GOVT. POLYTECHNIC KANDHAMAL , PHULBANI

(State Council for technical education and vocational training , Odisha)

Th-2 Structural Design -II

5TH Semester , Diploma

Lecture Notes

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INTRODUCTION:-

A steel structure is an assemblage of a group of members are elements expected to sustain their share of applied of force.

The design of steel structure involves

(i) Functional design

(ii) Structural design

The firest part takes into consideration the purpose it is to serve like requirments of ventilation, lighting asthetics etc. The second part consist in proportion various elements of the structure for safe transmission of loads with due consideration of economy of material & labour.

Common steel structures:-

Steel has high strengh per unit mass hence it is used in constructing large column - free structures.

Following are the common steel structures in use.

→ Roof trusses for factories, cinema halls auditorium etc.

→ Trussed bents, crans girders, columns etc in industrial structures.

→ Roof truss & column to cover platforms in railway station and bus stands.

→ Single layer and double layer domes for auditorium exhibition halls. indoor stadium etc.

→ plate girder and truss bridge for railway & roads.

→ Transmission tower for microwave & electric power.

→ Water tanks

→ chimneys etc.

Advantage of steel as a structural Material :-

1. Smaller weight to strength ratio - It has smaller weight to strength ratio resulting in light weight structure for covering large spans.
2. Speed of erection - steel structures can be specily construction due to prefabrication in the workshop or at site out of standard avaible section.
3. Addition, alteration & strengthening - Addition and alteration of steel structure can be easily accomplished by welding and hence steel structures can be strengthened at any later.
4. Easy dismantalling and transportation By using bolted connection steel structure can be easily dismantled & conveniently handled. It can be easily transported to other sites being light weight & smaller value.
5. Gas and water tight joints - carefully made joints result in water tanks & pipe lines.
6. Assured quality & high durability - Being manufactured in the factory desired quality could be assured & if properly maintained, steel structures have long life.
7. Material is Reusable.
8. High ductility & toughness.
9. Resistance to freezing & thawing at low temperature.
10. Not combustible - It does not catch fire

Dis-Advantage :-

1. It is susceptible to corrosion.
2. Maintance cost is high since it neart to prevent corrosin.
3. Steel members are costly.
4. Skilled labour is required.

Stainless steel - steel + Nickel + chromium

TYPE OF STEEL :-

→ Steel is an alloy of iron & carbon.

→ Apart from carbon by adding small percentage of maganese, sulphar, phosphorus, chrome, nickels and coppers, special proparties can be imparted to iron and a variety of steels can be produced.

The effect of diffrent chemical constituents on steel are as follows.

(I) Incresed quanity of carbon & manganese impacts higher tensile strength & yields properties but lower ductility, which is more diffrent to weld.

(II) Incresed sulphar and phosphorus beyond 0.067. impact brittleness affects weldability and fatigue strength.

(iii) chrome and nickel impact corrosion resistance propoaty to steel. It improves resistance to high temperature also.

→ Carbon steel
→ High strength carbon steel
→ High strength microalloyed steel
→ Medium & high strength quenched & tempred steels.
→ High strength quenched
→ weathering steels
→ Stainless steel -
→ Fire resistance steels

Properties of steel:-

Classified as

(i) Physical properties
(ii) Mechanical properties

Physical properties:-

1. Unit mass of steel ρ = 7850 kg/m^3
2. Modulus of elasticity $E = 2.0 \times 10^5$ N/mm
3. Poisson's ratio μ = 0.3
4. Modulus of rigidity $G = 0.769 \times 10^5$ N/S
5. Co-efficient of thermal expansion $\alpha_t = 12 \times 10^{-6}$ /°C

Mechanical properties:-

(a) Yield stress f_y
(b) Ultimate stress f_u
(c) The minimum percentage along acted on standard gauge length
(d) Notch toughness

Rolled steel section:-

(i) Rolled steel I-section
(ii) Rolled steel channel section
(iii) Rolled steel Angle section
(iv) Rolled steel T-section
(v) Rolled steel bars
(vi) Rolled steel flats

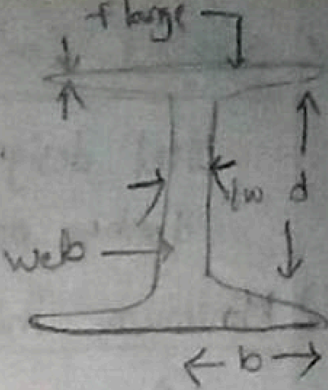
I SECTION:-

India Standard Junior beams

India Standard Light Beams (ISLB)

India Standard Medium beams (ISMB)
India Standard wide flange beams (ISWB)
Indian Standard heavy Beams (ISHB)
ISLB 500 @ 735.6 N/m ← wt per running meter

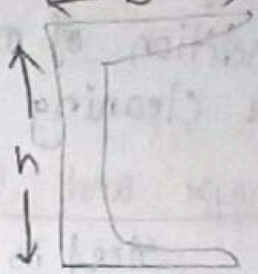
Depth of I-Section = 500mm



Angle Section:-

~~a. Indian Standard Equal Angle (ISA)~~
~~b. India Standard Unequal Angle (ISA)~~

Channel Section



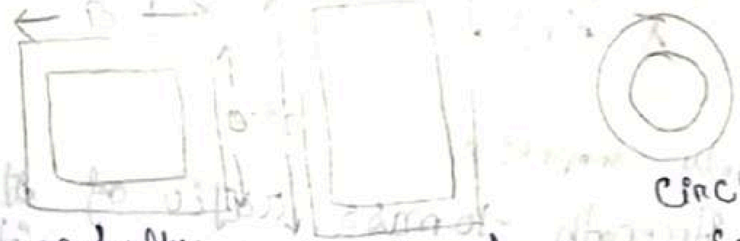
(1) ISJC
(2) ISLC
(3) ISMC
(4) ISSC (Indian Standard Special channel)

ISJC 200 @ 0.42 KN/m
└ height └ weight

Angle section (ISA)

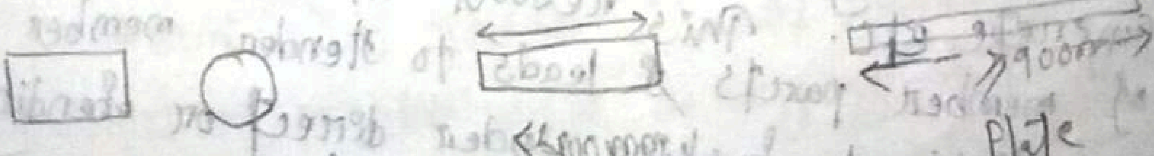
Indian Standard Round Bars - ISRO
Indian Standard Square Bars - ISSO

ISA 90 × 80 × 8 └ thickness
Length1 └(L 2)



Square hollow section
Rectangular hollow section
Circular hollow Section

hollow section



Square Circular Bars
Flat

Rolled steel structure shape

Special Consideration in steel design:-

Steel design differs from other design methods (Like RCC timber etc) in following aspects.

(i) Minimum thickness:-

In view of corrosion, the minimum thickness of the structural steel member are to be specified, otherwise a very small amount of corrosion may result into reduction of large percentage of effective area, if very thin section are used. It will also depend up the exposive condition of member to weather as well as accessibility for cleaning & painting.

(2) Shape and size:-

Steel is manufactured in rolling mills and are avaible in standard shape size. Have depending upon the site requi and loading condition, steel structure are designed. considering any of the avaible section or their combination

(3) Connection Design:-

During fabrication and assembling, various standard section in a member and the member themselves in a structure are to be suitably connected by welding boting rivetting & pins.

(4) Bucking:- Mainly compression

As the strength to mass ratio of stage is very high, the permissible load per unit area is much higher in steel compared to timber or concrete etc. This result in smaller cross-section of member parts & leads to slender member which are liable to bucking under direct or bending compression as the case may be.

Loads :-

(1) Dead loads (DL) Parmanent load

(2) Imposed loads (IL) / Live loads / Temporary

(3) Wind loads (WL)

(4) Earthquake loads (EL)

(5) Errection loads (EL)

(6) Accidental loads (AL) load due to fire, colapse

(7) Secondary effects (SE)

Load Combination :-

DL	DL + IL + EL
DL + IL	DL + IL + TL
DL + WL	DL + EL + TL
DL + EL	DL + IL + WL + TL
DL + TL	DL + IL + EL + TL
DL + IL + WL	

Design philosophies :-

Working stress Method / yield stress Mtd

Elastic Theory Method (WSM)

This is oldest Method of Design. In this method stress strain relationship is assumed to be linear within elastic range.

Permissible stress ≤ (yield stress / FOS)

This method provide safe desinging but highly uneconomical.

Plastic load Method / Ultimate load Method

This theory provaids highly economical designing but does not satisfy servicibility criteria.

Plastic load method is applicable to indeterminate members.

Limit State Method (LSM):-

In limit state design method, the structure is designed in such a way that it can safely withstand all kind of loads that may act on the structure under consideration in its entire design life.

This theory is based on probabilistic approach LSM.

Rectifies WSM by introducing partial safety factor for loads (γ_f).

TYPES OF LIMIT STATE :-

1. Limit state of strength
2. Limit state of serviceability

Limit state of strength	Limit state servicebility
→ strength (against yielding bucking)	→ Deflection
→ stability	→ Vibration
→ fatigue strength	→ corrosion
→ Plastic strength	→ fire etc.

FOS - (factor of safety) is defind combinely for variation in material property & variation in Dead load & live load.

$$\text{Permeciable stress} = \frac{fy}{fos}$$

Particle safety factor for loads (γ_f)
(variation in D.L & L.L)

Design action = γ_f × (working load / service load)

Partial safety factor for material (γ_m) γ_m accounts for variation in material properties & variation in member size.

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

Design action ≤ Design strength perm.

Structural steel fastenats And Connection

Connection :-

→ Connection of the weekest point of follower in a structure and thus needs to be proporly analysised and designed.

→ The various ~~parts~~ type of fastnumets avaible for making connection ribets, bolts, pins and the welds. bulting has become some much popular the high strength board has almost replaced ribets now.

Bolted Connection :-

A bolt is a short up threded pin week head at one and threded on the otherend of the shank to recipe the nut as shown in fig.

→ Bolt are used for the purpose of joining together pieces of metal having hones through which these inserted and the nuts are tighended at the threded ends. however steel washers an usally provaided under the bolts as well as under the nuts. To distribut claping pressure all the bolted member as well as to prevent threded parson of the bolt form bearing on the connected parts

Method of Drawing

Description

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TYPES OF BOLTS

The following types bolts are

(a) Unfinshed bolt or black bolt

(b) finish bolt or turn bolt

(c) High strength friction grip bolt

Unfinshed Bolt:-

These are also know as ordinary and common bolt these bolts are made from low carbon and the shank is left unfinshed or rough.

→ specification of these bolt are given him IS 1367 part one. The diameter of the shank is know as nominal diameter of the bolts. These bolts are avaible in nominal diameters of 5 to 36 mm being design as M_5 to M_{36}. the commonly used hence for structural steel work bolt diameters are 6, 20, 24, 36 mm.

M_{30} Where 'M' represent Matric unit in mm and '30' represent nominal diameter of bolts

$A_s = \pi/4 \times d^2$

$A_{net} = 0.78 \times \pi/4 \times d^2 (A_s)$

Where shnk - S

Grade 4.6

$f_{ub} = 4 \times 100 = 400$ f_{ub} = Ultimate bearing

$F_{yb} = 400 \times 0.6 = 240 N/mm^2$ F_{yb} = Yield bearing

Structural Steel Fasteners And Connection : Bolts

Introduction:- Different element / members of steel structure are required to be joined to one another either at their ends or at some intermediate length in order to facilitate the transmission distribution of member forces or for the purpose of stability as the case may be which is know as connection.

→ Connection are the weakest point of failure in a structure and thus need to be properly analyzed and designed.

→ The various types of fasteners avaible for making connections.

(a) Bolt (b) welds (c) Rivets (d) Pins.

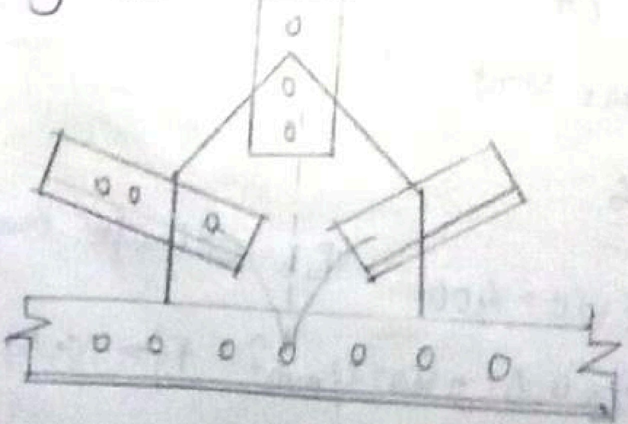
→ Bolting has become so much popular that high strength bolts has almost replaced rivetes now.

GUSSET PLATE:-

The places where more than one member are to be jointed such as truss joints or similar structures, gusset plate are provided to make connection

→ Gusset plates are provided to transfer the load to one member to another member

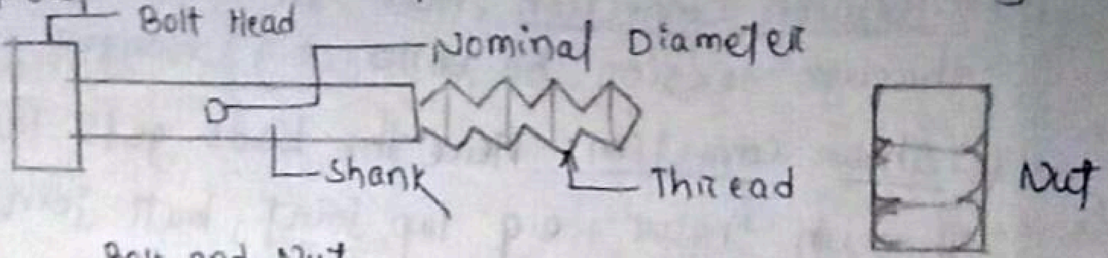
→ Gusset plate are designed for normal stresses shear stress, bending stresses.



BOLTED CONNECTION:-

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

→ A typical bolt and nut are shown in the fig.



Bolt and Nut

Bolts are used for the purpose of joining together pieces of metals having holes through which these are inserted ends. However, steel washers are usally provaid under the bolt as well as under the nut to distribute clamping pressure on the bolted member as well as to prevent threaded portion of the bolt from bearing on the connected parts.

Uses of Bolts

(a) Connection of tension and compression members.

(b) fabrication of compound and built-up section consisting of two or more section.

(c) As hold down bolts to hold the colums bases in position joining the culumn caps with shoe plate (of truesses) etc.

Classification of Bolted connection:-

(a) Classification based on line of action of resultant force transferred.

(i) Concentric connection :- Here the load line passes through the CG of the section e.g Axially loaded tension or compression member.

(ii) Ecentric connection:- Here the loaded line is away from the CG of the connection.

e.g Bracket connection, moment resisting connection, seat connection etc.

(b) Classification based on the type of force

(i) <u>Tension Connection</u>: Here the load gets transferred through tension on bolts e.g Hanger connection.

(ii) <u>Shear Connection</u>:- Here the loads gets transferred through shear e.g lap joint, butt joint etc.

(iii) <u>Combined shear and tension connection</u>:- This type of connection is required when an inclined members is to be connected to a column through bracket e.g connection to bracings.

(c) Classification based on the type of force Mechanism

(i) <u>Bearing connection</u>:- Here the bolts bear against the holes to transfer the load e.g slip type connection.

(2) <u>Friction connection</u>:- Here the load is transfered by friction betn the plates due to tensioning of bolts. e.g Slip critical connection

ADVANTAGES OF BOLTED CONNECTION:-

The following are the advantages of bolted connection over other type of connections.

(i) Use of simple tools and less skilled labour and working area.

(ii) Speedy & noiseless errection

(iii) Economical due to reduced labour and equipment costs.

(iv) Minimum strength reduction at joint due to less number of holes or bolts.

(v) Easy alteration or dismantling of connection.

Disadvantage of Bolted connection:-

(i) High cost of material.

(ii) Reduce tensile strength due to area reduction at the root of threads.

(iii) Normally loose fit reduce joints rigidity leading to excessive deflection

(iv) susceptibility of loosing of bolts under vibration and dynamic loads.

(v) Large joint space, when heavy loads are req.

Type of Bolts:-

The following type of bolts are in common use.

(a) Unfinished bolts or Black bolts

(b) Finished bolts or Turned bolts.

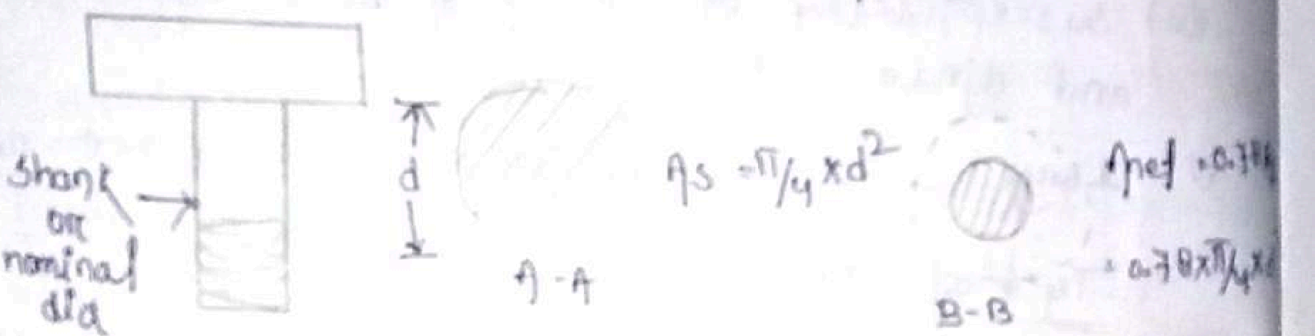
(c) High strength friction grip (HSFG) bolts.

Unfinished or Black bolts:-

These are also know as ordinary or common bolts. These bolts are made from low carbon mild steel round rods with square or hexagonal head & the shank is left unfinished or rough. specification of these bolts are given is IS: 1367 part -1. The diameter of the shank is know as nominal diameter of the bolt.

These bolts are avaible in nominal diameter of 5 to 36mm being designated as M_5 to M_{36}. Hence for structoral steel work, commonly used bolt diameters are 16, 20, 24, 30, and 36mm. These are used for light structures subject to static loads as well as for secondary members such as purlins, bracing, roof trusses etc. but not recommended for structures subject to vibration & fatigue.

M30, M represent metric unit in mm & 30 represent nominal dia meter of bolts.



GRADE 4.6

$f_{ub} = 400 N/mm^2$

$f_{yb} = 400 \times 0.6$

$= 240 N/mm^2$

Grade 8.8

$f_{ub} = 800 N/mm^2$

$f_{yb} = 640 N/mm^2$

f_{ub} = Ultimate tensile stress (Mpa)

f_{yb} = Yield stress (Mpa)

b. Finished and turned bolts:-

These are close tolerance bolts which are formed from mild steel hexagonal rods and are made by turning to circular shape. Turned bolts may be either precision bolts or semi-precision bolts. Tolerance allowed are about 0.15 mm to 0.5mm since the tolerance available is small, special method are employed to align the holes properly before bolting. of course, these are used where accurate alignment of components are necessary such as machine parts, structures subjected to dynamic loading.

6. High strength Friction Grip Bolts (HSFG):-

These bolts are made from high strength steel rods like black bolts, but the surface of the shank of these bolts is kept unfurnished and these bolts are tightened until very high speed tensile stress are devloped using calibrated wrenchy so that the connected parts are clapled devices. These are now being used in high rised buildings, Bridges etc.

Bolt holes:-

The holes made for placing the bolts in members may be either drilled or punched. punched holes may be either full size or under size and remain steel fabricator prefer punched holes because punching is simple & saves time and cost.

