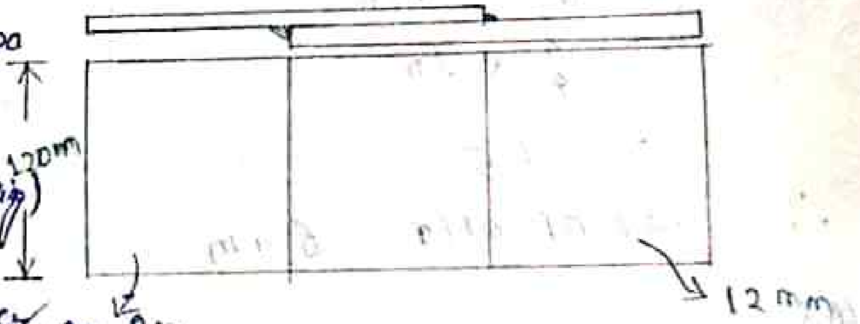
2 plates (120×8) mm

$f_y = 250$ MPa (120×12) mm

f_e 410 ⇒ $f_u = 410$ MPa

$\gamma_{m1} = 1.25$

$\gamma_{mw} = 1.25$ (shop welding)



Soln

For maximum efficiency, $\eta = 100\%$

$$\eta = \frac{\text{Design strength of joint}}{\text{Design strength of solid plate welded}}\times100$$

$$\Rightarrow 100\% = \frac{\text{Design strength of joint}}{\text{Design strength of solid plate}}\times100$$

$$\boxed{\Rightarrow \text{Design strength of welded joint} = \text{Design strength of solid plate}}$$

design strength of solid plate = $T_{dsp} = A_g \dfrac{f_y}{\gamma_{m0}}$

$= (120\times8)\dfrac{250}{1.1}$

$= 218181.82\,N$

$= 218.18\,kN$

design strength of welded joint = $P_{dw} = (l_w t_t)\dfrac{f_u'}{\sqrt{3}\,\gamma_{mw}}$

for maximum efficiency.

$\Rightarrow \boxed{P_{dw} = 218.2\,KN}$

$t_t = KS$

$K = 0.7$

$S_{min} = 5 min$

$S_{max} = 8 - 1.5$

$= 6.5\,m$

$\therefore S = 6\,mm$

$t_t = KS$

$= 0.7\times6$

$\Rightarrow \boxed{t_t = 4.2}$

$\Rightarrow P_{dw} \Rightarrow (l_w t_t)\dfrac{f_u'}{\sqrt{3}\gamma_{mw}} = 218.2\times10^3$

$\Rightarrow P_{dw} \Rightarrow (l_w\times4.2)\dfrac{410}{\sqrt{3}\times1.25} = 218.2\times10^3$

$\Rightarrow P_{dw} \Rightarrow (l_w\times4.2)\times\overset{189.37}{\cancel{328}} = 218.2\times10^3$

$\Rightarrow P_{dw} \Rightarrow l_w = 795.35 = 218.2\times10^3$

$\Rightarrow \boxed{l_w = 274.34\,kN}$

lap welded joint will be done at both top & bottom.

~~274.32 > l~~

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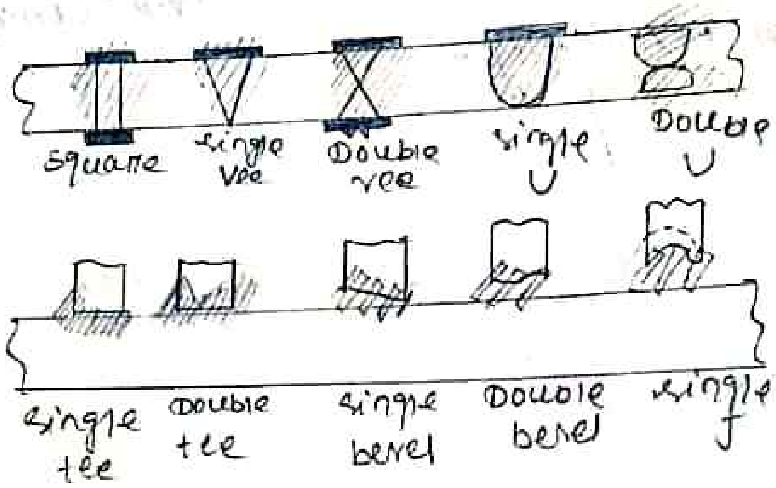
D.31.01.2020

Butt weld (or) Grove weld

> This type of bolt is usually design for direct tension compression and shear also. but they are design for axial stresses for only under concentric or axial load.



NOTE

> Butt weld connection is free from eccentricity

> Butt weld fails only due to axial stresses.

3) ① Design axial strength of butt weld (or) Design strength of butt weld :-

$$\boxed{P_{dw} = (l_w t_e)\left(\frac{f_y'}{\gamma_{mw}}\right)}$$

where,

f_y' = Smaller of yield stress of weld (f_{yw}) and yield stress of parent material (f_y)

t_e = Effective throat thickness

l_w = Effective length of weld.

② Design shear strength of butt weld (V_{dw}) (if any)

$$\boxed{V_{dw} = (L_w t_e) \frac{f_y}{\sqrt{3}\, \gamma_{mw}}}$$

Size of Butt weld

→ Size of Butt weld specified by throat dimension, is called effective throat thickness (t_e)

$\boxed{t_e = \frac{5}{8} t}$ { For single weld, partially penetrated weld like single vee, single U, single J weld etc.

$\boxed{t_e = t}$ [For double V, U weld ; fully penetrated weld]

problem

Two plates of 16mm and 14mm thickness are to be joint by a groove weld. The joint is subjected to a factored tensile force of 430 kN. Due to some reasons the effective length of weld that could be provided was 175 mm only. Check the safety of the joint, if

(a) single V groove weld is provided

(b) Double V groove weld is provided

Assume the plate should be shop weld.

sol^n

Given data

Two plates, thickness = 16 mm & 14 mm

factored tensile force = 430 kN = T_d

Assume Fe410 grade steel

f_u = 410MPa, shop weld ⇒ γ_{mw} = 1.25

f_y = 250 mpa

Scanned by CamScanner

Effective length of ~~bolt~~ weld = l_w = 175 mm

(a) Single - V groove weld is provided

$t = 14$ mm

$t_e = \frac{5}{8} t$ (Always single v weld is treated as a partially penetrated weld.

t_e = effective throat thickness

t = thickness of thinner plate.

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

$\Rightarrow \boxed{t_e = 8.75 \text{ mm}}$

⇒ Design axial tensile strength of butt weld

$$T_{dw} = (l_w t_e)\left(\frac{f_y'}{\gamma_{mw}}\right)$$

$$= (175 \times 8.75)\left(\frac{250}{1.25}\right)$$

$$= 306250 \text{ N}$$

$$\Rightarrow \boxed{T_{dw} = 306.25 \text{ kN}} < 430.$$

∴ So, it is unsafe or which is inadequate.

(b) Double - V - groove weld is provided

effective throat thickness (t_e) = thickness of thinner plate

$t_e = t$

$t_e = 14$ mm

Design ~~st~~ axial tensile strength of butt weld

$$T_{dw} = (l_w t_e)\left(\frac{f_y'}{\gamma_{mw}}\right)$$

$$= (175 \times 14)\left(\frac{250}{1.25}\right)$$

$$\Rightarrow \boxed{T_{dw} = 490 \text{ KN}} < 430$$

∴ So, it is safe (or) which is adequate.

D·03·02·2020

Tension Members

A structural member subjected to two pulling tensile forces applied at its ends is called tension member. The member & connections are arranged that eccentricity in the connection & bending stresses on the member are not developed.

