STRUCTURAL DESIGN-II

TH-2

5th SEM

CIVIL ENGG.

Under SCTE&VT,Odisha

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TARAPUR,JAJPUR ROAD

Introduction

Design of building consists of 2 parts
1. Functional design 2. structural design
→ The 1st part consists of planning the building to serve its requirement taking into account ventilation, lighting, asthetic view etc.
→ The 2nd part consist of various element of the building such that load acting on it transfer safely to the ground. ~~and~~
• structure consists of RCC or steel. In tall structure ~~constru~~ composite structure of steel and concrete is used.

Common steel structure :

- Steel has high strength per unit mass. Hence it used in constructing large columns.

1. Roof trusses for factories, cinemahalls, auditoriums etc.
2. Trussed bents, crane griders; columns etc. industrial structure.
3. Roof trusses and columns to cover plattform in railway station and bus stands.
4. plate grider and truss bridge for railway road.
5. Water tanks, chimmeys etc.

Advantage :

1. It has high strength per unit mass. Hence even for large structure, the size of steel structure element is small, saving space in construction and improving aesthetic view etc.
2. It has assured quality and high durability.
3. Material is reusable.
4. By using bolted connections, steel structures, can be easily dismantled and transported to other sites quickly.
5. Speed of construction is another important advantage of steel structure. Since steel is available workshop site. Hence there is lot of saving in construction time.

Disadvantage :-

- It is suspectible to corrosion.
- Maintemance cost is high, since it needs to help to prevent corrosion.
- steel membere are costly.

Types of steel :-

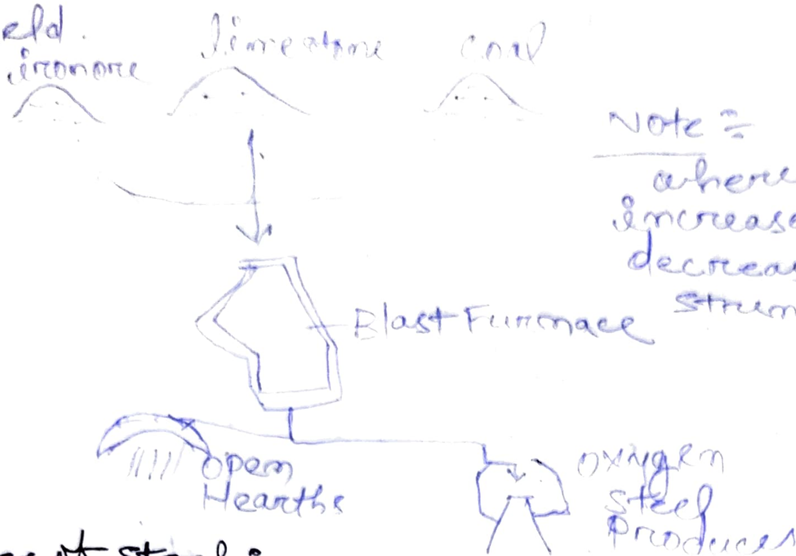
- Steel is an alloy of iron and carbon by adding sm percantage of manganese, sulpher, phosphorus, chrome, nickel and copper special properties imparted to iron and various steel can be produced.
- Increase quantity of carbon and manganese imparts higher tensile strength and yields properties but lower ductility, which is more difficult to weld.
- Increased sulphur and phosphorous 0.06% imparts brittleness, affects fatigue strength.
- chrome and nickel impart corrosion resistance property of steel.
- Addition of small quantity of copper also increase the resistance to corrosion.

~~Steel~~ :-

→ **steel making :-**

- First iron is extracted from iron ores like haematite, limestone, magnetite in furnace.
- oxygen is passed through molten iron to remove carbon and impurities make to steel.
- Magnese is added to strengthen the steel.
- Adding chrome, nickel, phosphorus can imparts special properties ~~to~~ in steel.
- Semi finished product from the machine is hot rolled to diffrent section like bars, plates, angles, section etc.
- Adding carbon increases the tensile strength and hardness but lower ductility and toughness

• In building we use structural steel which has low carbon of upto 0.1% have ductility and yield.



Note ÷ wherever % carbon increases, the ductility decreases but the strength increases

Types of steel ÷

1. Carbon steel
2. Alloy steel
3. Stainless steel
4. Tool steel

1. **Carbon steel ÷**

• Carbon steel contains trace amount of alloying elements and account for 90% of total steel production.

• Carbon steel can be further divided into 3 groups, depending on their carbon content

• Low carbon steel / mild steel contains up to 0.3% carbon.

• medium Carbon steel ÷ contains 0.3 – 0.6% carbon.

• High carbon steel ÷ It contains more than 0.6% Carbon.

Low carbon steel ÷

1. Low carbon steels are soft and weak.
2. They cannot be hardened appreciably by heat treatment but it can be strengthening by cold work.
3. They posses good formability and weldability.

uses ÷ sheets for tin cants, sheets for bridge, automobile parts

Medium Carbon steel:-

- They have high strength and hardness properties with the absence of ductility and toughness.

Ex:- Railway wheel. Railway track, Gears etc

High carbon steel:-

1. They are least ductile (more brittle) of the carbon steel.
2. They have more wear resistant.
3. They are capable of holding a sharp cutting edge (It is the important property of making tools)

Ex:- cutting tools, Dies, Hacksaw Blades, High strength wires, Spring, Razor Blades.

2. Alloy Steels :-

- Alloy steels contains alloying elements. (ex:- manganese, silicon, nickel, Titanium, copper, chromium and aluminum in varying proportion in order to manipulate's the steel properties, such as its hardenability, corrosion resistance, strength, formability, weldability or ductility.
- Application for alloy steel include pipelines, autoparts, transformers, power generators and electric motors.

3. Stainless steel:-

- Stainless steel is the most resistant and commonly used steel of all the types. It apart from carbon contains 11% chromium and some amount of nickel. It is probably the most resistant steel of all types.
- Although all the types of steel are generally resistant to rust and corrosion, the stainless steel in particular is resistant

to any sort of external attack.

- This steel is used in the making of crockery, wrist, watches, kitchen, utensils, cutlery and surgical equipments.

physical properties of steel :-

- unit mass of steel (ρ) = 7850 kg/m³.
- Modulus of elasticity (E) = 2.0×10^5 N/mm²
- poission's ratio (μ) = 0.3
- Modulus of rigidity (C) = 0.769×10^5 N/mm²
- co-efficent of thermal expansion (α) = 12×10^{-6}/°c

Market forms of steel :-

- Following are the standard shapes in which the steel section are available in the market.

1. Angle section
2. channel section
3. I-section
4. T-section
5. Round bar
6. Square bar
7. corrugated sheets
8. Expanded metal
9. flat bars
10. plates
11. Ribbed for steel bars

Mechanical properties of steel :-

a. yield stress
b. The tensile ultimate strength (Fu)
c. The maximum % standard gauge length
d. Notch Toughness

Rolled steel angle section :- (ISA)

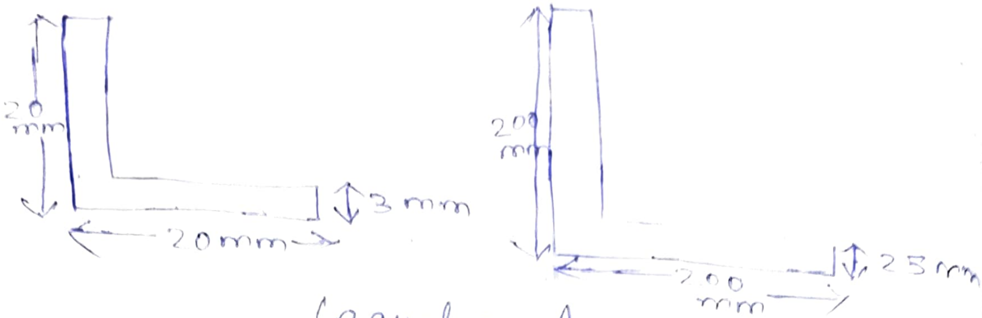
- Designated as indian standard angle (ISA) and width and length of legs.
- Available as

→ Equal angle section

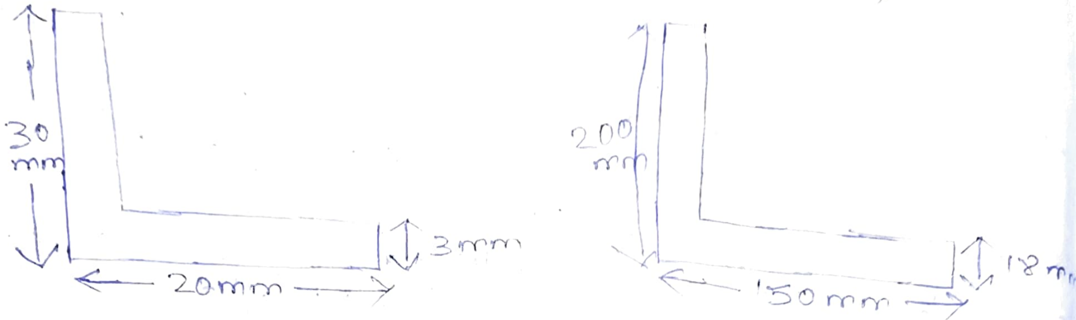
- Two legs will be equal in length.
- Available in sizes varying from 20mm × 20mm × 3 mm to 200 mm × 200 mm × 25 mm.

→ <u>unequal angle sections</u> :-

- Two legs will be unequal in length.
- Available sizes varying from 30 mm × 20 mm × 3 mm to 200 mm × 150 mm × 18 mm



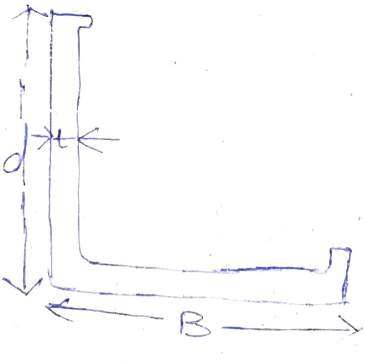
(equal angle section)



(unequal angle section)

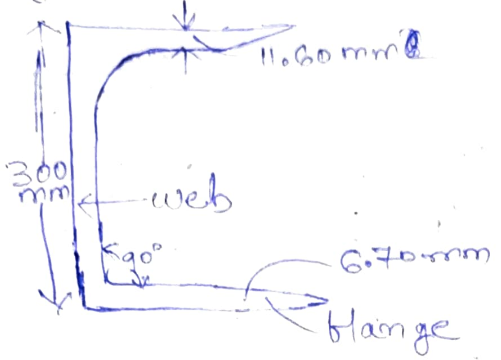
- <u>Bulb angle Section</u> :-

→ Extensively used in structural steel works like roof trusses, and as connecting member for different structures.



• Rolled steel channel Section :-

→ It consists of web and 2 equal flanges.
→ Designated by height of web and width of flange.
→ Available in sizes varying from 100mm×45mm to 400 mm × 100 mm.
→ widely used for beams and columns.

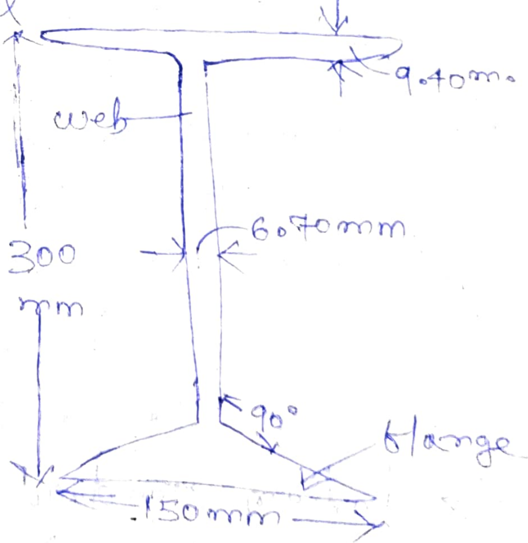


The diffrent types available are
→ Indian standard Junior channel (ISJC)
→ Indian standard light channel (ISLC)
→ Indian standard medium channel (ISMC)

(channel section)

• Rolled steel I-section :-

→ It is also known as steel Joists or beams.
→ It consists of 2 flanges connected by a web.
→ Designated ~~of~~ overall depth and width of flange.
→ Available in sizes varying from 75mm × 50 mm to 600 mm × 210 mm.



I-section

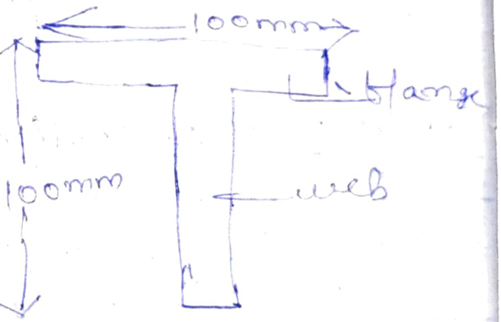
Diffrent types are:

→ Indian standard Junior Beam (ISJB)
→ Indian Standard light Beam (ISLB)
→ Indian Standard medium Beam (ISMB)
→ Indian standard wide flange Beam (ISWB)
→ Indian standard Heavy Beam (ISHB)

- It is strongest and most economical secti
- It is used as columns, purlins in trusses and grillage foundations.

Rolled steel Tee section:

→ Resembles the alphabet 'T'.
→ It consist of web and flange.
→ Designated by overall dimension and thickness.
→ Available in sizes varying from 20mm×20mm× 3mm to 150mm×150mm×10mm.
→ widely used as members of the steel roof truss and form built up section.



Diffrent types available are,

→ Indian standard Normal Tee (ISNT)
→ Indian standard Heavy Flanged Tee (ISHT)
→ Indian Standard ~~Short~~ Light Tee bar (ISLT)
→ Indian Standard Junior Tee bar (ISJT)
→ Indian standard Special legged tee bar (ISLT)

As per IS 808-1984, T-beem

- Indian standard deep legged tee bar - ISDT
- indian standard slit medium weight Tee bar - ISMT
- Indian standard heavy tee bars from I-section (ISHT)

Rolled steel bar Section:-

- Indian standard round bar (ISRO)

→ Designated as ISRO 10 (round bars having diameter 10mm)

→ Available in diameter varying from 6mm to 25mm.

Indian standard Square Bar (ISSQ)

→ Designated by ISSQ 10 (Square bars of size 10mm).

→ It is used for grillwork, handrails for staircase etc.

7. Corrugated sheets:-

- These are formed by passing sheet through grooves.
- These grooves bend and press steel sheets and corrugation are formed on the sheets.
- These corrugated sheets are usually galvanised and they are reffered to as galvanised iron sheets or G.I. sheets.
- These sheets are widely used for roof covering.

8. Expanded metals:-

- Formed by cutting and expanding either plain sheets or ribbed sheets of mild steel.
- A diamond mesh apperance is formed throughout the area.
- The manufacture sheets known as diamond mesh or rib mesh.
- Diamond mesh sizes from 30 to 150mm across the shorter length of the mesh and is available in the length of 1 to 3m. and width 5m.
- expanded metal is used for roads, floors, bridges, reinforcing concrete foundation etc. It is also used for as lathing material and for partitions.

9. Flat bars :-

- These are available in the
  → width varying from 10 mm to 400 mm
  → thickness varying from 5 mm to 40 mm
  → They are widely used in the construction of steelwork, grillwork for windows and gates.

Indian standard flats (ISF)

→ Designated as ISF 10×3 (flat of width 10 mm and thickness of 3 mm).

→ Available in suitable width varying from 10 mm to 400 mm.

→ Thickness varying from 3 mm to 40 mm.

→ used for steelwork or grillwork for windows and gates.

- Rolled steel plate section :- (ISPL)

→ Designated as ISPL 500×5 (500 mm width and 5 mm thickness).
(ISPL - Indian Standard plate Section)

→ It is used for construction of water tanks and others storage structures, built up beams, columns, baseplate for foundation etc.

- Rolled steel sheet Section :- (ISSH)

→ plates having thickness less than 5 mm.

→ Designated as ISSH 1800×600×4 (sheet having length 1800 mm, breadth 600 mm and thickness 4 mm).

→ It is used for construction of boxes and vehicle bodies etc.

ISSH - Indian standard steel sheet section

bar
Indian

Indian standard strips :- (ISST)

→ Mainly used as buildings.
→ It is designated as ISST 100×2 (steel strip with a width of 100mm and thickness 2mm

Rolled steel tubes :-

→ Inner diameter varying from 15 to 150 mm.
→ Thickness varying from 2 to 5.4 mm
→ efficient structural section for formwork and trusses

Built up sections :-

- It is composed of a combination of available basic sections like plates, angles, channel etc
- for increased strength and stability
- Different sections are jointed by welding or riveting.

Special considerations in steel design :-

The following special design consideration are required in the steel design.

1. Size and shape
2. Buckling
3. Minimum Thickness
4. Connection design

1. Size and shape :-

→ steel is manufactured ~~are required~~ in steel ~~design~~ mills and is available in certain shapes ~~& size~~ and sizes. Hence the member of steel structure, should be designed to consist of any of the available section or a combination of them. For example, a beam section may be a standard I-section, or it may consist of build up section.

P.T.O

Sometimes the choice of the Section of a memb. is governed by the shape of the other member and the type of Joint between the 2 members.

2. Buckling consideration:-

- The permissible load per unit area in steel is much higher as compared to permissible values in concrete. Therefore for the same load, the cross sectional area of steel member is smaller.
- As the member in a steel structure are more slender, the compression member in steel structure are liable to buckling.
- In case of beams, there are chances of lateral buckling which creates special members problem. As a steel member consists of a number of thin plates, the stability of each part is to be considered.

3. Minimum Thickness:-

- Corrosion needs special consideration in steel design. If very thin section are used a small amount of corrosion may result into a large percentage reduction in an effective area. Hence design practice specify minimum thickness to be used in structural members.

a. If fully accessible for cleaning and painting = 6 mm

b. If not accessible for cleaning and painting = 8 mm

c. The above limitation do not apply for rolled steel section, tubes and cold formed light gauge section, However IS 800-2007, has dropped the specification for minimum thickness

Need for design of a section :-

The following 3 types of connection are govened below.

1. Riveted connection
2. Bolted connection
3. Welded connection

Loads :-

The various load on a structure are given below.

a. Dead loads (DL)
b. Imposed loads (LL)
c. Wind loads (WL)
d. Earthquake loads (EL)
e. Erection loads (ER)
f. Accidental loads (AL) due to blast, impact of vehicle
g. Secondary effects due to contraction, differential settlement

a. Dead loads :-

- Dead loads include the weight of all permanent structure. Ex :- Building weight of roofs, floors, floor finishes, wall, beams, columns, footing, architectural material etc.

b. Imposed loads :-

IS 800 : 2007 groups the following loads are imposed loads.

1. Live load
2. crane load
3. Snow load
4. Dust load
5. Hydrostatic and earth pressure
6. Impact load / Accidental load.
7. ~~Horizontal~~ Second

Live load :-

The loads which keep changing from time to time. Ex:- Such loads in the building are weight of the persons, weight of movable partition or weight of furniture etc.

Crane load :-

These load include load from cranes and other machine acting on the structure. The loads may be taken as suppliers.

Snow load :-

It deals with the loads on the roof of the buildings. This load is to be considered for the buildings to be located in the region where snow is fall. The snow loads acts vertically downward.

Dust load :-

~~It~~ In areas prone to settlement of dust on the roof (ex:- steel plants, cement plant provision for dust load equivalent to probable thickness of accumulation of dust to made.

Hydrostatic and earth pressure :-

In the design of structure partly or fully ground level, the pressure exerted by soil or water both shall be accounted. ~~for~~ All foundation slabs and other footing subjected to resist pressure to design uplift hydrostatic pressure.

impact load :-

For structure supporting moving loads suitable additional allowance of load made by increasing impact load.

Horizontal load :

Parapets, balustrades and their supporting structure shall be designed for the horizontal forces acting at the hand rail. These load considered to act vertically also but not simultaneously with horizontal forces.

c. wind load :

These forces act on a exerted by the horizontal component of wind considered as design of building, towers etc. The wind force depends upon the velocity of wind, shape, size and location of building.

d. Earthquake load :

Earthquake shocks cause movement of foundation structures. The total vibration caused by earthquake, They are taken usually vertical and two horizontal direction.

e. Errection load :

Precast members are subjected to different types of load during errection compared supports and types of loads after errection.

F. Accidental load :

IS 875 gives the certain guidelines take care of accidental load on the structure.

1. Impact and collision
2. Explosions
3. fire

g. Secondary effects :

→ Secondary effects due to contraction or expansion resulting from temperature change differential settlement.

→ expansion or contraction due to change in temperature of the members and elements on the structure.

Load combination :-

• Load combination for design purposes shall be produce maximum forces and effects and consequently maximum stresses and deform

→ Dead load + imposed load

→ Dead load + imposed load + wind or earthquake load

→ Dead load + wind load or earthquake load

→ Dead load + erection load.

1. DL	7. DL+IL+EL
2. DL+IL	8. DL+IL+TL
3. DL+WL	9. DL+WL+TL
4. DL+EL	10. DL+EL+TL
5. DL+TL	11. DL+IL+WL+TL
6. DL+IL+WL	12. DL+IL+EL+TL

where DL = Dead load, IL = Imposed load, WL = wind load

EL = Earthquake load, TL = Temperature load.

Structural analysis :-

IS code permits the following analysis

a. Elastic Analysis

b. Plastic analysis

c. Advanced analysis

d. Dynamic analysis

Elastic analysis :-

It is based on the assumption that no fibre of member has yielded for design load whenever the stress is directly proportional to strain in proportional limit.

Plastic Analysis :-

- In this method when every fibre at a section reaches yield stress a plastic hinge formed.
- In plastic analysis the maximum or ultimate point reaches.

Design philoshophy :-

- The aim to design is to decide shape, size and connection details of members so that structure being intended life.
  - → Sustain all loads expected on it.
  - → Sustain deformation during after construct
  - → should be adequate durability.
  - → Should be adequate resistance and exposure to moisture condition.
- There are 3 types design philoshophy.

1. working stress Method (WSM)
2. ultimate load design (ULD)
3. Limit state Design (LSM)

WSM :-

- This method is simple.
- It provides serviceability.
- This is reliable reasonably.

limitations :-

→ It gives uneconomical sections.

ULD :-

The limitation of working stress method to asses the actual load carrying capacity to develop ultimate load factor method

LSM :-

In this method we will take care of both strength and serviceability requirements.

• Life Span of permanent steel structure = 120 years.

Limit state method

(It shouldn't collapse fail suddenly, it should gave some inditation before failure)

Serviceability
Service + ability
due to
— deflection &
— Vibration

Collapse generally due to
• torsion
• Shear
• Bending / compression

Partial factor of Safety
For concrete = 1.5
for steel = 1.15

→ Design load = partial F.O.S × Load

→ Design strength = $\frac{\text{strength}}{\text{partial factor of safety}}$

Limit state of strength :

1. strength
2. collapse
3. Fatigue
4. Brittle failure

Limit state of Serviceability :

Deflection
Vibration
resistance to fire

• partial Safety factor
For material = $\frac{\text{strength}}{\text{partial F.O.S}}$

For load = P.F.O.S × load

Principle of limit state design :

- Aim of a design to see that structure is a built is safe and it serves to safe purpose for which it is built.
- The structure may become unfit for use not only when it collapses but also it violates the serviceability requirements of deflection, vibration, cracks due to fatigue, corrosion and fire.
- The phillosophy of the limit state design method to see the structure remain within the acceptable limit of safety and serviceability requirements based on risks involved.

Design requirements :

- Steel structure design constructed should satisfy the requirements regards stability.

a. Remain fit with adequate reliability and be able to sustain all loads and other influence experienced during construction and use.

b. Have adequate durability under normal maintenance.

c. i. Avoids eliminating or reducing exposure to hazards which the structure is sustain.

d. Choosing the structural form, layout and detailing and designing such that.

i. The structure has low sensitivity to hazardous conditions.

ii. The structure survives with only local damage even after serious damage to individual element by the hazard.

e. choosing suitable material, designing and detailing such as to the particular structure.

→ The collapse is considered disproportionate if more than 15% of the floor or roof area 70 m^2 collapses at that level and one adjoining level either above or below it, under a load equal to 1.09 or 0.9 times the DL, 0.33 times temporary or full imposed load or permanent nature and 0.33 time the wind load acting together.

a. The building should be effectively tied together at each principle floor level and each column should be effectively held it positions.
These ties may be steel members such as beams which may be designed. For other purposes or the ~~steel~~ shear connector floor with beams and columns.
These connection should be capable of resisting i. expected tensile force subjected to a minimum of 75 KN.
ii. One percent of the maximum axial compression in the column.

b. Floor or roof units should be effectively anchored in the direction of their span either to each other or directly to the support.

c. All columns splices should be capable to resist a tensile forces equal to the largest of a factored ~~load~~ dead and live load on the floor above or below the splice.

d. Lateral load resisting horizontal loads should distributed throughout the building nearly orthogonal directions.