Terms used in Bolted connection:-

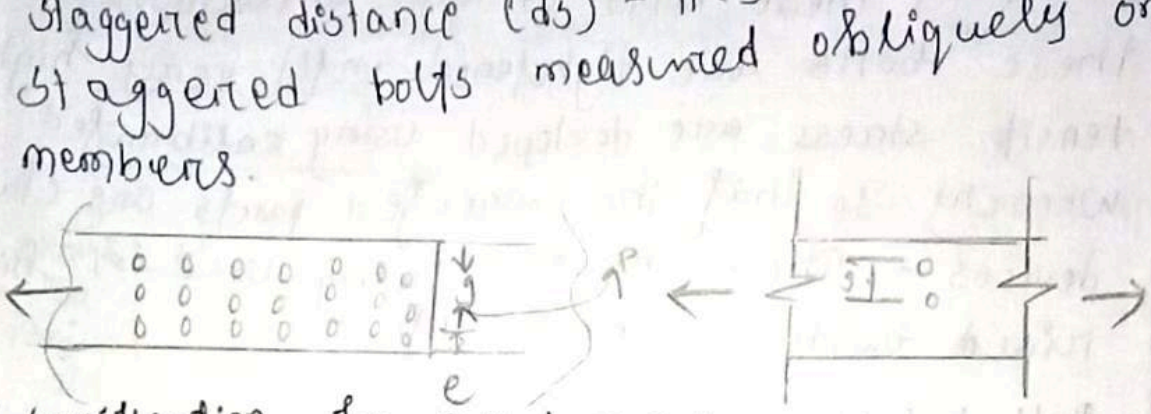
1. Pitch of bolts (P) → It is the distance between the centre of two consecutive bolts in a row measured along the direction of load.

2. Gauge distance (g) → It is the distance betn two consecutive bolts of adjacent rows and in measured at right angles to the direction of load.

3. Edge distance / End distance (e) → It is the distance of centre of bolt hole from the ~~advantage~~ adjacent edge or end of the plate.

4. Staggered pitch (P) - It is the centre to centre distance of staggered bolts measured along the direction of load.

5. Staggered distance (ds) - It is the centre distance of staggered bolts measured obliquely on the staggered members.



✓ Specification for Bolted Joint :-

1. Minimum pitch :- pitch shall not be less than 2.5d where 'd' is the nominal diameter of bolt.

2. Maximum pitch - pitch shall not be more than .

a. 16t or 200m, whichever is less, in case of tensile members.

b. 12t or 200mm whichever is less in case on compression member where 't' is the thickness of thinnest member.

c. In case of staggered pitch may be increased by 50 percent of values specified above provided gauge distance is less then 75mm.

3. In case of butt joints, maximum pitch is to be restricted to 4.5d of for a distance of 1.5 times the width of plate from the butting

surface

4. The gauge length g should not be more than (100+4t) or 200mm which ever is less.

5. Minimum edge distance shall be

(i) Not less then 1.7 times hole diameter in case of sheared or hand flame cut edges.

(ii) Not less than 1.5 times diameter in case of rolled machine flam cut, sawn and planed edges.

6. Maximum edge distance (e) should not exceed.

(a) $16 t\varepsilon$, where $\varepsilon = \sqrt{\frac{250}{f_y}}$ and t is the thickness of thinner outer plate.

(b) $40 + 4t$, wher t is the thickness of thinner connected plate.

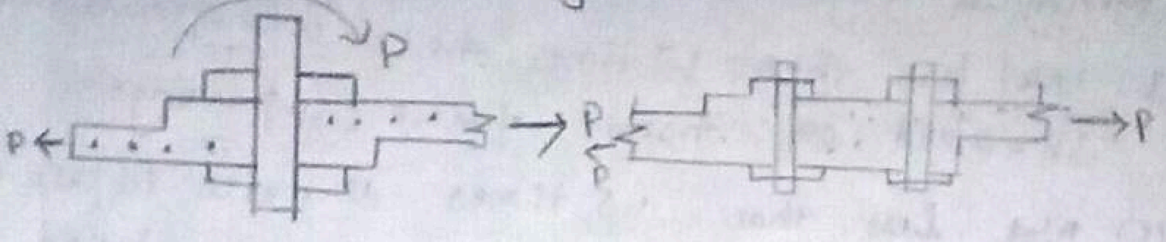
Types of Bolted Connections / Joints :-

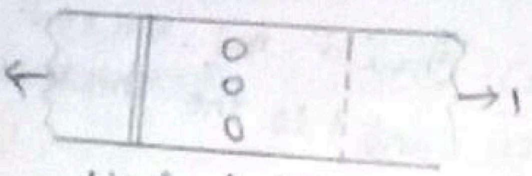
(a) Lap Joint (b) Butt Joint

(a) Lap Joint :-

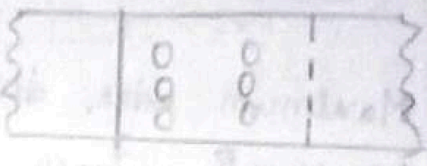
In this type of joint, the two members to be connected over lap one another. This constitutes the simplest type of joint requiring no extra accessories like cover plates. If their is one line of bolts, it is called single bolted lap joint & if there are two lines of bolt it is called a double bolted lap joint.

In lap joint there will be some amount of eccentricity in connection that will ~~be~~ generate a couple that leads to earlier failure of connection to reduce effect of eccentricity no of bolts should be increase to ~~insert~~ increase rigidity in the connection.

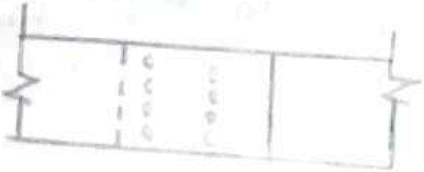




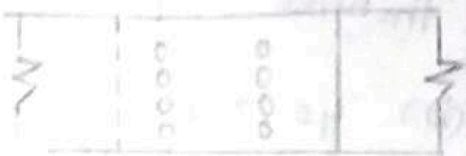
single bolted lap joint



Double bolted lap joint

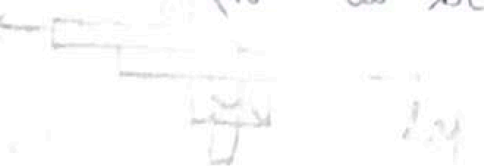


Chain bolting



Zig zag bolting

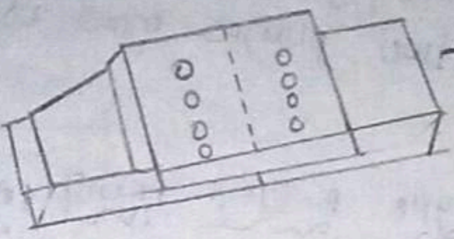
In this case, the bolts subjected to shear in one plane and hence know as bolts in single shear.



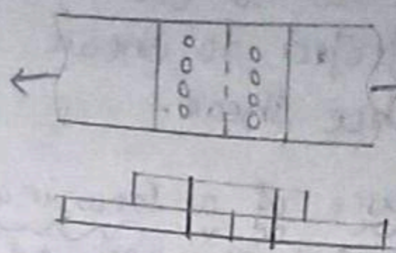
in single shear

Butt joint:-

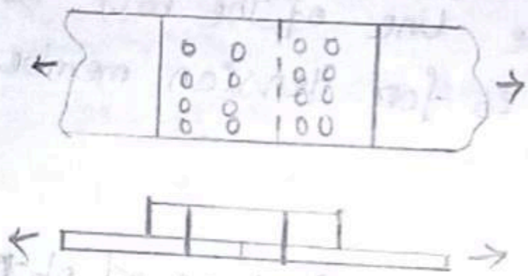
In this type of joint, the two members to be connected are placed end to end i.e butt against each other and the connection is made by providing additional plate/plates either on one side (single cover butt joint) or on both the sides (double cover butt joint). These additional plates are called cover plates whereas the members to be joined are called main plates.



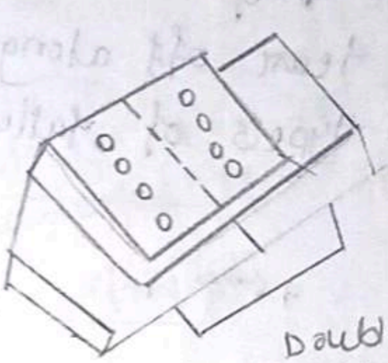
(a) Single - cover butt joint



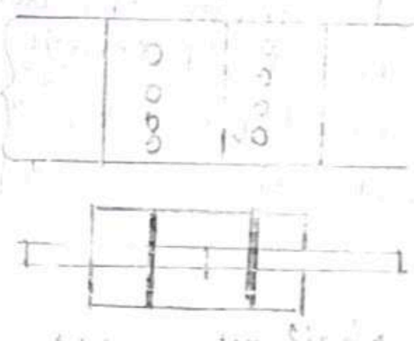
(b) Single - cover single bolted butt joint



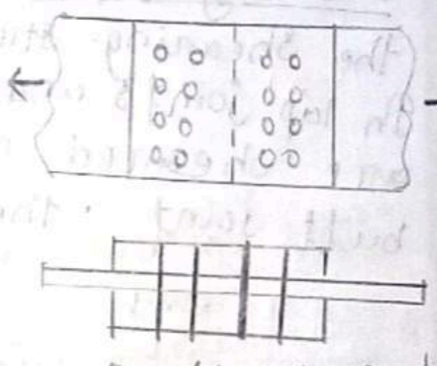
Single-cover double butt joint



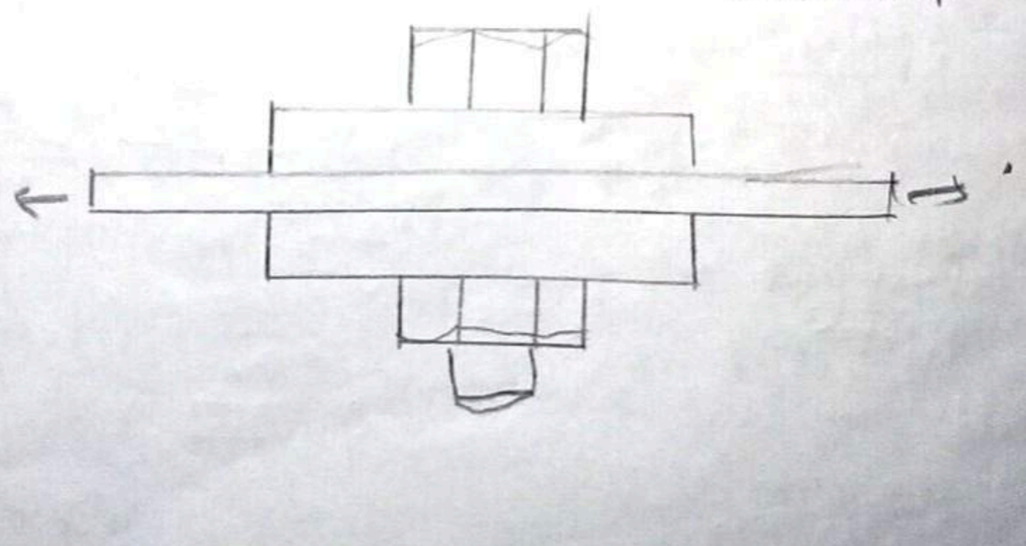
Double cover butt joint



Double - cover single bolted butt joint



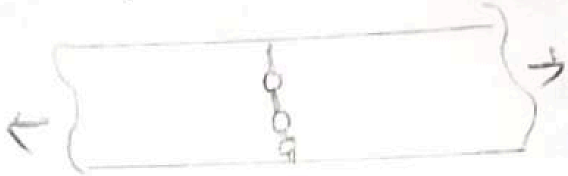
Double - cover double bolted butt joint



→ In case of double cover butt joint, bending eliminted as their is no eccentricity of forces and the bolts are subjected to shear in two planes know as bolts in double shear.

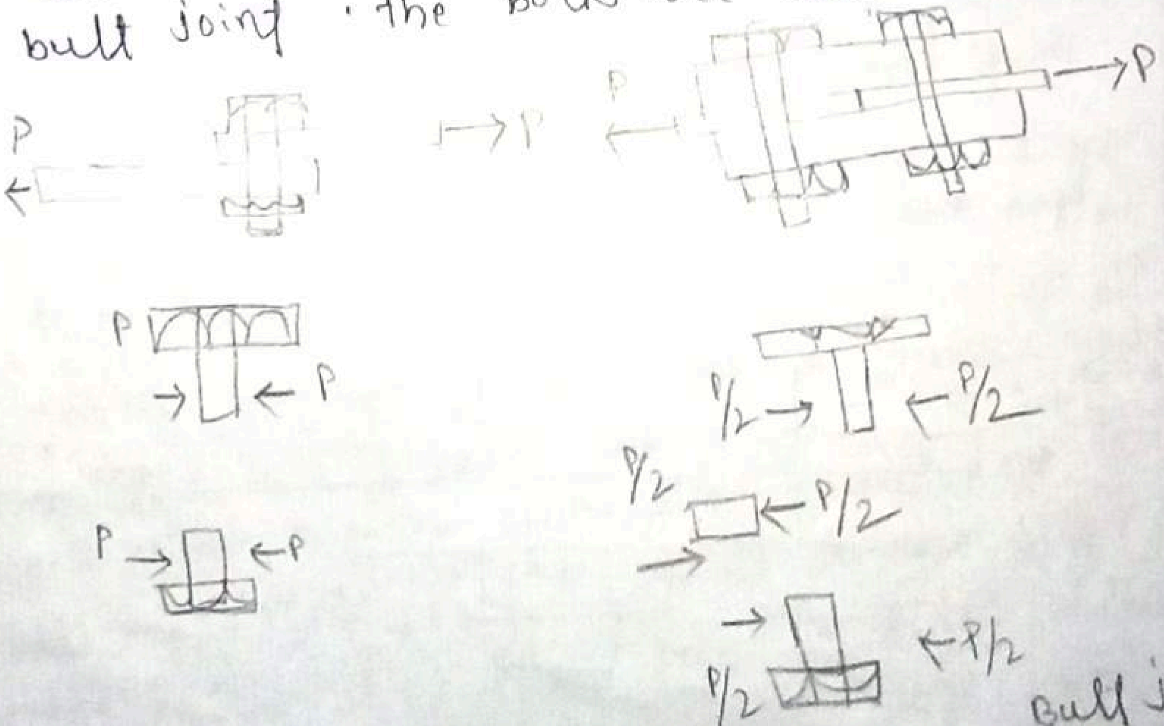
* Failure of a Bearing type Bolted joint A bolted joint may fail in any of the following manners.

(i) Repture of the plate btn bolt holes - The strength of the plate is reduced by bolt holes and the plate may tear off along the line of the bolt holes. Such types of failure is for tension member only.



Repture of plate

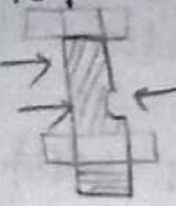
(ii) Shearing of bolt :- The bolt may fail by shearing if the shearing stress exceeds their shearing strength In lap joints and single cover butt joints the bolts are sheared at one plane only. In a double cover butt joint the bolts are sheared at two plane

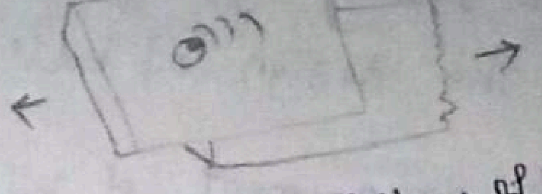


Bull joint

bolt in double shear

(iii) Bearing of bolt or plate → The plate or bolt is crushed if the compressive stress exceeds the bearing strength of the plate or bolt.





Bearing failure of bolt

Bearing failure of plate

(iv) Bursting or cracking of the edge–

The plate will crack at the back of a bolt, if it is placed very near to the edge of the plate such type of failure can be prevented if the provision of the minimum edge distance are satisfied.

Assumption in the Analysis of Bearing Bolts:

(i) The stress distribution on the plates betn the bolt holes is uniform.

(ii) The friction betn the plates is negligible.

(iii) The shearing stress is uniformly distributed over the cross-section of the bolts.

(iv) The bolts in a group shear the direct load equally.

(v) Bending stress devloved in bolts is neglected.

Principles of Bolted joint connections:-

The following guiding principles are to be observed in the design of bolted connection.

(i) The arrangment of bolts should satisfy the gauge, Pitch and the edge distance requirements.

(ii) To effect economy of material cost in respect of cover plates, gusset plates etc. the length of the connection is kept as small as possible.

(iii) The c.g of the bolt group should coincide with the centre line of connected members.

(iv) The centre line of all members meeting at a joint should coincid at ~~the~~ one point only to avoid twisting the joint out of position.

(v) The number of bolts should be gradually increased towards the joint for uniform stress distribution in the bolts.

(vi) Where possible, zig, zag or diamond from of bolting may be adopted so as to affect minimum strength reduction due to holes.

Design strength of plates in a joint.

Plates in a joint made with bearing type of bolts may fails due to (i) bursting of the edges (ii) crushing of plates in bearing or (iii) rupture of plates.

The design tensile strength of a plate in the joint is the strength of the thinner member against rupture which is given by .

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{mi}} \quad [Cl. 6.3.1 of IS800:2007]$$

γ_{m1} = partial safety factor for failure at ultimate stress = 1.25

f_u = ultimate stress of the material

where,

A_n = net effective area of the plate at critical section.

$$= \left[b - n d_n + \sum \frac{p_s^2}{4 g_i}\right] t$$

b = width of the plate

t = thickness of thinner plate in the joint

d_n = diameter of the bolt hole.

g = gauge distance between the bolt holes

p_s = length of staggested pitch between lines of bolt

n = number of bolt holes in the critical section

i = subscript for summation of all inclined legs.

When there is no staggering $p_s = 0$ & hence the formula reduces to $A_n = (b - n d_n) t$

Design Strength of Bearing types of Bolts in a joint:-

The Design strength of bearing type of bolts is the least of the (a) shear capacity (b) bearing capacity

Shear capacity or shear strength:-

The design shear strength of the bolt V_{dsb} is given by $V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}}$ [cl - 10.3.3 of IS 800-2007]

Where γ_{mb} = partial safety factor of material of bolt

V_{nsb} = nominal shear capacity of bolt

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

Where f_{ub} = ultimate tensile strength of a bolt

n_n = number of shear planes with threads intercepting the shear plane.

n_s = Numbers of shear planes, without threads intercepting the shear plane.

A_{sb} = nominal plain shank area of the bolt

A_{nb} = net shear area of the bolt at threads

Which may be taken as the area corresponding to root diameter at the thread.

$$A_{nb} = 0.78 \, \pi/4 \, d^2.$$

* Reduction factors for shear capacity of bolts:-

Under following circumstances, the code suggests the use of reduction factor for shear capacity of bolts:

(i) Long joints (ii) Larger grip lengths (iii) Packing plates of thickness more than 6mm.

(i) Reduction factor for Long joints:-

When the length of the joints lj (that is the distance between the first and last rows of bolts in the joints measured in the direction of the load transfer exceeds 15d, the nominal shear capacity V_{nsb} shall be reduced by the factor B_{ij} given by

$$B_{ij} = 1.075 - \frac{l_j}{200d} \qquad 0.75 \le B_{ij} \le 1.0$$

Where d = nominal diameter of the bolt.

Reduction factor for large grip length (B_{lg}):-

If the thickness of packing plates is more than 6mm in a joint, the shear capacity is decreased by a factor B_{pk} given by $B_{pk} = 1 - 0.0125\, t_{pk}$.

where t_{pk} = thickness of the thickest packing in mm.

Thus the complete formula for nominal shear capacity of bolts is given by.

$$V_{nsb} = \frac{f_u}{\sqrt{3}}(n_n A_{nb} + n_s A_{sb}) \cdot B_{ij} \cdot B_{lg} \cdot B_{pk}$$

Bearing Capacity or bearing strength:-

The design bearing strength of the bolt V_{dpb} is given by

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

Where V_{npb} = nominal bearing strength of a bolt $= 2.5\ k_b d t f_u$

Where K_b is smaller of

$$\left.\begin{array}{l} e/3d_o \\ p/3d_o - 0.25 \\ f_{ub}/f_u \\ 1 \end{array}\right\} \text{Min}^m$$

in which e, p = end and pith distance along bearing direction

d_o = diameter of the bolt hole.

f_{ub}, f_u = Ultimate tensile stress of the bolt and the plate respectively.

d = nominal diameter of the bolt

t = Summation of the thickess of the connected plates experiencing bearing stress in the same direction or if the bolts are counter sunk it is to be reduce by the half of the depth.