Example of tension joint

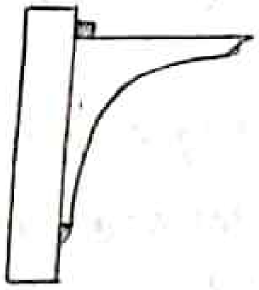
(i) wire & cable, (ii) bars & rods (iii) ties (iv) plates & flat bars.

common shapes of tension member

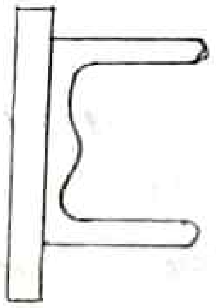
> open section such as angle, channel & I-section.

> closed section such as circular, square, rectangular, & hollow section.

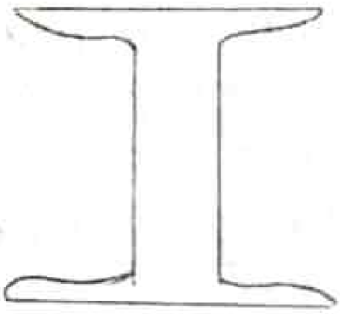
> compound & up sections such as double angle double channel with or without additional plates.



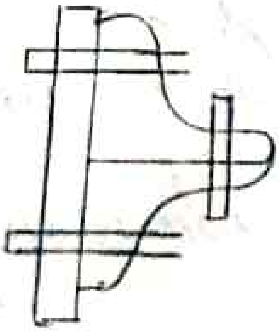
(open section)

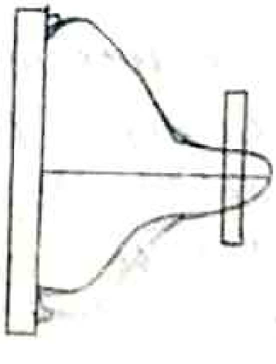


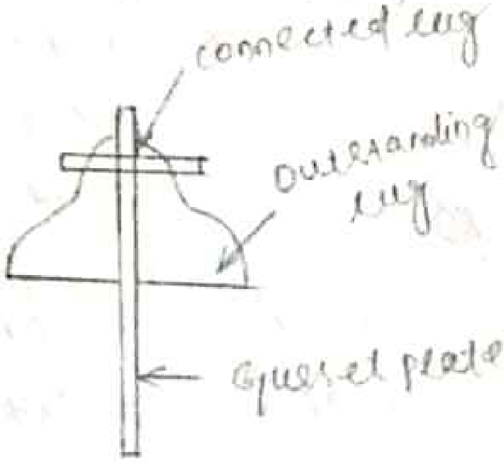
(closed section)



(I-section)







Types of failure in tension member

A tension member is subjected to factored or desig. tension load may change to fail by 3 possible modes.

(1) Gross section yielding failure.

(2) Net section rupture failure (tearing or fracture failure)

(3) Block shear failure.

D. 06.02.2020

NOTE

→ If a flat is connected by a single row of bolts the various possible failure are: Gross section yielding & Net section rupture failure.

→ If a flat is connected by two rows of bolts, the various possible failure are gross section yielding failure, Net section rupture failure & block shear failure.

→ If an angle section is connected by a single row of ~~column~~ bolt, the various possible failure are yielding, rupture & Block shear failure.

→ If an angle section is connected by two rows of bolts, the various possible failure are yielding, rupture & Blockshear failure in two ways.

→ In tension member generally Flat section is best ~~B~~ section & angle section longer ~~cake~~ leg connected to plate is most suitable for taking care of reversal load.

Design strength of Tensile member

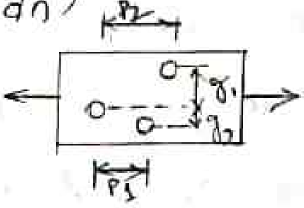
(1) Due to gross section yielding (T_{dg})

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

where, A_g = Gross area = $B \times t$

(2) Due to Net section rupture (T_{dn})

(a) For Flats & plates

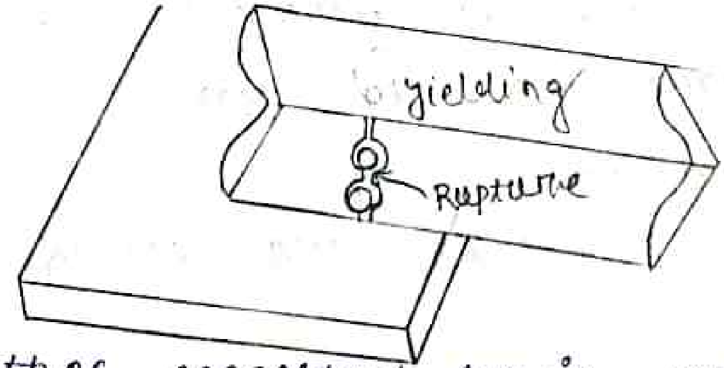


$$\boxed{T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}}}$$

$A_n = (B - n d_h)\, t$ (For chain pattern bolting)

$A_n = \left[B - n d_h + \frac{P_1^2}{4g_1} + \frac{P_2^2}{4g_2} + \cdots\right]$ (For staggered pattern bolting)

(b) For angle, tee section, channel section etc



T_{dn} = (strength of connected leg in net section rupture) + (strength of outstanding leg in gross-section yielding)

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

where, A_{nc} = connected net area

β = shear lag factor

A_{go} = gross area of the outstanding leg

$$\boxed{\beta = 1.4 - 0.076 \left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L_c}\right)}$$

w = width of the outstanding leg

b_s = shear lag width

L_c = length of end connection, that is the distance between the outermost bolts in the end joint measured along the load direction or length of the weld along the load direction.

for priliminary design, the rupture strength of net section may be approximately taken as:

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

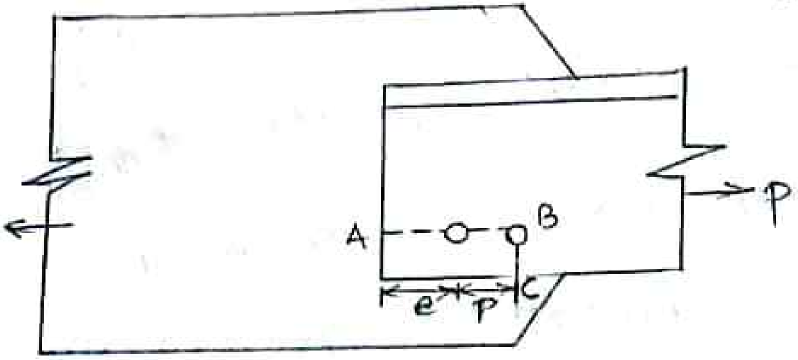
where, α = 0.6 for one two bolts, 0.7 for three bolts & 0. for four or more bolts along the length in the end connection or equivalent weld length.

t = thicness of leg

A_n = net area of total cross section.

(3) Due to Block Shear (T_{db})

AB = Shear plane
BC = tension plane



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

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

where,

A_{vg}, A_{vn}, ~~A_{tg}~~ = Minimum gross & minimum net sectional area of shear planes respectively

A_{tg}, A_{tn} = Minimum gross & net ~~area~~ sectional area of tension planes respectively

∴ T_{db} = Smaller of T_{db_1} & T_{db_2}

$$A_{vg} = \left[(e+P)\,t\right] \times 2$$

$$A_{vn} = \left[\left(e+P-d_h-\frac{d_h}{4}\right) t\right] \times 2 \quad , A_{tg} = p \times t$$

$$A_{tn} = \left[P \times t - \left(\frac{d_h}{4} + \frac{d_h}{4}\right) \times t\right]$$

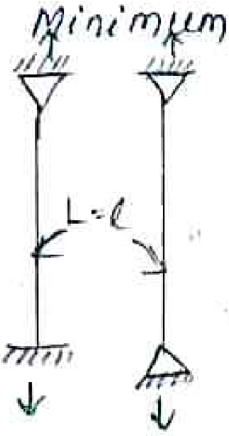
∴ Design strength of a tension member = ~~Least~~ smaller of T_{dg}, T_{dn} & T_{db}