• Limit states are the states beyond which the structure no longer satisfied the specified performance as the requirements.
a. Limit state of strength
b. Limit state of serviceability.

a. **Limit state of strength :-**

• The limit state of strength prescribed to avoid collapse of structure which may be endanger the safety of life any property under this category.
• The limit state of strength includes.
i. Loss of equilibrium of whole or part of structure.
ii. Loss of stability of structure as whole or part of it.
iii. failure by excessive deformation.
iv. fracture due to fatigue.
v. Brittle fracture.

b. **Limit State of Serviceability :-**

The limit state of serviceability include:
i. Deformations and defection are adversly affecting the appearance or effective use of structure or causing improper function of equipment of services or causing damage to finishi
ii. Vibration in structure of any component limiting of functional effectiveness.
iii. Repairable damage or crack due to fatigue
iv. Corrosion.
v. fire.

• Rolled Steel Section :-

→ Steel Section of standard shapes, sizes and length are rolled in steel mills and marketed.

→ Many steel Section are readily available in the market and are frequent demand. Such steel Sections are known as Regular steel Section.

→ Some steel Section are not in use commonly but the steel mills can roll them if order are placed. Such steel Sections are known as Special Steel Sections.

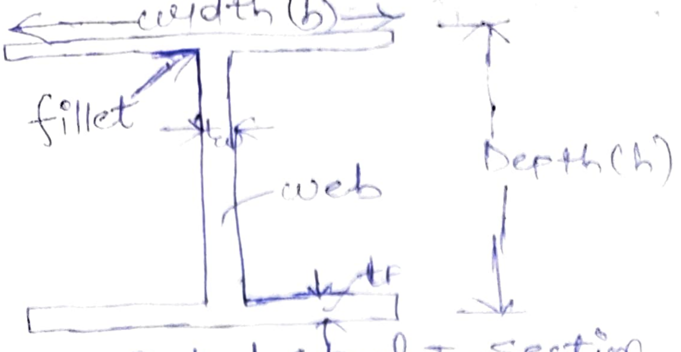
→ Various types of Steel Section manufactured list are below.

i. Rolled Steel I-Section (Beam Section)
ii. Rolled steel channel Section
iii. Rolled Steel angle Section
iv. Rolled Steel Tee Section
v. Rolled steel bar
vi. Rolled Steel tubes
vii. Rolled Steel plates
viii. Rolled steel flats
ix. Rolled steel sheets and strips.

1. Rolled Steel I Section :-

The following 5 Series of rolled steel I-Sections are manufactured in India

a. Indian Standard Junior beam – ISJB
b. Indian Standard light beam – ISLB
c. Indian Standard wide flange beam – ISWB
d. Indian Standard medium Beam – ISMB
e. Indian standard Heavy Beam – ISHB



Rolled steel I-section

- The section are followed by depth (mm) and weight per metre ex: ISMB 300 @ 0.852 KN/m
- It may not matter much if weight per metre length is not written case in ISJB, ISLB and ISMB sections.

ISWB 600 @ 1.423 KN/m

ISWB 600 @ 1.312 KN/m

ISHB 450 @ 0.855 KN/m

ISHB 450 @ 0.907 KN/m

Rolled steel channel Section :

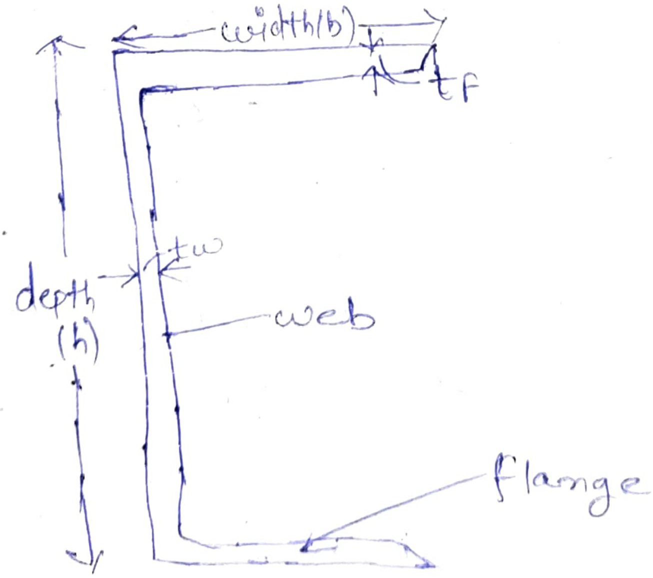
The Section are divided in 4 types.

a. Indian Standard Junior channel – ISJC

b. Indian Standard light channel – ISLC

c. Indian Standard Medium weight channel – ISMC

d. Indian Standard Special channel – ISSC



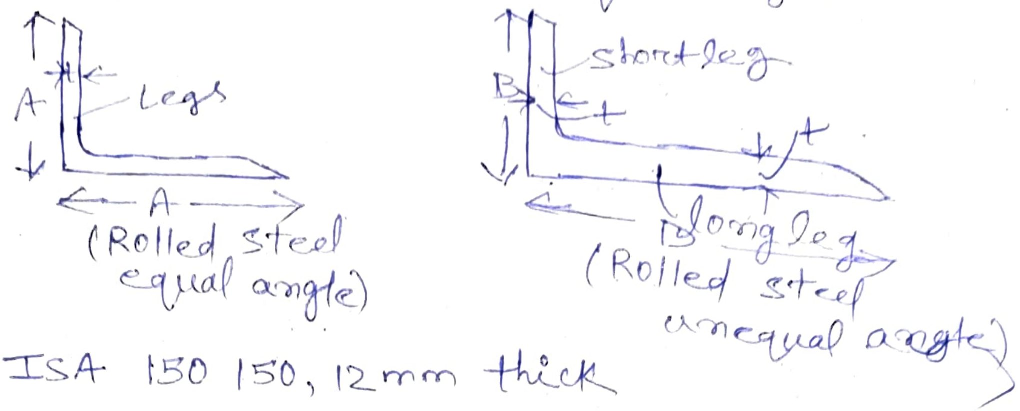
Rolled steel channel Section are followed by depth (mm) weight (KN/m) ex: ISMC 300 @ 0.351 KN/m.

Rolled Steel Angle Section :-

There are classified in 2 groups.

a. Indian Standard Equal angle – ISA

b. Indian Standard Unequal angle – ISA



- ISA 150 150, 12 mm thick

  or

  ISA 150×150×12

- ISA 150 115, 10 mm thick

  or

  ISA 150×115×10 mm

Rolled Steel Tee Section :-

There are 5 types

a. Indian standard Normal Tee bar = ISNT

b. Indian Standard heavy flanged tee bar (ISHT)

c. Indian standard Special legged tee bar (ISLT)

d. Indian standard light Tee bar – ISLT

e. Indian Standard Junior tee bar – ISJT

- The rolled steel section are followed by depth or weight per metre length

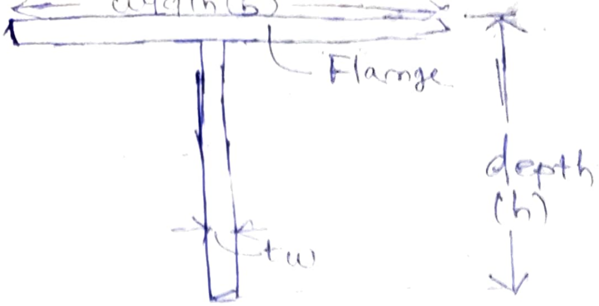
  Ex:- ISNT 60 @ 53 N/m.

As per IS 808-1984, T-Section has been adopted

a. Indian standard Deep legged bar (ISDT)

b. Indian standard silt medium weight Tee bar (ISMT)

c. Indian standard heavy tee bar (ISHT)



Rolled steel bars :-

These are divided into following 2 groups

a. Indian standard Round bar - ISRO

b. Indian standard Square bar - ISSQ

Rolled steel bar are designated by ISRO followed by round bar and ISSQ followed by Square bar. ex:- ISRO 16, ISRQ 20.

Rolled steel tubes :-

These sections are designated by their normal base sizes, In size there are 3 classes; Light, heavy & Medium.

Ex:- 40 mm tube has 3 type and their Sectional properties are given below.

Nominal Bore	Outer dia.	class	Thickness	Weight per length	Area (cm^2)	MI	K
		Light	2.90	31.9 N	414	107×10^3	16
40	48.3	Medium	3.25	35.4 N	460	117.3×10^3	16.
		Heavy	4.05	43.5 N	563	139.3×10^3	1[illegible]

Rolled steel plates :-

- Rolled steel plate following thickness are availabl. 5, 6, 8, 10, 12, 14, 16, 18, 20, 22, 25, 28, 32, 36, 40, 45, 50, 56, 63, 71, 80 mm.
- They are rolled in width: 160, 180, 200, 220, 250, 280, 320, 355, 400, 450, 500, 560, 630, 710, 800, 900, 1000, 1100, 1400, 1600, 1800, 2000, 2200, 2500 mm
- These plates are designated by ISPL followed by length, width and thickness. Ex:- ISPL 2000 × 1000 × 6.

Rolled Steel strips :

- Rolled Steel strip are designated by ISST followed by width and thickness. These sections available the following width and thickness.

- width : 100, 110, 125, 140, 160, 180, 200, 220, 250, 280, 320, 355, 400, 450, 500, 560, 630, 710, 800, 900, 1000 mm.

- Thickness : 0.8, 0.9, 1.0, 1.1, 1.2, 1.4, 1.6, 1.8, 2.0, 2.2, 2.5, 2.8, 3.2, 3.5, 4.0, 4.5 mm.

- Thickness of strip less than 5 mm. Rolled steel is followed by width & Thickness ex : ISST 250 × 2.5 mm.

Rolled Steel Flats :

Flats differ from strips in sense of thickness of flats is 5 mm and the width is limited.

- width : 12, 16, 20, 25, 32, 40, 50, 63, 80, 100, 125, 160, 200, 250 mm

Thickness : 5, 5.5, 6, 7, 8, 9, 10, 11, 12, 14, 16, 18, 20, 22, 25 mm.

They are designated by ISF letter and thickness.

ex : 80 ISF 10 means

80 mm wide indian standard flat of thickness 10 mm.

2nd chapter Structural Steel fasteners and connection

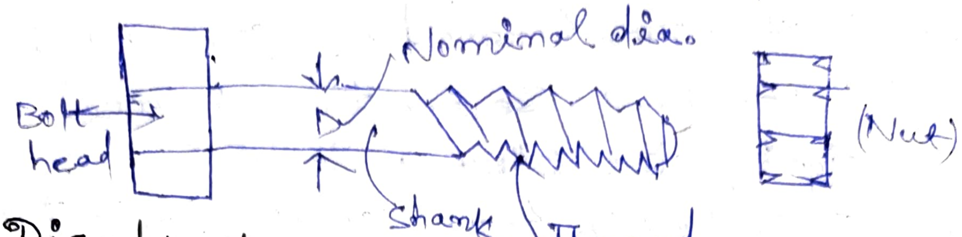
Bolted connection :-

- As steel structure formed by connecting available standard Section there is need for designing the following Sections.
  a. Diffrent Section to form composite Section of a member. (ex:- Connecting plates, channels, angles, I-Section etc.)
  b. Diffrent members at their ends (ex:- secondary beam to main beam, beam to columns and column to footing etc.)
- The design of connection is very important because the failure of Joint is sudden and Castrophic.
- The following 3 types of connection.
  a. Riveted
  b. Bolted
  c. Welded

Riveted connection :-

- Riveting is a method of Joining together pieces of metal by inserting ductile material pins called rivets into holes of pieces to be connected and forming a head at the end of the rivet to prevent each metal piece from coming out.
- Riveted holes are made in structural member to be connected by punching or by drilling. The size of rivet hole is kept slightly more (1.5 to 2mm) than the size of the rivet.
- After the rivet holes in the members, a red hot rivet is inserted which is made of one side and length of which more than combined thickness of the members are connected.

• Then holding red hot rivet at head end, and hammering is made. It result into expansion of the rivet completely fill up rivet hole and also form the driven head.



Disadvantages of Rivets :-

i. It is associated with high level of noise pollution.

ii. It needs heating the rivet to red hot.

iii. Inspection of connection is skilled work.

iv. Removing poorly installed rivets is costly.

v. Labour cost is high.

→ production of weldable qualities of steel ~~rivets~~ & introduction of high strength friction grip bolts (HSFG) replaced use of rivets.

Bolted connection :-

• A bolt is a metal pin with a head formed at one end and shank threaded at the other in order to recieve a nut.

• Bolts are used to Joining together pieces of metals by inserting them into the holes and tightening the threaded ends.

• Bolts are classified as

a. Unfinished bolt (Black Bolt)

b. Finished bolt (Turned Bolt)

c. High strength friction grip bolt (HSFG bolts)

1. Unfinished / Black Bolt :-

- These bolts are made up of mild steel rods with a square or hexagonal head. The shank is left unfinished i.e. rough or rolled. Through the black bolts are nominal dia (dia. of shank) 12, 16, 20, 22, 24, 27, 30 and 36 mm are available. Commonly used dia. are 16, 24, 30 and 36mm
- These bolts are designated by M16, M20, M24 as per (IS 1364 part-1) given specificat for such bolt.
- In structural elements to be connected holes may be larger than the nominal dia. bolts.
- Joint remain quite loose resulting into large deflection.
- The yield strength of commonly used black bolts is 240 N/mm², and ultimat strength is 400 N/mm²
- These bolts are used in light structure under static loads such as trusses, bracing and also temporary connection required.

2. Finished / Turned Bolts :-

- These bolts are also made from mild steel, but they are formed from hexagonal rod, which are finished by turning its circular shape.
- Actual dimension of the bolts are kept 1.2mm to 1.3mm larger than the nominal dia.
- As connection is more tight, it results into much better bearing contact between the bolts and holes.
- These bolts are used in special job connect ~~much better bearing~~ contact machine parts subjected to dynamic loading.

3. High strength friction grip bolt (HSFG bolt)

- The HSFG Bolt are made up from high strength steel rods. The surface of shank is kept as unfinished as in the case of black bolts. These bolts are tightened to a pro load using calibrated wrenches. Hence the grips are the members tightly.
- If the joint is subjected to shearing load it is primarily resisted by frictional force between the members and washers.
- The shank of the bolt is not subjected to shearing. Hence such bolt can be used to connect members subjected to dynamic loads.
- The successful introduction of HSFG Bolt resulted into replacement of rivets.
- IS 3747 varies specifies dimension for bolts and their washer and nuts. commonly available nominal diameter of HSFG Bolts are 16, 20, 24, 30 and 36 mm.

Classification of Bolts Based on Type of load Transfer:

- On the basis of load transfer in the connection bolts may be classified as the,

a. Bearing type
b. Friction type.

- Unfinished (black) bolts and finished (turned) bolts belong to bearing type since they transfer shear force from one member to other member by bearing. whereas HSFG Bolts belong to friction grip type since they transfer shear by friction.

Advantages of HSFG Bolts:

- HSFG Bolts have the following advantages over unfinished or finished bolts.

1. Joints are rigid, i.e., no slip takes place in the joint.
2. As load transfer is mainly by friction, the bolts are not subjected to shearing and bearing stresses.
3. High static strength due to high frictional resistance.
4. High fatigue strength since nuts are prevented from loosening and stress concentration avoided due to friction grip.
5. Smaller number of bolt result into smaller size of gusset plates.

Disadvantages of HSFG Bolt:

- The following disadvantages of HSFG Bolts over bearing type Bolts.

1. Material cost is high.
2. The special attention is to be given workmanship especially to give them right amount to tension.

Advantages of Bolted Section:

1. Making joints is noiseless.
2. Do not need skilled labour.
3. Needs less labour.
4. Connection can be made quickly.
5. Structure can be put to use immediately.
6. Accomodates minor discrepancies in dimension.
7. Alteration, if any, can be done easily.
8. Working area required in the field is less.

Disadvantages of Bolted Section :

1. Tensile strength is reduced considerably due to stress concentration and reduction in the area at the root of the threads.
2. Rigidity of joints is reduced due to loose fit, resulting into excessive deflections.
3. Due to vibrations nuts are likely to loosen, endangering the safety of the structures.

—x—

Terminology :-

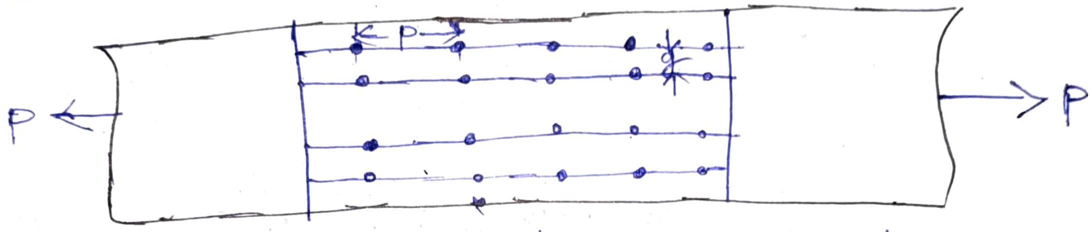
The following terms used in the bolted connection are defined below.

1. pitch of the bolt : (p)

It is the centre to centre spacing of the bolts in a row, measured along the direction of load.

2. Gauge distance : (g)

It is the distance between the 2 consecutive bolts of adjacent rows and is measured at right angles to the direction load.

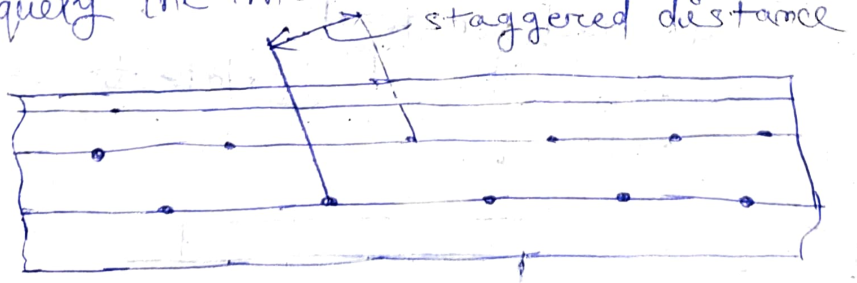


(pitch, gauge distance & edge distance)

3. Edge distance :- It is the distance of centre of bolt hole from the adjacent edge of plate.

4. End distance :- It is the distance of the nearest bolt hole from the end of the plate.

5. Staggered Distance :- It is the centre to centre distance of staggered bolts measured obliquely the member.

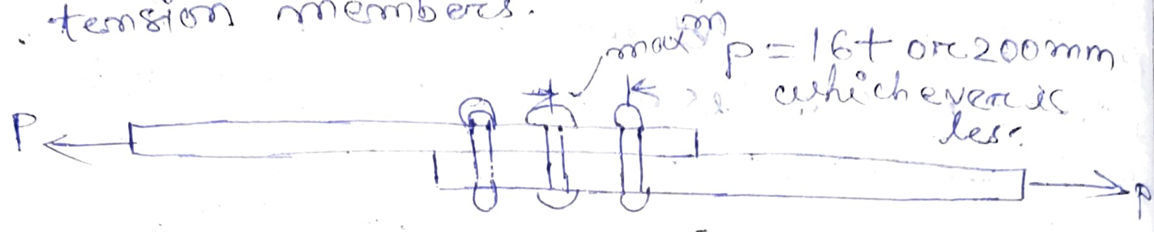


IS 800 : 2007 Spacing from Spacing and End distance of Bolt holes :

1. pitch 'p' shall not be less than 2.5d, where 'd' is the nominal diameter of the bolt.

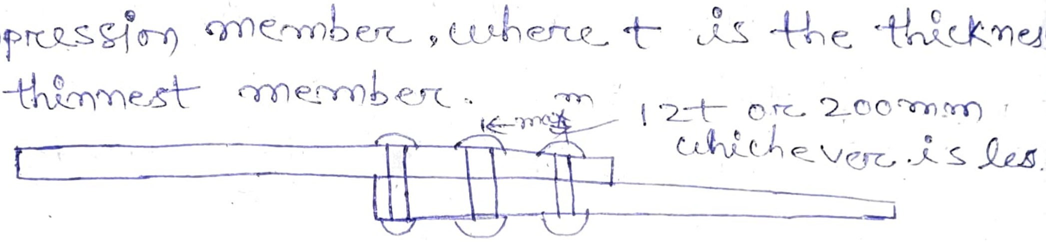
2. pitch 'p' shall not be more than

a. 16t or 200mm, whichever is less, in tension members.



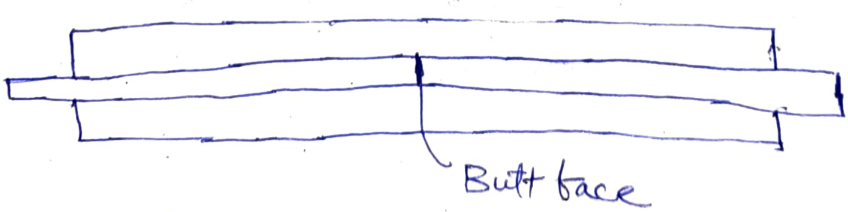
where t = thickness of thinnest member.

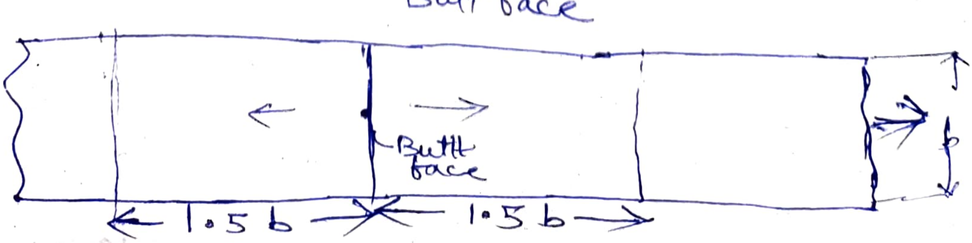
b. 12t or 200mm, whichever is less, in case of compression member, where t is the thickness of thinnest member.



c. In case of staggered pitch, pitch may be increased by 50% of the values specified above provide gauge distance is less than 75 mm.

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





4. The gauge length 'g' should not be more than 100+4t or 200 mm whichever is less.

5. Minium edge distance shall not be

i. Less than 1.7 X hole diameter in case of sheared or hand flame cut edges.

ii. Less than 1.5 x hole diameter in case of rolled, machine flame cut, Sawn and planed edges.

6. Minium edge distance (e) should not exceed

a. $16t\epsilon$, where $\epsilon = \sqrt{\frac{250}{f_y}}$

where t = thickness of thinner outer plate

b. 40+4t, where t is the thickness of thinner connected plate.

7. Tacking bolts: Apart from the required bolt from the consideration of design forces, additional bolts is called teacking fasteners, should be specified below.

a. If value of gauge length exceeds after providing design fasteners at maximum edge distances tracking rivets should be provided,

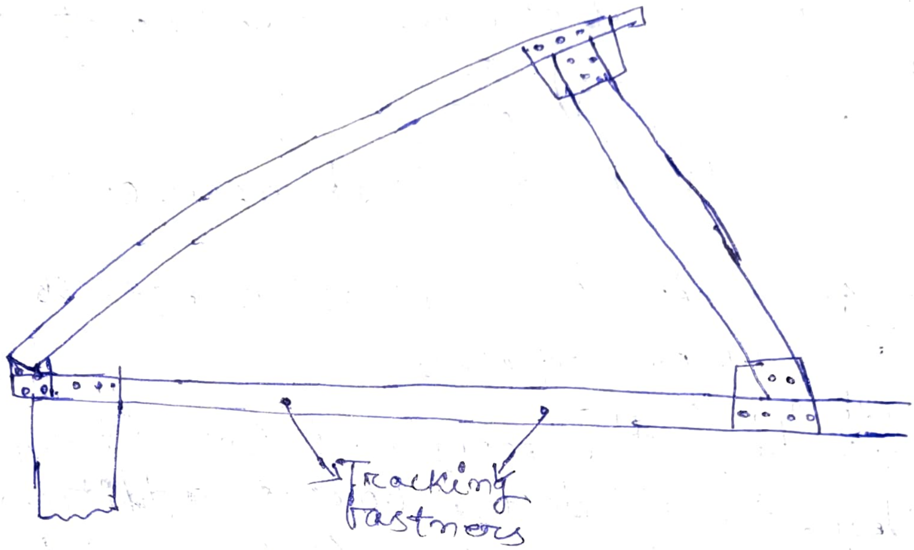
i. At 32t or 300 mm, whichever is less. If plates are not exposed to weather.

ii. At 16t or 200 mm, whichever is less, If plates.