Efficiency of a joint:-

The original strength of a section is reduced by bolt holes. The efficiency of a joint is the ratio of the strength of the joint and the original strength of member without bolt holes.

It is usally expressed as a percentage

Thus, efficiency (η) $= \dfrac{\text{strength of the joint}}{\text{strength of solid plate}} \times 100$

The strength of solid plate is governed by its strength in yeilding because it is less compared to rupture strength of solid plate. The strength of the joint is minimum of shear or bearing strength of bolts.

exm:- consider Fe415 plate

$f_{yp} = 415$

$f_y = 250 KN/mm^2$

$\gamma_{m0} = 1.1$

$\gamma_{m1} = 1.25$

Design ~~Tensile~~ strength of solid plate per unit area

$T_{dg} = A_g \times f_y / 1.1$

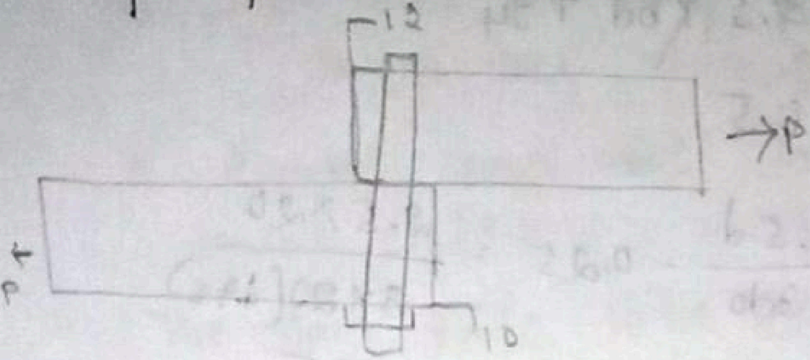
$= \dfrac{1 \times 250}{1.1} = 227.27$

$T_{dn} = 0.9 A_n \dfrac{f_u}{1.25} = 0.9 \times 1 \times \dfrac{415}{1.25}$

$= 298.8 KN$

hence the strength of solid plate by is covered by is strength yielding strength of joint is the smaller strength of shearing and strength of bearing.

* Calculate bolt strength for the given arrangment M20 grade of bolt are used to transpart the load n distan- 1.5 x do and pitch distance = 2.5 x d, grade of steel Fe410.



Given data :-

Fe 410 steel

$f_u = 410\ mm^2$

$f_y = 250\ N/mm^2$

4.6 grade bolt

$f_{ub} = 400\ mm^2$

$f_{yb} = 240\ N/mm^2$

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

Nominal dia of bolt d = 20mm

Minimum thickness $t_{min} = (12,10) = 10mm$

* Shearing capacity of bolt

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

$$V_{nsb} = \frac{\cancel{410} f_u}{\sqrt{3}}(n_n A_{nb} + n_s A_{sb})$$

$$= \frac{410}{\sqrt{3}}(1 \times 0.78 \times \pi/4 \times (d)^2)$$

$$= 45249.48 \Rightarrow \frac{245.04}{1.25} = V_{dsb} = 4827\ KN$$

* Bearing Capacity of bolt

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

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

(i) $\frac{e}{3d_0} = \frac{1.5 \cancel{d_0}}{3 \cancel{d_0}} = 0.5$

(ii) $\frac{p}{3d_0} - 0.25 = \frac{2.5d}{3d_0} - 0.25 = \frac{2.5 \times 20}{3 \times 22 (d+2)}$

$= 0.75$

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

(iv) $1.0 = 1$

$K_b = 0.5$

$V_{npb} = 2.5\, K_b \times d \times t_{min} \times f_u$

$= 2.5 \times 0.5 \times 20 \times 10 \times 410$

$= 102.500 KN$

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

$= \frac{102.500}{1.25}$

$= 82.3 KN$

* Determine the strength 20mm diameter bolt of grade 4.6 for the following

(a) Lap joint

(b) Single cover bolt joint with 10mm single cover butt 10 mm thick cover plate

(c) Double cover bott joint with 8mm thick cover plates.

The main plate to be join are 14mm thick used Fe 410 grade of steel.

Given data:-

(a) Lap joint

$d = 20mm$

4.6 grade bolt

$f_u = 400$

$f_y = 240mm^2$

$Fe = 410 N/mm^2$

$t = 14mm$

$d_0 = 20+2 = 22$

Partial factor of safety $\gamma_{mb} = 1.25$

* Shear capacity of bolt

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

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

$$= \frac{410}{\sqrt{3}}\left(1 \times 0.78 \times \pi/4\,(20)^2\right) = \frac{56.59}{1.25} = 45.27 kN$$

Bearing Capacity of bolt

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

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

$k_b = 33$

$V_{npd} = 2.5 k_b \times d \times t \times f_u$

$= 2.5 \times 0.5 \times 20 \times 14 \times 410$

$= 143.5\,kN$

$= \frac{143.5}{1.25} = 114.8\,kN$

Thus strength of bolt $= 45.27\,kN$

(b) single cover bolt joint

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

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

$$= \frac{400}{\sqrt{3}} \left(1 \times 0.78 \times \frac{\pi}{4} (20)^2 \right)$$

$$= \frac{565.54}{1.25} = 45.27$$

Bearing Capacity of bolt

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

$$V_{npb} = 2.5\, k_b d \cdot t_{min} \times f_u$$

$K_b = \frac{e}{3d_0}$

$= \frac{1.5 \cancel{d_0}}{3 \times \cancel{d_0}}$

$= 0.5$

$P/3d_0 - 25 = \frac{2.5d}{3d_0} - 0.25$

$= \frac{2.5 \times 20}{3 \times 22} - 0.75$

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

$1.0 = 1$

$K_b = 0.5$

$V_{npb} = 2.5\, K_b \times d \times t_{min} \times f_u$

$= 2.5 \times 0.5 \times 20 \times 10 \times 410$

$= 102.500 KN$

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

$= \frac{102.500}{1.25}$

$= 82.3 KN$

$k_b = \frac{e}{3d_0} = \frac{1.5\cancel{d_0}}{3 \times \cancel{d_0}} = 0.5$

$P/_{3d_0} - 2.5 = 2.5d/_{3d_0} - 0.25 = \frac{2.5 \times 20}{3 \times 22} = 0.75$

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

1.0 = 1

$k_b = 0.5$

$V_{npd} = 2.5\ k_b \times d \times t_{min} \times f_u$

$= 2.5 \times 0.5 \times 20 \times 10 \times 410$

$= 102.500 kN$

$V_{dpb} = V_{npb}/\gamma_{mb} = \frac{102.500}{1.25}$

$= 82.3 kN$

(ii) Double cover bolt Joint :-

Shearing Capacity of bolt

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

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

$= \frac{400}{\sqrt{3}} (2 \times 0.78 \times \pi/4 \times (d)^2)$

$$= \frac{113.08}{1.25}$$

$$= 90.54\,kN$$

Bearing Capacity of bolt

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

$$V_{npb} = 2.5\, K_b \times d \times t_{min} \times f_u$$

$$K_b \begin{cases} = \dfrac{1.5\,\cancel{d_0}}{3 \times \cancel{d_0}} = 0.5 \\ \dfrac{P}{3d_0} - 0.25 = \dfrac{2.5}{3d_0} - 0.25 = \dfrac{2.5 \times 20}{3 \times 22} = 0.75 \\ \dfrac{f_{ub}}{f_u} = \dfrac{400}{410} = 0.97 \\ 1.0 = 1 \end{cases}$$

$$K_b = 0.5$$

$$V_{npb} = 2.5 \times 0.5 \times 20 \times 14 \times 410$$

$$= \frac{143.500}{1.25}$$

$$= 114.8\,kN$$

Assuming that the threads intersec the shear plane.

* In an Industrial building an ISA 65 x 65 x 6 is connected with 10m thick guss plate. Angle section subjected to 150KN working load use 16mm dia bolt of grade 4.6 to transport the load.

$\gamma_f = 1.5$.

Imp

No of bolt = $\frac{\text{Total load}}{\text{Bolt strength}}$. $1.5 \times 150 \times 10^3$

Given data:-

$L_1 = 65$

$L_2 = 65$

$t = 6$

$d = 16$

$f_u = 400$

$\gamma_m = 1.5$

load = 150

$d_0 = 16 + 2 = 18$

Shearing capacity of bolt

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

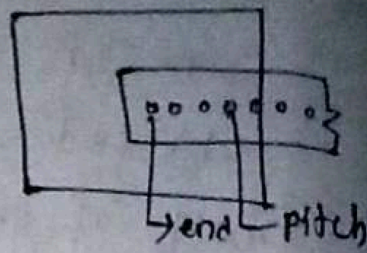
$$V_{nsb} = \frac{f_u}{\sqrt{3}} (n_n A_{nb} + n_s A_{sb})$$

$$= \frac{400}{\sqrt{3}} \left(1 \times 0.78 \times \pi/4 \, (d)^2\right)$$

$$= \frac{400}{\sqrt{3}} \left(1 \times 0.78 \times \pi/4 \, (16)^2\right)$$

$$= \frac{277.36}{1.5} = 184.90 \text{KN}$$

bearing Capacity of bolt



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

$$V_{npb} = 2.5 \times k_b \times d \times t_{min} \times f_u$$

k_b $\begin{cases} \frac{1.5\, \cancel{d_0}}{3\, \cancel{d_0}} = 0.5 \\ \frac{p}{3d_0} - 0.25 = \frac{2.5 \times 16}{3 \times 18} = 0.72 \\ \frac{f_{ub}}{f_u} = 0.97 \\ \quad 0.1 = 1 \\ k_b = 0.5 \end{cases}$

$$V_{npb} = 2.5 \times 0.5 \times 16 \times 6 \times \frac{400}{1.25}$$

$$= 39360$$

No of bolt = $\frac{\text{Total load}}{\text{Bolt strength}}$

$$= \frac{1.5 \times 150 \times 10^3}{\cancel{2898}\ 65 \times 10^3}$$

$$= 1216 \simeq 121$$

$$= \cancel{1217} = 2.58 \simeq 3$$

Total overlap length on guss plate

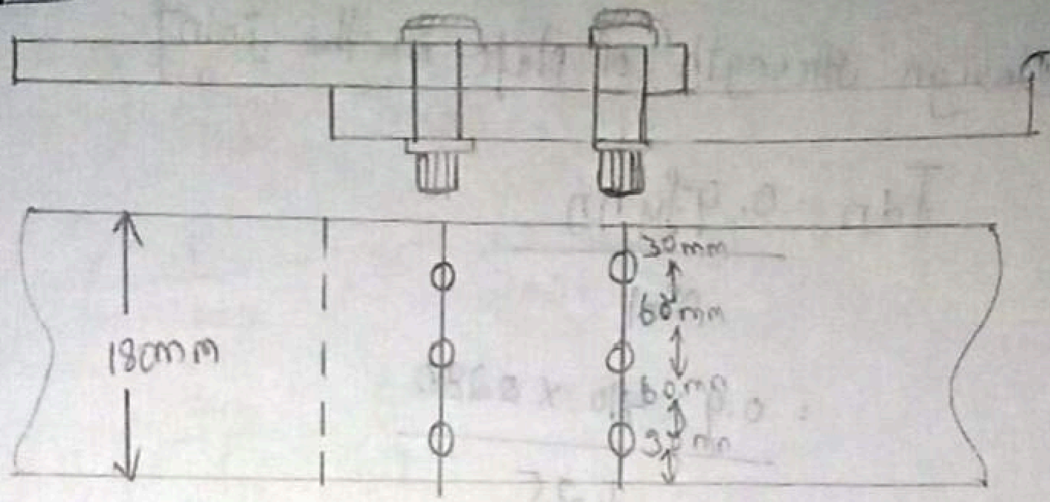
$$= 27 \times 20 + \cancel{2 \times 27}\ 8.427$$

$$= 13.84\,mm$$

$$= 1217/334 = 77.7 \simeq 8$$

Find the efficiency of the joint show in fig given M20 bolts of grade 4.6 and Fe 410 (E250) plate are used.

Solution:-



Given data:-

Bolt : M20 Grade = 4.6

$d = 20mm$

$d_0 = d+2 = 20+2 = 22mm$

ultimate strength $f_{ub} = 400mpa$

$\gamma_{mb} = 1.25$

Fe = 410 (E 250) plates

ultimate stress $f_u = 410$

$\gamma_{m1} = 1.25$

* strength of plate in the joint

$t = 20mm$

$b = 180mm$

There is no staggering $P_{si} = 0$

No of bolt holes in the weakest section = 3

Net area at weakest section

$A_n = [b - n d_0] t$

$A_n = [180 - 3.22] \times 20$

$= 2280 \text{ mm}$

* Design strength of plate in the joint

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

$$= \frac{0.9 \times 410 \times 2280}{1.25}$$

$= 673050N = 673.056 kN$

* Strength of bolt

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

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

$$= \frac{400}{\sqrt{3}} \left(6 \times 0.78 \times \pi/4 (d)^2\right)$$ ($\because$ Assume net area of shear strength)

$$= \frac{400}{\sqrt{3}} \left(6 \times 0.78 \times \pi/4 (20)^2\right)$$

$= 339.543 kN$

$$V_{dsb} = \frac{339.543}{1.25}$$

$= 271.586 kN$

* Bearing Capacity

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

$K_b : \begin{cases} \dfrac{e}{3d_0} = \dfrac{30}{3\times22} = 0.45 \\[2ex] \dfrac{p}{3d_0} - 0.25 = \dfrac{60}{3\times22} - 0.25 = 0.659 \\[2ex] \dfrac{f_{ub}}{f_u} = \dfrac{400}{410} = 0.97 \\[2ex] 0.1 = 1 \end{cases}$

$K_b = 0.45$

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

$= 2.5 \times 0.45 \cdot 20 \times 20 \times 410$

$= 186140$

$V_{npb} = \dfrac{186140}{1.25}$

$= 148912\ N$

Design strength of bolt in joint $= 6 \times 148.912$

$= 893472$

$= 893.47\ kN$

Design strength of bolt in joint $= 271.586\ kN$

efficiency of joint

Area of solid plate $= \dfrac{f_y}{\gamma_{m0}} \times A_g$

$= \frac{250}{1.1} \times 3600$

$= 818.8 kN$

Efficiency of the joint $= \frac{\text{Strength of joint}}{\text{strength of solid plate}} \times 100$

$= \frac{271.586}{818.182} \times 100$

$= 33.19\%$.

Find the efficiency of the joint if in the above example instead of lap butt joint is made using two cover plates each of size 12 mm and 6 no of butt on each side.

no. of bolt = 6

$t = 12mm$

$d_o = 20+2 = 22$

$Fe = 410$

grade = 4.6

$f_y = 400$

$f_{ub} = 240$

Design strength of plate $T_{dn} = 0.9 A_n f_u / m)$

$A_n = [b - nd_o + 0]$

$= [180 - 3 \times 22] \times 20$

$= 2280$

$T_{dn} = 0.9 A_n f_u / m_l$

$= 0.9 \times 2280 \times \frac{410}{1.25}$

$= 673050 N = 673.05 N$

Nominal shear strength of bolt

$V_{dsb} = \frac{f_u}{\sqrt{3}} (n_n A_{nb} + n_s A_{sb})$

$= \frac{400}{\sqrt{3}} \left(6 \times 0.78 \times \pi/4 (d)^2 + 6 \times \pi/4 (d)^2\right)$

$= \frac{400}{\sqrt{3}} (6 \times 245 + 6 \times 314.15)$

$= 774.78 kN$

$V_{dsb} = \frac{774.78}{1.25} = 619.82 kN$

Design strength of bearing

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

$= 2.5 \times 0.45 \times 20 \times 20 \times 410$

$= 18614$

$V_{npb} = \frac{186140}{1.25}$

$= 148912 N$

Design strength of bolt joint $= 6 \times 148912$

$= 893472$

$= 893.47 kN$

Design strength of bolt = 619.82

Total thickness of plate = 12 X 2 = 24mm

Design strength of plate $= \frac{250}{1.1} \times 180 \times 20$

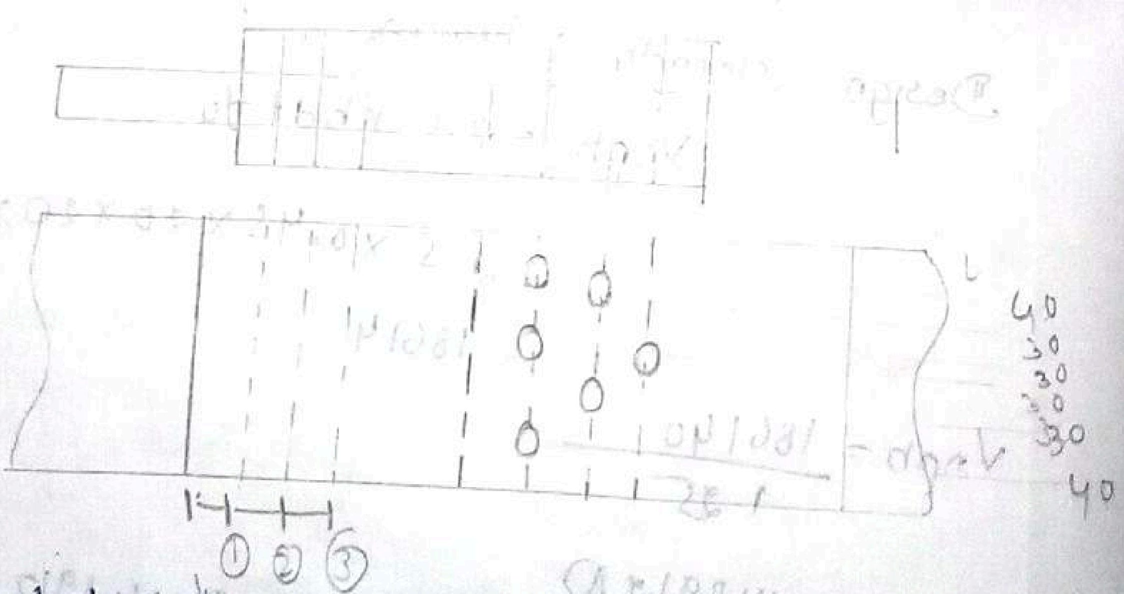
$= 8181.81$

Efficiency of joint $= \frac{\text{Strength of joint}}{\text{Strength of solid plate}} \times 100$

$= \frac{619.82}{8181.81} \times 100$

$= 75.76\%$

find the maximum force which can be transfer through double cover butt joint shown in fig. find the efficiency of joint also given M_{20} bolt of grade 4.6 and Fe 410 steel plate are used.