D.10.02.2020

Maximum values of slenderness ratio (λ_{max}) of tension member

$$\lambda = \frac{\text{effective length of tension member } (l)}{\text{minimum radius gyration } (r_{min})}$$

$$= \frac{\text{unsupported length of tension member } (l)}{\text{Minimum radious of gyration } (r_{min})}$$



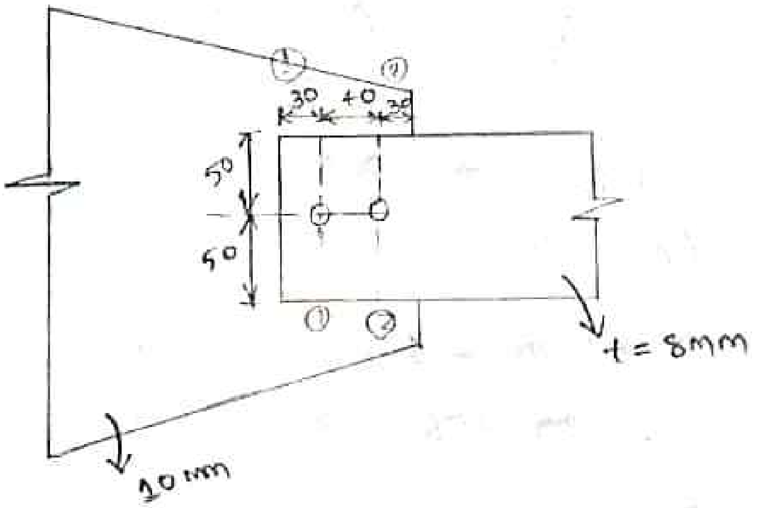
$$\boxed{r_{min} = \sqrt{\frac{I_{min}}{A}}}$$

Member	λ_{max} (or) λ_{limit}
(i) If a tension member is subjected to reversal of direct stresses / loads due to loads other than wind (or) seismic forces	180
(ii) If a tension member (tie in roof trusses & bracing system) subjected to reversal of stresses due to the effect of wind (or) seismic forces.	350
(iii) If any other tension member (other than pretensioned member)	400

problem

D. 11.02.2020

Q1 A tension member consist of ~~a~~ flat 100mm × 8 mm which is connected to a gusset plate of 10mm thick by ~~two~~ 2 numbers 16mm dia bolt. As shown in fig. Determine the strength of the flat against yielding, rupture & block shear. Also determine the maximum load the joi ~~can~~ can carry safely. Assume steel of grade Fe410 & bearing bolts of property class 4.6 in the field



Given data

Width of flat = B = 100mm

thickness of flat = t = 8mm

Gusset plate thickness = 10mm

2 no. of 16mm dia.

$d = 16\ mm \Rightarrow d_h = 16 + 2 = 18mm$

$Fe\ 410 \Rightarrow f_u = 410\ \frac{N}{mm^2}$

$f_y = 250\ \frac{N}{mm^2}$

$\text{property class} = 4.6 \Rightarrow f_{ub} = 400\ MPa$

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

(a)

$A_{sb} = \frac{\pi}{4} \times 10^2 = 201$

(i) Design strength of flat against yielding

$$T_{dg} = \frac{A_g f_y}{\gamma_{m0}} = \frac{(B \times t) \times f_y}{\gamma_{m0}} = (100 \times 8) \times \frac{250}{1.1}$$

$$T_{dg} = 181818.18 \text{ N}$$

$$\Rightarrow \boxed{T_{dg} = 181.82 \text{ kN}}$$

(ii) Design the strength of the flat against rupture

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

$$d_h = 18 \text{ mm}$$

$$A_n = (B - n d_h)\, t$$

$$= (100 - 1 \times 18) \times 8 \quad [\because n = 1]$$

$$= 656 \text{ mm}^2$$

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}} = \frac{0.9 \times 656 \times 410}{1.25}$$

$$= \cancel{0.9 \times \overset{656}{64} \times 410}$$

$$= 193651.2 \text{ N}$$

$$\Rightarrow \boxed{T_{dn} = 193.65 \text{ KN}}$$

ii) Design strength of flat against Block shear :-

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

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

~~$A_{tg} = (40\times100)\,8$~~

$A_{tg} = (40)8 = 320\ mm^2 \Rightarrow \boxed{A_{tg} = 320\ mm^2}$

$A_{tn} = p\times t - \left(\frac{d_h}{4} + \frac{d_n}{4}\right)\times t$

$= 40\times 8 - \left(\frac{18}{4} + \frac{18}{4}\right)\times 8$

$\Rightarrow \boxed{A_{tn} = 248\ mm^2}$

$\Rightarrow A_{vg} = \left[(e+p)\,t\right]\times 2$

$= \left[(30+40)\,8\right]\times 2$

$\Rightarrow \boxed{A_{vg} = 1120\ mm^2}$

$A_{vn} = \left[\left(e+p-d_h-\frac{d_h}{4}\right)t\right]\times 2$

$= \left[\left(30+40-18-\frac{18}{4}\right)\times 8\right]\times 2$

$\Rightarrow \boxed{A_{vn} = 760\ Mm^2}$

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

$$= \left[\frac{1120\times 250}{\sqrt{3}\times 1.1} + \frac{0.9\times 248\times 410}{1.25}\right]$$

$= 220171.48\ kN$

$\Rightarrow \boxed{T_{db1} = 220.17\ KN}$

$$T_{db_2} = \left[\frac{A_{vg} f_y}{\gamma_{m0}} + \frac{0.9\, A_{vn} f_u}{\sqrt{3}\, \gamma_{m1}}\right]$$

$$= \left[\frac{320 \times 250}{1.1} + \frac{0.9 \times 760 \times 410}{\sqrt{3} \times 1.25}\right]$$

$= \cancel{297079.27\ N} = 202256.76$ N

$\Rightarrow \cancel{T_{db_2} = 297\ KN} \Rightarrow \boxed{T_{db_2} = 202.025\ KN}$

$\therefore$ T_{db} = smaller of T_{bd_1} & T_{bd_2}

$\Rightarrow \boxed{T_{db} = 202.25 KN}$

$\therefore$ Design strength of tension member = smaller of T_{dg}, T_{dn} & T_{db}

$\therefore$ Design strength of tension member = T_{dg}
= 181.82 kN

Determine the maximum load of the joint can carry safely.

(1) Shear strength of 1 bolt = $V_{dsb} = \frac{f_u}{\sqrt{3}\, \gamma_{mb}} (n_n A_{nb} + n_s A_{sb})$

$= \frac{400}{\sqrt{3} \times 1.25} (0 + 1 \times 201)$

$= 37135.17$ N $= 37.13$ kN

$\Rightarrow$ shear strength of joint $V_{dsb}' = 2 \times V_{dsb} = 2 \times 37.13 = 74.26$ kN

(ii) Bearing strength of 1 Bolt

$$V_{dpB} = \frac{2.5 K_b d t f_u}{\gamma_{m1}}$$

K_b = smaller (i) $= \frac{e}{3d_0} = \frac{30}{3 \times 18} = \frac{5}{9} = 0.54$

(ii) $\frac{p}{3d_0} - 0.25 = \frac{40}{3 \times 18} - 0.25 = 0.49$

D.12.02.2020
1st half

Ch-05 STEEL COLUMN BASES && FOUNDATIONS

Design compressive stress in a concrete footing is much smaller than it is ~~a~~ in a steel column, so it is necessary to distribute the load from steel column and evenly to the footing below.

Types of column Bases

(1) Slab base
(2) Gusseted base
(3) Grillage foundation

(1) Slab Base

Suitability

→ It is most suitable for the column subjected to axial load.

→ It is suitable for column subjected to light or less intensity of loads.

→ A slab base is assumed to be hinged (Pinned) base which can not resist lateral load.