8. In case of member made up of 2 flats or angles or tees or channels, taking rivet are to be provided along the length to connect its tracking fastners.

a. Not exceeding 1000 mm, If it is tension number.

b. Not exceeding 600 mm, If it is compression member.



Types of Bolted connection :-

- There are 2 types of Bolted connections.

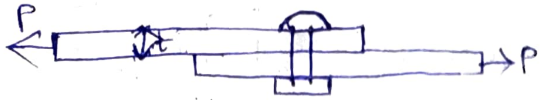
1. Lap Joint
2. Butt Joint

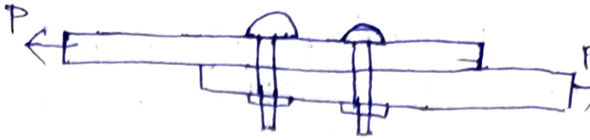
Lap Joint :-

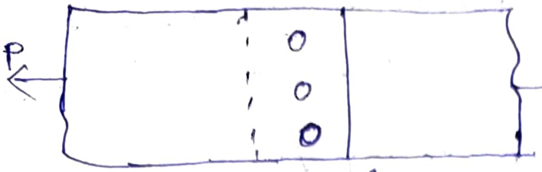
- It is a simplest type of Joints. In this the plates to be connected overlap one another.

Butt Joint :-

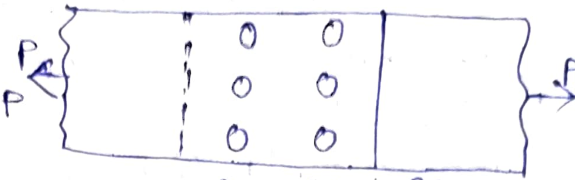
- In this type of connection, the 2 main plates a butt against each other and the connection is mainly by providing a single cover plate connected by a main plate or by double cover plates, one or other is connected by main plates.



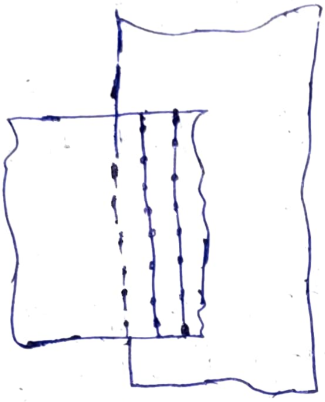




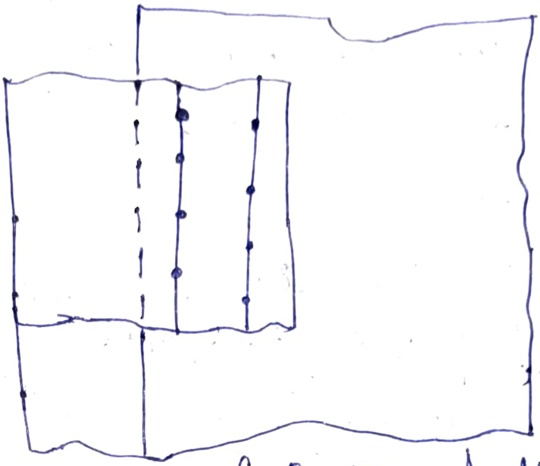
Single line bolting



Double line bolting

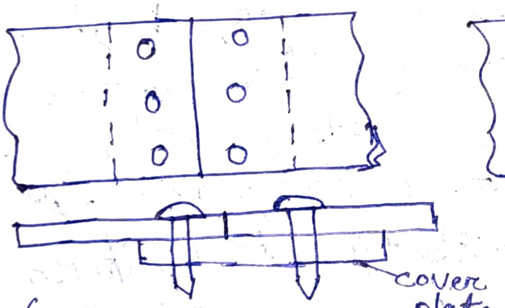


(Chain Bolted)

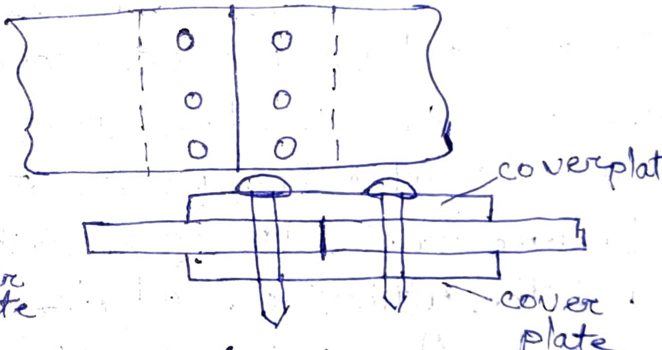


(Zig Zag bolted)

(Types of Lap Joint)



(Single cover butt Joint)



(Double cover butt Joint)

Types of Action ON fastners :

Depending on Types of load & connection, bolts.

1. Only one plane Subjected to shear (Single shear)
2. Two planes Subjected to shear (Double shear)
3. pure tension
4. pure moment
5. Shear and moments in plane of connection
6. Shear and tension.

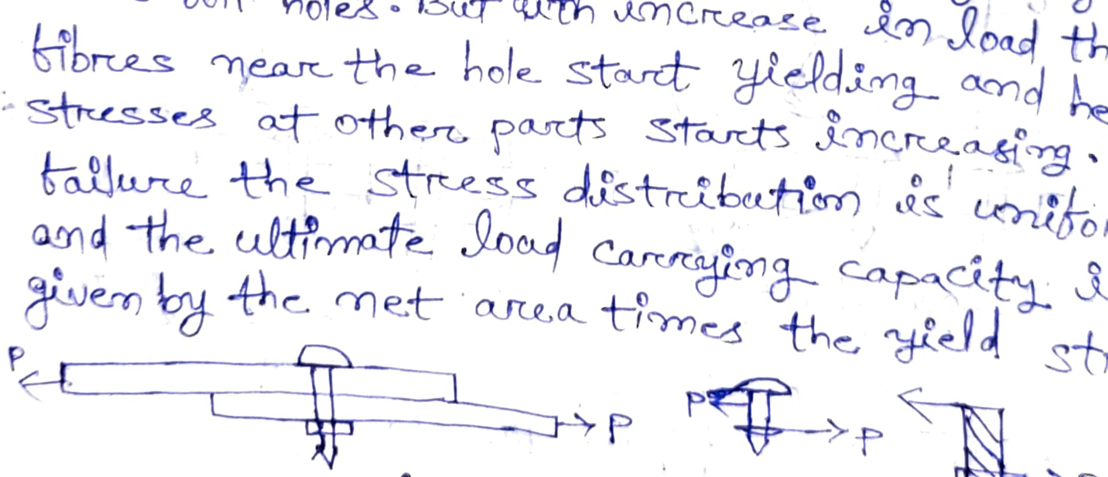
Assumptions in Design of Bearing Bolts :

• The following assumption made in design of bearing (finished or Unfinished) bolted connection
Grade Varies - 4.6 to 4.8

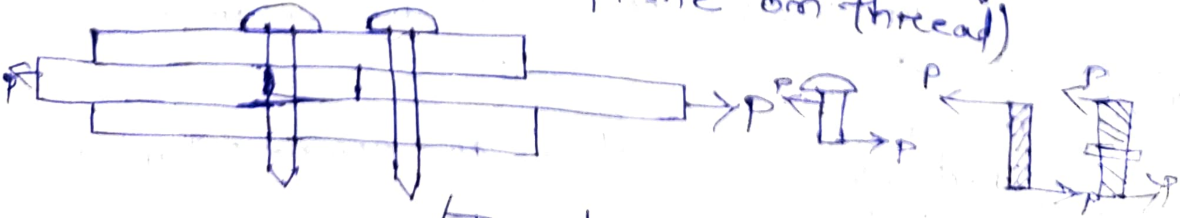
1. The friction between the plates are negligible.
2. The shear is uniform over the cross Section of the bolt.
3. The distribution of stress on the plates between the bolt hole is uniform.
4. Bolts in group subjected to direct ~~stress~~ load share the load equally.
5. Bending stresses developed in the bolts is neglected.

→ Assumption 1 is not correct because friction exists between the plates as they held tightly by bolts. But this assumption result in safer side in the design.

Actual stress distribution is not uniform in working Conditions. Stresses are very high near bolt holes. But with increase in load the fibres near the hole start yielding and hence stresses at other parts starts increasing. At failure the stress distribution is uniform and the ultimate load carrying capacity is given by the net area times the yield stress.



(Shear plane on thread)



(Two plane subjected to Shear)

principles of observed in design :-

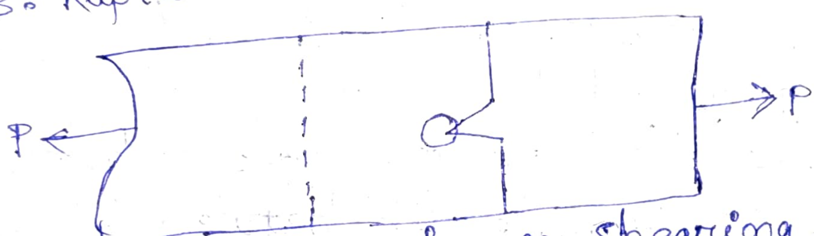
The following principles are observed in design connection.

1. The centre of gravity of bolts should coincide with the centre of gravity of the connected member.
2. The length of the connection should be kept as small as possible.

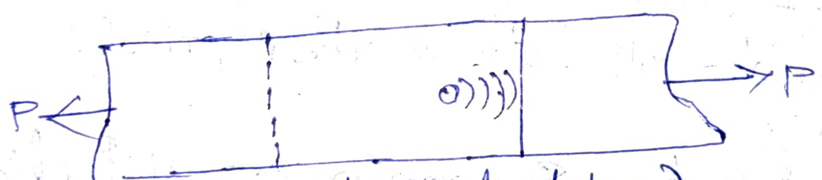
Design tensile strength plates of a Joint :-

Plates are Joint made with bearing bolts may fail under tensile force due to any following.

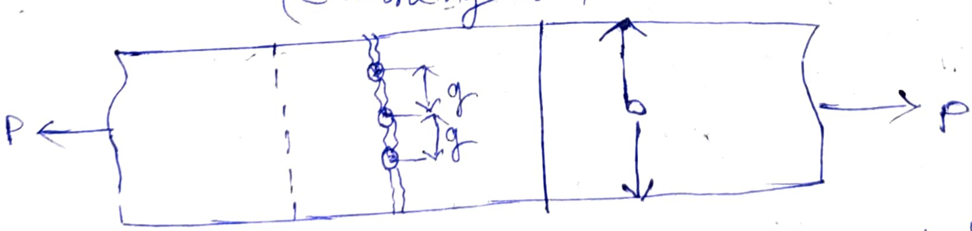
1. Brusting or shearing of the edge.
2. Crushing of plates.
3. Rupture of plates.



(Bursting or shearing of plates)



(Crushing of plates)



• If the minimum distances are ensured in a Joint, the design tensile strength of plate in the Joint is the strength of the thinnest member against rupture.

This strength is given by $T_{dn} = \dfrac{0.9 A_n f_u}{\gamma_{m1}}$

where
γ_{m1} = partial factor of safety for failure at ultimate stress = 1.25
f_u = ultimate stress of the material
A_n = net effective area of the plate at critical section.

$A_n = [b - nd_o]$ used where the connection is chain connection/or not staggered section
wherever this is staggered,

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

where b = width of the plate
t = thickness of thinner plate in Joint
d_o = diameter of the bolt hole (2mm addition to the diameter of hole, in case of directly punched holes)
g = gauge lengths between the bolt holes
P_s = staggered pitch length between lines of bolt holes.
n = number of bolt holes in the critical sections.
i = Subscript for summation of all inclined legs.

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Design strength of Bearing Bolts:

The design strength of bearing bolts under shear is the least of the following

a. shear capacity (strength)
b. Bearing capacity (strength)

a. shear Capacity of Bearing Bolts at a Joint:

Design strength of the bolt, V_{dsb} is given by, $V_{Dsb} = \frac{V_{nsb}}{\gamma_{mb}}$

where, V_{Dsb} = Design of shearing strength of the bolt.

V_{nsb} = Nominal shear capacity of the bolt.

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

In the above expression is given by,

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

where f_{ub} = ultimate tensile strength of the bolt

n_n = number of shear planes with threads intercepting shear planes

~~Asb = no~~

n_s = number of shear planes in shank intercepting the shear plane.

A_{sb} = Nominal shank area of the bolt.

A_{nb} = Net shear area of the thread.

$$A_{nb} = \frac{\pi}{4} \times (d - 0.9832\,P)^2$$

where p = pitch of the thread

$$\boxed{A_{nb} = 0.78\,\frac{\pi}{4} \times d^2}$$ For Area of

Reduction factors for shear capacity of the Bolt :

The code suggests the reduction factors for shearing capacity in the following situations.

1. If the Joint is too long.
2. If the grip length is large.
3. If the packing plates of thickness more than 6mm is used.

Reduction factors for long Joints :

If the distance between the first and last bolt of the Joint (l_j) measured in the direction of load exceeds 15d, the shear capacity V_{db} shall be reduced by the factors, β_{lj} given by

$$\beta_{lj} = 1.075 - 0.005\frac{l_j}{d}$$

Subjected to the limits $0.75 \leq \beta_{lj} \leq 1.0$, where d is the nominal diameter of the Bolt.

Reduction factor in grip length :

- If the total thickness of the connected plates exceed 5 times of the diameter of the bolts, the design shear capacity V_{db}, shall be reduced by, $\beta_{lg} = \frac{8d}{3d + l_g}$

subjected to condition maximum value $= \beta_{lj}$, where l_g = grip length = total thickness of the connected plates.

- In no case l_g be greater than 8d.

Reduction factor if packaging plates are used (B_{pk})

If packaging plates of thickness more than 6mm are used in the Joint, then shear capacity is to be reduced by a factor,

$$B_{pk} = 1 - 0.0125\, t_{pk}$$

where t_{pk} = thickness of the thicker packaging in mm.

Thus bearing capacity of the bolts in shear is $\frac{f_u}{\sqrt{3}}(n_n A_{nb} + n_s A_{sb})\, B_{IJ}\ B_{ig}\ B_{pk}$

Bearing Capacity of Bolts :- (V_{dpb})

IS 800:2007 suggest the procedure to find bearing strength of the bolts:

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

where V_{dpb} = design bearing strength

V_{npb} = nominal bearing strength

γ_{mb} = partial safety factor of material = 1.25

Nominal shearing strength may be found from the following relation.

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

where K_b is the smaller of $\frac{e}{3d_0}, \frac{P}{3d_0} - 0.25, \frac{F_{ub}}{f_u}, 1.0$ whichever is less.

In which e, p are end distance, pitch distance:

d_0 = diameter of hole

F_{ub} = ultimate stress of the bolt

F_u = ultimate stress of plate

d = nominal dia. of the bolt.
t = Summation of thickness of the Connected plates experiencing bearing stress in the Same direction. If bolts are Counter Sunk, it is to be reduced by half depth of Counter sinking.

Design procedure with Bearing type Bolts Subjected to shearing forces :

Determine the design action acting on the Joint. Then Select Connection with Suitable diameter of the bolts. Determine the strength of Connection and ensure that design strength is not less than the design action.

1. Diameter of bolt hole :

Nominal size of bolts (d)	12	14	16	20	22	24	30	36
Dia. of bolt hole (d_0)	13	15	18	22	24	26	33	39
Outer dia. of washer	—	—	30	37	—	44	56	60

2. Area of the bolt at root (A_{nb}) :

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

where A_{sb} = Area of the bolt of shank $= \frac{\pi}{4} \times d^2$

3. properties of material of bolts :

Grade 4.6 $F_{yb} = 240$ Mpa $F_{ub} = 400$ mpa
Grade 4.8 $F_{yb} = 320$ Mpa $F_{ub} = 420$ Mpa
Grade 5.6 $F_{yb} = 300$ Mpa $F_{ub} = 500$ Mpa
Grade 5.8 $F_{yb} = 400$ Mpa $F_{ub} = 520$ Mpa

Efficiency of a Joint :

It is defined as the ratio of strength of a Joint and strength of Solid plate in tension. It is expressed in %.

$$\text{efficiency} = \frac{\text{strength of Joint}}{\text{Strength of Solid plate}} \times 100$$

strength of solid plates is less in yielding compared to tearing of solid plate.

For ex :-

Consider Fe_{415} plates

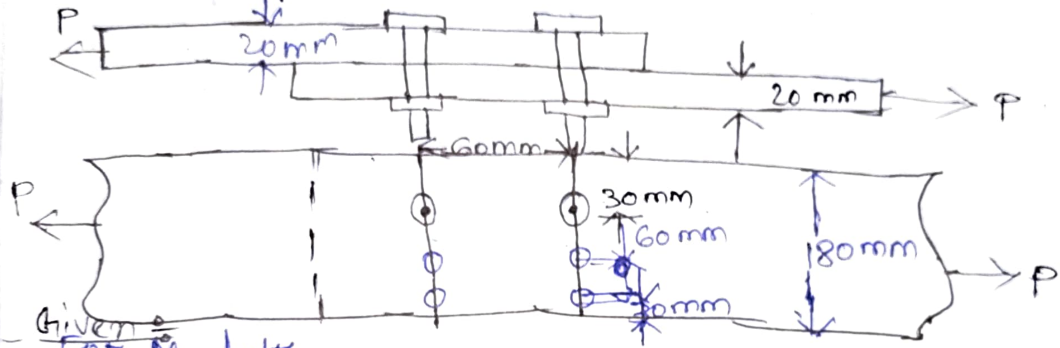
$F_y = 250\ N/mm^2$, $F_u = 410\ N/mm^2$

$\gamma_{m_0} = 1.1$, $\gamma_{ml} = 1.25$

Note :-

Strength of Joint = less of strength of bolt & st. of plate

Q. Find the efficiency of the lap joint shown in 3.16. Given M_{20} bolts and grade of 4.6 and Fe 410 (E 250) plates are used.



Given :-

For M_{20} bolts

dia. of bolt = 20 mm

d_0 (size of hole) $20 + 2 = 22$ mm

For Grade 4.6

F_{ub} = ultimate strength of plate
$= 4 \times 100 = 400\ N/mm^2$

F_{yb} = yield strength of plate
$= 0.6 \times 4 \times 100 = 240\ N/mm^2$

Thickness of thinner the plate (t) = 20 mm

edge distance = (e) = 30 mm

pitch distance (p) = 60 mm

gauge distance = 60 mm

γ_{mb} (partial safety factor) = 1.25

$\gamma_{ml} = 1.25$

Grade of Fe 410 (E 250) plates

Ultimate strength (F_u) = $410\ N/mm^2$

yield strength (F_y) = $250\ N/mm^2$

Strength of the plate in the Joint

Thickness of thinner plate = 20 mm

width of the plate (b) = 180 mm

There is no staggered (P_{st}) = 0

No. of bolt holes in weakest Section = 3

Design strength of plate in the Joint

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

where A_n = net effective area of the member.

In case of chain bolted

$$A_n = [b - n d_0] \times t$$

where b = width of the plate = 180 mm

n = no. of bolts in weakest secn = 3 no.

t = thickness of thinner plate = 20 mm

d_0 = size of hole = 22 mm

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

$$= 2280 \text{ mm}^2$$

Design strength of plate (T_{dn})

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

$$= 673056 \text{ N}$$

$$T_{dn} \simeq 673.056 \text{ KN}$$

Strength of bolts:-

Strength of bolts = Shearing strength of bolt or bearing strength of the bolt, whichever is less.

Shearing strength of bolt (V_{dsb})

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

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

Note:-

shear strength of the bolt for 1 bolt, and lapp Joint n_n = (Area of thread) = 1

n_s (Area of shank) = 0 depend upon the no. of bolts

1st we calculate $A_{nb} = 0.78 \times \frac{\pi}{4} \times d^2$

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

$$= 245.04 \ mm^2$$

No. per bolts

$$V_{nsb} = \frac{400}{\sqrt{3}} (1 \times 245.04)$$ (V_{nsb} = Nominal shear strength)

$$= 56589.563 \ N$$

$$V_{nsb} \simeq 56.589 \ KN$$

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{56589.563}{1.25}$$

$$= 45271.65 \ N$$

$$\simeq 45.271 \ KN$$

Other

No. of Total bolts = 6

$$V_{nsb} = \frac{400}{\sqrt{3}} \times (6 \times 245.04)$$

$$= 339537.38 \ N \simeq 339.537 \ KN$$

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{339537.38}{1.25} = 271.629 \ KN$$

Bearing Capacity of the bolt (V_{dpb})

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

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

K_b is the least of the following,

1. $\frac{e}{3d_0}$ 2. $\frac{p}{3d_0} - 0.25$ 3. $\frac{f_{ub}}{f_u}$ 4. 1.0

- $\frac{e}{3d_0} = \frac{30}{3 \times 22} = 0.4545$
- $\frac{p}{3d_0} - 0.25 = \frac{60}{3 \times 22} - 0.25 = \cancel{0.4540}\ 0.6590$
- $\frac{f_{ub}}{f_u} = 0.9756$
- 1.0

So $K_b = 0.4545$

$$V_{npb} = 2.5 K_b d t f_u$$
$$= 2.5 \times 0.4545 \times 20 \times 20 \times 410$$
$$= 186345 \text{ N}$$

$$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}} = \frac{186345}{1.25}$$
each bolt ≈ 149076 N

Design bearing strength of the bolt
$= 149076 \times 6$
$= 894456$ N.
$\simeq 894.456$ KN

Strength of the bolt = less of shear strength and bearing strength of the bolt.

Design
• Strength of the bolt = 271.629 KN.

strength of Joint = less of strength of bolt and Rupture strength of ~~bolt~~ plate

Strength of bolt = 271.629 KN

Rupture strength of the plate = 673.056 KN

Strength of Joint = 271.629 KN

Efficency of Joint (η)

$$\text{Efficency of Joint} = \frac{\text{strength of Joint}}{\text{Strength of Solid plate}} \times 100$$

strength of solid plate =

$$(T_{dn}) = A_g F_y / \gamma_{m0}$$

Area of the plate = $b \times t$

$$(A_g) = 180 \times 20 = 3600\,mm^2$$

$$T_{dn} = \frac{3600 \times 250}{1.1} = 818181.81\,N$$

$$\simeq 818.181\,KN$$

$$\eta = \frac{271.629}{818.181} \times 100 = 33.19\%$$

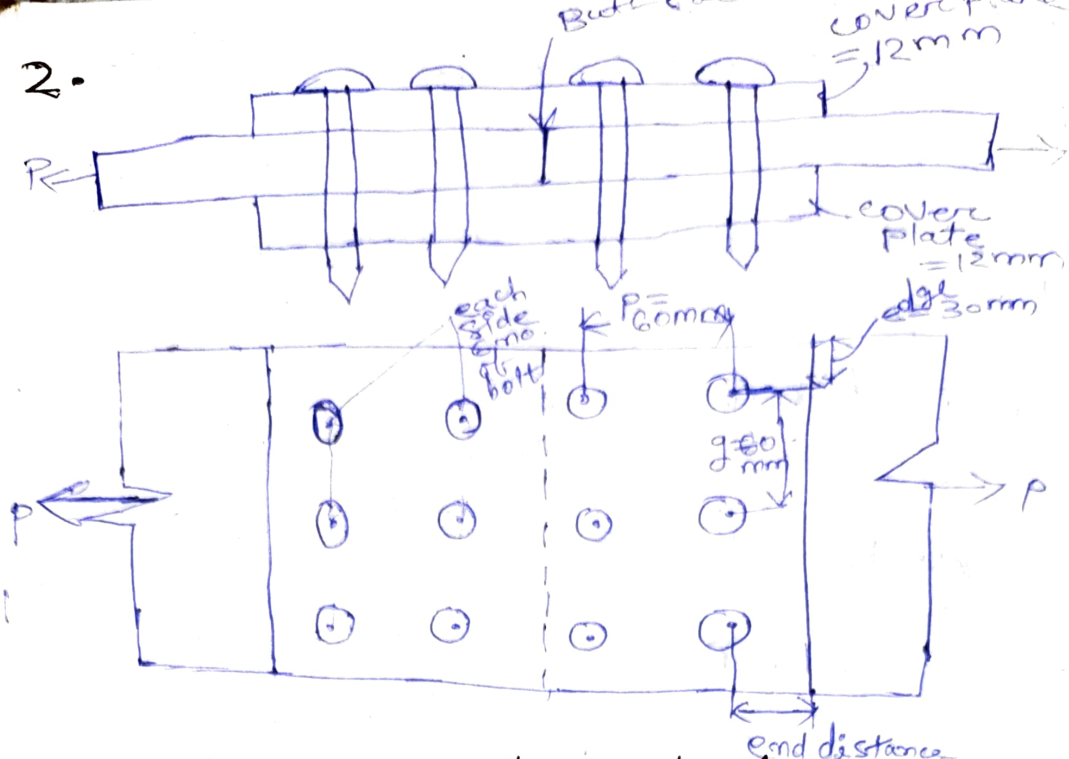
For Single cover butt Joint :-

$n_n = 1,\ n_s = 0$

For double Cover butt Joint

$n_n = 1,\ n_s = 1$

2.