Given data:-

bolt = M_{20}

grade = 4.6

$f_u = 400$

$Fe = 410$

$b = 200$

$\gamma_{mb} = 1.25$

$d = 20$

$e = 40$

$p = 60$

It is to be check along all three section

Shearing strength of bolt

$$V_{dsb} = \frac{f_u}{\sqrt{3}}\left(n_n A_{nb} + n_s A_{sb}\right)$$

$$= \frac{400}{\sqrt{3}}\left(1 \times 0.78 \times \pi/4 \, (d)^2 + 1 \times \pi/4 \, (d)^2\right)$$

$$= \frac{400}{\sqrt{3}}\left(1 \times 0.78 \times \pi/4 \times (20)^2 + 1 \times \pi/4 \, (20)^2\right)$$

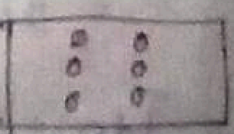
1 bolt $= \frac{129142}{1.25} = 103314 \, N$

Design strength of joint in double shear

no. of bolt $= 6 \times 103314 \, N$

$= 619.884 \, kN$

Bearing strength of bolt



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

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

$$k_b \begin{cases} \dfrac{e}{3d_0} = \dfrac{40}{3\times22} = 0.6060 \\[2ex] \dfrac{p}{3d_0} - 0.25 = \dfrac{60}{3\times22} - 0.25 = 0.6590 \\[2ex] \dfrac{f_{ub}}{f_u} = \dfrac{400}{410} = 0.97 \\[2ex] 0.1 = 1 \end{cases}$$

$k_b = 0.6060$

$2.5\, k_b d t f_u$

$= 2.5 \times 0.6060 \times 20 \times 16 \times 410$

$= 198.768$ kN

design strength of 6 bolt joint =

$= 3(2.5 \times 0.6061 \times 20 \times 16 \times 410) + 3(2.5 \times 0.659 \times 20 \times 16 \times 410)$

$= 1244957$ N

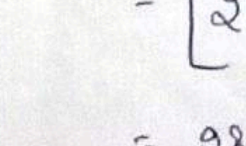
Design strength of joint plate at (section A-A)

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

$A_n = [b - n d_0]t$

$= [200 - 1\times22]\,16$

$= 2848$



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

$= 0.9 \times 2848 \dfrac{410}{1.25} = 840729.6$ kN

(b) When this section fail bolt in section (1-1) also has to fail hence strength plate of section (2-2)

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

$A_n = [b - n d_0] t$

$= [200 - 2 \times 22] 16$

$= 2496$

$T_{dn} = 0.9 \times 2496 \times \frac{410}{1.25} + 103314$

$= 942851 N$

(c) Section [3 - 3]

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

$A_n = [b - n d_0] t$

$= [200 - 3 \times 22] 16$

$= 2144$

T_{dn} = plate strength + strength of 3 bolts

$= 0.9 A_n f_u / \gamma_{m1}$

$= 0.9 \times 2144 \times \frac{410}{1.25} + 3 \times 103314$

$= 942851 N$

Strength of plate in the joint = 840133.2N

Force at working condition = $\frac{\text{Design Shear Strength}}{1.5}$

$$= \frac{619.884}{1.5}$$

$$= 413.256 \text{ KN}$$

Design Strength of solid plate

$$\frac{f_y}{\gamma_{mo}} \times A_g$$

$$= \frac{250}{1.1} \times 200 \times 16$$

$$= 7272.72$$

Efficiency of joint = $\frac{\text{Strength of joint}}{\text{Strength of solid plate}} \times 100$

$$= \frac{619.884}{7272.72} \times 100$$

$$= 85.23\% \text{ (Ans)}$$

WELDED CONNECTION

INTRODUCTION:-

→ Welding consist of joining two piecee of metal by establishing and Metallurgical bond between them through the application of pressure and ion through fusion.

→ In other wards welding is method of connecting two pieces of metal by heating to plastic or fluid stage (fusion)

→ The elements to be connected are brought closer and the metal is melted by means of electric arc or oxyacetylene flam along with weld rod which adds metal to the joint. The bond is established between the two elements after cooling.

TYPES WELD AND WELDED JOINTS:-

The basic type of welded joints are classified depending open the type of weld. Mainly there are three types of welds

1. Butt weld
2. fillet weld
3. Slot weld & Plug well

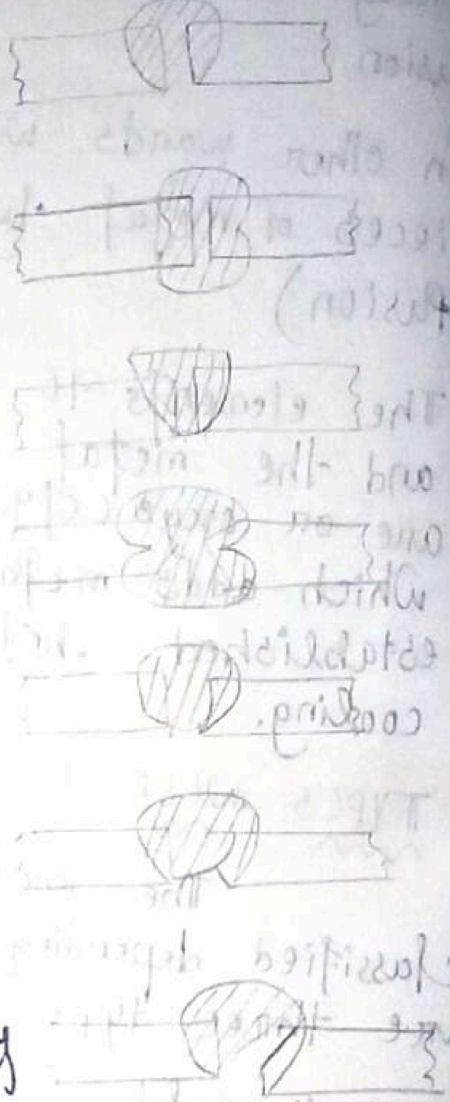
Butt weld:-

This is also otherwisely know as groov weld. Butt welds are provaided when the member to be joined are Placed end to end or aligned in the same plane.

⇒ Depending upon the shape of the groov made for welding various type of groove welds are listed as follow

Type of butt weld

Sl.no	Type of butt weld on one side	Sketch
(a) (a)	Square butt weld on one side	
(b)	square butt weld both side	
(c)	singl v butt joint	
(d)	Double - V - butt joint	
(e)	single U butt joint	
(f)	single J butt joint	
(g)	Single bevel butt joint	



⇒ Butt / Groove weld require edge preparation and single V, U, J etc. butt joints are cheaper to form the weld metal then the but require double the double grooved joints

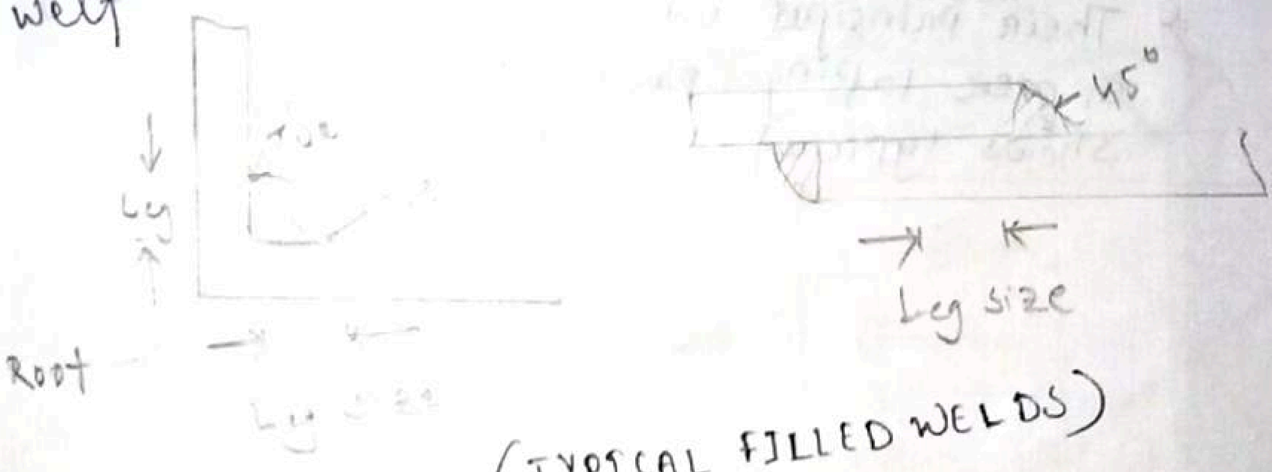
⇒ The choice between the single and double groove depends wheather cost of surface perparation is offeset by saving in the weld metal, square butt welds are used for plates upto 8mm thickness only.

> It is must suitable for transmitting alternating stresses since there will be no stress concentration due to absence of abrupt change of section.

Fillet Weld:-

> Fillet welds are provaided when two member to be joined are in diffirent planes.

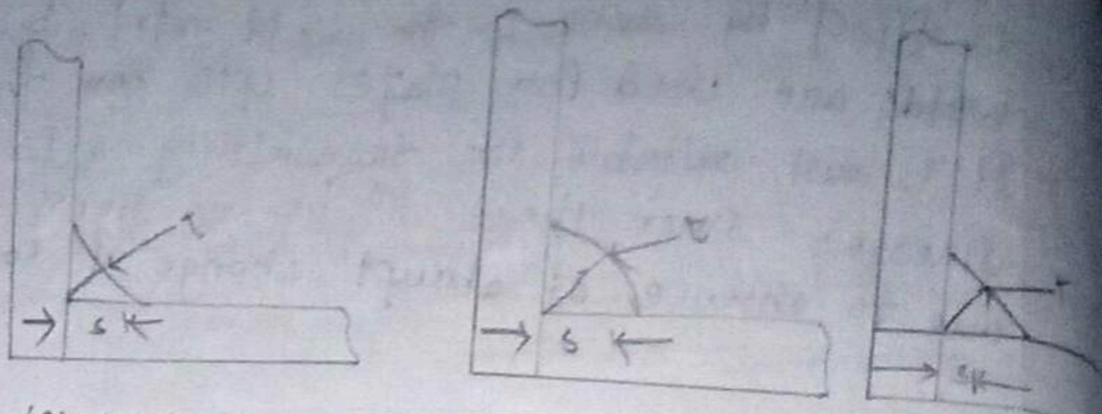
> This type of situation is frequently met within the steel structures. fillet weld is a weld of approximately triangular cross-section joining two surface nearly at right angle to each other in lap tee or corner types of joint. fig show typical fillt welt



(TYPICAL FILLED WELDS)

> When the cross-section of fillt weld is isoceles tringle with face at 45°. It is called as a standard fillet weld.

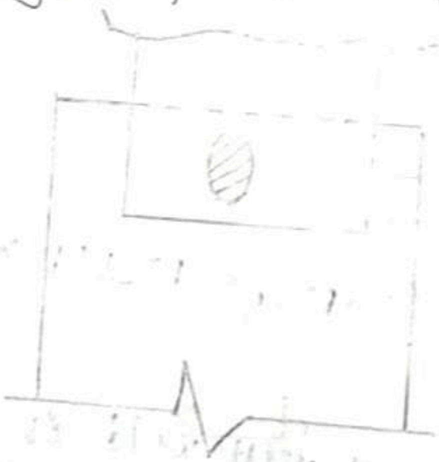
> In special circumstance 30° & 60° angle may be used, Depending upon the shape of weld face, a fillet weld is know as concave fillet weld, convex fillet weld or as mitre fillet weld.



(a) concave (b) convex (c) MITRE

Slot and plug welds :-

- Slot and plug welds are used to supplement the fillt welds, when the required length of fillt weld can not be provaid.
- The penetration of these weld into base metal is difficult to ascertain and the inspection of these weld is difficult.
- Their principal use is prevention of buckling in over laping parts fig. shows typical slot or plug welds.



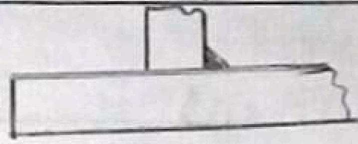
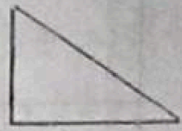
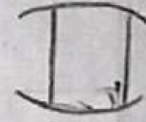
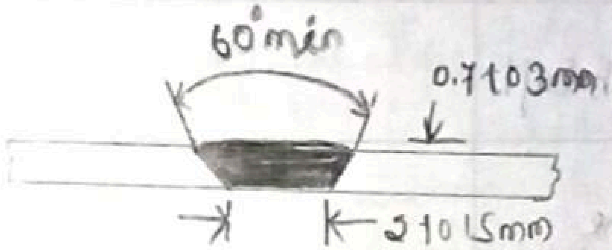
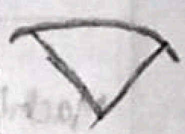
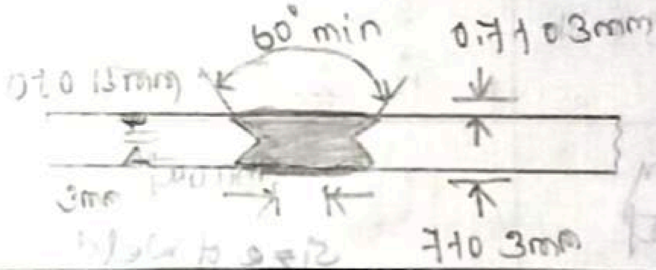
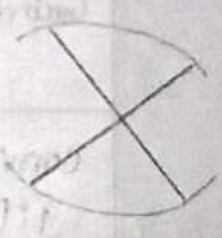
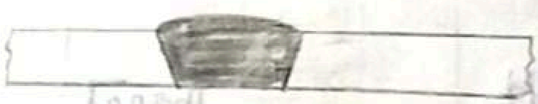
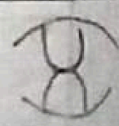
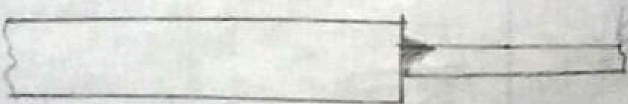
(Slot weld)

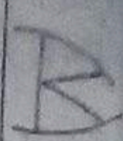
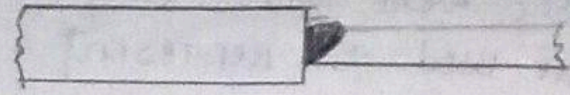
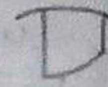
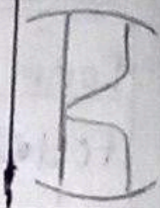
- In slot weld, slots (rectangular or circular holes) are made on one of the plates, which is kept with another plate and then fillet welding is made along the peciphery of the hole.
- In case of plug welds, small holes are made in one plate and is kept over another plate to be connected and then entire hole is

filled with filler material.

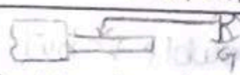
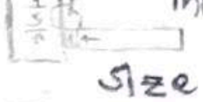
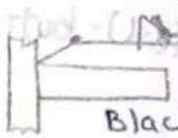
Weld symbols And supplimentary Drawings

Some basic weld symbols along with supplimentary drawing are used to represent a welded joint in a drawing.

form of weld	section	symbol
Fillet		
Square butt		
Single-V-butt		
Double V-butt		
Single-U-butt		
Double-U-butt		
Single-bevel-butt		

Double-belev-butt		
Single-J-butt		
Double-J-butt		

Supplymentary drawing representation

Form of weld	Section	Drawing representation
Flush butt		 Flush squre butt
Machine finish		 Double V-butt finish flush on other side
Grinding finish		 Double beleve butt finished then arrow side. Arrow mark on one side only indicate the section prepared for
convex fillet	 Throat Size of weld	
Concave fillet using field weld	 Throat	 Black circle indicates field weld
weld all round		 circle indicates filled well all around the section.

WELDING PROCESS:-

Differrent welding process are in use namely (?)

(i) Forge welding

(ii) Thermit welding

(iii) Gas welding

(iv) Resistance welding

(v) Electric arc welding

> Structural steel work, only electric are welding is used which is essentially a fusion process. In this process intense heat is produced by means of an electric arc struck between the parts to be welded, when an electrods is held manually or by automatic. During welding, the electrode melts and fills the gap of the joint.

=> Generally, the electrode used for welding is stonger then the parent metal. For example the electrode used for welding structural steel Fe410 is E41, which gives a weld deposit of yield strength of 330 Mpa and ultimate tensile strength of 410 to 510 Mpa.

ADVANTAGES OF WELDED CONNECTION:-

1. Welding is more adaptable then bolting or riveting as even circular sections can be easily connected by welding.
2. Full strength of a joint can be devloped i.e 100% efficiency can be achived in contrast to bolted or riveted connection which can reach a maximum of 70-80% efficiency.

3. Since there is deduction for holes, the gross section is effective in carrying loads and their is no problem of mismatching.
4. Better resistance against fatigue impact and vibration loads.
5. Result in lighter structures due to absence of connecting plates, gusset plates etc.
6. Quicker & speedier as these is no necessity of making holes for fasteners.
7. Noise pollution is nearly eliminated.
8. Presents good asthetic apperance
9. connections are water and air tight.
10. Welded joints are rigid and alteration in the connection can be easily made in the design.

Dis-advantages of welded connection:-

1. Skilled labour and electricity is necessary for welding.
2. Due to uneven heating and cooling, internal stresses and warping devlops.
3. Welded joints are more brittle and their fatigue strength is less.
4. Difficult to detect defect like internal air pockets, slag inclusion and incomplete penetration.
5. Welded joints are over rigid and proper welding in field condition is a difficult task.
6. Proper welding in ~~field condition~~ is

I.S CODE PROVISION FOR WELDING :-

For provision regarding the requirement of welds and welding IS : 816 & IS : 9595 as appropriate may be refered to. However, some important provision regarding butt weld, fillet weld and plug or slot welds as per IS - 800 : 2007 are as follow:

BUTT WELD:-

(i) Reinforcement :-

⇒ It is the extra weld metal deposited above the surface level of the parent metal to be connected.

⇒ It increases efficiency of the joint and makes the butt weld stronger for static loads. But in case of repeated or vibrating loads, stress concentration develops in the reinforcement leading to early failure. In any case the reinforcement should not exceed 3 mm.

SIZE :-

⇒ The size of the butt weld is specified by the throat dimension, which is also called the effective throat thickness.

⇒ The butt weld may be of partial penetration or complete penetration butt weld penetration. In case of a complete penetration the size of the weld shall be taken as thickness of the thinner part joined. Double U, Double V, double J and double bevel butt welds come under this category. But effective throat thickness in case of partial or incomplete penetration butt weld shall be taken as the minimum thickness of the weld metal common to the parts joined, excluding reinforcement.

⇒ In the absence of actual data, it may be taken as 3/8th of thickness and the effective of thinner material to be joined.

Effective Length:-

⇒ The effective length of butt weld shall be taken as the length of full size weld.

⇒ The effective area of the butt weld is the product of the effective throat thickness and the effective Length of the butt weld.

⇒ The minimum length of butt weld shall be four times the size of weld.

⇒ Where intermittent butt welding is used, it shall have an effective length of not less then four times the weld size and the spacing between the two welds shall be not more than 16 times the thickness of the thinner part joined.

fillet weld:-

SIZE:-

The size of fillet weld is specified as the minimum leg length of the weld i.e the length of the sides of the largest right angled tringle in the cross-section of the fillet weld.

(a) The size of normal fillet weld shall be taken as the minimum weld leg size.

(b) for deep penetration welds when the penetration is not less then 2.4mm. size of the weld is minimum leg size plus 2.4mm.

(c) for fillet welds made by semi-automatic and automatic process with deep penetration more than 2.4mm, the size of the weld (S) = minimum leg size + actual penetration

Minimum size:-

To avoid the risk of cracking in the absence of pre-heating the minimum size of fillet weld specified are as follows.

Thickness of thinner part	minimum size of weld
upto and including 10mm	3mm
over 10mm upto and including 20mm	5mm
over 20mm upto and including 32mm	6mm
over 32mm upto and including 50mm	8mm first run
	10mm minimum

when the minimum size of the fillet weld is greater then the thickness of the thiner part, the minimum size of the weld should be equal to the thickness of thiner part. The thicker part shall be adequatly preheated to prvent cracking of the weld

(iii) Maximum size:-

The maximum size of fillet weld applied to the square edges of a plate or shape should be 1.5mm less then the nominal thickness of the edges.

=) The size of the fillet weld used along the toe of an ange or the roundede edgs of flange should not exceed three fourths the nominal thickness of an angle and or flung leg.

=) In members subjected to dynamic loading the fillet weld shall be of full size with its leg length equal to the thickness of the plate.

(iv) Throat thickness:-

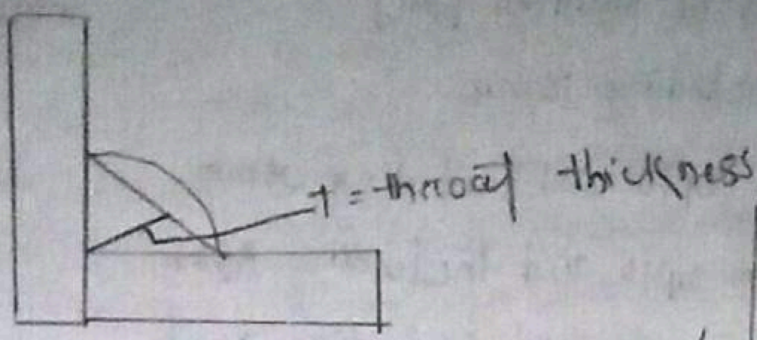
The throat of a fillet is the length of perpendicular

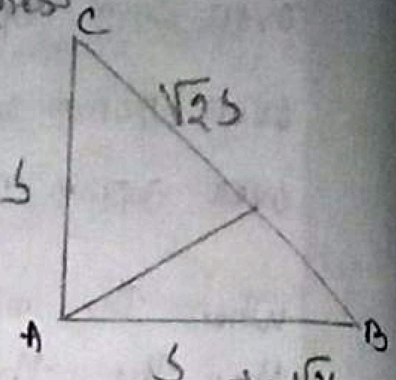
=) from the right angle corner to the hypotenuse.