D.13.02.2020

NOTE

Design bearing strength of concrete = 0.45 fck

~~Procedure for slab base~~

$A_{nb} = 0.78\ A_{sb}$

$= 0.78 \times \frac{\pi}{4} \times 20^2$

$= 245$

Design strength against yielding $= T_{dg} = T_d$

$\Rightarrow \frac{A_g f_y}{\gamma_{m0}} = T_d$

$\Rightarrow \frac{A_g \times 250}{1.1} = 225 \times 10^3 N$

$\Rightarrow 250 A_g = 247500$

$A_g = 990\ mm^2$

$\Rightarrow \boxed{A_g = 9.9 cm^2}$

Let us check the section ISA (100× 75×6) mm

shear strength of one bolt in single shear $= V_{dsb}$

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

$\Rightarrow V_{dsb} = \frac{400}{\sqrt{3} \times 1.25} (1 \times 245 + 0)$

$\Rightarrow V_{dsb} = 45264.26 N$

$\Rightarrow \boxed{V_{dsb} = 45.26\ KN}$

Bearing strength of one bolt $= V_{dpb} = \frac{2.5 k_b d t f_u}{\gamma_{mb}}$

Assume $e = 33$, $p = 60$

$\Rightarrow k_b$ smaller of

(i) $\frac{e}{3d_0} = \frac{33}{3\times22} = 0.5$

(ii) $\frac{p}{3d_0} - 0.25 = \frac{60}{3\times22} - 0.25 = 0.66$

(iii) $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.97$

(iv) 1.0

$\boxed{k_b = 0.5}$

$V_{dpb} = \frac{2.5 \times 0.5 \times 20 \times 8 \times 400}{1.25} = 64000N$

$\Rightarrow \boxed{V_{dpb} = 64\ KN}$

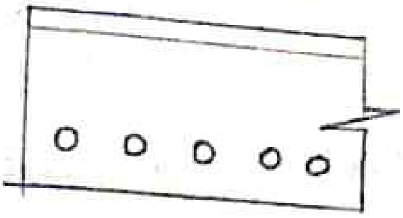
$V_{dpb} = \frac{2.5 \times 0.5 \times 20 \times 6 \times 400}{1.25} = 48000 N \Rightarrow \boxed{V_{dpb} = 48 kN}$

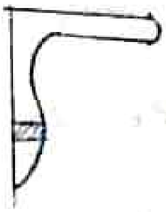
∴ Design strength of bolt = smaller of V_{dsb} & V_{dpb}

⇒ Design strength of bolt = 45.26 kN

No. of bolt required to resist the factored tensile force

$(225 kN) = \frac{225}{45.26} = 4.97 \simeq 5 nos$.





Check

$A_g = 13.36 cm^2$

$= 1336 mm^2$

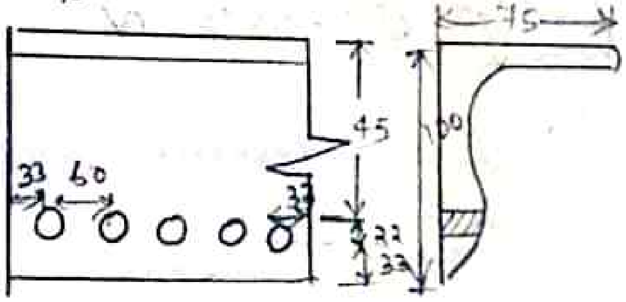
(1) Strength against yield $= T_{dg} = \frac{A_g f_y}{\gamma_{m0}} = \frac{1336 \times 250}{1.1}$

$\boxed{T_{dg} = 303636.36 N}$

$\boxed{T_{dg} = 303.63 kN} > 225 kN$

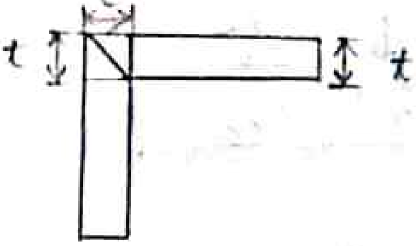
(2) Design strength against net section rupture $= T_{dn}$

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



$A_{nc} = \left(100 - 22 - \frac{8}{2}\right) 8 = 592 mm^2$

$A_{go} = \left(75 - \frac{8}{2}\right) 8 = 568 mm^2$



$\beta = 1.4 - 0.076 \left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L_c}\right)$

$b_s = w + w_1 - t$

$= 75 + 45 - 8 = 112 mm$

$L_c = 33 + 60 + 60 + 60 + 60 - (5 \times 22) = 163 mm$

$\beta = 1.4 - 0.076 \left(\frac{75}{8}\right)\left(\frac{250}{410}\right)\left(\frac{112}{163}\right)$

$\Rightarrow \boxed{\beta = 1.1}$

↓

D. 03.02..

↓

D. 03.03.2020

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

$$= \frac{0.9 \times 592 \times 410}{1.25} + \frac{1.1 \times 568 \times 250}{1.1}$$

$$= 316758.4 \text{ N}$$

$\Rightarrow T_{dn} = 316.75$ KN > 225 KN. (so safe)

Strength against block shear

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

$$A_{vg} = [(e+p)\, t] \times 2$$

$$= [(33+60)\, 8] \times 2$$

$\Rightarrow \boxed{A_{vg} = 1488 \text{ mm}^2}$

~~$A_{vn} = (e+p)$ Avg~~

$$A_{vn} = \left[\left(e+p-d_h-\frac{d_h}{4}\right) t\right] \times 2$$

$$= \left[\left(33+60-22-\frac{22}{4}\right) \times 8\right] \times 2$$

$\Rightarrow \boxed{A_{vn} = 1048 \text{ mm}^2}$

$$A_{tn} = \left[p \times t - \frac{d_h}{4} + \frac{d_h}{4}\right) \times t\Big]$$

$$= \left[60 \times 8 - \frac{22}{4} + \frac{22}{4}\right) 8\Big]$$

$\Rightarrow \boxed{A_{tn} = \overset{392}{\cancel{3840}} \text{ mm}^2}$

$$T_{db1} = \frac{1488 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times \overset{392}{\cancel{3840}} \times 410}{1.25}$$

$\Rightarrow \overset{T_{db1}}{\cancel{43288}} = 310967.76$ N

$\Rightarrow T_{db1} = 310.97$ KN →

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

$$A_{tg} = P \times t$$
$$= 60 \times 8$$
$$= 480 \text{ mm}^2$$

$$T_{db2} = \frac{0.9 \times 1048 \times 410}{\sqrt{3} \times 1.25} + \frac{480 \times 250}{1.1}$$

$$\Rightarrow T_{db_2} = 287705.53 \text{ N}$$

$$\Rightarrow \boxed{T_{db2} = 287.7 \text{ KN}}$$

T_{db} = smaller of T_{db1} & T_{db2}

$$T_{db1} = 310.97 \text{ kN}$$

$$T_{db2} = 287.7 \text{ kN}$$

$$\boxed{T_{db} = T_{db2} = 287.7 \text{ kN}}$$

Strength of beam shear = 287.7 kN > 225 kN

so, it is safe.

D. 10.02.2020

Compression Members

→ If two equal & opposite compressive forces applied at any structural member's end then that member is called as compression member

Ex - strut, principal rafter / strut, top chord of truss, bracing member, boom in crane, and column/post (stanchi

General failure of compression Member

> very short column or short strut is subjected to axial compressive load. May generally fail by yielding or crushing of material.

> very long column fail by elastic buckling in the Euler mode.