Find the efficency of Joint, If the above example in ~~Lap~~ butt Joint in use two cover plate of each size of 12mm and 6 no. of bolts in each side.

Ans :

For M_{20} bolts grade 4.6,

diameter of bolt = 20 mm

diameter of bolt hole (d_0) = 20+2 = 22 mm

ultimate strength of bolt (F_{ub}) = 4×100
= 400 N/mm^2

yield strength of bolt (F_{yb}) = $0.6 \times 4 \times 100$
= 240 N/mm^2

For Fe 410 (E 250) plates

ultimate strength of plate (F_u) = 410 N/mm^2

yield strength of plate (F_y) = 250 N/mm^2

partial factor of safety = (γ_{mb}) = 1.25

Shear strength of bolt :

Design shear strength (V_{dsb}) : $\frac{V_{nsb}}{\gamma_{mb}}$

$V_{nsb} = (n_n A_{nb} + n_s A_{sb})$

For butt Joint

$n_n = 1$ for each bolt

$n_s = 1$ for each bolt

$A_{sb} = \frac{\pi}{4} \times d^2 = \frac{\pi}{4} \times 20^2 = 314.159 \ mm^2$

$A_{nb} = 0.78 \frac{\pi}{4} \times d^2 = 0.78 \times \frac{\pi}{4} \times 20^2 = 245.04 \ mm^2$

For

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

$= (6 \times 245.04) + (6 \times 314.159) \times \frac{400}{\sqrt{3}}$

$= \cancel{341422} \ 774848.8636 \ N$

$= 774.848 \ KN$

Design shear strength (V_{dsb}) = $\frac{\text{Nominal shear strength } (V_{nsb})}{\text{partial factor of Safety } (\gamma_{mb})}$

$= \frac{774848.8636}{1.25}$

$= 619879.09$

$= 619.87 \ KN$

Bearing strength of bolt :

Design Bearing Strength (V_{dpb}) = $\frac{V_{npb}}{\gamma_{mb}}$

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

K_b is the least of following

- $\frac{edge}{3d_0} = \frac{30}{3 \times 22} = 0.4545$
- $\frac{P}{3d_0} - 0.25 = \frac{60}{3 \times 22} - 0.25 = 0.659$
- $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.976$
- 1.0

So the Value of K_b is 0.4545.

$V_{npb} = 2.5 \times K_b \times d \times t \times f_u$
$= 2.5 \times 0.4545 \times 20 \times 20 \times 410$
$= 186345$ N

$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}} = \frac{186345}{1.25} = 149076$ N for each bolt

Total 6 no. of bolt $V_{dpb} = 149076 \times 6$
$V_{dpb} = 894456$ N.
$\simeq 894.456$ KN

∴ So strength of bolt = less of strength of Shearing Capacity or Bearing Capacity

Strength of bolt = 619.87 / 894.456 KN

∴ So, Hence the strength of bolt 619.87 KN.

Strength of plate in the Joint :

Design strength of plate in Joint
$(T_{dn}) = \frac{0.9 f_u A_n}{\gamma_{ml}}$

A_n = net effective area of member

$A_n = (b - n d_0) t$ n = no. of bolt in weakest Section = 3
b = width of the plate = 180 mm

$A_n = (180 - 3 \times 22) \times 20$
$= 2280$ mm²

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

$= 673056$ N
Design strength of plate $\simeq 673.056$ KN

Strength of Joint = less of strength of bolt or Rupture strength of plate.

Hence strength of bolt = 619.87 KN

Rupture strength of plate = 673.056 KN

strength of Joint = 619.87 KN

Efficiency of Joint : (η)

$$\text{efficiency of Joint} = \frac{\text{strength of Joint}}{\text{strength of Solid plate}}$$

strength of Joint = 619.87 KN

$$\text{strength of Solid plate } (T_{dn}) = \frac{A_g \cdot f_y}{\gamma_{mo}}$$

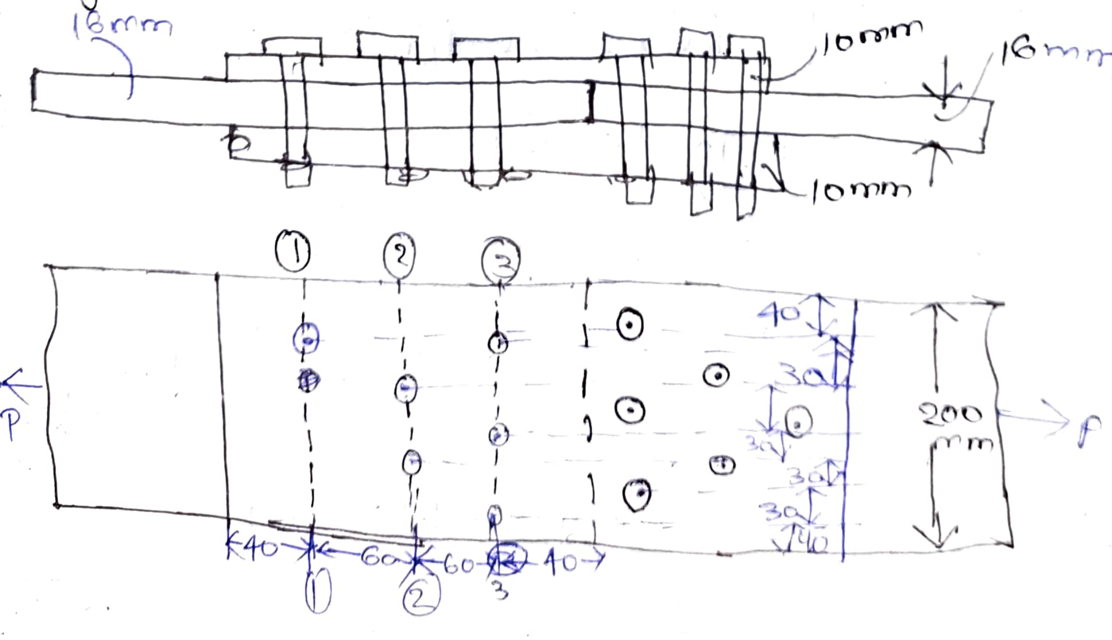
$(\gamma_{mo} = 1.1)$

Area of gross plate $(A_g) = b \times t$
$= 180 \times 20$
$= 3600 \, mm^2$

$$T_{dn} = \frac{3600 \times 250}{1.1} = 818181.8182 \, N$$
$$= 818.181 \, KN$$

$$\eta = \frac{619.87}{818.181} \times 100 = 75.76\%$$

3. Find the maximum force which is transferred through the double covered butt Joint shown in fig. Find the efficiency of Joint also (Given M20 bolts of grade 4.6 & Fe 410 Steel plates used.



Given data :

For M_{20} bolts of grade 4.6

diameter = 20 mm

dia. of hole (d_0) = 20 + 2 = 22 mm

$F_{ub} = 400\ N/mm^2$, $F_{yb} = 0.6 \times 4 \times 100 = 240\ N/mm^2$

For grade Fe 410 plates,

$F_u = 410\ N/mm^2$

pitch distance (p) = 60 mm

edge distance (e) = 40 mm

• Shear Strength of the bolt :

→ $V_{dsb} = \dfrac{V_{nsb}}{\gamma_{mb}}$

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

For butt Joint

$n_s = 1$ for each bolt

$n_n = 1$ for each bolt

$A_{nb} = 0.78 \times \dfrac{\pi}{4} \times d^2 = 245.044\ mm^2$

$A_{sb} = \dfrac{\pi}{4} \times d^2 = 314.159$

$\approx 314.16\ mm^2$

$V_{nsb} = \dfrac{400}{\sqrt{3}} \times \{(6 \times 245.044) + (6 \times 314.16)\}$

$= 774854.4062\ N$

$= 774.85\ KN$

$V_{dsb} = \dfrac{V_{nsb}}{\gamma_{mb}} = \dfrac{774854.4062}{1.25} = 619883.525\ N$

$= 619.883\ KN$

$V_{dsb} = 619.883\ KN$

Bearing strength of the bolt :

- $V_{dpb} = \dfrac{V_{npb}}{\gamma_{mb}}$

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

where k_b is the smaller of

- $\dfrac{e}{3d_0} = \dfrac{40}{3\times22} = 0.6060$

- $\dfrac{P}{3d_0} - 0.25 = \dfrac{60}{3\times22} - 0.25 = 0.6590$

- $\dfrac{f_{ub}}{f_u} = \dfrac{400}{410} = 0.975$

- 1.0

k_b Value is 0.6060

$V_{npb} = 2.5\times0.6060\times20\times16\times410$
$= 198768$ N

$V_{dpb} = \dfrac{V_{npb}}{\gamma_{mb}} = \dfrac{198768}{1.25} = \cancel{159014.4}\ 157440$ N

For Sec 3-3 ~~3~~ no. of bolt $= \cancel{159014.4}\ 157440 \times 3$
$= \cancel{954086.4\ N}$
$= \cancel{954.0864\ KN}$
472.32 KN

For Sec-2-2 k_b is the less of, edge distance = 70

- $\dfrac{e}{3d_0} = \dfrac{70}{3\times22} = 1.060$

- $\dfrac{P}{3d_0} - 0.25 = \dfrac{60}{3\times22} - 0.25 = 0.6590$

- $\dfrac{F_{ub}}{F_u} = \dfrac{400}{410} = 0.975$

- 1.0

$\therefore$ k_b Value is 0.6590

For Section 1-1 K_b, whichever is smaller

- $\frac{e}{3d_0} = \frac{100}{3 \times 22} = 1.515$

- $\frac{p}{3d_0} - 0.25 = \frac{60}{3 \times 22} - 0.25 = 0.6590$

- 1.0

- $\frac{F_{ub}}{F_u} = \frac{400}{410} = 0.975$

∴ K_b value is 0.6590

V_{dpb} for Sec^n 2-2 & Sec^n (1-1) Total 3 no. of bolt

$$V_{dpb} = \frac{2.5 \times 0.65 \times 20 \times 16 \times 410}{1.25} \times 3$$

$= 170560 \text{ N} \times 3$

$= 511680 \text{ N}$

$= 511.68 \text{ KN}$

Total bearing strength of bolt

= bearing strength of bolt sec^n 3-3 + bearing strength of bolt Sec^n 1-1 & 2-2

$= 511.68 + 472.32 \text{ KN}$

Total bearing st. of bolt $= 984.00 \text{ KN}$

- Strength of bolt = less of shearing strength & bearing strength

$= 619.883 / 984 \text{ KN}$

Strength of bolt $= 619.883 \text{ KN}$

each bolt strength $= \frac{619.88}{6} = 103.31 KN$

• Strength of plate :-

$$T_{dn} = \frac{0.9 F_u A_n}{\gamma_{ml}}$$

$\cancel{A_n = (b - nd_0)t}$

At Secn 1-1 :- $T_{dn_1} = \frac{0.9 F_u A_n}{\gamma_{ml}}$

$\Rightarrow A_n = (b - nd_0) t$

$= (200 - 1 \times 22) 16$

$A_n = 2848 mm^2$

$$T_{dn_1} = \frac{0.9 \times 410 \times 2848}{1.25}$$

$= 840729.6 N$

$= 840.729 KN$

$$\boxed{T_{dn-1} = 840.73 KN}$$

At Section 2-2 :-

$$T_{dn-2} = \frac{0.9 F_u A_n}{\gamma_{ml}}$$

$A_n = (b - nd_0) t$

$= (200 - 2 \times 22) \times 16$

$A_n = 2496 mm^2$

$$T_{dn-2} = \frac{0.9 F_u A_n}{\gamma_{ml}} + \text{Sec}^n\text{-1 bolt strength}$$

(so each bolt strength 103.31 KN)

$$= \frac{0.9 \times 410 \times 2496}{1.25} + (103.31 KN \times 1)$$

$= \cancel{869.66 KN + 103.31 KN}$

$= 736819.2 N + 103.31 KN$

$= 736.819 KN + 103.31 KN$

$T_{dn-2} = 840.1292 KN$

At Section 3-3 :

$$T_{dn-3} = \frac{0.9 F_u A_n}{\gamma_{m1}} + \text{Sec}^n \text{ 1-1 bolt} + \text{Sec}^n \text{ 2-2 bolt}$$
(So strength of 3 bolt

$$A_n = (b - n d_0) \times t$$
$$= (200 - 3 \times 22) 16$$
$$A_n = 2144 \text{ mm}^2$$

$$T_{dn-3} = \frac{0.9 \times 410 \times 2144}{1.25} + (3 \times 103.31) \text{KN}$$

$$T_{dn-3} = 632.9088 \text{ KN} + 309.98 \text{ KN}$$

$$T_{dn-3} = 942.88 \text{ KN}$$

∴ Strength of the plate = less of T_{dn-1}, T_{dn-2} & T_{dn-3}

∴ So strength of plate = T_{dn-2} = 840.13 KN

Strength of Joint :

less of strength of plate and strength of bolt

∴ So strength of Joint = 619.883 KN

Efficiency of a Joint :

$$\eta = \frac{\text{Strength of Joint}}{\text{Strength of Solid plate}} \times 100$$

Strength of Solid plate $(T_{dg}) = \frac{F_y A_g}{\gamma_{m0}}$

$$A_g \Rightarrow 200 \times 16$$
$$= 3200$$

$$T_{dg} = \frac{250 \times 200 \times 16}{1.1}$$
$$= 727.272 \text{ KN}$$

$$\eta = \frac{619.883}{727.272} \times 100$$

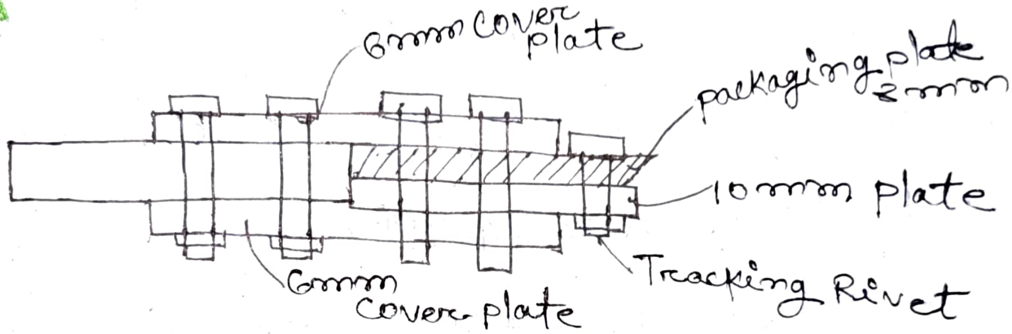
$$\eta = 85.23\%$$

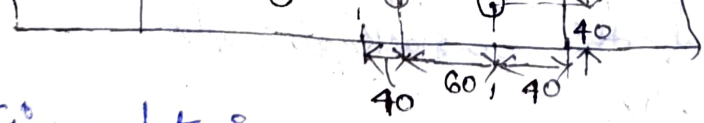
∴ Maximum design force that can be transferred Safely = 619.886 KN.

Design strength / permissible force of condition = $\frac{619.886}{1.5}$ = 413.257 KN ~~495.886 KN~~ plate can be carry safely.

∴ So efficiency is 85.23%.

4. Two cover plates 10 mm & 18 mm thick are connected by double cover butt joint using 6 mm & Fe 415 plates are used to find the strength of joint using M_{20} bolts grade 4.6 & packaging plate.





Given data:-

M_{20} bolts, dia. of bolts = 20 mm

Dia. of hole = 20 + 2 = 22 mm

M_{20} bolts of grade 4.6

ultimate strength of bolt (Fub) = 4 × 100 = 400 N/mm²

yield strength of bolt (Fyb) = 0.6 × 4 × 100 = 240 N/mm² ultimate strength of plate $(F_u) = 415\ N/mm^2$

pitch distance (p) = 60 mm

edge distance (ed) = 40 mm

end distance (e) = 40 mm

gauge distance (g) = 60 mm

There are 2 plates, i.e. 18 mm & 10 mm

So, thickness of the thinner plate (t) = 10 mm.

Shearing strength of bolt :-

- $V_{dsb} = \dfrac{V_{nsb}}{\gamma_{mb}}$, $V_{nsb} = \dfrac{F_{ub}}{\sqrt{3}} \times (n_n A_{nb} + n_s \cdot A_{sb})$

For double cover butt Joint,

$n_n = 1$ for one bolt, $n_s = 1$ for one bolt

$A_{nb} = 0.78 \times \frac{\pi}{4} \times d^2 = 0.78 \times \frac{\pi}{4} \times 20^2 = 245.04\ mm^2$,

$A_{sb} = \frac{\pi}{4} \times d^2 = \frac{\pi}{4} \times 20^2 = 314.159\ mm^2$

$V_{nsb} = \frac{400}{\sqrt{3}} \times ((6 \times 245.04) + (6 \times 314.159))$

$V_{nsb} = 774.84\ KN$, $V_{dsb} = \frac{774.84}{1.25} = 619.87\ KN$

Reduction for packaging plates,

$N_{dsb} = 619.87 \times B_{pk}$, where $B_{pk} = (1 - 0.0125) t_{pk}$

t_{pk} = thickness of thinner packaging plate

$B_{pk} = (1 - 0.0125) \times 8$

$= 0.9$

$N_{dsb} = 619.87 \times 0.9 = 557.883\ KN$

Bearing strength of bolt :

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

K_b is the smaller of

- $\frac{e}{3d_0} = \frac{40}{3 \times 22} = 0.6060$

- $\frac{P}{3d_0} - 0.25 = \frac{60}{3 \times 22} - 0.25 = 0.6590$

- $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.9756$ ∴ Hence K_b Value is 0.6060.

- 1.0

$V_{npb} = 2.5 \times 0.6060 \times 20 \times 10 \times 400$

$\gamma_{mb} = 1.25$

$$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}} = \frac{2.5 \times 0.6060 \times 20 \times 10 \times 410}{1.25}$$

$V_{dpb} = 96960\,N \simeq 96.96$ KN for 1 bolt

Design bearing strength of 6 bolts $= 6 \times 96.96$
$= 581.76$ KN

Strength of bolt = less of Shearing strength / Bearing strength of bolt

∴ strength of bolt = less of Shearing strength of bolt = 557.883 KN

Rupture Strength of plate :-

Design strength of plate $(T_{dn}) = \frac{0.9 A_n f_u}{\gamma_{ml}}$

$A_n = (b - n d_o) \times t$

$A_n = \{200 - (3 \times 22)\} \times 10$

$A_n = 1340 \text{ mm}^2$

$T_{dn} = \frac{0.9 \times 1340 \times 410}{1.25} = 395.568 \text{ KN}$

∴ Design strength of Joint = less of strength of bolt & strength of plate

∴ Hence strength of Joint = 395.568 KN.

5. Design a lap Joint between the 2 plates each of width 120mm. If the thickness of one plate 16 mm and other is 12 mm. The Joint has to transfer a design load of 160 KN. The plates are of Fe410 grade use bearing type bolts.

Given data :-

$b = 120$ mm (width of plate)

lap Joint, one is = 16 mm plate & another = 12 mm

thickness of the thinner plate $(t) = 12$ mm

load $(W) = 160$ KN

Fe410 plates, $F_u = 410 \text{ N/mm}^2$

Let, using M16 bolts of grade 4.6

dia. bolts = 16 mm, dia. of bolt hole = 16+2 = 18 mm

ultimate strength of bolt : $F_{ub} = 400 \text{ N/mm}^2$

yield strength of bolt = $F_{yb} = 240 \text{ N/mm}^2$

Strength of bolt :

- **Shear strength of bolt :**

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}}, \quad V_{nsb} = \frac{F_{ub}}{\sqrt{3}} \times (n_n A_{nb} + n_s A_{sb})$$

$n_n = 1$ for one bolt, $n_s = 0$

$$A_{nb} = 0.78 \times \frac{\pi}{4} \times d^2 = 0.78 \times \frac{\pi}{4} \times 16^2 = 156.8283 \text{ mm}$$

$$A_{sb} = \frac{\pi}{4} \times d^2 = \frac{\pi}{4} \times 16^2 = 201.0619 \text{ mm}^2$$

$$V_{nsb} = \frac{400}{\sqrt{3}} \times ((1 \times 156.823) + (0 \times 201.061))$$

$$= 36216.027$$

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{36216.027}{1.25} = 28.972 \text{ KN}$$

- **Bearing strength of bolt :**

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

$$V_{dpb} = \frac{2.5 K_b d t F_u}{\gamma_{ml}}$$

- K_b is the smaller of

- $\frac{e}{3d_0} = \frac{30}{3 \times 18} = 0.55$

- $\frac{P}{3d_0} - 0.25 = \frac{40}{3 \times 18} - 0.25 = 0.490$

∴ K_b is 0.490

- $\frac{F_{ub}}{F_u} = \frac{400}{410} = 0.9756$

- 1.0

Minimum edge distance $(e_{min}) = 1.5 \times d_0$
$= 1.5 \times 18 = 27$ mm

minimum pitch distance $(P_{min}) = 2.5 d_0$
$= 2.5 \times 16 = 40$ mm

∴ So provide pitch (P) = 40 mm, edge = 30 mm

$$V_{dpb} = \frac{2.5 \times 0.4907 \times 16 \times 12 \times 410}{1.25}$$

$V_{dpb} = 77145.6\ N = 77.1456\ KN$

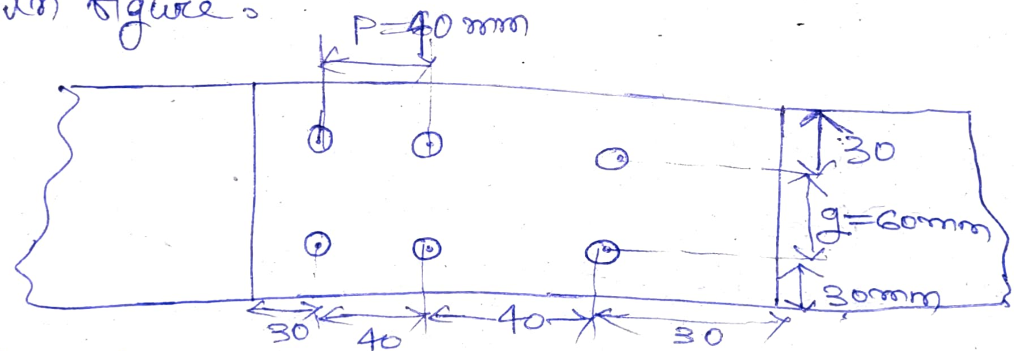
$\therefore$ Design bearing strength of bolt $= 77.1456$ KN

Strength of bolt $= 28.97$ KN for one bolt

Hence to transfer a design load of 160 KN,

No. of bolts Required $= \dfrac{160 \cancel{\times 10^3} KN}{28.97\ KN} = 5.52$

$\therefore$ so provide 6 no. of bolts, provide in 2 rows and pitch 40 mm, gauge 40 mm shown in figure.



check for strength of plate $A_n = (b - n d_0) \times t$

$= (120 - 2 \times 18) \times 12$

$= 1008\ mm^2$

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}} = \frac{0.9 \times 1008 \times 410}{1.25}$$

$= 297561.6\ N$

$\simeq 297.5616$ KN

$\therefore$ Hence $297.5616\ KN > 160\ KN$

Hence Structure is safe.

6. Design a single bolted double cover butt Joint to connect a boiler plates of thickness of 12 mm for maximum efficiency. use M16 bolts of grade 4.6. Boilers plate Fe410 grade. find the efficiency of the Joint?