=) The effective thickness of throat is calculated as

Throat thickness = K x fillet size

=) The value of K depends upon the angle between the fusion ~~fase~~





from Δ BOC

$$BD = \sqrt{\left(\frac{\sqrt{2}s}{2}\right)^2 - (s)^2}$$

$$BD = \sqrt{s^2 - \left(\frac{\sqrt{2}}{2}\right)^2 s^2}$$

$$= \sqrt{\left(1 - \left(\frac{\sqrt{2}}{2}\right)^2\right) s^2} = \frac{s}{\sqrt{2}} = 0.70s$$

$$t = \frac{\sqrt{2}s}{2} = \frac{s}{\sqrt{2}} = 0.7$$

Angle between fusion faces	60-90°	91-100	101-106	107-113	114-1
Constant k	0.7	0.65	0.60	0.55	0.50

In most cases, a right angled fillet is used, for which

$$\boxed{K = \frac{1}{\sqrt{2}} \approx 0.7}$$

The effective throat thickness shall not be less than 3mm and shall not generally exceed 0.7t (or t under special circumstances), where t is the thickness of the thinner plate of the elements being welded.

The effective thoat thickness shall not be less than 3mm and shall not generally exced 0.7t (or t under special circumstance), where t is the thickness of the thinner plate of the elemengs being welded

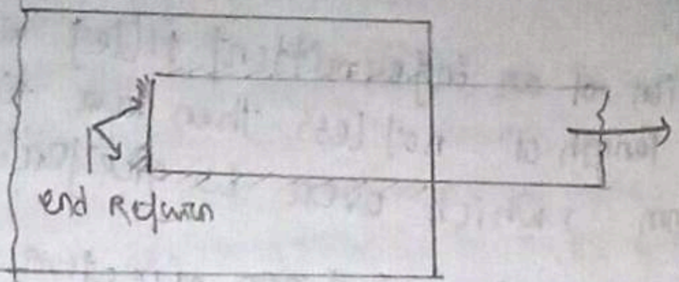
Effective length:-

The effective length of a fillet weld is equal to its overall length minus twice the weld size.

The effective length of a fillet weld designed to transmit load should not be less then 4 times weld size.

End Return:-

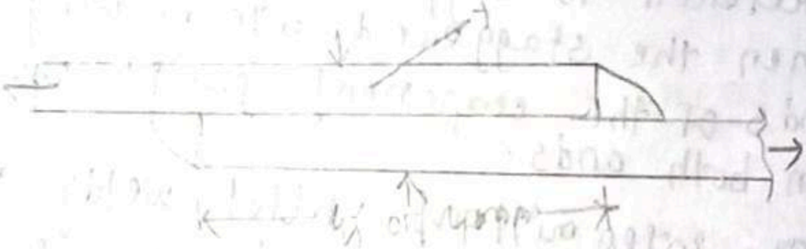
The fillet weld terming at the end or side of a member should be returned around the corner when - ever practicable for a distance not less than twice the weld size as shown in fig.



This provision applies in particular to fillet welds in tension.

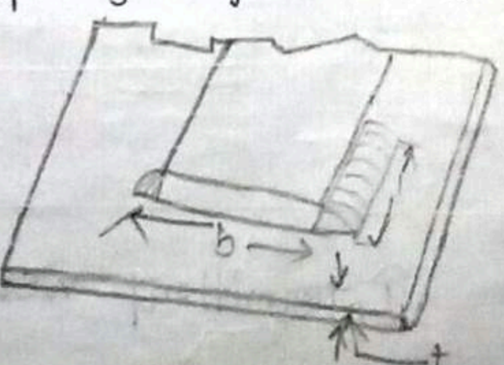
(vii) Over lap:-

The minimum lap in a lap joint should not be less then four times the thickness thinner plates joined or 40mm which ever is more.



(viii) Side fillet:-

When end of an element is connected only by parallel longitudinal welds, the length of the weld along either edge should not be less then the transverse spacing between longitudinal welds.



(iv) Intermittent fillet weld:-

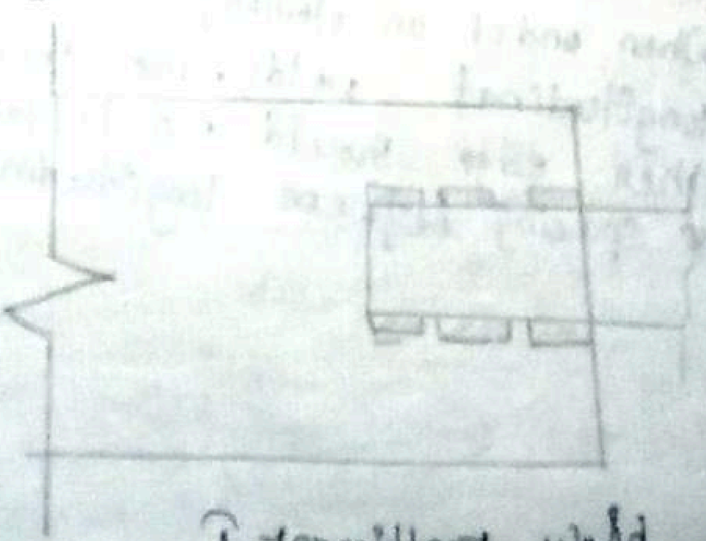
Intermittent fillet weld may be used when length of the smallest size fillet weld required to transmit stress is less then the continuous length of the joint.

(a) Any section of an intermittent fillet weld should have a effective length of not less then four times the weld size 40mm , which ever is greter.

(b) The clear spacing between effective lengths of intermittent fillet weld carrying stresses should not exced 12t for compression and 16t for tension and in no case be more than 200mm where t = thickness of the thinner part joined.

(c) Generally chain intermittent fillet welding is to be prefered to staggered intermittent to welding. when the staggered intermittent welding is used the ends of the component parts should be welded on both ends.

(d) The intermittent fillet welds shall not be used in position subjected to dynamic repreatative alternating stresses.



Intermittent weld

(x) Single fillet weld:-

A single line of fillet weld, when used, should not be subjected to moment about the longitudinal axis of the weld.

Plug and slot welds:-

(i) Size:- (a) Width or diameter should be not less then 3t or 25 mm which ever is more

(b) Corner radius in slotted holes should not be less then 2t or 25mm which ever is greater

(ii) Spacing:- Clean distance between the holes should not be less then 2t or 25mm which is more, where it is the thickness of the plate having a hole or slot.

(iii) Combination:- A combination of Plug weld and other types of weld is permissible and the strength of the joint is the sum of the individual capacities of the welds.

Design stresses & Design strength of WELDS:-

Like bolts, weld could be classified as shop welds and field or site welds. Since the shop welding is done under controlled condition, its strength is more than the site welds. This fact has been taken into account in the partial safety factor for weld material.

Fillet weld, slot or plug welds:-

It resistance the external load by shear action only. The tensile stress in the member is transferred to shearing stress in the weld. Design strength of weld shall be based on its throat area and is given by $f_{wd} = \frac{f_{wn}}{\gamma_{mw}}$

Where f_{wn} = nominal strength of the fillet weld $\frac{f_u}{\sqrt{3}}$, f_u

f_u being the ultimate stress of weld or the parent metal and γ_{mw} = partial safety factor of weld material = 1.25 for shop welds & 1.5 for field welds.

Butt welds:-

Butt welds shall be treated as parent metal with a thickness equal to the throat thickness and the stress shall not exced those permitted in the parent metal.

It can resist external load by tension or compression and shear action.

Design stress of the butt/groove weld in tension or compression $f_{w} = \frac{f_u}{\gamma_{mw}}$

where f_u = smaller of ultimate stress of weld and the parent metal.

γ_{mw} = partial safety factor for weld material

Design stress of butt weld in shear is given by $f_{w} = \frac{f_{yw}}{\sqrt{3}\gamma_{mw}}$

where f_{yw} = characteristic yeild stress of web material

This is to be multiplied with effective throat area $= (L_w \times t_e)$ where L_w = effective length of the weld and t_e = effective throat thickness.

Reduction of Design Stress for Long Joints:-

When the length of the welded joints in the direction of force transfer 'l' is greater than $150t_p$ the design capacity of the weld f_{wd} shall be reduce by a factor $\beta_{lw} = 1.2 - \frac{0.2l}{150t} \leq 1.0$.

STRESS DUE TO INDIVIDUAL FORCES:-

When subjected to either compressive, tensile or shear force alone, the stresses in the weld is given by

$$f_a \text{ or } q = \frac{P}{t_t \times l_w}$$

Where f_a - calculated normal stress due to axial forces

q = shear stress

P = force transmitted

t_t = effective throat thickness of the weld

l_w = effective length of the weld.

ASSUMPTION IN THE ANALYSIS OF WELDED JOINTS:-

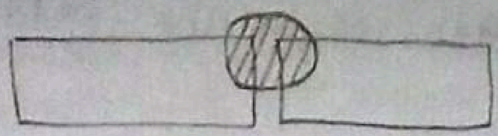
(a) Welds connecting various parts are homogeneous, isotropic and elastic.

(b) The parts connected by the welds being rigid, their deformation is usally neglected.

(c) only stress due to extranal loads are considered and the effects of residual stress, stress concentration and shape of the welds are neglected.

* **Case-1**

for single side penetration



Design strength = crossection area x permissible stress

$C/A = B \times 5/8 \times t_{min}$

Where

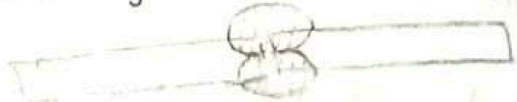
Effective throat thickness = $5/8\, t_{min}$

$P.S = \frac{f_y}{\gamma_{mw}}$ $\quad \gamma_{mw}$ → 1.25 (workshop) → 1.5 (field)

$\boxed{\text{Butt welded connection} = B \times 5/8 \times t_{min} \times \frac{f_y}{\gamma_{mw}}}$

Case-2

Double side penetration



Design strength = crossection area x permissible shear stress

$C/A = B \times t_{min}$

effective throat thickness = t_{min}

$P.S = \frac{f_y}{\gamma_{mw}}$ $\quad \gamma_{mw}$ → 1.25 (workshop) → 1.5 (field)

$\boxed{\text{Butt welded connection} = B \times t_{min} \times f_y / \gamma_{mw}}$

* Design a butt welded connection to connect 40mm 16mm thick plates, plates section are subjected to 450KN factor load the effective length for welded person 175 mm check the safety of the connection

(i) Single side penetration

(ii) Double side penetration

Given data:

(i) One side penetration

$B \times 5/8 \times t_{min} \times \frac{f_y}{\gamma_{mw}}$

$= 175 \times 5/8 \times 14 \times \frac{250}{1.25}$

$= 306.25$

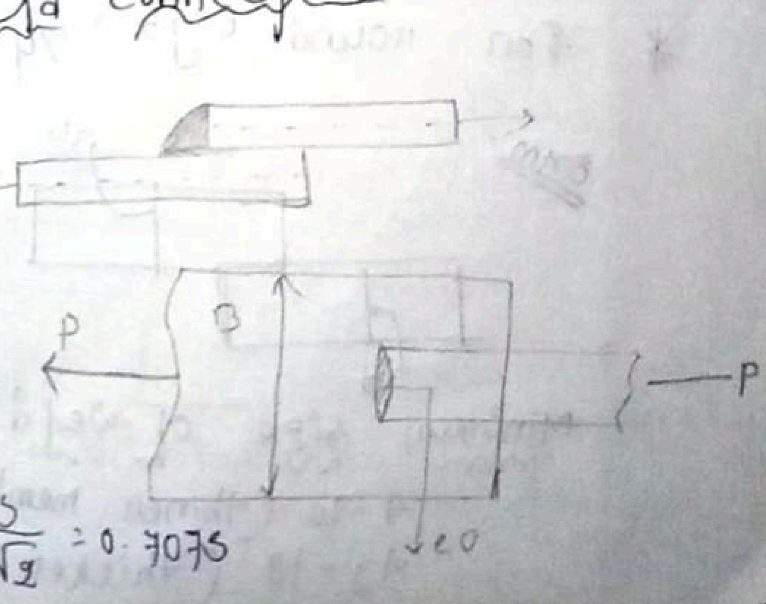
(ii) Double side penetration

$B \times t_{min} \times f_y/\gamma_{mw}$

$= 175 \times 14 \times \frac{250}{1.25}$

$= 490000$

Design of fillet weld connection:-



From ΔBDC

$BD = \sqrt{\left(\frac{\sqrt{2}s}{2}\right)^2 - (s)^2}$

$BD = \sqrt{s^2 - \left(\frac{\sqrt{2}}{2}\right)^2 s^2}$

$= \sqrt{\left(1 - \left(\frac{\sqrt{2}}{2}\right)^2 s^2\right.} = \frac{s}{\sqrt{2}} = 0.7075$

Effective throat thickness = K × S

where K = Design constant

, S = size of weld

Refer tabul 22 the value of K for diffrent angle between fusion.

Minimum size of weld (desaide by thicker Member)

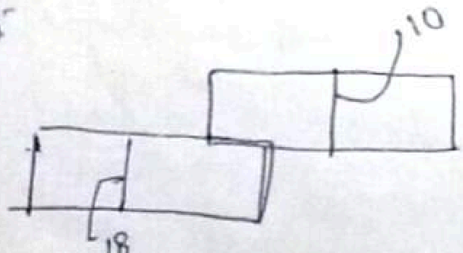
Tabul-2

Thickness of thicker member	min size of weld
0 - 10	3
10 - 20	5
20 - 30	6
32 - 50	8

Maximum size of weld (Deside by thinner Member)

* Max size of weld for square edgs = tmin - 1.5
* For round edgs = 3/4 × nominal thickness

Exm:-



Minimum size of weld :-

$t_1 = 10$ (thiner member)

$t_2 = 18$ (thicker member)

minimum size = 5 Square Edgs

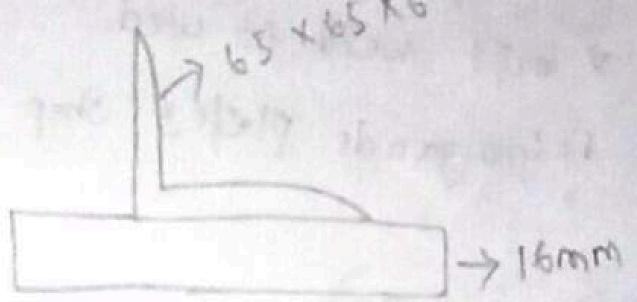
Maximum size = $t_{min} - 1.5$

$= 10 - 1.5$

$= 8.5$

size of weld = [5 to 8.5]

<u>Minimum size of weld:-</u>



$t_1 = 6$

$t_2 = 16$

minimum size decide by thicker member

min size = 5

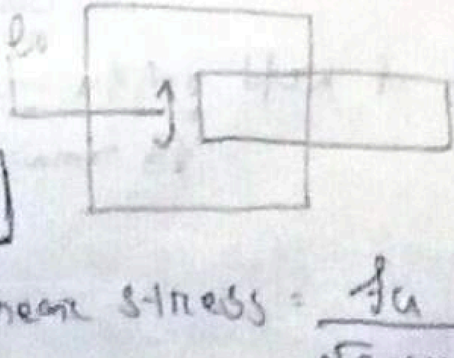
Minimum size = $t_{min} - 1.5$

$= 6 - 1.5$

$= 4.5$

Round B = $\frac{3}{4} \times 6$

effective length weld is required to transmit the apply force that is $l_{eff} = L_0 - 2S$



$\boxed{l_{eff} = L_0 - 2S}$

Permisible shear stress = $\frac{f_u}{\sqrt{3} \times \gamma_{mw}}$

where γ_{mw} — 1.5 (field), 1.25 (workshop)

Fillet weld connection $= k \times s \times l_{eff} \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}}$

cross section   permissible shear stress

* A 18mm thick plate is jointed to a 16mm plate by 200mm long (effective) butt weld. Determine the strength of joint if

(1) A double V butt weld is used

(2) A single V butt weld is used

Assume the Fe410 grade plates shop welds are used.

Given data:- (single)

$t_1 = 18mm$ $f_u = 410 N/m^2$

$t_2 = 16mm$ $\gamma_{mw} = 1.25$

Effective length $L_0 = 200$

effective throat thickness $= \frac{5}{8} \times t_{min}$

$= \frac{5}{8} \times 16$

$= 10$

effective area of weld = effective length x effective throat thickness

$= 200 \times 10$

$= 2000 mm^2$

Design strength of weld $= \frac{A f_u}{\sqrt{3}\, \gamma_{mw}} = \frac{2000 \times 410}{\sqrt{3} \times 1.25}$

$= 378.47 KN$

Double :-

Effective throat thickness = 16

effective area of weld = effective length x effective throat thickness

$= 200 \times 16$

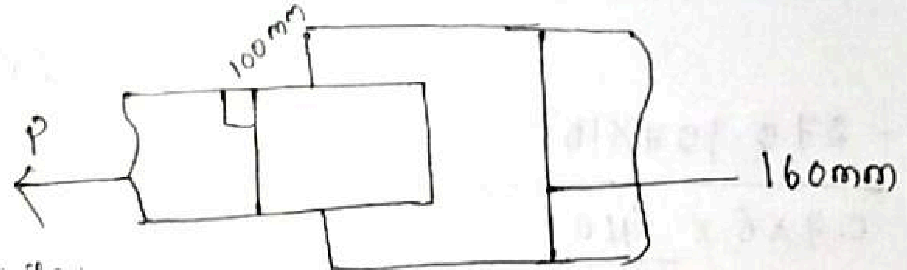
$= 3200 mm^2$

Design strength of weld $= \dfrac{A f_u}{\sqrt{3}\, \gamma_{mw}}$

$= \dfrac{3200 \times 410}{\sqrt{3} \times 1.25}$

$= 605.987 KN$

* Design a suitable longitudinal fillet weld to connect plate as shown in fig to transmit a pull equal to the full strength of small plate given plates fe 410 and welding to be made in workshop.



Given data :-

t = 12mm

fe 410 plates

fillet weld

size of weld :-

Minimum size of weld = 5
(thicker)

Maximum size of weld = tmin - 1.5
(thinner member) = 12 - 1.5
= 10.5 m

[5 to 10.5 mm]

Assum $S = 10mm$

Strength of plate

Full design strength of smaller plate $T_{dg} = A_g \times \frac{f_y}{\gamma_{m0}}$

$A_g = b \times t$

$= 100 \times 12$

$= 1200 \times \frac{250}{1.1}$

$= 272.727 kN$

Assum normal weld throat thickness

$t = KS$

$= 0.7 \times 10$

$= 7mm$

Design strength of weld $= KS\, l_{eff} \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}}$

$272.727 \times 10^3 = 0.7 \times 10 \times l_{ef} \times \frac{410}{\sqrt{3} \times 1.25}$

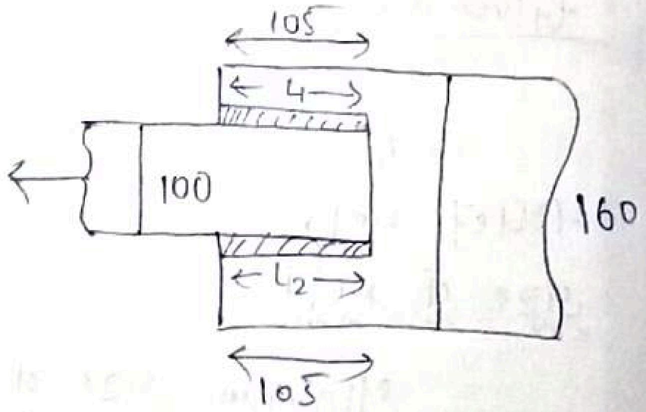
$l_{ef} = \frac{272.727 \times 10^3}{0.7 \times 6 \times \frac{410}{\sqrt{3} \times 1.25}}$

$l_{ef} = 205.73 \simeq 210$

$L_1 + L_2 = 210$

$L_1 = \frac{210}{2}$

$= 105mm$



Provaide effective length of 105 mm on each side.

* A tie member of a roof truss consist of 2ISA 100×75×8mm the angle are connected to either side of a 10mm gussplates and the member is subjected to a working full of 300KN. Design the welded connection assum the connection are made in workshop.