> intermediate column or strut generally fail by elastic buckling

Design compressive strength of Compression Members:

Design compressive strength $= P_d = f_{cd} \times A_e \quad (P < P_d)$

Where, P = External compressive load

$$\boxed{f_{cd} = \underset{\substack{\downarrow \\ xi}}{\chi} \cdot \frac{f_y}{\gamma_{m0}}} \leq \frac{f_y}{\gamma_{m0}}$$

(xi) → χ = stress reduction factor

$$\chi = \frac{1}{\phi + [\phi^2 - \lambda^2]^{0.5}}$$

ϕ = stress factor

λ = effective slenderness ratio

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

α = imperfection factor from table (pg. 35)

~~f_{cr}~~ =

λ = effective buckling slenderness ratio $= \sqrt{\frac{f_y}{f_{cc}}}$

$f_{cc} = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}$ = Euler buckling stress

Buckling classification (pg 44)

Q. What will be the buckling class of ISHB 400 @ 907 N/m about z-z & y-y axis?

ISHB 400 → overall depth; 907 N/m → weight/length of member

Ans from steel table ISHB 400 (taking 806 N/mm)

t_f - 12.7 mm

$h = 400$ mm

$b_f = 250$ mm

$\frac{h}{b_f} = \frac{400}{250} = 1.6 > 1.2$

for z-z axis buckling class — a

for y-y axis buckling class — b

Limiting (or) Maximum slenderness-ratio (By WSM & LSM; λ_{limit} (or) λ)

(1) A member carrying compressive loads ~~from load~~ from dead & imposed load. → 180

(2) A member carrying compressive loads due to only ~~weight or~~ wind load & seismic loads → 250

(3) for compressive flange or beam section or I-section, restrained against torsional buckling → 300

[long = slender (λ)]

D.04.03.2020

calculate the design compressive load for an ISHB 250 @ 536.6 $\frac{N}{m}$. 4m height. The column is restrained in erection only at both the ends. It is to be used as an uncased column in a single storey building.
uncased → open to the atmosphere

given data height of column = 4m = L
ISHB 250 @ 536.6 N/m

effective length = $l = 0.65L$
$= 0.65 \times 4$
$= 2.6$ m

ISHB 250 @ 536.6 N/m

⇒ Depth of section $(h) = 250$ mm
width of flange $= b_f = 250$ mm
thickness of flange $= t_f = 9.7$ mm
thickness of web $= t_w = 8.8$ mm

$$\frac{h}{b_f} = \frac{250}{\cancel{250}} = 1 \cancel{\leq} 1.2$$

$t_f = 9.7$ mm < 40 mm

$$\frac{h}{b_f} = \frac{250}{250} = 1 \leq 1.2$$

$t_f = 9.7$ mm < 100 mm

about zz axis ⇒ buckling class = b

about y-y axis ⇒ buckling class = c

$A_e = 69.71\ cm^2 = 0.006971\ m^2$

~~about Z-Z~~

about z-z axis (b)

Design compressive strength = $P_d = f_{cd} \times A_e$

$$f_{cd} = \chi \times \frac{f_y}{\gamma_{m0}}$$

$$\chi = \frac{1}{\left[\phi + (\phi^2 - \lambda)^{2^{0.5}}\right]}$$

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

$\alpha = 0.34$ (buckling class 'b')

$\lambda = \sqrt{\frac{f_y}{f_{cc}}}$ (Assume fe 410 steel $\Rightarrow f_u = 410 MPa$

$f_y = 250 N/mm^2$

$E = 2 \times 10^5$, $r_{zz} = 10.7 cm$
$= 107 mm$

$r_{yy} = 5.37 cm$
$= 53.7 cm$

$$f_{cc} = \frac{\pi^2 E}{\left(\frac{KL}{r_{zz}}\right)^2} = \frac{\pi^2 \times 2 \times 10^5}{\frac{0.65 \times 4}{107 mm}}$$

$$= \frac{\pi^2 \times 2 \times 10^5}{\left(\frac{0.65 \times 4 \times 1000}{107}\right)^2} = 3343.$$

$$\lambda = \sqrt{\frac{f_y}{f_{cc}}} = \sqrt{\frac{250}{3343}} = 0.27$$

$$\phi = 0.5\left[1 + \alpha(\lambda - 0.2) + \lambda^2)\right]$$
$$= 0.5\left[1 + 0.34(0.27 - 0.2) + 0.27^2\right]$$
$$= 0.548$$
$$\simeq 0.55$$

$$\chi = \frac{1}{\phi + (\phi^2 - \lambda^2)^{2^{0.5}}} = \frac{1}{0.55 + (0.55^2 - 0.27^2)^{0.5}} = 0.97$$

$$f_{cd} = \chi \times \frac{f_y}{\gamma_{m0}} = 0.97 \times \frac{250}{1.1} = 220.45 N/mm^2$$

$\gamma_{m0} = 1.1$

$$\boxed{f_{cd} = 220.45 N/mm^2}$$

$P_d = f_{cd} \times A_e$
$= 220.45 \times 6971$
$= 1536756.95 N = 1536 KN$

$A_e = 69.71 cm^2$
$= 6971 mm^2$

About YY axis $\Rightarrow$ buckling class = c $\alpha = 0.49$

$\lambda = \frac{f_y}{f_{cc}}$, $f_{cc} = \frac{\pi^2 E}{\left(\frac{KL}{r_{yy}}\right)^2} = \frac{\pi^2 \times 2 \times 10^5}{\left(\frac{2.6 \times 10^3}{53.7}\right)^2} = 842.$, $\lambda = \sqrt{\frac{f_y}{f_{cc}}} = \sqrt{\frac{250}{842}} = 0.54$

$\phi = 0.5\left[1 + \alpha(\lambda - 0.2) + \lambda^2\right] = 0.5\left[1 + 0.49(0.54 - 0.2) + 0.54^2\right] = 0.73$

$$f_{cd} = \frac{f_y/\gamma_{m0}}{\phi + [\phi^2 - \lambda^2]^{0.5}} = \frac{250/1.1}{0.73 + [0.73^2 - 0.54^2]^{0.5}} = 186.1.$$

Design compressive load $= P_d = f_{cd} \times A_e$
$= 186.1 \times 69.71 = 12973. = 12.97 N/mm^2$

D. 06.03.2020

Design a column section to ~~carrying~~ a working axial load of 400 kN. The column is 4 m long & effectively held in position & restrain against rotation at both ends. Consider fy-250 MPa

Given data

working axial Load = 400 KN

factored axis load = 400 × 1.5 = 600 KN

length of the column = 4m = 4 × 1000 = 4000 mm

f_y = 250 N/mm^2

fixed @ both end ⇒ K = 0.65

let us assume the column section is beam.

Assume slenderness ratio = 90 → λ

⇒ From code table → Table 9(a) ⇒ f_{cd} = 149 MPa about z-z axis

from table → Table 9(b) ⇒ f_{cd} = 134 MPa about y-y axis

∵ fy = 250 MPa
λ = 90
buckling class = a about zz axis
buckling class = b about y-y axis

∴ $\boxed{f_{cd} = 134 \text{ MPa}}$ (∵ Because about y-y axis buckling will occure first)

⇒ Required Area =

$$= \frac{\text{Factored Axial load}}{\text{Design compressive stress}} = \frac{600 \times 10^3 N}{134 N/mm^2} = 4477.61 mm^2 = 44.77 cm^2$$

let us take the section ISHB 200 @ 365.9 N/m

Area(A) = 47.54 cm^2, r_{zz} = 8.71 cm, r_{yy} = 4.51 cm

h = 200 mm, b_f = 200 mm, t_f = 9 mm

As $\frac{h}{b_f} = \frac{200}{200} = 1 \not> 1.2$ ⇒ It will not satisfied our assumption for buckling class

So again assume the section ISMB 250 @ 365.9 N/m

A = 47.55 cm^2, h = 250 mm, r_{zz} = 10.39 cm,

r_{yy} = 2.65 cm = 26.5 mm, b_f = 125 m, t_f = 12.5 mm

$$\frac{h}{b_f} = \frac{250}{125} = 2 > 1.2, \; t_f = 125 \leq 40 mm$$

Effective length of column = KL = 0.65 × 4000 = 2600

Effective slenderness ratio = $\frac{KL}{r_{yy}} = \frac{0.65 \times 4000}{26.5} = 98.11$

Scanned by CamScanner

buckling class (b) and f_y 250, $\frac{KL}{r_{yy}} = 98.11$

$\frac{KL}{r_{yy}}$		f_{cd}
90	$\longrightarrow$	134
98.11	$\longrightarrow$	
100	$\longrightarrow$	118

$$f_{cd} = 134 - \frac{134-118}{100-90} \times (98.11-90)$$
$$= 121.02\ N/mm^2$$

∴ the design compressive load $= P_d = f_{cd} \times A_e$

$$= 121.02 \times 4755$$
$$= 575450.1N$$
$$\Rightarrow \boxed{P_d = 575.\ KN} < 600 KN$$

so not safe
~~so not safe~~

∴ The section should be taken ISMB 300 @ 433.6 N/m

Then it will satisfied the load 600 KN.

effective length of column $= KL = 0.65 \times 4000 = 2600$.