Given data :-

M 16 bolts, dia. of bolts = (d) = 16 mm

dia. of hole = (d_0) = 16+2 = 18 mm

M16 bolts of grade 4.6

ultimate strength of bolt (F_{ub}) = 400 N/mm²

yield strength of bolt (F_{yb}) = 0.6×4×100
= 240 N/mm²

Boilers plate Fe 410 plates

ultimate strength of plate (F_u) = 410 N/mm²

thickness of the plate (t) = 12 mm

Strength of Bolt :-

1. shear strength of bolt :-

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

$n_n = 1$, for one bolt, $n_s = 1$ for one bolt, we used in this problem double cover butt Joint

$A_{nb} = 0.78 \times \frac{\pi}{4} \times 16^2 = 156.828$ mm²

$A_{sb} = \frac{\pi}{4} \times 16^2 = 201.061$ mm²

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{\frac{400}{\sqrt{3}} \times ((1 \times 156.828) + (1 \times 201.061))}{1.25}$$

$V_{dsb} = 66120.739$ N

$\boxed{V_{dsb} \simeq 66.120 \text{ KN}}$

Bearing strength of bolt :

Assume Bearing strength is more than the shearing strength. Hence the strength of bolt,

Strength of bolt = 66.12 KN

- To get maximum efficency, strength of plate or pitch width should be equal to strength of a bolt.

- To avoid failure of cover plates the total thickness of cover plates should be more than the thickness of plates.

Hence provide 8 mm plates.

- Design strength of plate per pitch width

$$(T_{dn}) = \frac{0.9 A_n F_y}{\gamma_{ml}}$$

$A_n = (b - n d_o) \times t$

$A_n = P - (1 \times 18) \times 12$

$$T_{dn} = \frac{0.9 \times \{(P-18) \times 12\} \times 410}{1.25} = 3542.4 (P-18) \text{ N}$$

∴ For maximum efficency, Design strength of plate = Strength of bolt

∴ $3542.4 (P - 18) = 66120$

$= \quad P - 18 = \frac{66120}{3542.4} = 18.66 + 18$
$= 36.66$

$P = 36.66$ mm

∴ So P minimum $= 2.5 \times d = 2.5 \times 16$
$= 40$ mm

∴ So provide pitch 40 mm.

e_{min} (edge) $= 1.5 \times d_o$
$= 1.5 \times 18 = 27$ mm

So provide edge distance 30 mm.

Bearing strength of bolt :

$$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}} = \frac{2.5 K_b d t F_u}{1.25}$$

K_b is the less of

- $\frac{e}{3d_0} = \frac{30}{3\times18} = 0.555$
- $\frac{P}{3d_0} - 0.25 = \frac{40}{3\times18} - 0.25 = 0.4907$
- $\frac{F_{ub}}{F_u} = \frac{400}{410} = 0.9756$ $\therefore K_b = 0.4907$
- 1.0

$$V_{dpb} = \frac{2.5\times0.4907\times16\times12\times400}{1.25}$$

$= 75264$ KN

efficiency ~~of a Joint~~ $= \frac{\text{strength of Joint}}{\text{Strength of solidplate}}$

Strength of solid plate $= (T_{dg}) = \frac{A_g F_y}{\gamma_{mo}}$

$A_g = p \times t$

$$T_{dg} = \frac{40\times250\times12}{1.10}$$

$= 109090.90N \simeq 109.09$ KN

$$\eta = \frac{\cancel{66012}\ 66120.7}{109090.90}\times100$$

$= 60.610\%$

HSFG Bolts :-

HSFG bolts are mainly divided into 2 types.

1. parallel shank 2. Waisted shank.

1. parallel shank :-

It gives slip resistance at service load, but not at ultimate load.

2. Waisted shank :-

It gives slip resistance even at ultimate load (as well as service load)

IS 800-2007 Recommend use to nominal shear capacity of HSFG bolts.

$$V_{nsf} = \mu_f \, n_e \, K_h \, F_o$$

μ_f = co-efficient of friction (called slip factor)

n_e = number of effective interfaces offering frictional resistance to slip

K_h = 1.0 for fasteners in clearance holes

F_o = Minimum bolt tension, $A_{nb} f_o$

A_{nb} = net area of bolt threads $(0.78 \times \frac{\pi}{4} \times d^2)$

f_o = proof stress = 0.70 f_{ub}

f_{ub} = ultimate strength of bolt = 800 N/mm²

The slip resistance should be taken as,

$$V_{dsf} = \frac{V_{nsf}}{\gamma_{mf}}$$

where γ_{mf} = 1.10, If the slip resistance is design at service load (parallel shank HSFG)

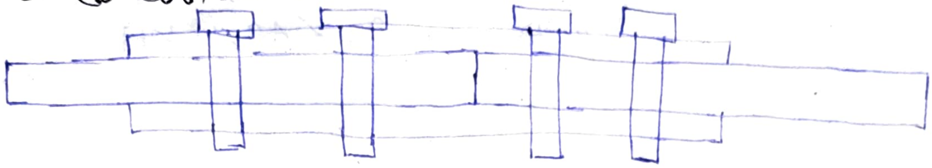
γ_{mf} = 1.20, If the slip resistance is designed at ultimate load (Waisted shank HSFG)

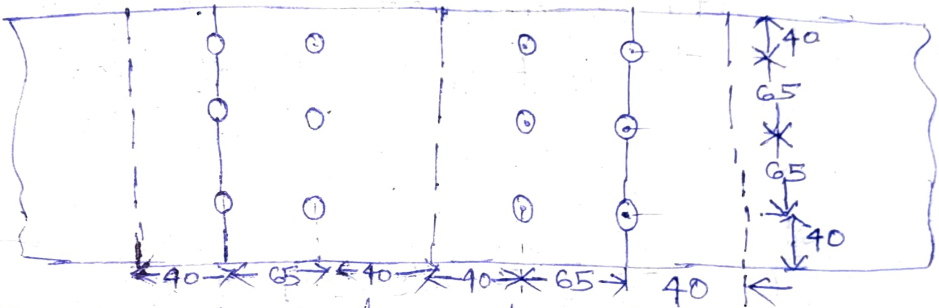
1. Determine the shear capacity of bolts used in connecting two plates as shown in fig 3.30.1b

- slip resistance is designed at service load.
- Slip resistance is designated at ultimate load.

- Given

1. HSFG bolts of grade 8.8 are used.
2. Fasteners in clearance hole
3. Co-efficient of friction = ($\mu_f = 0.3$)





For HSFG bolts of grade 8.8

$F_{ub} = 8 \times 100 = 800\ N/mm^2$

- For fasteners of clearance holes, $K_h = 1.0$
- Co-efficient of friction (μ_F) = 0.3

Nominal shear capacity of bolt

$(V_{nsF}) = \mu_F n_e K_h F_o$

$n_e = 2$, Hence it is a double cover butt joint

$F_o = 0.70 f_{ub} A_{nb}$

$= 0.70 \times 800 \times \frac{\pi}{4} \times 20^2$

$= 137224.76\ N$

$V_{nsF} = 0.3 \times 2 \times 1 \times 137224.76$

$= 82334.85\ N$

$V_{nsF} = 82335\ N$

i. Design capacity of one bolt, If it is slip resistance is designated at Service load, so $\gamma_{mf} = 1.10$

$$V_{dsf} = \frac{V_{nsf}}{\gamma_{mf}} = \frac{82335}{1.10} = 74850\ N$$

$\simeq 74.85$ KN for one bolt.

Design capacity of Joint (for 6 no. of bolts) $= 6 \times 74850$

$= 449100\ N$

$V_{dsf} \simeq 449.1$ KN

ii. Design capacity of one bolt, If it is slip resistance is designated, at ultimate load, So $\gamma_{mf} = 1.25$

$$V_{dsf} = \frac{V_{nsf}}{\gamma_{mf}} = \frac{82335}{1.25} = 65868\ N$$ for one bolt.

Design Capacity of Joint (for 6 no. bolts) $= 6 \times 65868$

$= 395208\ N$

$V_{dsf} = 395.208$ KN

chapter-4 **Welded Connection**

• Welded consists of Joining two pieces of metal metallurgical bond between them. The elements to be connected are brought closer and the metal is melted by means of electric arc or oxy acetylene flame along with weld rod which adds metal to the Joint After cooling the bond is established between the 2 element

Advantages of welded connection :-

1. Due to the absence of gusset plates, connecting angles etc. welded structure are lighter.
2. The absence of making holes for fasteners, making welding process quicker.
3. Welding is more adoptable than bolting and riveting. for ex: circular tubes can be easily connected by welding.
4. It is possible to acheive 100% efficency in the Joint whereas the bolted can connection can reach a maximum 70-80% only.
5. Noise produced a welding process is relatively less.
6. Welded connection have good asthetic apperance
7. Welded connection is air tight and water tight. Hence there is corrosion of steel structure and welded connection making for water tanks.
8. Welded Joints are rigid.
9. There is no problem of mismatching of holes in welded connection whereas in bolted connection mismatching of bolt holes creates considerable problem.

Alteration in consideration can be easily made in the design of welded connection.

Disadvantages of welded connection:

1. Due to uneven heating and cooling members are likely to distort in the process of welding
2. There is a greater possiblity of brittle fracture in welding.
3. A welded Joint fails earlier than a bolted Joint If the structure is under fatigue stress.
4. The inspection of welding Joint is difficult and expensive. It needs non destructive testing
5. Highly skilled person is required for welding
6. proper welding in field connection is difficult.
7. Welded Joints are over Joint.

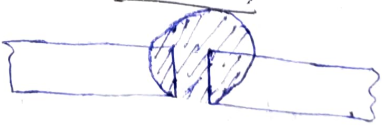
Types of welded Joints:

There are 3 types of welded Joint.

1. Butt weld
2. Fillet weld
3. Slot weld or plug weld.

Butt weld:

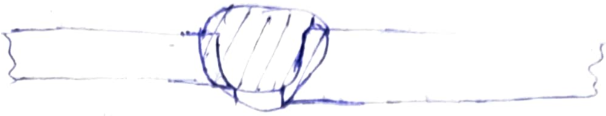
Butt weld is also known as groove weld. Depending upon the shape of groove made for welding welds are classified.

SL. NO.	Types of Butt weld	Sketch
1.	Square butt weld, on one side	
2.	Square butt weld, both sides	

3. Single V butt Joint

4. Double V butt Joint

5. Single U butt Joint

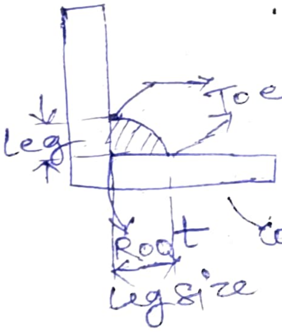


6. Single J butt Joint

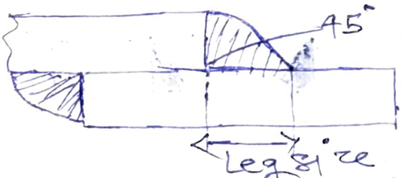
7. Single bevel butt Joint

Fillet weld :-

- Fillet weld is a weld of approximately triangular cross Section Joining two Surfaces approximately right angles to each other in lap Joint, tee Joint or Corner Joint.



Corner Joint

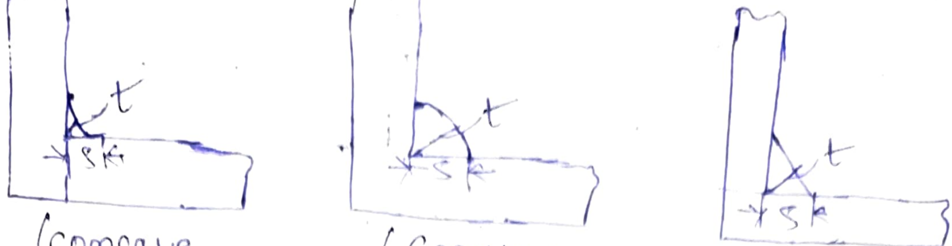


(Lap Joint)

- When the cross Section of fillet weld is isosceles triangle with face at 45°, it is known as a standard fillet weld.

In Special Consideration/circumstances 60° and 30° angles are also used.

- A fillet weld is known as Concave fillet weld. Convex fillet weld or as mitre fillet weld depending upon the shape of weld face.



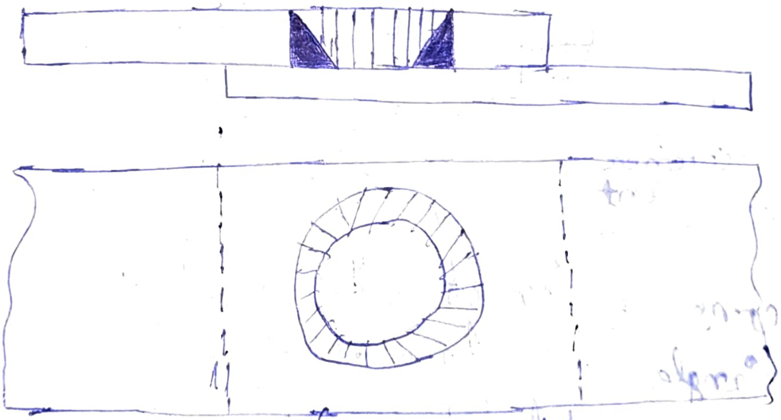
(concave fillet weld) (convex fillet weld) (mitre fillet weld)

where s = size of weld
t = throat thickness.

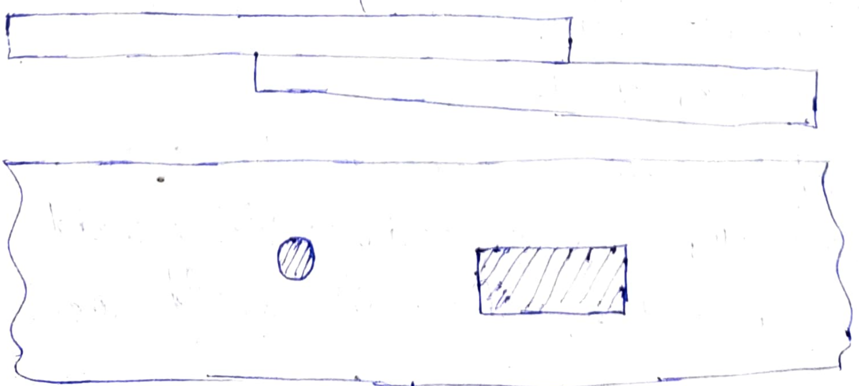
Slot weld and plug weld :

- A slot weld which a plate with circular hole is kept with another plate to be Joined. and then fillet welding is made along the periphery of the hole.

- A plug weld in which 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.



(Slot weld)



(plug weld)

Important Specifications for welding :

- Requirements of welds and welding shall confirm to IS 816 & IS 9595, as appropriate. Some important specification regarding butt weld and plug and slot welds as per IS 800-2007 listed below.

Butt weld :

1. The size of Butt weld shall be specified by the effective throat thickness. In case of complete penetration butt weld shall be taken as thickness of the thinner part Joined. Double U, Double V, double J and double bevel butt welds may be generally regarded as the completion penetration of butt welds.

OR

- The size of butt weld (S) = thickness of the thinner plate, in case of complete penetration
- The effective throat thickness in case of in complete penetration, butt weld shall be taken as the minimum thickness of weld metals part Joined, excluding reinforcement the absence of actual data, taken as the $\frac{5}{8}$th of thickness of thinner plate.

2. The effective length of butt weld shall be taken as the length of full size weld.
3. The minimum length of butt weld, shall be four times the size of weld.
4. If intermittent butt welding is used, the effective length of not less than 4 times of the weld size and space between the 2 welds shall not be more than 16 times the thickness of thinner part Joined.

Fillet weld :

1. Size of weld = Minimum leg size

- For deep penetration weld with penetration not less than 2.4 mm, then

Size of weld = Minimum leg size + 2.4 mm

- For fillet welds made by semi automatic or automatic process with deep penetration more than 2.4 mm, then

Size of weld = Minimum leg size + actual penetration

2. Minimum size of fillet weld (S_{min}) = 3 mm

To avoid risk of cracking in the absence of preheating the minimum size specified are,

For less than 10 mm plate = 3 mm
For 10 to 20 mm plate = 5 mm
For 20 to 32 mm plate = 6 mm
For 32 to 50 mm plate = 8 mm

3. Effective throat thickness shall not be less than 3 mm and shall not generally exceed 0.7t, where t = thickness of thinner plate of elements being welded.

- If the faces of plates being welded are inclined to each other, the effective throat thickness shall be taken as K times as per fillet size.

Angle between fusion faces	60°–90°	91°–100°	101°–106°	107°–113°
Constant (K)	0.70	0.65	0.60	0.55

$t = KS$

where K = constant, S = size of weld

4. Effective length :- In drawing only effective length is shown. While welding length is made equal to effective length should not be less than four times the size of the weld.

- $L_w = L_{eff} + 2S$

$L_{eff} \min = 4S$

where L_w = welding length

where S = size of weld

5. Lap Joint :- The minimum lap should be four times the thickness of thinner part or 40mm whichever is more. The length of weld along either edge should not be less than in the transverse spacing of the welds.

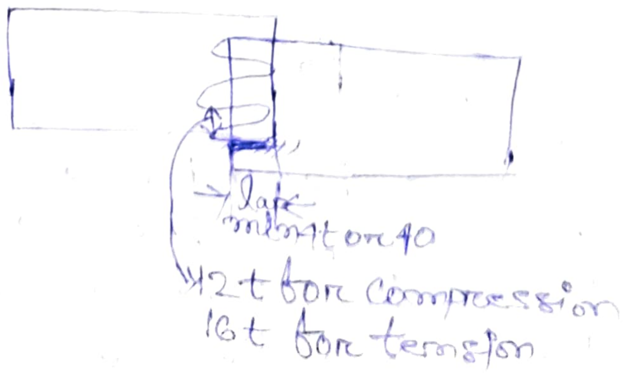
Lap min = 4 × thickness of thinner part or 40mm whichever is more

6. Intermittent welds :-

- Length shall not be less than 4 times the weld size or 40mm whichever is more. The minimum clear spacing of intermittent weld shall be 12t for compression joints and 16t for tension joints, where t is the thickness of thinner part.
- The intermittent welds shall not be used in position to subject to dynamic, repetitive and alternative stresses.

plug welds :-

The



Plug welds :

The effective area of the plug weld shall be considered nominal area of the hole.

Design stress in welds :

• **Butt welds :**

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

Fillet weld, slot or plug welds :

Design strength shall be based on its throat area shall be given by,

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

where $f_{wn} = \frac{f_u}{\sqrt{3}}$

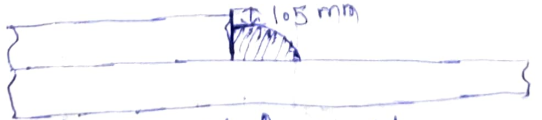
f_u = Smaller of the ultimate stress of the weld or of the parent metal.

γ_{mw} = 1.25 for shop welds

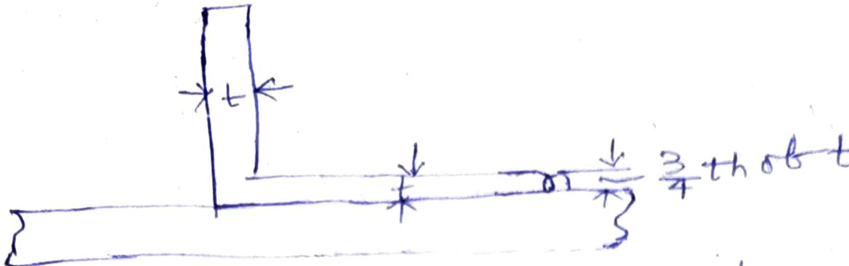
1.5 for field welds.

• The following provision are made in the Code for the fillet welds applied to the edge of a plate or Section :

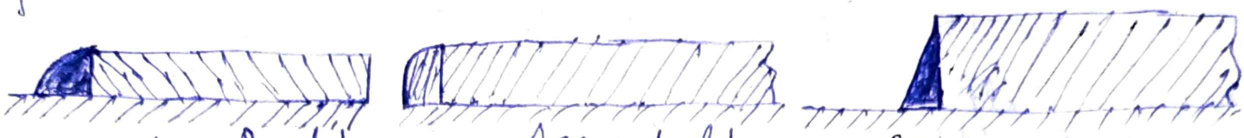
1. If a fillet weld is to the Square edge of a part, the Specified size of weld should be generally atleast 1.5mm less than the edge thickness.



2. If a fillet weld is to the rounded toe of rolled Section, the Specified size of the weld should generally not exceed $\frac{3}{4}$th of thickness of the Section at the toe.



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



4. End fillet weld, normal to the direction of force shall be unequal size with throat thickness not less than 0.5t. The difference in the thickness of weld shall be negotiated at a uniform slope.

Reduction in design stresses for long joints:

If the length of the welded joint (l_j) is greater than 150t, where t is the throat thickness, the design capacity of weld fwd shall be reduced by the factor

$$B_{tw} = 1.2 - \frac{0.2 l_j}{150t} \leq 1.0$$

1. A 18 mm plate is joined to a 16 mm plate by 200 mm long (effective) butt weld. Determine the strength of joint.

If,
i. a double V butt weld is used.
ii. a single V butt weld is used.
Assume that Fe 410 grade plates and shop welds are used.

Given data:

i. Case-I Double V butt weld Joint

Thickness of one plate = 16 mm

and thickness of another plate = 18 mm

∴ thickness of thinner plate = 16 mm

Length of weld (L_w) = 200 mm

$F_u = 410 \, N/mm^2$

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

Design strength of weld = $\frac{L_w \, t \, F_u}{\sqrt{3} \times \gamma_{mw}}$

$\sqrt{3}$ = Filet weld

$\sqrt{3}$ = Butt weld in shear

$= \frac{200 \times 16 \times 410}{\sqrt{3} \times 1.25} = 605986.84 \, N$

$= 605987 \, N$

$\simeq 605.98 \, kN$

Since in such case complete penetration taken place, throat thickness (t) = thickness of thinner plate

So (t) = 16 mm

Case-2

In case of single V butt weld Joint, since penetration is not complete, so effective throat thickness (t) = $\frac{5}{8}$th of the thickness of the thinner plate

throat thickness (t) = $\frac{5}{8} \times 16 = 10$ mm

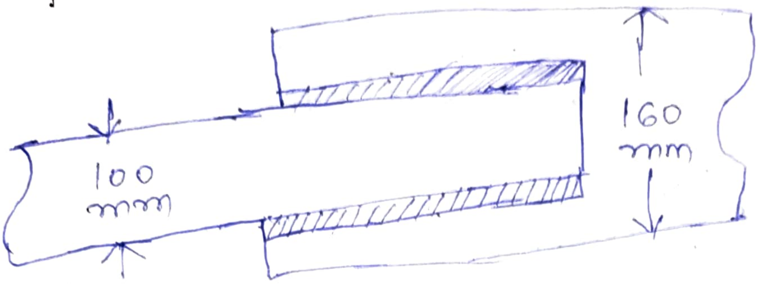
So, Design strength = $\frac{L_w \times t \times F_u}{\sqrt{3} \times \gamma_{mw}}$

$= \frac{200 \times 10 \times 410}{\sqrt{3} \times 1.25}$

Design strength = 378741.77 N

$\simeq$ 378.74177 kN.

2. Design a ~~stre~~ Suitable longitudinal fillet weld to ~~connect~~ connect the plates as shown in fig. to transmit a pull equal to the full strength of small plate. Given plates are 12 mm thick, grade of plates Fe 410 and welding to be made in workshop.



Given data :-

ultimate Strength (F_u) = 410 mpa

welding shop made work (γ_{mw}) = 1.25

thickness of thinner plate = 12 mm

Minimum size of weld (S_{min}) = 5 mm ← seen from IS code

Maximum size of weld (S_{max}) = $t - 1.5$ (for Square edge maximum)

S_{max} = 12 − 1.5
= 10.5 mm

S_{max} = $t - 1.5$ for Square edge
$S_{max} = \frac{3}{4} \times t$ for round edge

use S (size of weld) = 10 mm fillet weld.

Breadth of plate = 100 mm

full design strength of smaller plate = $\frac{A_g F_y}{\gamma_{m0}}$

$= \frac{100 \times 12 \times 250}{1.1}$

Design strength of the smaller plate = 272727.27 N

Here Given the design strength of plate = strength of weld

$272727.27 = \frac{L_w t f_u}{\sqrt{3}\, \gamma_{mw}}$

throat thickness :

$(t) = KS$

where $t_k = 0.7 \times S$ where $K = 0.7$

$= 0.7 \times 10$

$= 7$ mm

So design strength of plate = strength of weld

$$\Rightarrow \frac{L_w \times 7 \times 410}{\sqrt{3} \times 1.25} = 272727.27$$

$L_w = 205.73$ mm

provide effective length of 105 mm on each side.