Given data:-

2ISA 100×75×8mm

Working load = 300KN

factor load = 1.5×WL

= 1.5×300

= 450 KN

Thickness of gusst plate = 10mm

γ_{mw} = 1.25

Size of weld:-

At the rounded to of the angle section size of the weld should not exit = 3/4×t

= 3/4×8

= 6mm

At top thickness should not exceed (S) = t_{min} - 1.5

= 8 - 1.5

= 6.5mm

Hence provaided S=6mm weld

Assuming normall well effective throat thickness

t = K×S

= 0.7×6

= 4.2

Each angle carries a factor pull of $\frac{450}{2}$ = 225KN

Let, Lw be the total length of the weld required

Lw is the required

$$= K \times S \times l_{eff} \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}}$$

$$= 0.7 \times 6 \times l_{eff} \times \frac{410}{\sqrt{3} \times 1.25}$$

$$l_{eff} = \frac{225 \times 10^3 \times \sqrt{3} \times 1.25}{0.7 \times 6 \times 410}$$

$$= 282.89 \simeq 283$$

centre of gravity of the angle section at a distance 31mm from top

Let L_1 be the length of the top weld

L_2 be the Length of the lower web, to make the centre of gravity of the weld to coinside with datam angle.

$$L_1 \times 31 = L_2 \times (100 - 31)$$

$$\Rightarrow L_1 \times 31 = L_2 \times 69$$

$$\Rightarrow L_1 = \frac{L_2 (69)}{31}$$

$$\Rightarrow L_1 + L_2 = (L_w) = 283$$

$$\Rightarrow L_2 \left(\frac{69}{31}\right) + L_2 = 283$$

$$\Rightarrow L_2 \left(\frac{69}{31} + 1\right) = 283$$

$$L_2 = 88 \text{ mm}$$

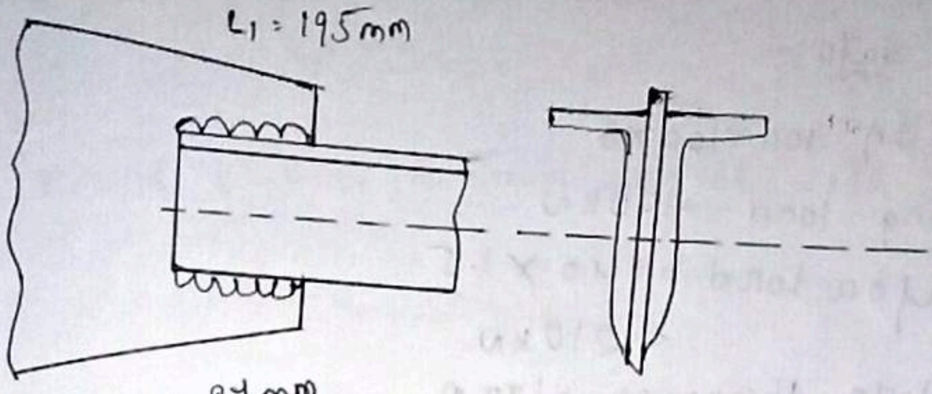
$$L_1 = 283 - L_2$$

$$= 195 \text{ mm}$$

Provide 6mm weld of $L_1 = 195\,mm$

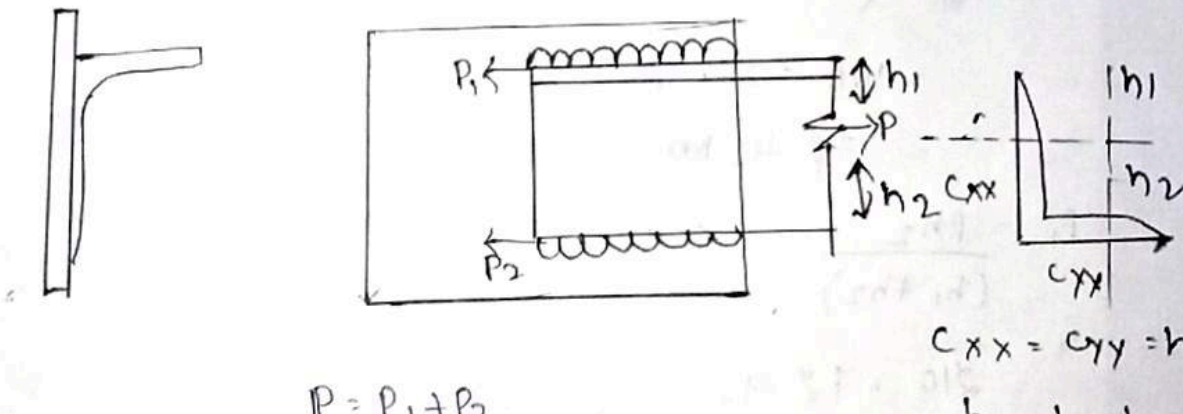
$L_2 = 87$

as shown in fig.



Case-1

When single section is connected with gussplate and welding is provaided on two side of angle section



$C_{xx} = C_{yy} = h_1$

$h_2 = b - h_1$

$$P = P_1 + P_2$$

$$b = h_1 + h_2$$

Moment about 2

$$Ph_2 - P_1(h_1 + h_2) = 0$$

$$\boxed{P_1 = \frac{Ph_2}{(h_1 + h_2)}}$$

* A member of truss consist of NISA 100x100x8mm angle section is subjected to 140KN working load thickness of gussplate is 12mm design the welded connection $C_{xx} = C_{yy} = 27.6$ mm, $\gamma_f = 1.5$.

Given data:-

NISA 100x100x8

working load - 140KN

factor load = 140 x 1.5

= 210 kN

gussplate thickness = 12mm

$C_{xx} = C_{yy} = 27.6$ mm

$\gamma_f = 1.5$

size of weld:-

$h_2 = (b - h_1)$

$= 100 - 27.6$

$= 72.40$ mm

$P_1 = \dfrac{P h_2}{(h_1 + h_2)}$

$= \dfrac{210 \times 72.40}{27.6 + 72.40}$

$= 152.04$ km

$P_2' = b - P_1$

$= 210 - 152.04$

$= 57.96 \approx 58$

~~Provaide 6mm weld~~

~~Design Streng~~

<u>size of weld</u>:-

squre edgs maximum size of weld $= t_{min} - 1.5$

$= 8 - 1.5$

$= 6.5$

Round edgs of maximum size of weld $= 3/4 \times t$

$= 3/4 \times 8$

$= 6$

$[6 \text{ to } 6.5]$

Assum $s = 6$

Provaide 6mm weld

Design strength of weld $= ks\, l_{eff} \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}}$

$$P_1 = ksL_1 \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}}$$

$$152.04 = 0.7 \times 6 \times L_1 \times \frac{410}{\sqrt{3} \times 1.25}$$

$= 191.15$ mm

$$P_2 = ksL_2 \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}}$$

$$57.96 = 0.7 \times 6 \times L_2 \times \frac{410}{\sqrt{3} \times 1.25}$$

$= 72.87$

$L_w = L_1 + L_2$

$= 191.15 + 72.87$

$= 264.02$ mm

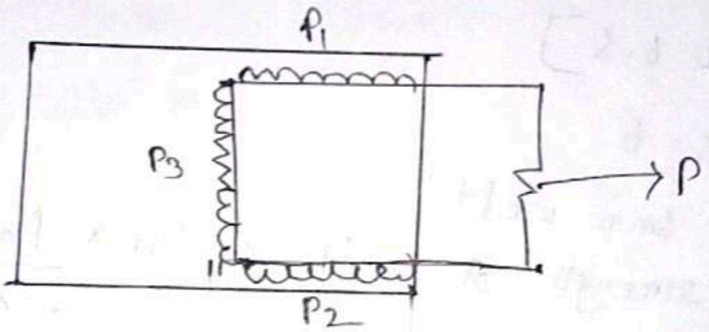
$$P = 0.7 \times 6 \times l_{eff} \frac{410}{\sqrt{3} \times 1.25}$$

$$210 = 0.7 \times 6 \times l_{eff} \frac{410}{\sqrt{3} \times 1.25}$$

$$= 264.03 mm$$

Case - 2

When single angle section is connected with gussplate and welding is provided on 3 side angle section.



$$P = P_1 + P_2 + P_3$$

Moment about Point (2)

$$Ph_2 - P_1 (h_1 + h_2) - P_3 \left(\frac{h_1 + h_2}{2}\right) = 0$$

$$P_3 = KS\, l_{ef} \frac{f_u}{\sqrt{3} \times 1.25}$$

$$KS (h_1 + h_2) \frac{f_u}{\sqrt{3} \times 1.25}$$

* A steel plate 200 x 12mm is welded to 10mm thick gussplate such that the overlap of the member is 250mm if fillet weld of size 6mm is used for the connection determine the design strength of joint given that shop welding is done on 3side and grade of steel is Fe 410.

steel plate = 200 x 12 mm

$t_1 = 12\,mm$

$b = 200\,mm$

gussplate thick = 10mm

size of weld (s) = 6mm

$\gamma_{mw} = 1.25$

$f_u = 410\ KN/mm^2$ $\quad f_y = 250$

length of weld $l_{eff} = 250 + 200 + 250 = 700$

effective throat thickness $t_e = K \times S$

$= 0.7 \times 6$

$= 4.2\,mm$

Design strength of weld $= K \times S\ l_{eff} \times \dfrac{f_u}{\sqrt{3} \times \gamma_{mw}}$

$= 0.7 \times 6 \times 700 \times \dfrac{410}{\sqrt{3} \times 1.25}$

$= 556.750\ KN$

Design strength of plate on yielding $T_{dg} = A_g \times \dfrac{f_y}{\gamma_{m0}}$

$A_g = b \times t$

$= 200 \times 12$

$= 2400\,mm^2$

$= 2400 \times \dfrac{250}{1.1}$

$= 545.45\,KN$

Strength of joint = minimum of weld or plate

strength = 545.45KN.

* A tie member of a roof truss consist of 2ISA 90 X 60 X 8mm the angles are connected on either side of gussplate and the members are subjeged to a factor pull of 360KN, design the weld connection assum welding is to be made in field

<u>Given data</u>:-

2ISA 90 X 60 X 8

$P = 360KN$

$\gamma_{mw} = 1.5$

$t = 8$

size of weld in round edgs $= 3/4 \times t$

$= 3/4 \times 8$

$= 6mm$

size of weld in squre edgs $= t_{min} - 1.5$

$= 8 - 1.5$

$= 6.5mm$

~~leff~~ strength of weld $= \dfrac{K s\, l_{eff}\, f_u}{\sqrt{3} \times \gamma_{mw}}$

$l_{eff} = \dfrac{180 \times 10^3 \times \sqrt{3} \times 1.5}{0.7 \times 6 \times 410}$

$= 271.57\ mm = 280mm$

$L_1 + L_2 = 280$ —— (1)

$29.6 L_1 - (90 - 29.6) L_2 = 0$ (∵ 2.96 center of grav... from steel table pg...

$\Rightarrow L_1 = 187mm$

$L_2 = 280 - 187$

$= 93mm$

Tension Members

→ Tension member are linear member predominatly subjected to pulling (Direct axial tensile load which ten to streech or elongate the member)

→ Exmple of tension member :- tension member in a truss know as tie.

(ii) Suspenders of cable stayed and suspension bridges

(iii) Sag rod of roof purlins and suspenders of building system hung from central core

(iv) Tension member are also used as brassing for lateral load resistance in transmission line satelite tawer, braced frames

Common shapes of Tension Members:-

Tension members can have varity of cross section fig source various forms of tension members.

(1) Wire cables / ropes

(2) circular,

(3) squre

(4) flat bars of the simplest for of tension members in use for light bridges and roof truss.

(ii) Steel section such as induction section I section Chhanel section and T-section provaide more rigidity towardws buckling in compression when reversal load takeplace under wind load.

→ Single angle section devloped bedning stress due to esntji between and connection and position of their centre of gravity.

→ Double angle and chhanel section devloped reduction less essency.

→ Builtup section are use for heavy loads. builtup section made with the arrangement up bittup section and cover plate eg angle or chhanel section and cover plate in order to provaide sufficient cross-section and to be suitable connection with adjoining member

Types of failure of tension members:-

A tension member may fail in any of the following modes

(a) Yielding of the gross section

(b) Repture of the critical section

(c) Block shear

(a) Yielding of the gross section:-

Yielding of the gross section occure when considerable deformation of in longitudinal direction take place before it rapture making the structure ~~bad~~ serveeible.

(b) Rapture of the critical section take place when the net sectional area of the member reaches the ultimate stress.

Block shear

A segment of block of material at the end of members shears out due to the possible use of high great steel and high strength of bolt resulting in smaller connection.

Design strength of Tension Members:-

Design strength of tension member is the least of the following

(1) Design strength of yielding of gross section

$$T_{dg} = A_g f_y / \gamma_{m0}$$

where T_{dg}: The design strength of member under axial tension

A_g : gross area of cross-section

f_y : yield stress of the material

γ_{m0} : partial safety factor for failure in tension by yielding $\gamma_{m0} = 1.1$

Design strength Due to Rupture of critical section

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

γ_{m1} = partial safety factor for failure of ultimate stress

f_u : ultimate stress of the material

A_n : net effective area of the member given by

$$A_n = \left[b - n d_0 + \sum \frac{p_{si}^2}{4 g_i} \right] t$$

Where,

b, t = width and thickness of the plate respectively

d_0 - diameter of the bolt hole

g - gauge length between the bolt holes as shown in fig

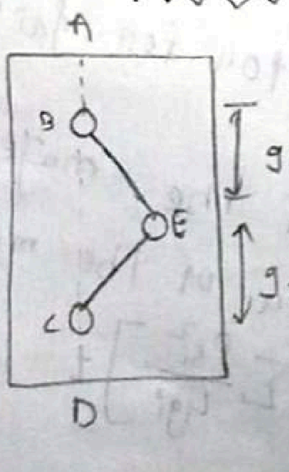
p_s = staggered pitch length between line of bolt holes, as shown in fig

n = number of bolt holes in the critical section and

i - subscript for summation of all the inclined legs.

fig:-

Net area calculation for plate section :-



A_{net}

$\rightarrow ABCD \rightarrow A_{net} \; (B - 2 \times d_0) t$

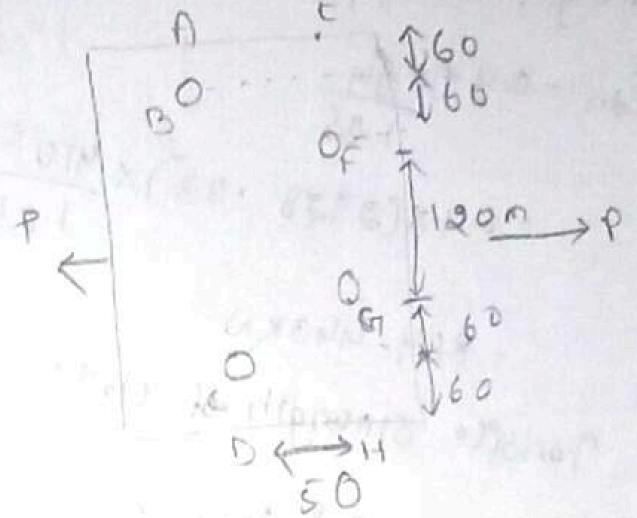
$\rightarrow ABECD \rightarrow A_{net} \left(B - 3 \times d_0 + \frac{p^2}{4g_1} + \frac{p^2}{4g_2}\right) t$

Tensile strength of plate is minimum of

(i) Net strength of critical section $T_{dn} = 0.9 A_n f_u / \gamma_{m1}$

(ii) gross strength of critical section $T_{dg} = A_g f_y / \gamma_{m0} (1.1)$

(Q) calculate tensile strength of plate. Tickness of plate is 10mm and bolt hole diameter is 22mm



$$ABCD = (B - 2 \times d_0) t$$

$$= (360 - 2 \times 22) 10$$

$$= 3160$$

$$ABFGCD = \left[B - n d_0 + \frac{p^2}{4g_1} + \frac{p^2}{4g_2} \right] t$$

$$= \left[360 - 4 \times 22 + \frac{50^2}{4 \times 60} + \frac{50^2}{4 \times 60} \right] 10$$

$$= 2928.33$$

$$ABFGH = \left[B - n d_0 + \frac{p^2}{4g_1} + \frac{p^2}{4g_2} \right] t$$

$$= \left[360 - 3 \times 22 + \frac{(50)^2}{4 \times 60} + \frac{(50)^2}{4 \times 60} \right] \times 10$$

$$= 3148.33$$

$$EFGH = (B - 2 \times d_0) t$$

$$= [360 - 2 \times 22] \times 10 = 3160$$

Design strength of gross section

$$T_{dg} = A_g \frac{f_y}{\gamma_{mo}}$$

$$= (360 \times 10) \times \frac{250}{1.1} = 818.18 KN$$

Design strength of Rapture

$$T_{dn} = 0.9 A_n \frac{f_u}{1.25}$$

$$= 0.9 \times (2928.33) \times \frac{410}{1.25}$$

$$= 864.443 KN.$$

Minimum Tensile strength of plate is 818.18 KN.

Two steel plate in axial tension are to connected by bolts as shown in fig determine its tensile stress bolt diameter 25mm and hole diameter 28mm, plate thickness 16mm each yield stress $f_y = 290 MPa$

(i) Shear failure in DEBFG section $= \left[B - nd_0 + \frac{P^2}{4g_1} + \frac{P^2}{4g_2} \right] \times t$

$$= \left[390 - 3 \times 25 + \frac{65}{4 \times 60} + \frac{65}{4 \times 60} \right] \times 16$$

$$= 4483$$

(2) Shear failure in ABC section

$ABC = (B - 1 \times d_0)t$

$= (320 - 1 \times 25)16$

$= 4720$

(3) Section on DEFG

$(B - nd_0)t$

$= (320 - 2 \times 25)16$

$= 4320$

Section HIJKL

$(B - nd_0)t$

$= (320 - 3 \times 25)16$

$= 3920$

HIEBJKL section

$= \left[320 - 5 \times 25 + \frac{65^2}{4 \times 50} + \frac{65^2}{4 \times 60}\right] \times 16$

$= 3739.667$ mm

Minimum of $A_n = 3739.66$

Design strength of gross section

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

$= (320 \times 16) \times \frac{290}{1.1}$

$= 134.98$ kN

Design strength of Rapture

$T_{dn} = 0.9 A_n \frac{f_u}{1.25} = 0.9 \times (3736.66) \times \frac{410}{1.25}$

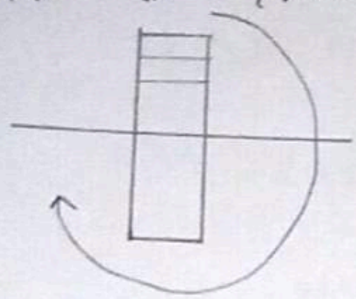
$= 110.306$ kN

Minimum tensile strength of plate 110.306 kN.

Slenderness Ratio : (λ) $\left(\frac{l_{eff}}{r_{min}}\right)$ $l_{eff} = KL$

→ The ratio of the effective length of a member to the radius of gyration of the cross-section about the axis under consideration.

→ Radius of gyration :-



It is the distance from neutral axis about which total area is placed in small strate to prevent to rotation of the section.

→ Slenderness ratio is neccery for tension member because tension member may be subjected to compresive forces due to wind load and earthquak load and live load.