$A = 56.26 mm^2$, $h = 300m$, $b_f = 140 mm$, $t_f = 12.4 mm$

$r_{zz} = 12.37 cm$, $r_{yy} = 2.84 cm = 28.4 mm$

effective slenderness ratio $= \frac{KL}{r_{yy}} = \frac{0.65 \times 4000}{28.4} = 91.55$

buckling class (b) of f_y 250, $\frac{KL}{r_{yy}} = 91.55$

$\frac{KL}{r_{yy}}$		f_{cd}
90	$\longrightarrow$	134
91.55		
100	$\longrightarrow$	118

$$f_{cd} = 134 - \frac{134-118}{100-90} \times (91.55-90)$$
$$= 131.51\ ~~KN~~\ N/mm^2$$

∴ the design compressive ~~load~~

$$P_d = f_{cd} \times A_e$$

$$= 131.51 \times \overset{5626}{~~4755~~}\ ~~56.26~~ = \overset{}{\cancel{625}}\ \underset{739}{} KN$$

~~625330N~~

$$\boxed{P_d = \overset{739}{\cancel{625}}\ KN} > 600 KN$$

D.12.02.2020
1st half

Ch-05 STEEL COLUMN BASES ~~&~~ & FOUNDATIONS

Design compressive stress in a concrete footing is much smaller than it is ~~a~~ in a steel column, so it is necessary to distribute the load from steel column and evenly to the footing below.

Types of column Bases

(1) Slab base
(2) Gusseted base
(3) Grillage foundation

(1) Slab Base

Suitability

→ It is most suitable for the column subjected to axial load.

→ It is suitable for column subjected to light or less intensity of loads.

→ A slab base is assumed to be hinged (pinned) base which can not resist lateral load.

D.13.02.2020

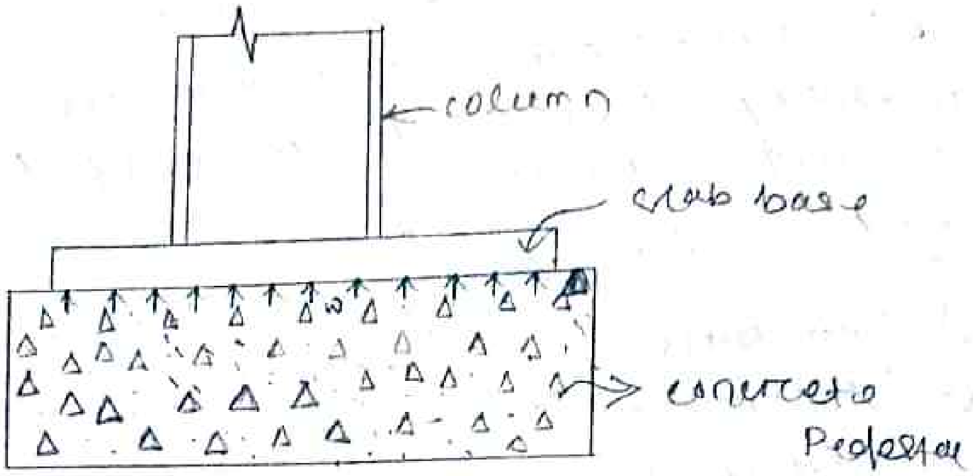
NOTE

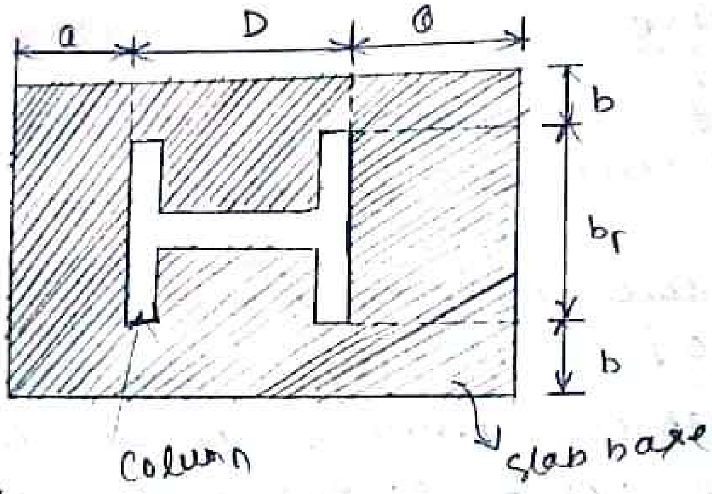
Design bearing strength of concrete = 0.45 fck

~~for slab base~~

Imp

Design procedure for slab base





D = overall ~~width~~ depth of column section

a = larger projection of slab base plate beyond column in mm

b = smaller projection of slab base plate beyond column in mm

b_f = width of the flange.

(i) Area of base plate required = $\boxed{A_{base} = \frac{P}{0.45 f_{ck}} = \frac{\text{Load}}{\text{Bearing strn}}}$

(ii) If a square base plate is provided, then the side of slab base = $\sqrt{A_{base}}$

(iii) If a rectangular slab base plate is provided, then

Area $\boxed{A = L \times B = D+2a)(b_f+2b)}$

To have optimum thickness of slab base, the condition should be $a = b$, so area - $A = (D+2a)(b_f+2a)$

$\Rightarrow a = ?$

(ii) upward pressure from concrete pedestal below the slab base (w) = factored axial load of the column (P) / Provided area of the slab base ($A_{provided}$)

$$w = \frac{P}{A_{provided}} \leq 0.45 f_{ck}$$

(iii) Minimum thickness of rectangular slab base =

$$t_s = \sqrt{\frac{2.5\, w\, (a^2 - 0.3b^2)\, \gamma_{m0}}{f_y}}$$

Q) Design a slab base for a column ISHB 350 @ 710.2 $\frac{N}{mm}$ subjected to a factored load of 1500 KN. The base plate is to raised on a concrete pedestal of M_{25} grade of concrete. provided welded connection between column & base plate @

Given data

ISHB 350 @ 710.2 $\frac{N}{mm}$

overall depth - D = 350 mm

Width of flange (b_f) = 250mm

Factored load = $P = 1500\ KN = 1500 \times 10^3\ N$

M25 concrete $\Rightarrow f_{ck} =$ ~~20~~ 25 MPa

Assume $f_u = 410$ MPa, $f_y = 250$ MPa

Area of base required $= A_{base} = \frac{P}{0.45 f_{ck}}$

$$= \frac{1500 \times 10^3}{0.45 \times 25}$$

$$\Rightarrow A_{base} = 133333.33\ mm^2$$

Let us provide a square slab~~to~~ base base

∴ ⇒ side of base plate = $\sqrt{A_{base}}$

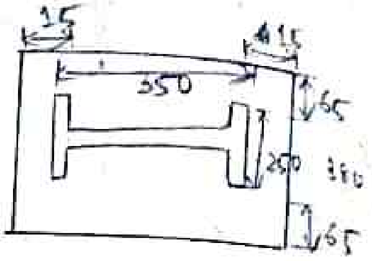
$= \sqrt{133333.33}$

$= 365.15$ mm

~~Let~~ Now provide a square slab base of size = 380 mm

⇒ provided Area = $(380)^2$

(Aprovided) = 144400 mm²



∴ Larger projection of slab base = a = $\frac{380-250}{2}$ = 65 mm

smaller projection of slab base = b = $\frac{380-350}{2}$ = 15 mm

~~Minimum thickness of rectangular~~

upward pressure from concrete pedestal below slab base = w = $\frac{P}{A_{provided}}$ (provided Area of the slab base)