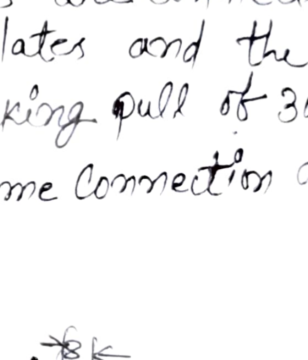
$(L_{eff}) = \frac{205.73}{2} = 102.865$ mm
$\simeq 105$ mm

3. A tie (tension) member of a roof truss consists of 2 ISA 100, 75, 8mm. The angles are connected to either of a 10 mm gusset plates and the member is subjected to a working pull of 300 k[N] Design the weld connection. Assume connection are made in workshop.

Given data :

working load = 300 KN

ultimate/factored load = 300×1.5
$= 450$ KN



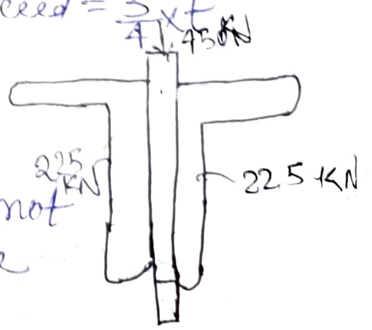
Size of weld :

• $S_{min} = 3$ mm, < 10 mm

• At the rounded toe of an angle section size of weld shouldn't exceed $= \frac{3}{4} \times t$

$S_{max} = \frac{3}{4} \times 8$ mm

$S_{max} = 6$ mm



• At top the thickness should not be exceed at the square edge

$= t - 1.5$

$= 8 - 1.5$

$= 6.5$ mm

Hence provide size of weld (S) = 6 mm

∴ Each angle carries a factored load of $= \frac{450}{2} = 225$ KN

Let Lw be the total weld length required

So, throat thickness (t) $= 0.7 \times 6$ ($t = KS$)

$= 4.2$ mm

Design strength of weld $= \frac{L_w \times t \times f_u}{\gamma_{mw} \times \sqrt{3}}$

$= \frac{L_w \times 4.2 \times 410}{\sqrt{3} \times 1.25}$

$= \frac{L_w \times 4.2 \times 410}{\sqrt{3} \times 1.25}$

Hence, as we know the,

Design strength of weld = Factored load of each angle

$\Rightarrow \frac{L_w \times 4.2 \times 410}{\sqrt{3} \times 1.25} = 225 \times 10^3$

$\Rightarrow L_w = 282.89$ mm

$L_w \simeq 283$ mm

$L_w = L_1 + L_2 = 283$ mm

centre of gravity of the angle section is at a distance 31 mm from the top. (see on steel code Cxx)

Let L_1 be the length of top weld and L_2 be the length of bottom weld. To make centre of gravity of weld, to coincide with that of angle.

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

$L_1 = L_2 \times \frac{69}{31}$ —— (i)

$L_1 + L_2 = 283$

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

$\Rightarrow 2.225 L_2 + L_2 = 283$

$3.225 L_2 = 283$

$L_2 = 283 / 3.225$

$L_2 = 87.75$

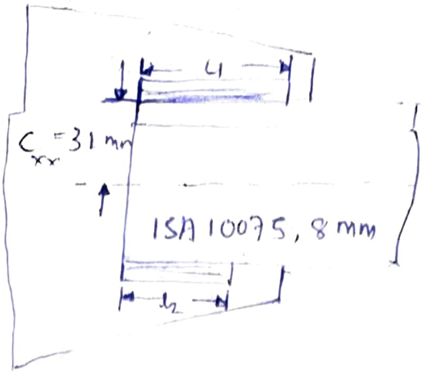
$L_2 = 87$ mm

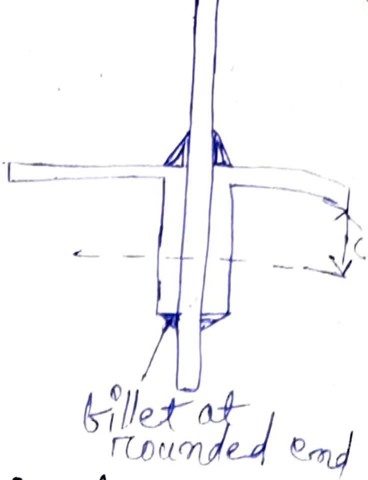
$L_1 = L_2 \times \frac{69}{31}$

$= 87 \times \frac{69}{31}$

$= 193.645$

$L_1 = 195$ mm





4. Design the welded connection to connect two plates of width 200 mm and thickness 10mm ~~Fore~~ 100% efficency.

Given data :-

width of plate = 200 mm

thickness of plate = 10 mm

As per our 100% efficency,

Strength of weld = strength of plate

Strength of plate $= \frac{A_g f_y}{\gamma_{m0}} = \frac{200 \times 10 \times 250}{1.1}$

$= 454545.45$ N

$\simeq 454.54$ KN

Minimum size of weld = 5 mm

Maximum size = t − 1.5

= 10 − 1.5

= 8.5 mm

So, use size of weld = (S) = 8 mm

Effective length of fillet weld (Lw)

$= (B - 2S) \times 2$

$= (200 - 2 \times 8) \times 2$

$L_w = 368$ mm

throat thickness $(t) = KS$

$t = 0.7 \times 8$
$= 5.6$ mm

Design strength of fillet weld

$$= \frac{L_w t f_u}{\sqrt{3} \times \gamma_{mw}}$$

$$= \frac{368 \times 5.6 \times 410}{\sqrt{3} \times 1.25}$$

$= 390255.526$ N
$\simeq 390256$ N

Hence as per our requirement the strength of fillet weld is not equal to the strength of plate, so we may be provide slot weld.

So, slot welds are provided to resist force of $= 454545 - 390256$
$= 64289$ N.

Strength of slot weld $= \frac{f_{wn}}{\gamma_{mw}} = \frac{f_u}{\sqrt{3}\,\gamma_{mw}}$

$$= \frac{410}{\sqrt{3} \times 1.25}$$

$= 189.370$ N/mm^2

So we required per 1 mm^2 = 189.370 N.

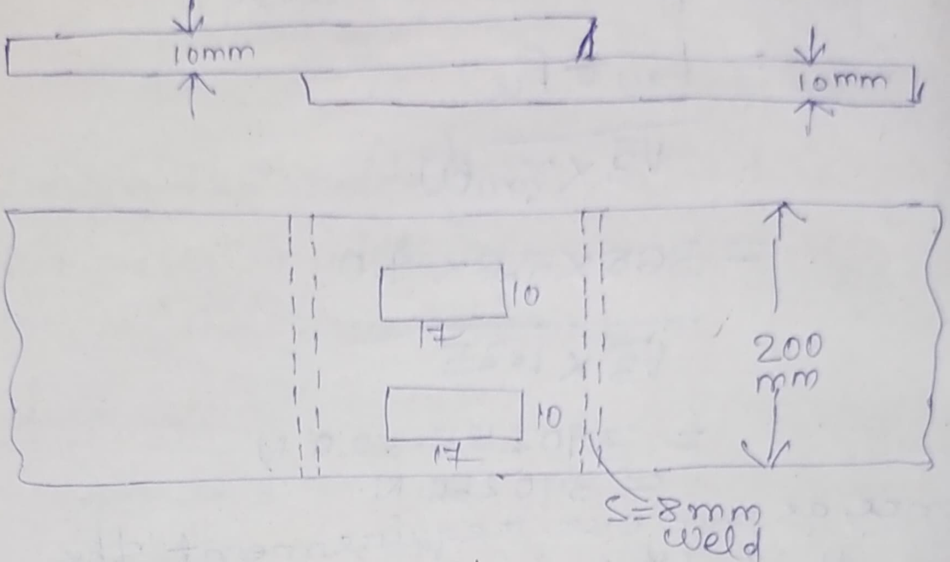
189.370 N $= 1$ mm^2

1 N $= \frac{1}{189.370}$ mm^2

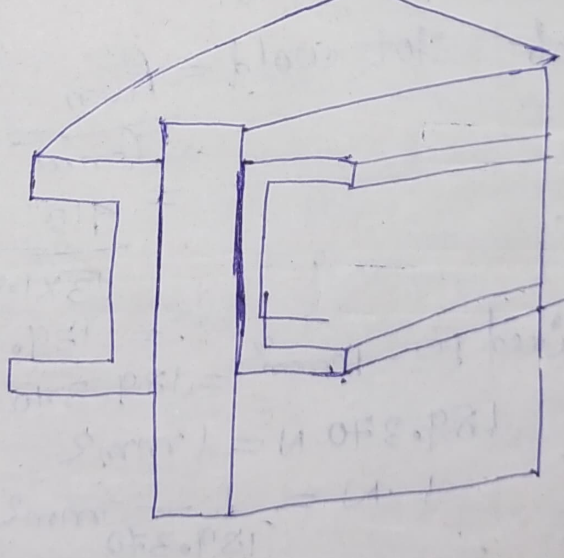
$\cancel{390256}$ 64289 N $= \frac{\cancel{390256}\ 64289}{189.370}$ mm^2

Hence Area of the slot required $= 339.48$ mm^2
$= 340$ mm^2

Hence provide two slot welds of size 10×17 mm of 2 slot hole.



5. A tie member of ~~2~~ two ISMC 250, The channel are connected on either side of 12 mm thick ~~g~~ gusset plates. Design the welded Joint to develop the full strength of ~~each~~ the tie. However the overlap is be limited to 400 mm.



Given data :-

For ISMC 250, from steel tables,

Thickness of web = 7.1 mm

Thickness of flange = 14.1 mm

Sectional area = 3867 mm^2

Tensile design strength of each channel,

$(T_{dg}) = \dfrac{A_g f_y}{\gamma_{mo}}$

$= \dfrac{3867 \times 250}{1.1}$

$= 878863.63$ N

Design strength of each channel $= 878864$ N

Size of weld :-

Minimum thickness, size of weld (S_{min}) = 3 mm (7, <10mm Hence)

Maximum thickness of weld (S_{max}) = $t - 1.5$

$= 7.1 - 1.5$

$= 5.6$ mm

Hence provide size of weld (S) = 4 mm

throat thickness (t) $t = KS$

$= 0.7 \times S$

$= 0.7 \times 4$

$= 2.8$ mm

Strength of weld $= L_w \times t \times \dfrac{f_u}{\sqrt{3}} \times \dfrac{1}{\gamma_{mw}}$

$= L_w \times 2.8 \times \dfrac{410}{\sqrt{3}} \times \dfrac{1}{1.25}$

Hence, strength of weld = Strength of channel

$878864 = L_w \times 2.8 \times \dfrac{410}{\sqrt{3}} \times \dfrac{1}{1.25}$

$L_w = 1657.48$ mm

∴ Length of weld (L_w) = 1658 mm.

Since allowable limited to 400+400 mm not equal to 1658 mm. So it needs slot weld. two slots length is 'x'.

$$400+400+(250-2\times30)+4x=1658$$

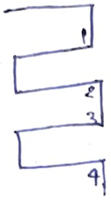
$$\Rightarrow 800+190+4x=1858$$

$$\Rightarrow 990+4x=1658$$

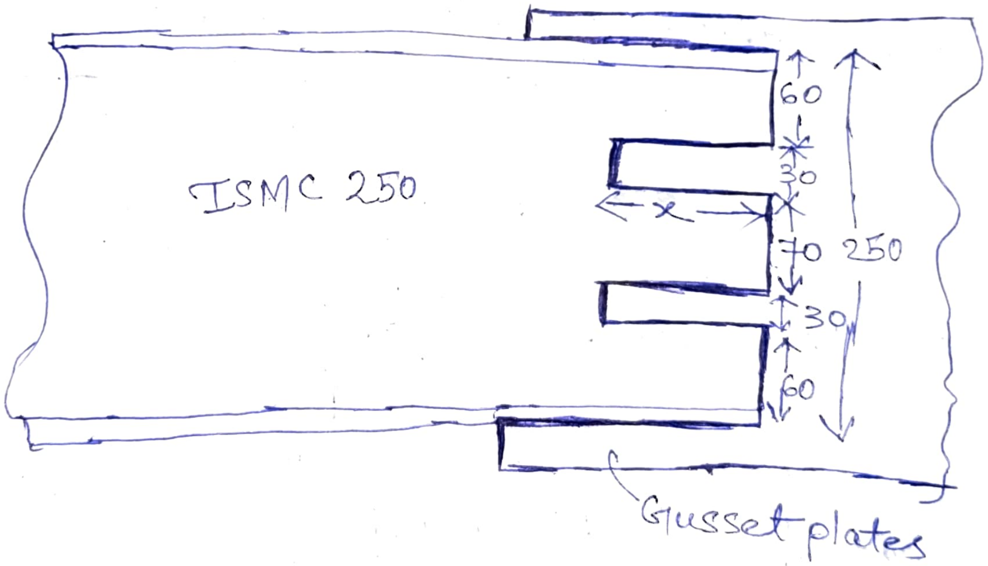
$$4x=1658-990$$

$$4x=668$$

$$x=\frac{668}{4}=167 \text{ mm}$$



– Hence, S = 4,



Design of Tension member

- Tension members are also known as tie members.
- Tension members are linear member predominatly Subjected to pulling (direct axial tensile load) which stretch/elongate the member.

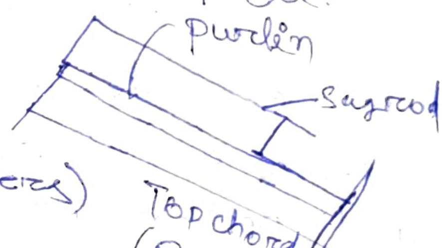
Ex:- A rope is a tension member.

- Tension member in a truss is known as tie

Ex:- Suspenders of Cable stayed and Suspended bridges, Sag rods of roof purlins and Suspenders of building System hung from central Core are other resistance in transmission line/ Satelite towers, braced framed etc.

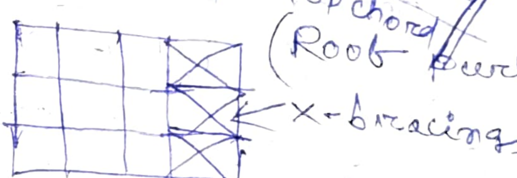
(Roof truss)

(Suspenders)



(Roof purlin)

(Stayed bridge)

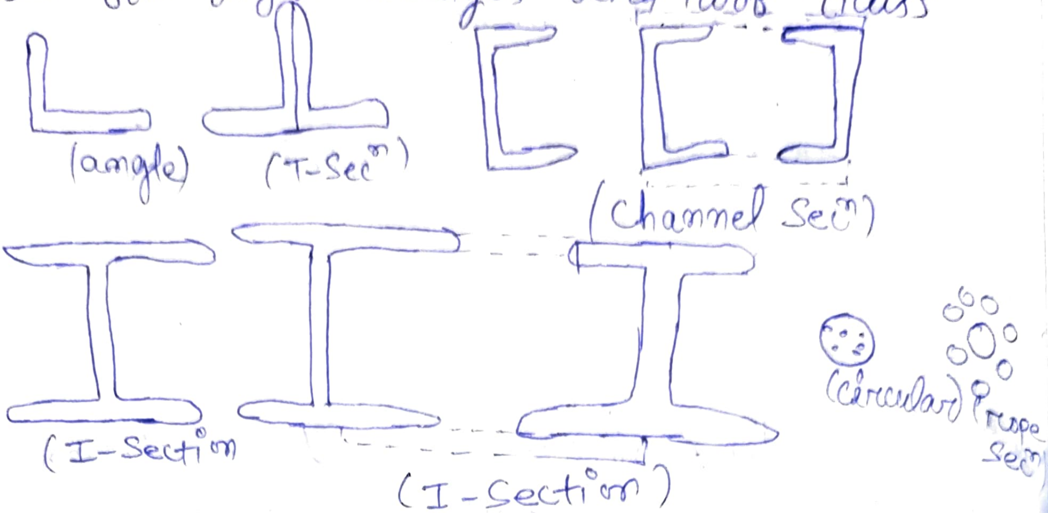


(Braced frame)

- Tension members are pure efficent because a member in pure tension can be stressed upto and yield limit overal buckling.
- under tension due to lateral load in one direct
- With a view to avoid stress Concentration, a tension member should arranged to large portion if it is possible is Connected to the gusset plates.

Common Shapes of tension member :-

Tension members can be Variety of cross sections Various forms of tension member used for light bridges and roof truss



- Steel section such as angles, I-channels and tee-Section provide more rigidity towards buckling in Compression when reversal of load takes place under wind load.
- Single angle Section develop bending stress due to eccentricity between and connection and position of their Centre of gravity.
- Double angle and channel Section develop relatively less eccentricity.
- Built up Section are used for heavy loads. The arrangement of built up. Section made up
- The arrangement of angle and channel Suitable for Section and Cover plates etc. In order to provide sufficent cross Sectional areas and to be Suitable for Connection with adJoining member

Types of failure of tension member :-

A tension member fail any following methods.
a. yielding of the gross section
b. Rupture of the net critical section
c. Block shear

a. yielding of gross section occurs when considerable deformation of the member in longitudinal direction takes place before it fractures, making the structure unserviceable.

b. Rupture of critical section takes place when the net sectional area of the member reaches the ultimate stress.

c. Block shear failure :- A segment of the block of the material at the end of member out of the member shears out due to possible use of high grade steel and high strength of bolts resulting in smaller connection length.

- To avoid the failure, the factored design load T in the member should be less than the design strength in tension Td of the member

Design strength of a tension member :

Design strength of a tension member is the least of followings.

a. Design strength of yielding of gross section :

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

where f_y = yield stress of material

A_g = gross area of the gross section.

γ_{m0} = partial safety factor for failure = 1.1

b. Design strength due to rupture of critical section :

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

γ_{m1} = partial safety factor ultimate stress = 1.25

f_u = ultimate stress of the material

A_n = net effective area of the member

$A_n = (b - n d_0) \times t$

b = width of the plate

d_0 = diameter of hole

n = no. of bolt holes in critical section

b. Threaded rods :

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

A_n = net root area at the threaded section

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

c. Design strength due to block shear :

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

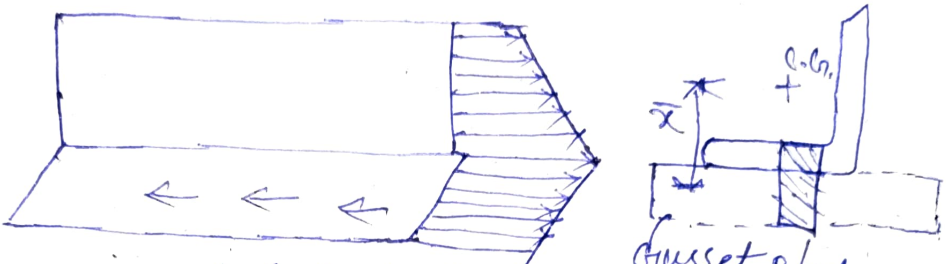
- $$T_{db_2} = \frac{A_{tg} f_y}{\gamma_{m0}} + 0.9 \frac{A_{vn} f_u}{\sqrt{3}\,\gamma_{m1}}$$

Concept of Shear lag:

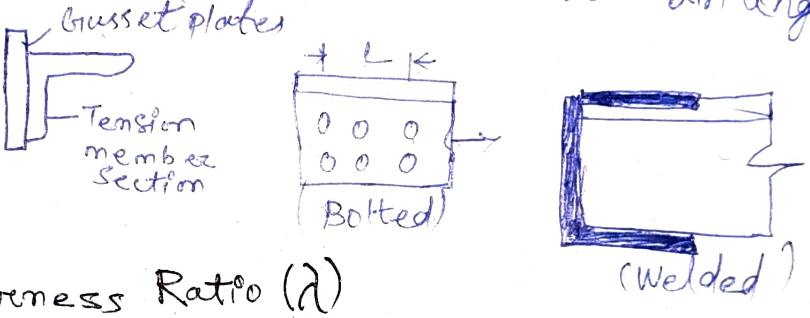
- This effect is considered when all elements of a member are not directly connected at a joint.
- When the force transfer takes place between a tension member (angle, channels, T section, I-Section) and gusset or adjacent member connected to one of the legs or to the flanges of I-Section either by bolting and welding, the tensile stress in the section from the 1st bolt hole upto the last bolt hole or from the start of the weld to the end of the weld will not be uniform.
- The connected legs or flange will have higher stresses at failure of ultimate stress and the outstanding legs stress may be below yield stress.
- Hence the internal transfer of forces from one leg in case of angles or from the flanges to web in case of I-sections.
- So that only at a distance away from the connecti[on] the uniform stress throughout the section, thus one part of the lags behind the other as far as internal stress development is concerned and the phenomenon is known as shear lag.
- Since the shear lag reduces the effectiveness of a component part of tension member, that are not directly to the gusset plates, the outstanding leg kept shorter in length, for this reason, In case of unequal angles the connection with long legs are preffered.
- The shear lag effect can be reduced by increasing the length of the end connection, because in longer connection a major portion of the load is distribute more or less uniform over the cross section and a smaller portion of load transfer in transistion zone.

- The shear lag canbe accounted for using reducti
factor $= 1 - \frac{\bar{x}}{L}$

where $\bar{x}$ = the distance from the face of the gusset plate to the centroid of the Connected area.
L = Length of Connection in the direction of load.
which is equal to the distance between outer most bolts in bolted Joint.



(Distribution of stress in angle)

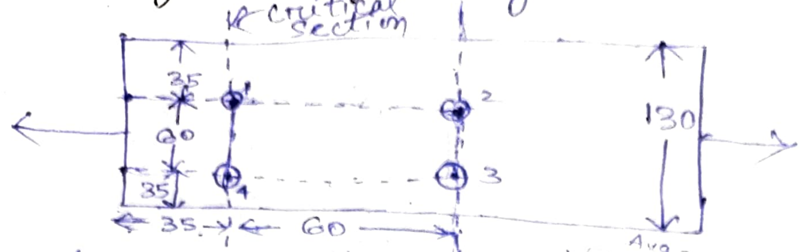


Slenderness Ratio (λ)

The effective slenderness ratio of a tension member is the ratio of its effective length (KL) to the least radius of gyration (r), where k is co-efficient depending on end condition.

—x—

1. Determine the design tensile strength of plate 130 mm × 12 mm with the holes for 16 mm diameter bolts as shown in fig. Steel used is Fe 415 grade quantity.



Given data :

Strength of plate is the least of
a. yielding of gross section
b. Rupture of critical section
c. Block shear strength

a. yielding of gross section :

$$T_{dg} = \frac{A_g f_y}{\gamma_{m0}} = \frac{130 \times 12 \times 250}{1.1} = 354545.45 \text{ N}$$
$$= 354.54 \text{ KN}$$

b. Rupture of critical section :

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

$A_n = (b - n d_0) \times t$

$d_0 = 16 + 2 = 18$ mm

$A_n = \{130 - (2 \times 18)\} \times 12$

$A_n = 1128 \text{ mm}^2$

$$T_{dn} = \frac{0.9 \times 1128 \times 410}{1.25} = 332985.6 \text{ N} = 332.98 \text{ KN}$$

c. Block shear strength :

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

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

where A_{vg} = shear gross area / minimum gross shear area
A_{tn} = Minimum net area in tension
A_{vn} = Minimum net shear area
A_{tg} = Minimum gross area on tension

$A_{vg} = (35+60) \times 12 \times 2$ — shear gross area for 1-2 & 3-4

$= 2280\ mm^2$

A_{tg} = tension gross area, 2-3

$= 60 \times 12 = 720\ mm^2$

$A_{vn} = \{(35+60) - (1.5 \times 18)\} \times 12 \times 2$

$A_{vn} = 1632\ mm^2$

$A_{tn} = (60-18) \times 12$

$= 504\ mm^2$

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

$T_{db} = \dfrac{2280 \times 250}{\sqrt{3} \times 1.1} + \dfrac{0.9 \times 504 \times 410}{1.25}$

$T_{db} = 447953.21\ N$

$= 447.953\ kN$

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

$T_{db} = \dfrac{0.9 \times 1632 \times 410}{\sqrt{3} \times 1.25} + \dfrac{720 \times 250}{1.1}$

$T_{db} = 441784.324\ N$

$= 441.784\ kN$

Block shear strength 441.784 kN.

∴ Hence strength of plate 332.98 kN.