Maximum value of effective slenderness ratio refer to (page no - 20. table no 3)

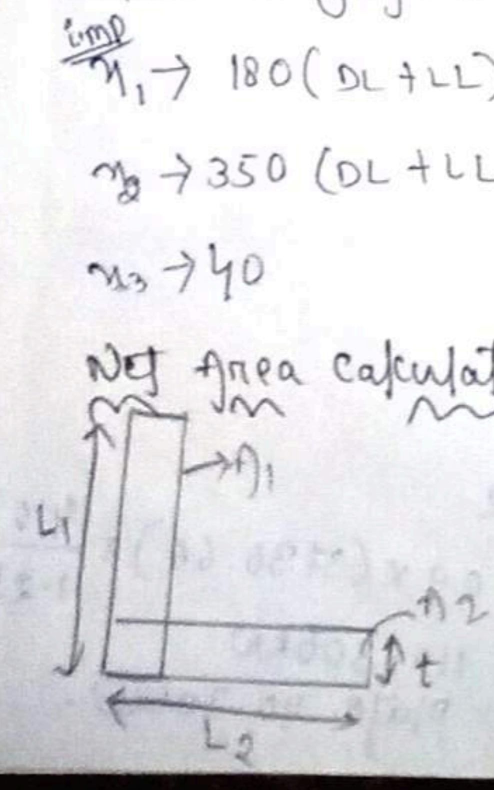
imp

$\lambda_1 \rightarrow 180$ (DL + LL)

$\lambda_2 \rightarrow 350$ (DL + LL + WL / EL)

$\lambda_3 \rightarrow 40$

Net Area calculation for angle section :-



$A_1 = (L_1 - t/2)t$

$A_2 = (L_2 - t/2)t$

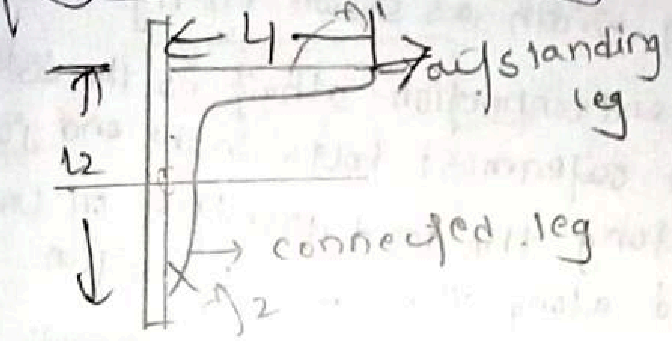
$Ag = (L_1 - t/2)t + (L_2 - t/2)t$

$= L_1 - t/2 \times t + L_2 - t/2 \times t$

$= L_1 t - \frac{t^2}{t} + L_2 t - \frac{t^2}{t}$

$= L_1 t + L_2 t - \cancel{2}\frac{t^2}{\cancel{2}}$

$$\boxed{A_g = (L_1 + L_2 - t) \times t}$$



connected leg = $\cancel{(L_2 - t/2 - d_0)t}$ → $(L_1 - t/2)t$

outstanding leg = $\cancel{(L_2 - t/2 - d_0)t}$

⇒ non-uniform stress distribution near the connection is know as shear leg effect provaide unequal angle section for increase the length of connection to eliminate shear leg effect provaide to gusset plate.

⇒ single angle section will behalf us plate section if two gusset plates are provaided to connect sigle angle section.

Threaded Rods :-

The design strength of threaded rods in tension, Tdn, as governed by rupture is given by

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

where A_n = net root area of the threaded section

Single Angle

The rupture strength of an angle connected through one leg is affected by shear lag. The design strength, T_{dn} as governed by rupture of net section is given by :-

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

where

$$\beta = 1.4 - 0.076 (w/t)(f_y/f_u)(b_s/L_c) \leq (f_u \gamma_{m0} / \gamma_{m1})$$

$$\geq 0.7$$

Where,

w = outstand leg width

b_s = shear leg width, as shown in fig 6. and

L_c = length of the 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 preliminary sizing, the rupture strength of net section may be approximately taken as:

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

where

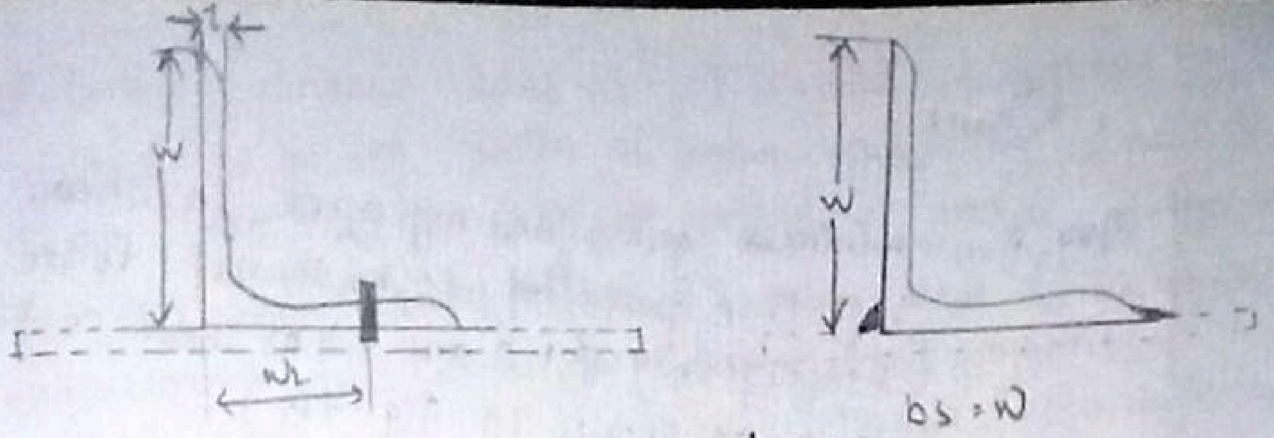
α = 0.6 for one two bolts, 0.7 for three bolts and 0.8 for four or more bolts along the length in the end connection or equivalent weld length.

A_n = net area of the total cross-section

A_{nc} = net area of the connected leg

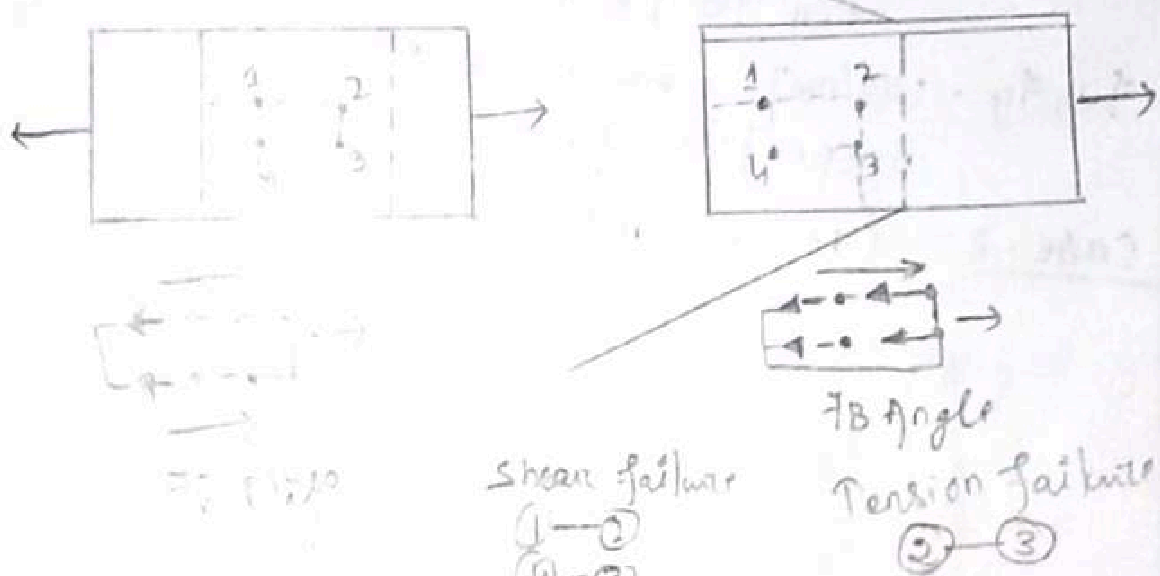
A_{go} = gross area of the outstanding leg; and

t = thickness of the leg.

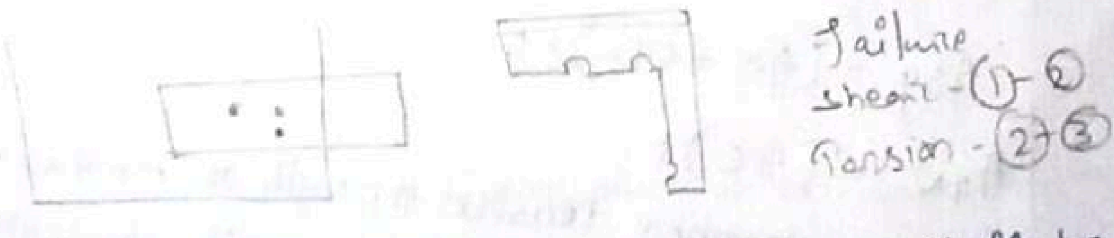


✓ Design strength Due to Block shear:-

- It is combination of shear and tensile strength of a member under tensile forces.
- Block shear strength depends upon spacing bolt holes.



Case - 1 (for bolted connection)



The block shear strength T_{db} of connection shall be taken as the smaller of

$$T_{db} = \left[A_{vg} f_y / (\sqrt{3}\gamma_{m0}) + 0.9 A_{tn} f_u / \gamma_{m1}\right]$$

$A_{vg} = BA \times t$
$A_{vn} = (BA - d_0 - \frac{d_0}{2}) t$ } shear failure

$A_{tg} = AO \times L$
$A_{tn} = (AO - d_0) t$ } tension failure

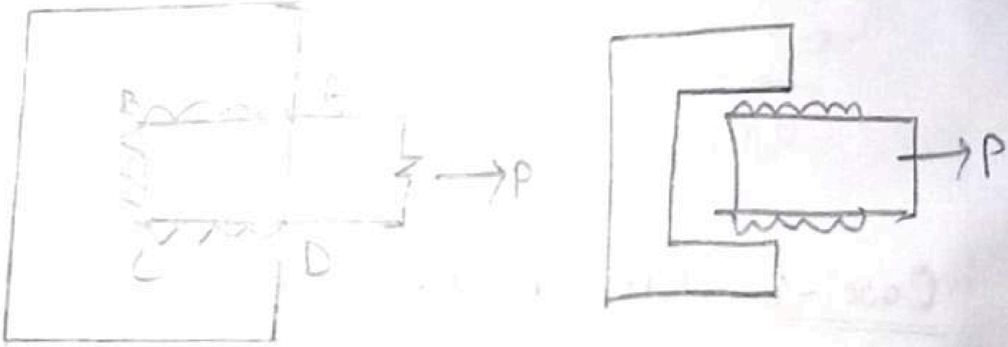
Where,

A_{vg}, A_{vn} = minimum gross and net area in shear along bolt line parallel to external force respectively (1-2 and 3-4 as shown in fig 7A and 1-2 as shown in fig 7B)

A_{tg}, A_{tn} = minimum gross and net area in tension from the bolt hole to the toe of the angle, end bolt line, perpendicular to the line of force respectively (2-3 as shown in fig 7B) and

f_u, f_y = Ultimate and yield stress of the material respectively.

Case - II (for welded connection)



$A_{sg} = (AB + CD) \times t$

$A_{TN} = (BC) t$

Tension member tensile strength of member is the minimum of gross strength, net strength of critical section and block shear failure

* A Tension member consist of a flat 100mm×8mm which is connect to gusset plate of 10mm thick by 2 no. of 16mm dia bolt. If the steel of grade fe410 and bearing bolt ~~parted~~ class 4.6 and used PS workshop determine the strength of the flat against yielding, rupture and block shear also determine the maximum load the joint can be transfer.

for Fe410 grade of steel :-

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

$\gamma_{m0} = 1.1$

$\gamma_{mb} = 1.25$

$d = 16mm$

$d_0 = 16+2 = 18mm$

(i) strength of the plate :

strength of the plate against yielding of gross cross-section

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

$$= (100\times8)\frac{250}{1.25}$$

$$= 181.81kN$$

(ii) strength of the plate against rupture of critical section

In this case section (1)-(1) is critical

Net effective area $A_n = (b - nd_0)t$

$$= (100 - 1\times18)8$$

$$= 656 mm^2$$

$$T_{dn} = 0.9\times656\times\frac{410}{1.25} = 193.65kN.$$

Strength of the plate against block shear the shaded portion may shear off

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

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

$A_{vg} = (30+40) \times 8 = 560\,mm^2$

$A_{vn} = (70 - 18 - 18/2)\,8 = 344\,mm^2$

$A_{tg} = 50 \times 8 = 400\,mm^2$

$A_{tn} = (50 - 18/2) \times 8 = 328$

for shear yield and tension fracture

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

$$= \left[560 \frac{250}{\sqrt{3} \times 1.1} + 0.9 \times \cancel{400}\ 328 \frac{410}{1.25} \right]$$

~~188.621 KN~~. 170.31 KN

for shear fracture and tension yield

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

$$\left[0.9 \times 344 \frac{410}{\sqrt{3} \times 1.25} + \cancel{328}\ 400 \frac{250}{1.1} \right]$$

$=$ ~~125.514 KN~~. 149.54

$\therefore$ Block shear strength = smaller of T_{db_1} & T_{db_2} = 149.5 ~~135~~

strength of the plate = minimum of yielding, rupture of block shear = ~~125.514 KN~~. 149.54 KN

Strength of bolts:

Strength of the bolt in single shear, Here $n_n = 1$ $n_s = 0$

Anb of

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

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

$= \frac{400}{\sqrt{3}} (1 \times \pi/4 \times (d)^2)$

$= \frac{400}{\sqrt{3}} (1 \times \pi/4 \times (16)^2)$

$= \frac{36257}{1.25} = 290.06 KN$

2 no. of bolt ~~bearing~~ shearing $= 2 \times 290.06 = 58.02 KN$.

Design strength of bolt bearing

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

$V_{npb} = 2.5 K_b d t f_u$

$K_b = \begin{cases} \frac{30}{3 \times 18} = 0.55 \\ \frac{P}{3d_0} - 0.25 = \frac{40}{3 \times 18} - 0.25 = 0.49 \\ \frac{f_{ub}}{f_u} = \frac{410}{400} = 0.97 \\ 1.0 = 1 \end{cases}$

$K_b = 0.49$

$V_{npb} = 2.5 \times 0.49 \times 18 \times 8 \times 410$

$= \frac{64288}{1.25}$

$= 514.30 KN$

Maximum load the joint can be carry safely = strength of plate or strength of bolt which is less is 58.02KN.

* Determine the design tensile strength of the plate 130 mm x 12 mm with the hole for 16mm diameter bolts as shown in fig. steel used is of fe410 grade quality.

Given:-

130 mm x 12 mm

fe 410 grade steel

$d = 16$ mm

$d_o = 18$

$f_y = 410$ mpa

$\gamma_{mb} = 1.25$

$\gamma_{m0} = 1.1$

Design strength due to yielding

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

$$= (130 \times 12) \times \frac{250}{1.1}$$

$$= 354.54 \, mm^2$$

Design strength due to Rupture

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

$$A_n = [b - n d_o] t$$

$$A_n = [130 - 2\times 18]\,12$$
$$= 1128\,mm^2$$
$$= 0.9 \times 1128 \frac{410}{1.25}$$
$$= 332.98\,mm^2.$$

Design strength due to block shear

$$A_{vg} = (35+60)\times 12 = 1140\times 2 = 2280\,mm^2$$
$$A_{vn} = (95 - 18 - 18/2)\,12 = 816\times 2 = 1632\,mm^2$$
$$A_{tg} = 60\times 12 = 720$$
$$A_{tn} = (60-18)\times 12 = 504\,mm^2$$

$$T_{db} = \left[A_{vg}\frac{f_y}{\sqrt{3}\,\gamma_{m0}} + 0.9 A_{tn}\frac{f_u}{\gamma_{m1}}\right]$$
$$= \left[\cancel{228}\ 1140\frac{250}{\sqrt{3}\times 1.1} + 0.9\times 504\frac{410}{1.25}\right]$$
$$= 350.570\,kN$$

$$T_{db_2} = \left[0.9 A_{vn}\frac{f_u}{(\sqrt{3}\times m1)} + A_{tg}\frac{f_y}{\gamma_{m0}}\right]$$
$$= \left[0.9\times\cancel{816}\frac{410}{\sqrt{3}\times 1.25} + 720\frac{250}{1.1}\right]$$
$$= 441.784\,kN.$$

Strength of plate = 332.986 kN.

Compression Member

A structural member carrying axial compressive force in general is called compression member.

Various types of compression members:-

- The vertical compressive member in RCC building is generally termed as column where as for steel building it is called stanchion.
- The compressive member in a roof truss or bracing is called as strut.
- The principle compression in a crane is called boom.

Modes of failure of column:-

- Crushing
- Buckling
- Mixed mode of crushing & buckling

CRUSHING FAILURE:-

A very short length compression member under compressive force.

BUCKLING FAILURE:-

A very long length compression member under compressive force.

Such member has a critical load which cause elastic instability due to which the member has a critical load which the member fails.

MIXED Mode of crushing & buckling:-

The above two failure occure in the extreme cases.

for all intermediate values of slenderness ratio, the column fails due to the combined effect of crushing & buckling. Most of the practical column fails due to this mixed mode.

Effective length leff:-

The effective length of a compression member is the length ~~of a compression member~~ between the two adjacent points of zero moments. The effective length of a member depends upon the end condition.

Radius of Gyration & slenderness ratio:-

Radius of gyration

The radius of gyration is given by

$$r = \sqrt{\frac{I}{A}} \quad r_{xx} = \sqrt{\frac{I_{xx}}{A}} \quad r_{yy} = \sqrt{\frac{I_{yy}}{A}}$$

Where

I = Moment of intertia

A = Cross sectional of section

→ Angle struts - 5.5

$$\sigma_{ac}' = 0.8\ \sigma_{ac} \quad \frac{l}{r} \leq 180$$

$$l_e = 0.85L \quad \sigma_{ac} = \sigma_{ac}$$

→ Double angle struts

→ continuous member - 5.5.3

for every section the values of radius of gyration about the principal axis, are obtained so that the least values of radius of gyration r_{min} may be obtained to find the maximum

Slenderness Ratio (λ_{max}) :-

The slenderness ratio (λ) of a compressive member is defind as the ratio of its effective length to the radius of gyration

$$\lambda = \frac{L_{eff}}{r} \cdot \lambda_{max} = \frac{L_{eff}}{r_{min}}$$

effective length - (Page - 45, T.2)
effective length of a compression member in the product of effective length factor 'K' & the actual length 'L'

Mathmatically, $\boxed{l = K \times L}$

→ The value of K depends upon the rational & relative translational condition of the end of the member.

For K values → (P-45, T-11)

→ The tendency of a member to buckle is usally measured by the slenderness ratio.

Maximum values of slendeness ratio (P-20, T-3)

Important pages

Pg. no-34 - Design compressive strength

Pg. no-44 - Buckling class

Pg. 40 & 41 - value of fcd
42 & 43

Step-1

Note down the properties of given section (using steel table)

Step-2

Calculate effective length
(In case of only height is given)

Step-3

calculate value of

$\lambda = \frac{l_{eff}}{r_{min}}$ $\boxed{l_{eff} = KL}$ Page no-45

Step-4

Select bucking class (Pg-44)

Step-5

find out the value of f_{cd} by using pg no- 40 & 41

Step-6

Calculate value of P_d (Pg-

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

(Q) Calculate design compressive strength for a column made up of ISHB 350 @ 710.2 N/m & 3.5 high. The column is restrained in direction end position of both end. Use steel grade Fe 410.

Given ISHB 350 @ 710.2 N/m

(350 → Height, 710.2 → wt per meter)

$f_u = 410 N/mm^2$

$f_y = 250 N/mm^2$

$l = 3.5 m$

Step-1 (Property of section)

$A = 9221 mm^2$, $h = 350 mm$, $b = 250 mm$

$r_{xx} = r_{zz} = 146.5 mm$, $r_{yy} = 52.2 mm$

Step-2

Effective length

$L_{eff} = KL = 0.65 \times 3.5m$

Step-3

calculation of slenderiness ratio

Here $r_{yy} < r_{zz}$

Buckling will occur about yy-axis

$\lambda = \frac{L_{eff}}{r_{min}} = \frac{2.275 \times 10^3}{52.2}$

$\lambda_{yy} = 43.50$

Step-4

selection of bucking class

$\frac{h}{b_s} = \frac{350}{250} = 1.4 > 1.2$

$t_f = 11.6 mm < 40 mm$

& we know that bucking will occur about yy-axis

It is a case of bucking class 'b'

Step-5

calculation of fcd

for $\frac{KL}{r} = 43.58$ & $f_y = 250 N/mm^2$

we gate

$40 - x_1 \qquad 206 - y_1$

$x - 43.58 \qquad f_{cd} - y$

$50 - x_2 \qquad 194 - y_2$

$$f_{cd} = 206 + \frac{(43.58 - 40)}{(50 - 40)}(194 - 206)$$

$= 201.704 N/mm^2$

Step-6

Compressive strength

$P_d = A_e \times f_{cd}$

$= 9221 \times 201.704 \times \frac{1}{1000}$

$= 1859.912 kN$

$$\text{working load} = \frac{1859.912}{1.5}$$

(2) Determine the design axial load capacity of the column ISHB 300 @ 577 N/m if the length of column is 3m and its both ends pinned.