$= \frac{1500 \times 10^3}{380^2} \leq 0.45 f_{ck}$

$= 10.38 \text{ N/mm}^2 \leq 0.45 \times 25$

$= 10.38 \text{ N/mm}^2 \leq 11.25$ (safe)

Minimum thickness of rectangular slab area

$$t_s = \sqrt{\frac{2.5w(a^2 - 0.3b^2)\cdot \gamma_{m0}}{f_y}}$$

$$= \sqrt{\frac{2.5 \times 10.38 (65^2 - 0.9 \times 15^2)\,1.1}{f_y}}$$

⇒ $\boxed{t_s = 21.8 \text{ mm}}$ ∴ size of the slab base $= (380)^2 \times 21.7$) mm $= 3133480$ mm

250

Actual effective length of weld (l_w) D. 15.02.2020

~~$= (250+250 \times 2 \times 11.6$~~ [thickness of flange $(t_f) = 11.6$]

$$= \left[\left(250+250+(350-2\times 11.6)\right)\right]\times 2$$

$$\Rightarrow \boxed{l_w = 1653.6 \text{ mm}}$$

effective throat thickness (t_t)

$$t_t = KS$$

$$S_{max} = \frac{3}{4} t_{min}$$

$$= \frac{3}{4} \times 11.6$$

$$= 8.7$$

$\therefore S_{min} = 5$

$\therefore$ size of weld $= 8$ mm

$$t_t = KS$$

$$= 0.7 \times 8$$

$$= 5.6 \text{ mm}$$

$\Rightarrow$ Strength of weld $= P_{dw}$

$$\therefore P_{dw} = (L_w t_t)\left(\frac{f_u}{\sqrt{3}\,\gamma_{mw}}\right)$$

$$\Rightarrow P_{dw} = P = (L_w t_t)\,\frac{f_u}{\sqrt{3}\,\gamma_{mw}}$$ [$\because P =$ factored load]

$$\Rightarrow 1500 \times 10^3 = (l_w \times 5.6)\cdot\frac{410}{\sqrt{3}\times 1.25}$$

$$\Rightarrow 1500\times 10^3 = (l_w \times 5.6) \times 189.37$$

$$\Rightarrow \frac{1500\times 10^3}{189.37} = (l_w \times 5.6)$$

⇒ 7921. = $l_w \times 5.6$

⇒ $l_w = \frac{7921}{5.6}$

⇒ l_w = 1414.46 mm

⇒ Required effective length of weld = 1414. mm ← (Say)

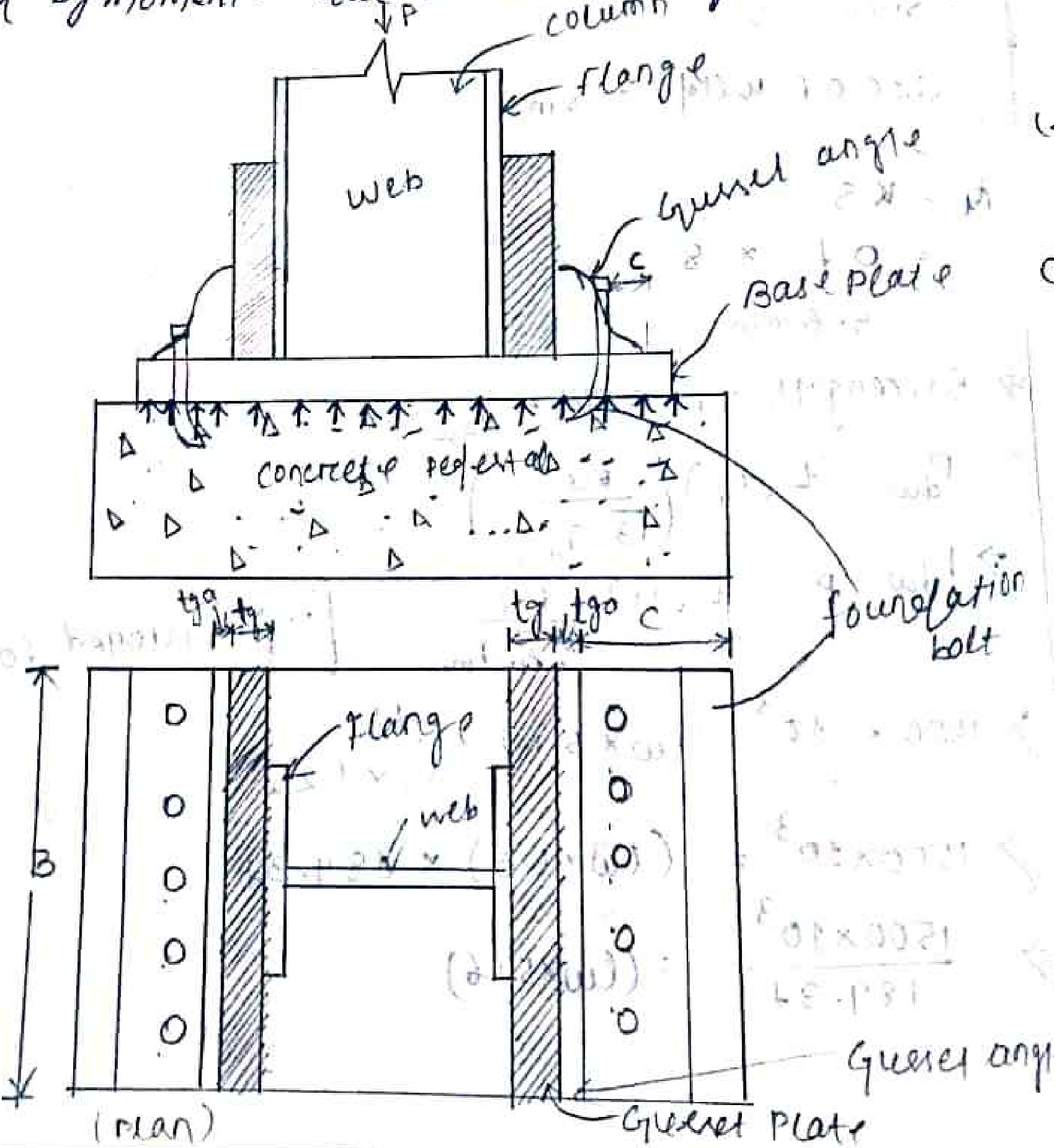
$< (l_w)_{actual}$ = 1653 mm

So, which is safe design.

D. 17.02.2020

(12) Gusseted Base

Gusseted base is normally used when steel column is subjected to heavy axial compressive loads as well as when steel column is subjected to axial loads & accompanied by moments due to eccentricity of loads.



B & L = width of and length of the base plate respectively

c = cantilever projection of the base plate from root of the gusset angle.

t_g = Thickness of gusset plate

t_{ga} = Thickness of gusset angle

D = Depth of steel column

b_f & t_f = width & thickness of column flange

(i) Area of base plate required = $A_{required}$

$$A_{required} = \frac{\text{Design axial compressive load (P)}}{\text{Design bearing strength of concrete}}$$

$$A_{base,\ required} = \frac{P}{0.45 f_{ck}}$$

$$\Rightarrow \left[A_{provided} = \text{more value than } A_{required}\right]^{*}$$

(ii) length of the base plate required

$$\boxed{L = D + 2(t_g + t_{ga} + e.)}$$

(iii) upward pressure from concrete pedestal below the base plate =

$$\boxed{w = \frac{\text{Design axial compressive load (P)}}{\text{Provided Area of the base plate (}A_{provided})}}$$

(iv) Thickness of gusset plate = $\boxed{t_g = c\sqrt{\frac{2.75w}{f_y}}}$

(v) Thickness of base plate =

$t_b = t_g$ (in case of welded connection)

$t_b = t_g - t_{ga}$ (in case of bolted connection

D. 18. 02. 2020

Ch-06

DESIGN OF STEEL BARS

→ A structural member subjected to transverse load (load ⊥ to its longitudinal axis) is called beam.