2. A tension member consists of a flat 100 mm x 8 mm which is connected to a gusset plate 10 mm thick by 2 nos. of 16 mm dia. bolts, If steel grade Fe 410 & bearing bolts property 4.6 are used in workshop. Determine strength of flat against yielding, Rupture and Block shear. Also determine the maximum load the Joint can carry safely.

Given data :-

plate size = 100 x 8 mm

thickness of plate(t) = ~~10~~ 8 mm

Bolts dia (d) = 16 mm

Dia. of hole (d_0) = 16 + 2 = 18 mm

steel grade ~~4.6~~ fe 410

ultimate strength of plate (F_u) = 410 N/mm²

Bearing bolts grade 4.6

ultimate tensile strength of bolt (F_{ub}) = 400 N/mm²

yield strength of bolt (F_{yb}) = 0.6 x 4 x 100 = 240 N/mm²

work shop bolt (γ_{m2}) = 1.25

Strength of plate :-

least of the following.

yielding :- $T_{dg} = \frac{A_g f_y}{\gamma_{m0}} = \frac{100 \times 8 \times 250}{1.1} = 181818.18\ N$

$\simeq 181.81\ KN$

Rupture strength of plate :-

$T_{dn} = \frac{0.9 A_n F_u}{\gamma_{m2}}$, A_n = no. of critical section of bolt hole

$A_n = (b - n d_0) \times t$

$A_n = (100 - \cancel{2}1 \times 18) \times \cancel{10}8$

$A_n = 656\ mm^2$

$T_{dn} = \frac{0.9 \times 656 \times 410}{1.25} = 193651.2\ N$

$= 193.65\ KN$

Block shear strength :-

• $A_{vg} = \overset{L\times t}{(30+40)\times 8}$

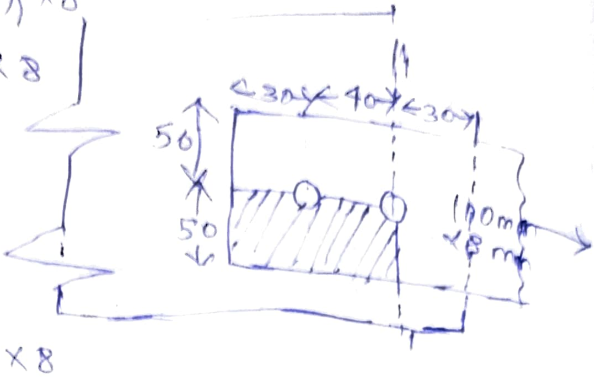
$= 70\times 8$

$A_{vg} = 560\ mm^2$

• $A_{vn} = \{(30+40)-(1.5\times 18)\}\times 8$

$= \{70-(1.5\times 18)\}\times 8$

$A_{vn} = 344\ mm^2$



• $A_{tg} = 50\times 8$

$A_{tg} = 400\ mm^2$

• $A_{tn} = \left(50 - \frac{1}{2}\times \overset{9}{18}\right)\times 8$

$= (50-9)\times 8$

$= 41\times 8$

$A_{tn} = 328\ mm^2$

$$T_{db_1} = \frac{0.9\, A_{vn}\, f_u}{\sqrt{3}\times \gamma_{ml}} + \frac{A_{tg}\, f_y}{\gamma_{m0}}$$

$$= \frac{0.9\times 344\times 410}{\sqrt{3}\times 1.25} + \frac{400\times 250}{1.1}$$

$= 149538.3179\ N$

$= 149.538\ KN$

$$T_{db_2} = \cancel{\frac{0.9\, A_{tg}\, f_u}{\sqrt{3}\times \gamma_{ml}}} + \frac{A_{vg}\, f_y}{\sqrt{3}\, \gamma_{m0}} + \frac{0.9\, A_{tn}\, f_u}{\gamma_{ml}}$$

$$= \frac{560\times 250}{\sqrt{3}\times 1.1} + \frac{0.9\times 328\times 410}{1.25}$$

$= 170306.54\ N$

$= 170.306\ KN$

$T_{db_2} = 170.306\ KN$

Block shear strength $= 149.54\ KN$

$\therefore$ Hence strength of plate $= 149.54\ KN$

strength of bolt :

shear strength of bolt :

$A_{sb} = \frac{\pi}{4} \times 16^2 = 201.06 \, mm^2$

lap Joint, so $n_n = 1$, $n_s = 0$, $A_{nb} = 0.78 \times \frac{\pi}{4} \times 16^2 = 156.82 \, mm^2$

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

$$= \frac{400 \times (1 \times 156.82) + (1 \times \cancel{201.06} \, 0)}{\sqrt{3} \times 1.25} \quad 1.25$$

$$V_{ds} = \frac{400 \times (156.82 + \cancel{201.06} \, 0)}{\sqrt{3} \times 1.25}$$

$V_{dsb} = \cancel{66119.07 \, N}$ $\frac{62728}{\sqrt{3} \times 1.25} = 28.972$ KN

$= \cancel{66.11 \, KN}$ $= 29.01$ KN

Bearing strength of bolt :

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

- K_b is less of
- $\frac{e}{3d_0} = \frac{30}{3 \times 18} = 0.555$
- $\frac{P}{3d_0} - 0.25 = \frac{40}{3 \times 18} - 0.25 = 0.4907$
- $\frac{F_{ub}}{F_u} = \frac{400}{410} = 0.9756$
- 1.0

$\therefore$ K_b is 0.4907

$$V_{dpb} = \frac{2.5 \times 0.4907 \times 16 \times 8 \times 410}{1.25}$$

$= 51503.872$ N For 1 bolt

$\simeq 51.503$ KN $= 51.54$ KN

~~For 2 bolt = 2 × 51503.872 N~~

~~= 103007.744 N~~

~~≃ 103.00 KN~~

Strength of Bolt/ Bolt Value = 29.01 KN

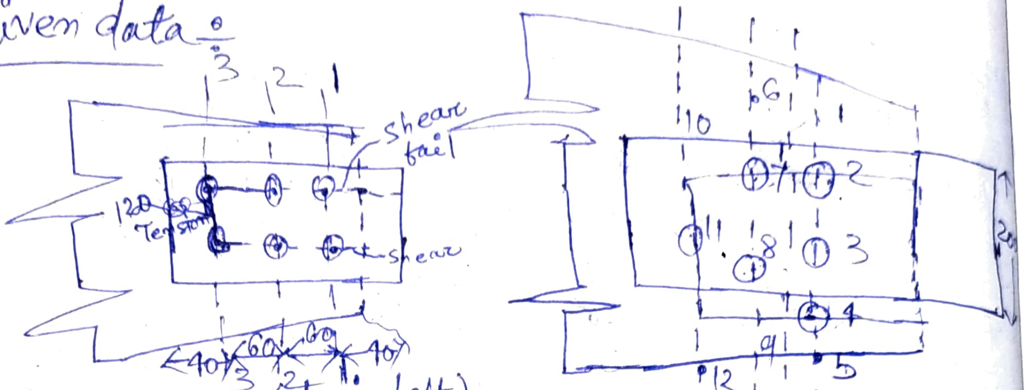
No. of Bolts in the Joint = 2 × 29.01 KN = 58.02 KN

Max^m load carry can safely = less of strength of plate and strength of bolts

$\therefore$ So load can be carry safely 58.02 KN.

2. A flat size 200 mm × 10 mm of grade Fe 410 used as a tension member is connected to a 12 mm gusset plate by 2 alternate methods of bolting as shown in fig. Calculate the strength in yielding, rupture and block shear.

Given data :



(chain bolt) (Diamond Bolting)

Size of plate = 200 mm × 10 mm

ultimate strength of plate (F_u) = 410 mpa

bolts grade = 4.6 F_y = 250 Mpa

ultimate strength of bolt = (F_{ub}) = 400 N/mm^2

yield strength of bolt = (F_{yb}) = 240 N/mm^2

dia. of bolt (d) = 20 mm

dia. of bolt hole (d_0) = 22 mm

Shop welds (γ_{ml}) = 1.25

$\gamma_{mo} = 1.1$

a. Chain Bolting :

i. Strength of bolt in yielding gross Section

$$T_{dg} = \frac{A_g F_y}{\gamma_{mo}} = \frac{200 \times 10 \times 250}{1.1} = 454.54 \text{ KN}$$

ii. Strength of plate against rupture of critical strength

$$T_{dn} = \frac{0.9\, F_u\, A_n}{\gamma_{ml}} \qquad A_n = (b - n d_o) \times t$$

n = no. of bolts at critical section (2)

$A_n = (200 - 2 \times 22) \times 10$
$= 1560\ mm^2$

$$T_{dn} = \frac{0.9 \times 410 \times 1560}{1.25}$$

- Block shear :-

$A_{vg} = L \times t$
$= \{\cancel{(60+40+60+40) \times 10}\} \times 2$
$= 2 \times (\overset{L}{60} + 60 + \overset{\times t}{40}) \times 10$
$= 3200\ mm^2$

$A_{vn} = 2 \times (160 - 2.5 \times 22) \times 10$
$= 2100\ mm^2$

$A_{tg} = 120 \times 10 = 1200\ mm^2$

$A_{tn} = (120 - 1 \times 22) \times 10$
$A_{tn} = 980\ mm^2$

$$T_{db_1} = \frac{0.9\, A_{vn}\, F_u}{\sqrt{3}\, \gamma_{ml}} + \frac{A_{tg}\, f_y}{\gamma_{mo}}$$

$$= \frac{0.9 \times 2100 \times 410}{\sqrt{3} \times 1.25} + \frac{1200 \times 250}{1.1}$$

$= 630.638\ kN$

$$T_{db2} = \frac{0.9\, A_{tn} f_u}{\gamma_{ml}} + \frac{A_{vg} f_y}{\sqrt{3}\,\gamma_{mo}}$$

$$= \frac{0.9 \times 980 \times 410}{1.25} + \frac{3200 \times 250}{\sqrt{3} \times 1.1}$$

$$= 709.187$$

∴ Block shear Strength = 630.638 KN

Diamond bolting :-

i. strength of bolt against yielding gross Secti

$$T_{dg} = \frac{A_g f_y}{\gamma_{mo}} = \frac{200 \times 10 \times 250}{1.1}$$

$$T_{dg} = 454.54 \text{ KN}$$

ii. Strength of bolt against rupture strength :-

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

A_n = net effective area

Along critical line 1-2-3-4-5

$$A_n = (b - n d_o) \times t$$

$$= (200 - 3 \times 22) \times 10$$

$$= 1340 \text{ mm}^2$$

Along 1-2-7-3-8-4-5,

$$A_n = \cancel{(b - n d_o) \times t} \left[b - n d_o + \sum \frac{P_{si}^2}{4 g_i} \right] \times t \quad \text{where } P_s = 60, \; g = 3, \; i = 4, \; n =$$

$$\cancel{(200 - 5 \times 22}$$

$$= \left[(200 - 5 \times 22) + \frac{P_{si4}^2}{4 g_{i2}} + \frac{P_{si3}^2}{4 g_{i3}^2} \right] \times t$$

$$A_n = \left[200 - 5 \times 22 + 4 \times \frac{60^2}{4 \times 30} \right] \times 10$$

$$A_n = 2100 \text{ mm}^2$$

Along 1-2-7-8-4-5

$A_n = \left[(b - n d_0) + \Sigma \frac{P_{si}^2}{4 g_i}\right] \times t$

$i = 2$, $P_s = 60$, $g = 30$, $n = 4$

$= \left[200 - 4 \times 22 + 2 \times \frac{60^2}{4 \times 30}\right] \times 10$

$= 1720\ mm^2$

Along 1-2-7-11-8-4-5

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

$i = 4$, $P_s = 60$, $n = 5$, $g = 30$

$A_n = \left[(200 - 5 \times 22) + 4 \times \frac{60^2}{4 \times 30}\right] \times 10$

$A_n = 2100\ mm^2$

Along 6-7-11-8-9

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

$i = 2$, $P_s = 60$, $n = 3$, $g = 30$

$= \left[200 - 3 \times 22 + 2 \times \frac{60^2}{4 \times 30}\right] \times 10$

$= 1940\ mm^2$

least of all the critical section is line 1-2-3-4-5, $A_n = 1340\ mm^2$

Rupture strength $= \frac{0.9\, F_u A_n}{\gamma_{m1}}$

$= \frac{0.9 \times 410 \times 1340}{1.25}$

$= 395.568$ KN

$T_{dn} = 395.57$ KN

iii. strength of plate against block shear:

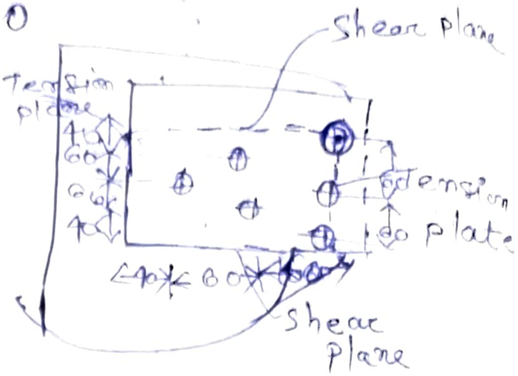
$A_{vg} = 2\times(2\times 60+40)\times 10$

$= 3200\ mm^2$

$A_{tg} = 120\times 10$

$= 1200\ mm^2$

~~Atn~~ $A_{vn} =$



~~Atn~~ $A_{vg} = 2\times(40+60+60)\times 10$

$= 3200\ mm^2$

$A_{vn} = 2\times(40+60+60) - \frac{22}{2})\times 10$

$= 2\times(40+60+60-11)\times 10$

$= 2980\ mm^2$

$A_{tg} = \overset{L\times t}{(60+60)\times 10}$

$= 120\times 10 = 1200\ mm^2$

$A_{tn} = \{120-(2\times 22)\}\times 10$

$= (120-44)\times 10 = 760\ mm^2$

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

$$= \frac{0.9\times 2980\times 410}{\sqrt{3}\times 1.25} + \frac{1200\times 250}{1.1}$$

$T_{db_1} = 780619.99N = 780.61\ KN$

$$T_{b_2} = \frac{0.9\,A_{tn} f_u}{\gamma_{m1}} + \frac{A_{vg} f_y}{\sqrt{3}\,\gamma_{m0}}$$

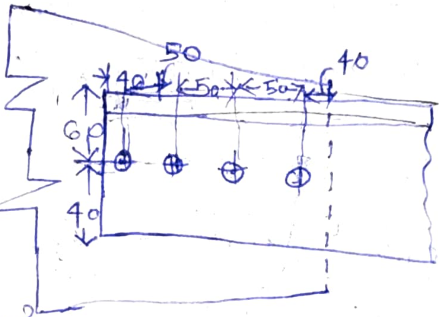
$$= \frac{0.9\times 760\times 410}{1.25} + \frac{3200\times 250}{\sqrt{3}\times 1.1}$$

$T_{db_2} = 644243.10\ N$

$T_{db_2} = 644.243\ KN$

∴ So block shear strength 644.243 KN

A single unequal angle ISA 100×75×8 mm is connected by longer leg to a 12 mm gusset plate at the ends with 4 nos. of 20 mm dia field bearing bolts of property class 4.6 to transfer the tension. Design the tensile strength of angle?



Given data:

For Fe410 plates of Steel

$F_u = 410\ N/mm^2$, $F_y = 250\ N/mm^2$

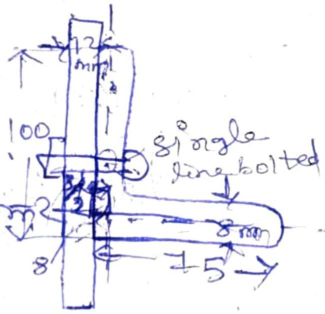
$\gamma_{mo} = 1.1$, $\gamma_{mb} = 1.25$

For 20 mm dia bolts property 4.6

$F_{ub} = 400\ N/mm^2$, $F_{yb} = 240\ N/mm^2$

dia of bolts $d = 20\ mm$

dia. of bolts hole $= 20+2 = 22\ mm$



- strength of angle:

i. Strength of angle against yielding of gross section:

For ISA 100×75×8 mm, $A_g = 1336\ mm^2$

$$T_{dg} = \frac{A_g f_y}{\gamma_{mo}} = \frac{1336 \times 250}{1.1} = 303636.36\ N$$

$$= 303.636\ KN$$

$$T_{dg} \simeq 303.64\ Kg$$

ii. Strength of angle against rupture of critical section:

Net area connected leg $(A_{nc}) = \left(100 - \frac{8}{2} - 22\right) \times 8$

$L \times t$

$(100 - \frac{8}{4} - 22) \times 8$

$A_{nc} = 592\ mm^2$

gross area of outstanding leg (A_{go})

$$= \left(75 - \frac{8}{2}\right) \times 8$$

$$A_{go} = 568\ mm^2$$

W = outstanding leg width = 75 mm

$W_f = 100 - 40 = 60$ mm , $W_i = 60$ mm (angle connected to both holes)

$b_s = W + W_f - t = 75 + 60 - 8 = 127$ mm

L_c = connected leg, (first bolt hole to last bolt hole)

$L_c = 3 \times 50 = 150$ mm

$$\beta = 1.4 - 0.076\,(W/t) \times (F_y/F_u)(b_s/L_c)$$

$$= 1.4 - 0.076\left(\frac{75}{8}\right) \times \left(\frac{250}{410}\right) \times \left(\frac{127}{150}\right)$$

$\beta = 1.0321$

$$\frac{F_u \gamma_{m0}}{F_y \gamma_{m1}} = \frac{410 \times 1.1}{250 \times 1.25} = 1.4432$$

0.7

Hence $\beta = 1.032$

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

$$T_{dn} = \frac{0.9 \times 592 \times 410}{1.25} + \frac{1.032 \times 568 \times 250}{1.1}$$

$T_{dn} = 307.980$ KN

iii. Strength of against block shear :-

$A_{vg} = L \times t$

$A_{vg} = (50 + 50 + 50 + 40) \times 8$

$A_{vg} = 1520$ mm^2

$A_{vn} = (50 + 50 + 50 + 40) - (3.5 \times 22) \times 8$

$= (190 - 3.5 \times 22) \times 8$

$= 904$ mm^2

$A_{tg} = L \times t$

$A_{tg} = 40 \times 8$

$A_{tg} = 320$ mm^2

$A_{tn} = \cancel{40\times}\left(40 - \frac{1}{2}\times 22\right)\times 8$
$= 232\ mm^2$

$$T_{db1} = \frac{0.9\ A_{vn} F_u}{\sqrt{3}\ \gamma_{m1}} + \frac{A_{tg} F_y}{\gamma_{m0}}$$

$$= \frac{0.9\times 904\times 410}{\sqrt{3}\times 1.25} + \frac{320\times 250}{1.1}$$

$T_{d2} = 226.79$ KN

$$T_{db2} = \frac{0.9\ A_{tn} F_u}{\gamma_{m1}} + \frac{A_{vg} F_y}{\sqrt{3}\times \gamma_{m0}}$$

$$= \frac{0.9\times 232\times 410}{1.25} + \frac{1520\times 250}{\sqrt{3}\times 1.1}$$

$T_{d2} = 267.93$ KN

block shear strength = less of T_{db1} & T_{db2}

∴ block shear strength = 226.79 KN

∴ Tensile strength of angle = 226.79 KN

Strength of bolts :

Strength of bolt :

i. Shear strength of bolt :

Assuming shear plane at threads $n_n = 1$, $n_s = 1$

$A_{nb} = 0.78\times \frac{\pi}{4}\times 20^2 = 245.04\ mm^2$

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

$$= \frac{400\times(1\times 245.04)}{\sqrt{3}\times 1.25} = \frac{400\times 245.04}{\sqrt{3}\times 1.25}$$

$= 45.27$ KN

∴ shearing strength of bolts
45.27 KN

Bearing strength of bolt :-

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

K_b is the less of

- $\frac{e}{3d_0} = \frac{40}{3\times22} = 0.6060$
- $\frac{P}{3d_0} - 0.25 = \frac{50}{3\times22} - 0.25 = 0.507$
- $\frac{F_{ub}}{F_u} = \frac{400}{410} = 0.9456$
- 1.0

∴ K_b value is 0.507

$$V_{dpb} = \frac{2.5\times0.507\times20\times8\times410}{1.25} = 66.51$$

No. of bolts in the Joint = 4

Bolt Value :- less of V_{dsb} & V_{dpb}

Bolt Value = 45.26 KN

Strength of Bolt in the Joint = 4×45.26
= 181.04 KN

Design tensile strength = minimum of strength of angle or strength of bolts

∴ Design tensile strength = 181.04 KN.

Design Determine the tensile strength of a bridge truss diagonal 2 ISA 100 75, 8 mm connected to a gusset plate of 10 mm thick by 5 mm size fillet by longer weld. The effective length of weld 180 mm and is provided on either end of the connected leg. Assume steel of grade Fe 410.

Solution :

Given data :

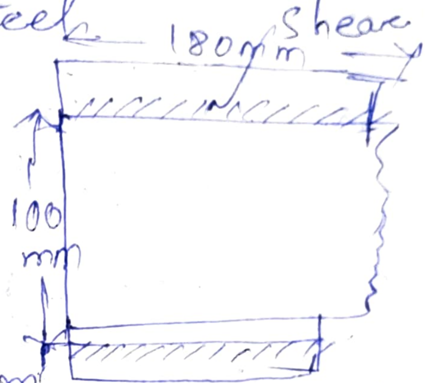
For Fe 410 grade steel

$F_u = 410\ N/mm^2$

$F_y = 250\ N/mm^2$

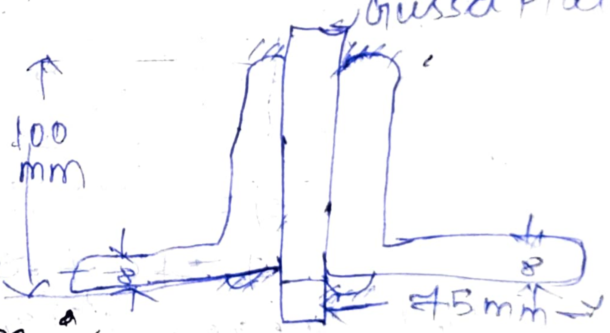
$\gamma_{m0} = 1.1$

$\gamma_{m1} = 1.25$



For 2 ISA 100, 75, 8 mm

$A_g = 2 \times 1336\ mm^2$
$= 2672\ mm^2$



i. Design strength of yielding gross section :

$T_{dg} = \frac{A_g f_y}{\gamma_{m0}} = \frac{2672 \times 250}{1.1} = 607273\ N$
$= 607.27\ KN$

ii. Design strength due to rupture of net section :

Here w = outstanding leg width = 75 mm

b_s = shear lag width = $(b_s = w) = 75$ mm

L_c = length of weld along load direction (welded connection)

$L_c = 180$ mm

$\beta = 1.4 - 0.076 \times \frac{w}{t} \times \frac{f_y}{f_u} \times \frac{b_s}{L_c}$

$$\beta = 1.4 - 0.076 \times \frac{75}{8} \times \frac{250}{410} \times \frac{75}{180}$$

$\beta = 1.218 > 0.7$

$$\frac{F_u}{F_y} \times \frac{\gamma_{m0}}{\gamma_{ml}} = \frac{410}{250} \times \frac{1.1}{1.25} = 1.4432$$

$\beta = 1.2182$

A_{nc} = net area of the connected length

$$= 2 \times \left(100 - \frac{8}{2}\right) \times 8 \quad (L \times t)$$

$A_{nc} = 1536 \text{ mm}^2$

A_{go} = gross area of the outstanding le

$$= 2 \times \left(75 - \frac{8}{2}\right) \times 8 \quad (L \times t)$$

$= 1136 \text{ mm}^2$

$$T_{dn} = \frac{0.9 F_u A_{nc}}{\gamma_{ml}} + \frac{\beta A_{go} F_y}{\gamma_{m0}}$$

$$= \frac{0.9 \times 410 \times 1536}{1.25} + \frac{1.218 \times 1136 \times 250}{1.1}$$

$= 767892.65 \text{ N}$

$= 767.89 \text{ KN}$

iii. Block shear strength of the member (T_{db})

The gusset fail by block shear & shaded portion may shear of

$A_{vg} = A_{vn} = 2 \times 180 \times 8 \quad (L \times t)$ angle width ~~Gusset plate width~~

$\therefore A_{vg} = A_{vn} = 2800 \text{ mm}^2$

$A_{tg} = A_{tn} = 100 \times 8 \quad (L \times t)$

$= 800 \text{ mm}^2$

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

$$= \frac{0.9 \times 2800 \times 410}{\sqrt{3} \times 1.25} + \frac{800 \times 250}{1.1}$$

$$= \cancel{229143.342}\ N$$

$$T_{db1} \simeq \cancel{229.143}\ KN$$
$$659.032$$

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

$$= \frac{0.9 \times 800 \times 410}{1.25} + \frac{2800 \times 250}{\sqrt{3} \times 1.1}$$

$$T_{db2} = \cancel{814815.24}\ N \quad 603.56\ KN$$

$$= \cancel{814.815}\ KN$$

Block shear strength = less of T_{db1} & T_{db2}

∴ Block shear strength = ~~229.143~~ KN

Design tensile strength of member = less of T_{dg}, T_{dn}, T_{db} = ~~229.143~~ KN

Design of Tension members Subjected to Axial load:

1. The gross area (Ag) required to carry the factored load Tu from consideration of yielding is given by, $A_g = \frac{T_u}{f_y/\gamma_{m0}}$

or The net area 'A_n' required to carry the factored load Tu is obtained by

$A_n = \frac{T_u}{0.9 f_u / \gamma_{m1}}$ plates or by $\frac{T_u}{\alpha f_u/\gamma_{m1}}$ (angle)

where α varies between 0.6 to 0.8 as appropriate.