Given data:-

ISHB 300 @ 577 N/m

$h = 300mm$ $b_f = 250\,mm$ $t_f = 10.6mm$

$A_e = A = 7485\ mm^2$

$r_{xx} = 129.5mm$ $r_{yy} = 54.1mm$

$r_{yy} < r_{xx}$

for rolled steel sections

$f_y = 250\ N/mm^2$, $f_u = 410\ N/mm^2$ $E = 2\times10^5\ N/mm^2$

for both end pinned column

$KL = l = 3m$

step-2 effective length

$l_{eff} = KL = 1l = 1\times3 = 3m$

step-3

calculation of slenderness ratio

Here $r_{yy} < r_{xx}$

Bucking will occure about y-axis

$r_{min} = 54.1mm$

$$f_{cc} = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2 \times 2\times10^5}{\left(\frac{3000}{54.1}\right)^2} = 641.92\ N/mm^2$$

non - dimensional effective slenderness ratio

$$\lambda = \sqrt{\frac{f_y}{f_{cc}}} = \sqrt{\frac{250}{641.92}} = 0.624$$

for buckling class b

$\alpha = 0.34$

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

$$= 0.5\left[1 + 0.34(0.624 - 0.2) + 0.624^2\right]$$

$$= 0.764$$

$$f_{cd} = \frac{f_y/\gamma_{m0}}{\phi + (\phi^2 - \lambda^2)^{0.5}} = \chi \frac{f_y}{\gamma_{m0}} \leq f_y/\gamma_{m0}$$

$$= \frac{250/1.1}{0.767 + (0.767^2 - 0.624^2)^{0.5}}$$

$$= 185.36 \, N/mm^2$$

strength of column

$$P_d = A_e f_{cd} = 7484 \times 187.36$$

$$= 1402237 \, N$$

$$= 1402.237 \, kN$$

working load $= \dfrac{1402.237}{1.5}$

$$= 934.823 \, kN$$

Design of Compression Members

The following are the usal steps in the design of commpression member:

1. Design stress in compression is to be assumed for rolled steel beam section the slenderness ratio various form 70 to 90; Hence Design stress may be assumed as 135 N/mm^2, for angle struts the slenderness ratio various form 110 to 130. Hence design stress fore such members may be assumed as 90N/mm^2. for compression members carrying large loads, the slenderness ratio is comparatively small. for such members design stress may be assumed as 200N/mm^2,

2. Effective sectional area necd^d is

$$A = \frac{Pd}{fcd}$$

3. select a section to give effective area required and calculate rmin.

4. Knowing the end condition & deciding the type of connection determine effective length.

5. ~~Knowing the end conditions & deciding the type of connection determine~~

5. Find the slenderness ratio, and hence design stress fcd & load carring capacited Pd

6. Revise the section if calculated Pd differs considerably from the design load.

(1)

(ii) Design a single angle strut connected to the gusset plate to carry 180KN factored load. The length of the strut betn centre to centre connection is 3m.

Assuming $f_{cd} = 90 N/mm^2$

$$A = \frac{180 \times 10^3}{90} = 2000 mm^2$$

Try ISA 9090, 12mm, which has $A = 2019 mm^2$

$$r_{min} = r_{vv} = 17.4 mm$$

Assuming the strut will be connected to the gusset plate with at least 2 bolts

(Strength of 20mm bolt in single shear is 45KN)

$$KL = 0.85 \; L = 0.85 \times 3000 = 2550 mm$$

$$\frac{KL}{r} = \frac{2550}{17.4} = 146.55$$

$$f_y = 250 N/mm^2$$

$$\frac{KL}{r} = 140, \; f_{cd} = 60.2$$

$$\frac{KL}{r} = 150, \; f_{cd} = 59.2$$

When $\frac{KL}{r} = 146.55$.

$$f_{cd} = 58.4 - \frac{6.55}{10}(58.4 - 52.16) = 54.6 N/mm^2$$

$$P_d = A f_{cd} = 2019 \times 54.6 = 110239 < 180000 N$$

Hence the revise the section

Provaide ISA 130, 130, 8mm

Area provaid $= 2022 mm^2$, $r = r_{vv} = 25.5$

$$\frac{KL}{r} = \frac{2550}{25.5} = 100$$

$$f_{cd} = 107 N/m^2$$

$$P_{cd} = 2022 \times 107 = 216353 > 180,000 N$$

Provaide ISA 130, 130, 8m

Area provaided $= 2022 mm^2$, $r = r_{xx} = 25.5$

$\frac{KL}{r} = \frac{2550}{25.5} = 100$

$f_{cd} = 107 \, N/mm^2$

$P_{cd} = 2022 \times 107 = 216354 > 180,000 N$

provaide ISA 130, 130, 8mm

$$\boxed{y - y_1 = \frac{y_2 - y_1}{x_2 - x_1}(x - x_1)}$$

(Q) Design a single angle discontinuous structure to carry a factored axial compressive load of 65 KN. The length of strut is 3m b/n intersection. It is connected to 12mm thick gusset plate by 20mm dia 4.6 grade bolts. use steel of grade Fe 410.

for Fe 410

$f_u = 410 N/mm^2$, $f_y = 250 N/mm^2$

$\gamma_{m1} = 1.25$, $\gamma_{m0} = 1.1$

for bolt of grade 4.6

$f_{ub} = 400 N/mm^2$

$d = 20mm$

$d_0 = 22mm$

Let's assume slenderness ratio $\lambda = 120$ & class C

form table $f_{cd} = 83.7 N/mm^2$

Area required $A = \frac{P}{f_{cd}} = \frac{65 \times 10^3}{83.7} = 777$

from steel table, let's provaide ISA 70×70×6mm

provaide Area = 806 mm² (P-4)

$r_{vv} = 13.6mm$

considering both end fixed (P-45)

effective length $\ell = K \times L$

$= 1 \times 3$

$= 3000\,mm$

<u>No. of bolts</u>

shearing strength of bolt

$$V_{dsb} = \frac{f_{ub}}{\sqrt{3} \times 1.25}\left(1 \times 0.78 \times \pi/4 \times (d)^2\right)$$

$= 45.25\,KN$

(1) Bearing strength of bolt

$$V_{dpb} = 2.5 \times K_b \times d \times t \times \frac{f_u}{1.25}$$

$$= 2.5 \times 1 \times 6 \times 20 \times \frac{410}{1.25} = 98.4\,KN$$

strength of bolt $= 45.25\,KN$

no. of bolt read for end connection

$$\frac{65}{45.25} = 1.436 \simeq 2\,Nos$$

~~For~~ Provaide 2. 20mm dia bolts for making the end connection of the strut

Considering the end fixes

$K_1 = 0.2$, $K_2 = 0.36$, $K_3 = 20$ (P - 48, T - 2)

$\alpha = 0.49$ for class C (P - 35, T - 7)

$$\lambda_{vv} = \frac{\ell / r_{vv}}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}} = \frac{3000/13.6}{1 \times \sqrt{\frac{\pi^2 \times 2 \times 10^5}{250}}}, \quad \epsilon = \sqrt{\frac{F_y}{250}} = 1$$

$$\lambda_\phi = \frac{b_1 + b_2}{\varepsilon\sqrt{\frac{\pi^2 E}{250}} \times 2t} = 2.482$$

$$\frac{70+70}{1\times\sqrt{\frac{\pi^2\times2\times10^5}{250}}\,(2\times6)} = 0.131$$

$$\lambda e = \sqrt{0.2+0.35\times2.482^2+20\times0.131^2}$$

$$= 1.642$$

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

$$= 0.5\times\left[1+0.49\times(1.642-0.2)+1.642^2\right]$$

$$= 2.20$$

$$f_{cd} = \frac{f_y/\gamma_{m0}}{\phi+(\phi^2-\lambda e^2)^{.5}}$$

$$= \frac{\frac{250}{1.1}}{2.26+(2.30^2-1.64)^{2}\,0.5}$$

$$= 62.02\ N/mm^2$$

So..

Design is not ok

Now, let's provide ISA $70\times70\times8$ mm

provide Area $= 1058\ mm^2$ (P.4)

$r_{vv} = 13.5$ mm

$$\lambda_{vv} = \frac{l/r_{w}}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}} \quad \frac{3000/13.5}{1\times\sqrt{\frac{\pi^2\times2\times10^5}{250}}} = 2.5$$

Tubular steel structures are used in trusses, members, scaffolding of building, stadium, exhibition halls, transmission towers.

Codes Required :- IS1161-1998, IS806-1968.

(1) These have small self weights. Also because of direct connection, gusset plates are eliminated further reducing dead loads.

(2) Torsional strength of these structures is more than any other rolled section.

(3) For the same load, the surface area of a tube is about 60 to 70% of that for other rolled section. Because of less area economy is achived in maintance, painting & fire proofing.

(4) Due to smooth finished surface dirt & moisture do not collect over the surface reducing the possibility of corrosion.

(5) The change in load with the floor levels can be accomodated by varying the tube thickness & the external tube demension may be maintained.

(6) The internal hollow space of tubular columns may be used for carrying drain pipe, wires cable etc. Also these space may be filled with concrete to increase the load carrying capacity & to improve fire resistance.

(1) They pose difficulty in connection among themselves or to any plate elements due to their shape problems

(2) Bolting & riveting on those section are not convenient.

(3) There light weight some times become responsible for the structural instability.

(4) Highly skilled man power & special welding techniques are required for their connection.

Steel tubes are designated by their nominal bore size

Classification of steel tube →

- based on wall thickness
  - Light
  - medium
  - heavy
- based on yield stress
  - Yst 22, Yst 210 Mpa
  - Yst 25, Yst 240 Mpa
  - Yst 32, Yst 310 Mpa

Y/S → Yield stress

t → tube

22, 25, 32 are on kgf/mm^2

Note:-

$1MPa = 1N/mm^2 = 0.102\ kgf/mm^2$

Tensile properties of steel tubes for structural purpose (Table-2, clause-11.2.1, IS code 1161)

Design of Steel Beams:-

1. Common cross section and their classification.
2. Deflection limit, web bucking and web clippling.
3. Design of laterally supported of beams against bending and shear.

Introduction:-

A beam is structural member subjected to bending it is subjected to transverse loads normal to its axis.

Beams are most critical members in any structure their design should therefore not only be economical but safe, the main considerations in the design of beams are :-

1. They should be proportioned for strength in bending keeping in view the lateral and local stability of the compression flange and the capacity of the selected shape to devlop the necessary strength in shear and local bearing.
2. They should be proportional for stiffness keeping in mind their deflection & deformation under service condition.
3. They should be proportioned for economy, paying attention to the size and grade of steel to yield the most economical design.

Types of Beams:-

Beams are generally classified according to their geometry and the manner in which they are supported. They may be straight or curved.

* Girders usally the most important beams which are frequently at wide spacing.

* Joist :- Beams of light section that supported floor construction.

* Stringers :- The beam supporting the stair steps (longitudinal beam spanning betn floor beams)

* Purline horizontal beam spanning betn the adjacent trusses

* Grits :- Horizontal wall beam serving principally to resist bending due to wind on the side of an industrial building.

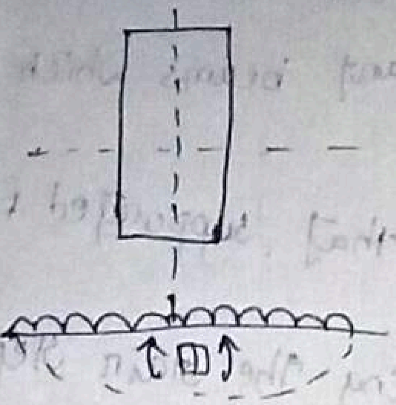
* Lintels :- Members supporting a well over window or door opening.

* Rolled I - section with and without cover plates are noramally used for floor beams. Channel tee and angle section are used in roof trusses as purlins & common rafters.

* Laterally stable beams can fail only by (a) flexure (b) shear (c) Bearing, assuming that local bucking of slender componates does not occur. These three condition are the criterial for limit state of collapse for steel beams, steel beems should also become unserve due to excessive deflection as a limit state of serviceability.

* this is classified as a

for most economical & efficient design of a beam member strength of the beam section modulus should maximum for given cross-section Area.



$$\frac{M}{I} = \frac{\sigma}{y_{max}}$$

$$Z = \frac{I}{y_{max}} \Rightarrow \boxed{M = Z \cdot \sigma}$$

1. $Z_{req} = \dfrac{B_{max}}{\sigma_{p.\,Bending}}$

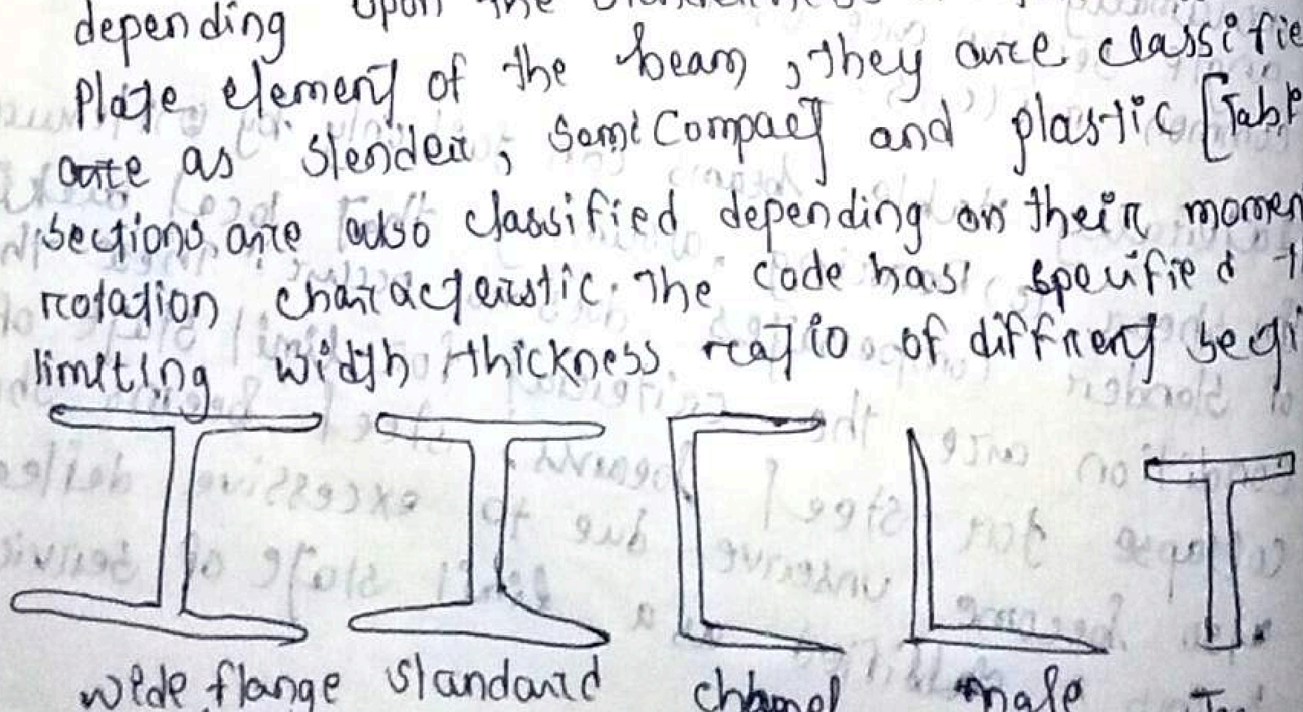
2. Moment of Resistance = $Z_{avaible} \times \sigma_{p.\,bending}$

3. σ_{cal} Bending = $B_{max} / Z_{avaible}$ (σ_{cal} Bending < $\sigma_{p.}$ Bending)

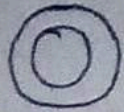
$\boxed{\sigma_p \text{ Bending} = \dfrac{f_y}{1.1}}$ [LSM].

Common cross-section and their classification.

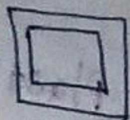
The Common cross-section, to be used as beam depending upon the slenderness of the constituent Plate element of the beam, they are classified are as Slender, Semi Compact and plastic [Table 8.1] sections are also classified depending on their moment rotation characteristic. The code has specified the limiting width thickness ratio of diffrent section



wide flange beam | Standard | channel | Angle | Tee



Tubular



Box

P.g - 17, 18

Table - 2

Plastic cross-section :-

Plastic cross-section are those which can devlop plastic hinges and have the rotation capacity reqd for failure of the structure by formation of plastic mechanism.

Compact cross-section :-

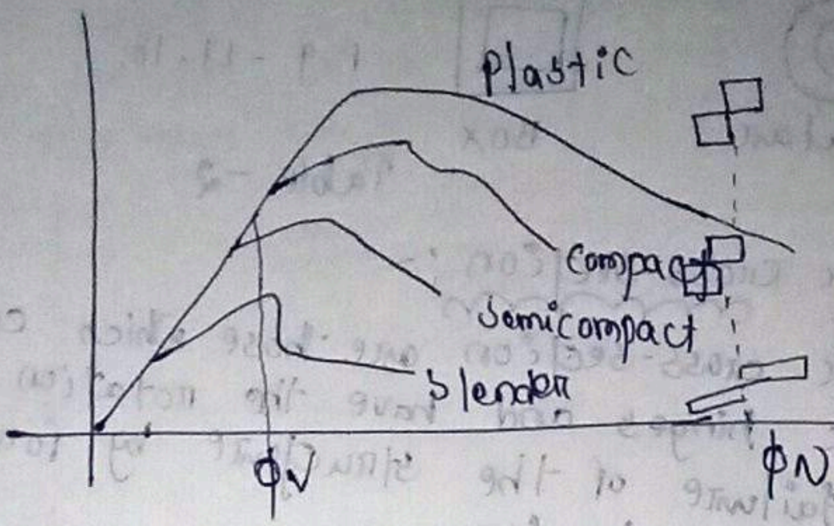
compact cross-sections are those which can devlop plastic moment of resistance, but have inadequate plastic hinge rotationcapacity for formation of plastic mechanism due to local buckling.

Semi-compact cross-section:-

Semi-compact cross-section are those in which the stress in the extreme fibers in compression should be limited to yield stress. These section can not develop the plastic moment of resistance due to local bucking.

Slender cross-section:-

Slender cross-sections are those in which the elements buckle locally even before reaching yield stress.



section classefication best on moment - Rotation charactericstics

Check for shear :-

D.S = C/SA X P.S

shear strength of member = $D \times t_w \times f_y / \sqrt{3} \times 1.1$

Check for deflection (C 15.6.1, 5 Table)

simply supported elements are brittle in nature then permissible deflection $l/360$.

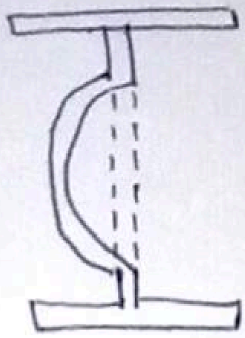
→ If supported elements are ductile on nature the permissible deflection $l/300$.

Web Bucking & Web Criping :

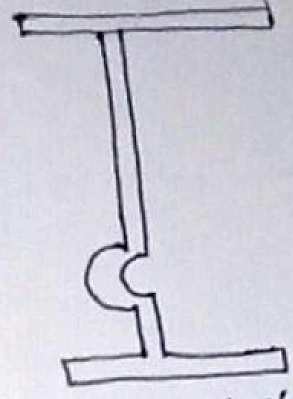
Web bucking is the failure of web under the action of concentrated local. The web is subject to column action during buckling. Buckling may occur below concentrated load. Web cripping is the failure of web in direct crushing under connentraged load.

Web Crippling phenomen:-

This phenomen occurs at the junction of web & flange due to suddern reduction in cross-section Area.



web buckling



web crippling

local buckling failure:-

local buckling of web is generated by shear & to avoid this slenderness ratio $\boxed{\lambda = 2.45\, d_1 / t_w}$