→ when provided in buildings to support roofs, they are called joint.

→ A large beam supporting a no. of joints is called girder.

→ Beam which carry the roof loads in trusses are called purlins.

→ Beam which support the loads from masonry over the openings are called lintels.

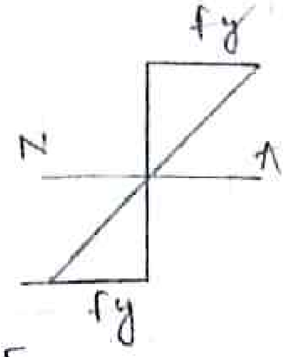
~~Beam~~

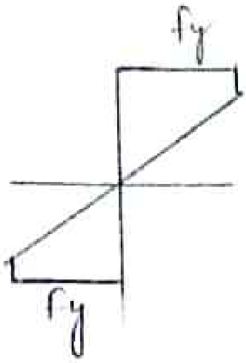
Plastic moment capacity of beam section (M_P) :-

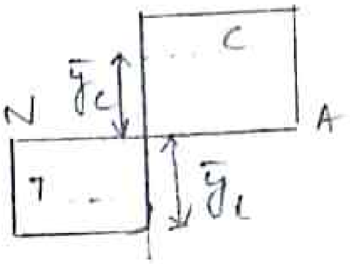
$$\boxed{M_P = f_y . Z_P}$$

where, Z_P = plastic section mod

f_y = yield stress material







$$\frac{M}{I} = \frac{f}{y} = \frac{E}{R}$$

$$\boxed{M = f\frac{I}{y} = f_y \cdot Z}$$

$$\boxed{M_p = f_y \cdot Z_p}$$

$$\boxed{\begin{aligned} Z_p &= \frac{A}{2}\left(\bar{y}_c + \bar{y}_t\right) \\ &= A_c\bar{y}_c + A_t\bar{y}_t \end{aligned}}$$

A_c & A_t are Area under compression and tension respectively.

$\bar{y}_c$ & $\bar{y}_t$ are centroidal distance of compression & tension area from the plastic neutral axis respectively.

Area under compression = Area under tension.

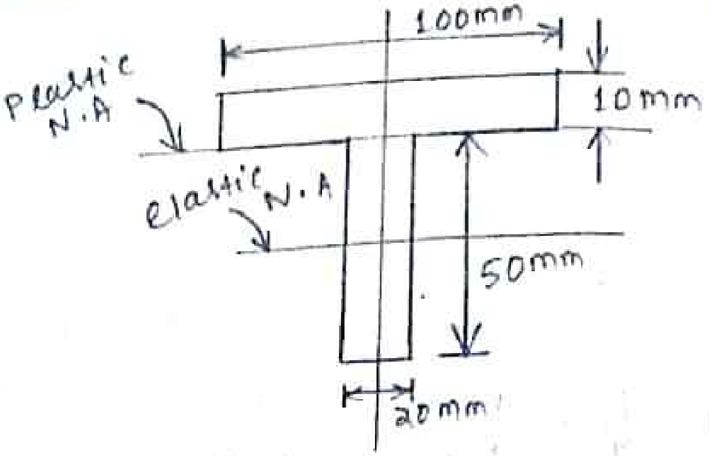
NOTE

→ Plastic neutral axis divides the total area into two equal halves.

→ Elastic neutral axis always pass through the C.G.

→ For symmetrical section (about x-axis) the elastic N-A & plastic N-A will coincide.

→ For unsymmetrical section (about x-axis) the elastic N-A and plastic N-A will not coincide.



Elastic N.A = $(\bar{y}) = \frac{A_1 y_1 + A_2 y_2}{A_1 + A_2}$

from bottom

$$= \frac{1000 \times (50 + \frac{10}{2}) + (50 \times 20) \times (\frac{50}{2})}{1000 + 1000}$$

= 40 mm

Plastic N.A from bottom = 50 mm from bottom

$A = (100 \times 10) + (50 \times 20) = 2000$

$\frac{A}{2} = \frac{2000}{2} = 1000$

Ex

$M_p = f_y Z_p$

$= f_y \left[\frac{A}{2} (\bar{y}_c + \bar{y}_t) \right]$

$= f_y \left[\frac{bh}{2} \left(\frac{\frac{h}{2}}{2} + \frac{\frac{h}{2}}{2} \right) \right]$

$M_p = f_y \left[\frac{bh}{2} \times \frac{h}{2} \right]$

$\Rightarrow \boxed{M_p = f_y \frac{bh^2}{4}}$

Problem Determine the plastic moment capacity of an unsymmetrical I-section. Given size :-

Top Flange = 100mm × 20mm

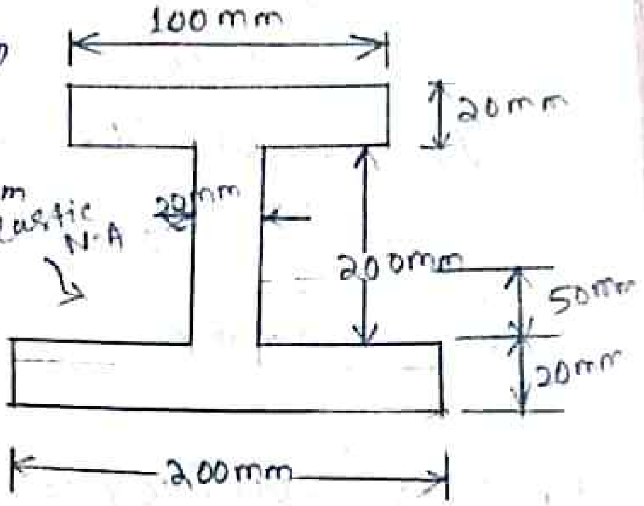
bottom flange = 200mm × 20mm

Web = 200 mm × 20 mm

<u>Solution</u>

Area of Top flange = 100mm × 20mm

= 2000mm2



Area of bottom flange = 200mm × 20mm

= 4000 mm2

Area of web = 200 mm × 20mm

= 400mm2

Total Area = 2000 + 4000 + 4000

= 10000 mm2

plastic N-A = $\frac{A}{2} = \frac{10,000}{2}$ = 5000mm 2

Area = bh

$\Rightarrow 1000 = 20h$

$\Rightarrow h = \frac{1000}{20}$

$\Rightarrow h = 50$ mm

$\frac{A}{2} = A_c = A_t$

$M_p = f_y Z_p$

$= f_y \left[\frac{A}{2} (\bar{y}_c + \bar{y}_t) \right]$

$= f_y (A_c \bar{y}_c + A_t \bar{y}_t)$

$= f_y \left[5000 (\bar{y}_c + \bar{y}_t) \right]$

$\bar{y}_c$ = centridal distance of compression area from plastic N-A

$$\bar{y}_c = \frac{A_1 y_1 + A_2 y_2}{A_1 + A_2}$$

$$\Rightarrow \bar{y}_c = \frac{(100 \times 20)(150 \overset{\times}{\text{[illegible]}} 10) + (150 \times 20)\left(\frac{150}{2}\right)}{(100 \times 20) + (150 \times 20)}$$

$\Rightarrow \boxed{\bar{y}_c = 109 \text{ mm}}$

$\bar{y}_t$ = centroidal distance of tension area from plastic N-A

$$\bar{y}_t = \frac{A_1 y_1 + A_2 y_2}{A_1 + A_2}$$

$$\Rightarrow \bar{y}_t = \frac{(200 \times 20)(50 \times 10) + (50 \times 20)(25)}{(200 \times 20) + (50 \times 20)}$$

$\Rightarrow \boxed{\bar{y}_t = 53 \text{ mm}}$

$$M_p = f_y [5000 (109 + 53)]$$

$\Rightarrow \boxed{M_p = 81 \times 10^4 f_y \quad \text{N.mm}}$