- The net area calculated may be suitably increased (25% to 40%) to arrive at gross section area.

2. Depending upon the type of structure and the location of the member, suitable shape and the area of the section is selected from steel table or IS Hand book.

3. The connection is designed by calculating the no. of bolts or the length of weld required which is suitable arranged as per requirement.

4. The design strength Td of the trail section consider minimum of T_{dg}, T_{dn} & T_{db}, which should be more than factored load.

5. It is too much higher side or unsafe, the section may be suitably revised.

G. The effective slenderness ratio of the member checked which should satisfy I.S. Specification

Q. Design a double angle tension member connected on each side of 10 mm thick gusset plate to carry an axial factored load of 480 KN. If the connection is made by a. shop connection of bearing bolts of property class 4.6.

b. Shop fillet welding on three sides.

Effective length of the member = 7.5 m. and the member is to be always under tension. Assume steel grade of Fe 410. ~~Effective length~~ ~~of~~ the member is to be always under tension.

Given data :-

For Fe 410 steel grade,

$F_u = 410\ N/mm^2$, $F_y = 250\ N/mm^2$, $\gamma_{mo} = 1.1$,

$\gamma_{mb} = 1.25$, $T_u = 480 \times 10^3\ N$

i. Calculate Sectional area (A_g)

$$A_g = \frac{T_u}{F_y/\gamma_{mo}} = \frac{480 \times 10^3}{250/1.1} = 2112\ mm^2$$

Let us adopt 2 ISA 100 x 100 x 8 mm

$A_g = 2 \times 1167 = 2334\ mm^2$

Shop connection bearing bolts :-

Design of bolted connection.

For unwin's formula, $d = 6\sqrt{t}$, t = gusset plate

$d = 6\sqrt{10}$

$= 18.97\ mm$

$d = 20\ mm$

dia. of hole $(d_0) = 20+2 = 22$ mm, bolt grade $= 4.6$

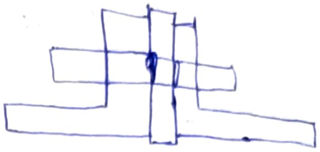
$\gamma_{mb} = 1.25$, $F_{ub} = 400$ N/mm², $F_{yb} = 240$ N/mm²

$A_{nb} = 0.78 \times \frac{\pi}{4} \times d^2 = 0.78 \times \frac{\pi}{4} \times 20^2 = 245.04$ mm²

$A_{sb} = \frac{\pi}{4} \times d^2 = \frac{\pi}{4} \times 20^2 = 314.15$ mm²

The bolts will be double shear intercepted thread as well as shank

$n_n = 1, n_s = 1$



Shear strength of bolt :

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{\frac{F_{ub}}{\sqrt{3}} \times (n_n A_{nb} + n_s A_{sb})}{1.25}$$

$$V_{dsb} = \frac{400 \times (1 \times 245.04) + (1 \times 314.15)}{\sqrt{3} \times 1.25}$$

$V_{dsb} = 103.311$ KN

Bearing strength of bolt :

$$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}} = \frac{2.5 K_b d t F_u}{1.25}$$

K_b is the less of,

- $\frac{e}{3d_0} = \frac{40}{3 \times 22} = 0.6060$
- $\frac{P}{3d_0} - 0.25 \quad \frac{60}{3 \times 22} - 0.25 = 0.6590$
- $\frac{F_{ub}}{F_u} = \frac{400}{410} = 0.9756$
- 1.0

$K_b = 0.6060$

$$V_{dpb} = \frac{2.5 \times 0.6060 \times 20 \times 10 \times 410}{1.25} = 99.384 \text{ KN}$$

Bolt Value = less of shear strength / bearing strength

Bolt Value = 99.384 KN

No. of bolt = $\frac{\text{Apply load}}{\text{Bolt Value}} = \frac{480}{99.384} = 4.82$
$\simeq 5$ nos.

Hence 5 no. of 20 mm dia bearing bolts.

So that length of Joint (l_c/l_j) = 4 × 60
= 240 mm

$l_c < 15d$

= 240 < 15 × 20

= 240 < 300

∴ OK

• Design tensile strength of member :

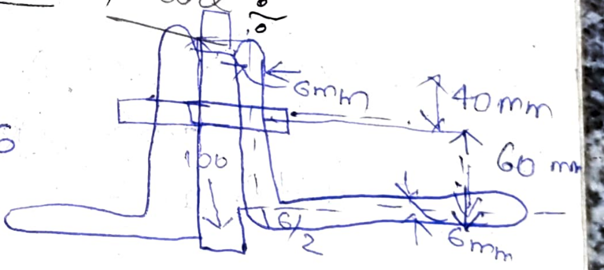
i. Design yielding gross section : (angle)

$T_{dg} = \frac{A_g F_y}{\gamma_{m0}} = \frac{2334 \times 250}{1.1} = 530.45$ KN

∴ 530.45 KN > 480 KN

∴ OK

ii. Strength ~~of~~ against rupture :



$A_{nc} = \cancel{100 - \frac{1 \times 22 - 6}{2}}$

$A_{nc} = (100 - 3 - 22) \times 6$

$A_{nc} = 450$ mm²

$A_{go} = (100 - 3) \times 6$
$= 97 \times 6 = 582$ mm²

$W = 100$ mm

$b_s = w + w_1 - t$

$= 100 + 60 - 6$

$= 160 - 6$

$b_s = 154$ mm

$L_c = 4 \times 60 = 240$ mm

$\beta = 1.4 - 0.076 \times \left(\frac{w}{t}\right) \times \left(\frac{F_y}{F_u}\right) \times \frac{b_s}{L_c} > 0.7$

$= 1.4 - 0.076 \times \left(\frac{100}{6}\right) \times \left(\frac{250}{410}\right) \times \left(\frac{154}{240}\right)$

$= \cancel{1.36}$ 0.904 > 0.7

$\frac{Fu}{Fy} \times \frac{\gamma_{m0}}{\gamma_{ml}} = \frac{410}{250} \times \frac{1.1}{1.25} = 1.4432.$

Hence $\beta = 0.904$

$T_{dn} = \frac{0.9\, F_u\, A_{nc}}{\gamma_{ml}} + \frac{\beta\, A_{go}\, F_y}{\gamma_{m0}}$

$T_{dn} = \frac{0.9 \times 410 \times 450}{1.25} + \frac{0.904 \times 582 \times 250}{1.1}$

$T_{dn} = 252.414$ KN

Rupture strength of double angle $= 2 \times 252.414$ KN

$= 504.829$ KN

$T_{dn} = 504.82 > 480$ KN

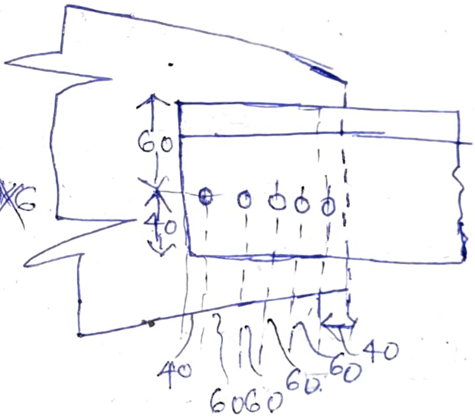
$\therefore$ ok

iii. Strength of angle in block shear :-

$A_{vg} = 2 \times (60+60+60+60+40) \times 6$ ($L \times t$)

$= 580 \times 6$

$A_{vg} = 3360 \text{ mm}^2$



$A_{vn} = 2 \times (60+60+60+60+40)$

$- 4.5 \times 22) \times 6$

$= 2172 \text{ mm}^2$

$A_{tg} = 2 \times (40) \times 6$

$= 80 \times 6$

$A_{tg} = 480 \text{ mm}^2$

$A_{tn} = 2 \times \{(40 - 0.5 \times 22) \times 6\}$

$= 2 \times 174$

$= 348 \text{ mm}^2$

$T_{db1} = \frac{0.9\, A_{vn} F_u}{\sqrt{3}\, \gamma_{ml}} + \frac{A_{tg}\, F_y}{\gamma_{m0}}$

$= \frac{0.9 \times 2172 \times 410}{\sqrt{3} \times 1.25} + \frac{480 \times 250}{1.1} = 479.273$ KN

$T_{db2} = \frac{0.9\ A_{tn} F_u}{\gamma_{m1}} + \frac{A_{vg} F_y}{\sqrt{3} \times \gamma_{m0}}$

$= \frac{0.9 \times 348 \times 410}{1.25} + \frac{3369 \times 250}{\sqrt{3} \times 1.1}$

$T_{db2} = 543.615$ KN > 480 KN ok

Block shear strength = less of T_{db1} & T_{db2}

∴ Block shear strength = 479.273 KN $\ngtr$ 480 KN ∴ not ok

≈ 481 KN > 480 KN

∴ Hence it is ok.

- Check for slenderness ratio :-

Slederness ratio :- (λ) $r_{xx} = 30.9$ mm, less

$r_{yy} = 44.3$ mm

$\lambda = \frac{KL}{R} = \frac{\text{effective length}}{\text{radius of gyration}} = \frac{7.5 \times 10^3}{30.9}$

$= 242.71 < 400$

∴ Hence it is ok

- Welding :-

- For shop welding = $\gamma_{mb} = 1.25$
- Factored load for each angle $= \frac{480}{2} = 240$ KN

- Size of weld :-

Minimum size for 10mm gusset plate = 5mm

Maximum size for rounded toe $= t \times \frac{3}{4}$

$= \cancel{6}^{3} \times \frac{3}{\cancel{4}_{2}}$

$= 4.5$ mm

Maximum size for square edge $= t - 1.5$

$= 6 - 1.5$

$= 4.5$ mm

∴ Hence size of weld = 5mm.

throat thickness (t) = KS

where K = 90° so K = 0.7

$t = 0.7 \times 5 = 3.5$ mm

Design strength of the weld $= \frac{l_w \times t \times F_u}{\sqrt{3} \times \gamma_{mw}}$

$= \frac{l_w \times 3.5 \times 410}{\sqrt{3} \times 1.25} = 662.79\ l_w$

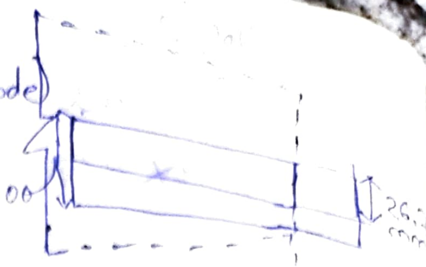
$240 \times 10^3 = 662.79\ l_w$ $l_w = 240 \times 10^3 / 662.79$

$l_w = 3$[illegible]

$l_w = 362.10$ mm

$l_w = 100 + l_1 + l_2$

$C_{xx} = 26.7$ mm (see from steel code)



$l_1 \times 26.7 = l_2(100-26.7)$

$\Rightarrow l_1 = \frac{l_2 \times (100-26.7)}{26.7}$

$l_1 = l_2 2.74$

$l_1 = 2.74 l_2$

$l_1 + l_2 = l_w$

$\Rightarrow 2.74 l_2 + l_2 = 362.10$

$\Rightarrow 3.74 l_2 = 362.10$

$l_2 = \frac{362.10}{3.74} = 96.818$ mm

$l_2 = 96.818$ mm

$\cancel{l_1 = 2.74 l_2} = \cancel{2.74 \times 96.818}$

$l_w = l_1 + l_2$

$362.10 = l_1 + 96.818$

$l_1 = 265.282$ mm

1. A bridge truss diagonal of length ... is subjected to a factored tensile load of 350 KN. It is connected to a gusset plate of 12mm thick by high strength of bolt of property 8.8. Design the member if grade of steel is Fe 410 for the following cases. Assume bolts clearance holes & slip factor 0.7
i. when slip permitted ii. No slip is permitted.

Given data:

For steel grade Fe 410

$F_u = 410$ Mpa, $\gamma_{mb} = 1.25$, $\gamma_{ml} = 1.25$,

$\gamma_{mo} = 1.1$ Bolts property class 8.8

Gusset plate thick = 250 N

Factored load = 350 KN, effective length = 3.25m.

Calculation of Sectional area required:

$$A_g = \frac{T_{dg}}{F_y/\gamma_{mo}} = \frac{350\times10^3}{(250/1.1)} = \cancel{1272.727}\ 1540\ mm^2$$

So let us adopt an angle Section 150×75×8 mm

So, $A_g = 1742\ mm^2$

1. when slip is permitted

Dia. of bolt by unwin's formula, $d = 6\sqrt{t} = 6\sqrt{8}$
$= 16.97 \simeq 17 \simeq 18\ mm$.

Let us adopted 18mm bolts of property class 8.8

$d_o = d+2 = 18+2 = 20mm$, $\gamma_{mb} = 1.25$

$F_{ub} = 800\ N/mm^2$, $A_{nb} = 0.78\times\frac{\pi}{4}\times\cancel{18}^2 = \cancel{245.04}\ 198.485\ mm^2$

$A_{sb} = \frac{\pi}{4}\times d_o^2 = \frac{\pi}{4}\times\cancel{18}^2 = \cancel{314.159}\ 254.469\ mm^2$

Shearing strength:

The bolts will single shear, $n_n = 1$, $n_s = 0$

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{\frac{F_{ub}}{\sqrt{3}}(n_n A_{nb} + n_s A_{sb})}{\gamma_{mb}} = \frac{\frac{800}{\sqrt{3}}\times(1\times198.48)}{1.25}$$

$V_{dsb} = \cancel{73.33}$ KN
73.33 KN

Bearing strength :-

$e = 1.5\,d_0 = 1.5 \times 20 = 30$ mm

$P = 2.5\,d_0 = 2.5 \times 18 = 45$ mm

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

K_b is the less of,

- $\frac{e}{3d_0} = \frac{30}{3\times20} = 0.5$
- $\frac{P}{3d_0} - 0.25 = \frac{45}{3\times20} - 0.25 = 0.5$
- $\frac{F_{ub}}{F_u} = \frac{800}{410} = 1.95$
- $1.0 = 1.0$ $\quad K_b = 0.5$

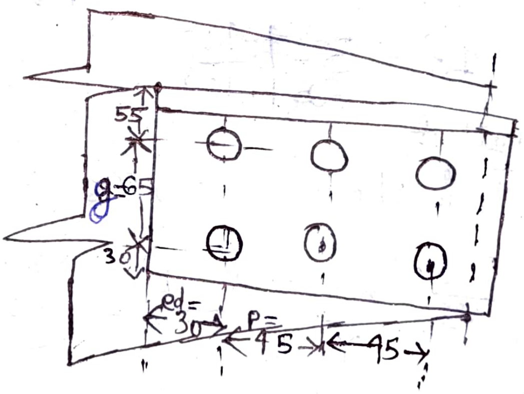
$$V_{dpb} = \frac{2.5\times0.5\times18\times8\times410}{1.25} = 59.04\ \text{KN}$$

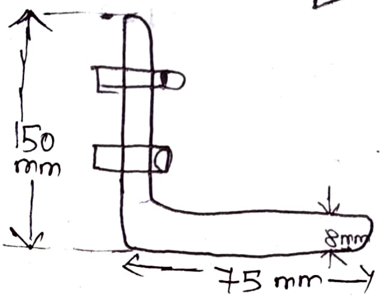
Bolt Value = ~~Minimum~~ less of shearing strength/bearing strength of Bolt

Bolt Value = 59.04 KN

$\therefore$ Hence No. of Bolts required $= \frac{350}{59.04} = 5.92 \simeq 6$ no.

Hence we provide 6 no. of 18 mm dia. bolts.





check for strength of the angle :

1. Strength against yielding gross Section :

$$T_{dg} = \frac{A_g F_y}{\gamma_{m0}} = \frac{1742 \times 250}{1.1} = 395.909 \text{ KN} > 350 \text{ KN}$$

$\therefore$ OK

2. Strength against rupture of critical Section :

$$A_{nc} = \left[150 - 2 \times 20 - \frac{8}{2}\right] \times 8$$

$A_{nc} = 848 \text{ mm}^2$

$$A_{go} = \left[75 - \frac{8}{2}\right] \times 8$$

$A_{go} = 568 \text{ mm}^2$

w = outstanding leg width $= 75$ mm, w_i = corner to 1st bolt hole = 55 mm

$b_s = w + w_i - t$

$= 75 + 55 - 8$

$b_s = 122$ mm

$L_c = 45 \times 2 = 90$ mm

(because 3 no. bolts)

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

$\beta = 1.4 - 0.076 \times \left(\frac{75}{8}\right) \times \left(\frac{250}{410}\right) \times \left(\frac{122}{90}\right)$

$\beta = 0.811 > 0.7$

- $\frac{F_u}{F_y} \times \frac{\gamma_{m0}}{\gamma_{m1}} = \frac{410}{250} \times \frac{1.1}{1.25} = 1.4432$

Hence $\beta = 0.811$

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

$$= \frac{0.9 \times 410 \times 848}{1.25} + \frac{0.811 \times 568 \times 250}{1.1}$$

$T_{dn} = 355.022$ KN > 350 KN $\therefore$ OK

3. Strength in block shear :

$A_{vg} = 45 + 45 + 30 = 120 \times 8 = 1600 \text{ mm}^2$

$$A_{vn} = \left[(45 + 45 + 30) - (3.5 \times 20)\right] \times 8$$

$A_{vn} = 400 \text{ mm}^2$

$A_{tg} = (65 + 30) \times 8 = 760 \text{ mm}^2$

$A_{tn} = \{(65 + 30) - (1 \times 20)\} \times 8 = (95 - 20) \times 8 = 75 \times$ [illegible]

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

$$= \frac{0.9 \times 400 \times 410}{\sqrt{3} \times 1.25} + \frac{760 \times 250}{1.1}$$

$$T_{db1} = 240.900 \text{ KN} < 350 \text{ KN}$$

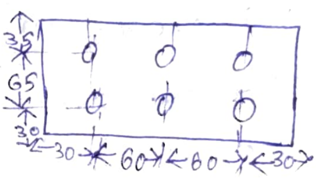
$$T_{db2} = \frac{0.9\, A_{tn} F_u}{\gamma_{m1}} + \frac{A_{vg} F_y}{\sqrt{3}\, \gamma_{m0}}$$

$$= \frac{0.9 \times 600 \times 410}{1.25} + \frac{1600 \times 250}{\sqrt{3} \times 1.1}$$

$$T_{db2} = 387.06 \text{ KN} > 350 \text{ KN}$$ ∴ Hence it is ok

Hence the design is unsafe. To increase resistance against block shear, let us increase the length of connection by increasing the pitch of bolts, Let $p = 60$ mm.

$$A_{vg} = (60+60+60+30) \times 8$$
$$= 1680 \text{ mm}^2$$



$$A_{vn} = \{(60+60+60+30) - (3.5 \times 20)\} \times 8$$
$$= 1280 \text{ mm}^2$$

$$A_{tg} = (65+30) \times 8 = 760 \text{ mm}^2$$

$$A_{tn} = \{(65+30) - (\tfrac{1}{2} \times 20)\} \times 8 = 680 \text{ mm}^2$$

$$T_{db1} = \frac{0.9 \times 1280 \times 410}{\sqrt{3} \times 1.25} + \frac{760 \times 250}{1.1}$$

$$= 390.882 \text{ KN} > 350 \text{ KN}$$ ∴ Hence ok

$$T_{db2} = \frac{0.9 \times 680 \times 410}{1.25} + \frac{1680 \times 250}{\sqrt{3} \times 1.1}$$

$$= 421.17 \text{ KN} > 350 \text{ KN}$$ ∴ Hence it is ok.

Check for slenderness ratio :-

For ISA 150×75×8 mm minimum radius of gyration (r_{min}) = 16 mm (see from steel code)

maximum slenderness ratio (λ) $= \frac{Kl}{r} = \frac{3250}{16} = 203.12 \not> 350$

∴ OK

ii. when slip is not permitted:

$$V_{dsf} = \frac{V_{nsf}}{\gamma_{mf}} = \frac{\mu_f \cdot n_e \cdot K_h \cdot F_o}{\gamma_{mf}}$$

μ_f = slip factor = 0.7

For $n_e = 1$ (for lap joint)

$K_h = 1.0$ for fastener clearance holes.

$\gamma_{mf} = 1.25$ for ultimate load

$A_{nb} = 0.78 \times \frac{\pi}{4} \times 18^2 = \cancel{2...}$ 198.48 mm^2

$F_o = 0.70\, f_{ub} \cdot A_{nb} = \cancel{137.224}$ $0.70 \times 800 \times 198.48$

$= 111148.8$

$$V_{dsf} = \frac{V_{nsf}}{\gamma_{mf}} = \frac{0.7 \times 1 \times 1.0 \times 111148.8}{1.25} = 62243.328\,N$$

$V_{dsf} = 62.243$ KN

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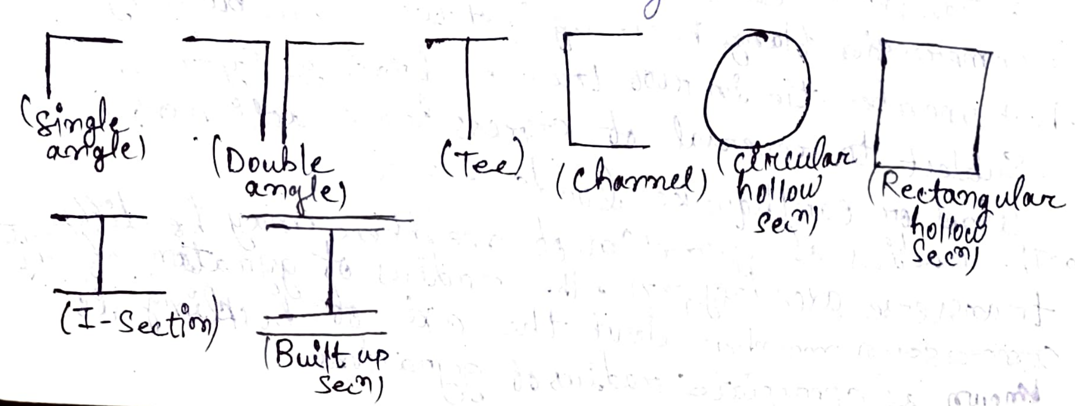
Chap-5th

Compression members

- Pure Compression members are structural elements subjected to axial compressive forces only. Axial compressive force means forces applied along the centroid of longitudinal axis of the cross section.
- A column is defined as the structural member whose longitudinal dimension is comparatively more than its lateral dimension subjected to compressive force in a direction parallel to its longitudinal axes.
- Endposts are the end compression member in truss bridge girders. The structural members carrying compressive load in a truss called struts. Heavy compression members in building are called stanchions. The compression member of a crane is called a boom. The compression members subject to both axial compression and bending.

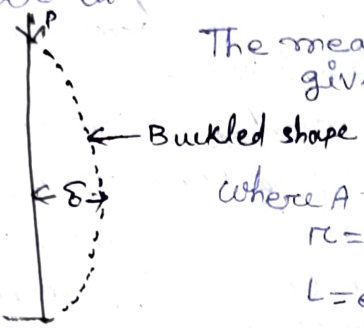
Common shapes of compression members :-

SL. NO.	Type of rolled steel section	uses
1.	Single angles	1. Light roof trusses, bracing in plate girders and built up columns.
2.	Double angle back to back	2. Top chrod member of roof truss.
3.	T- Section	3. Welded roof truss
4.	Single channels	4. Rarely used as Columns. Not Suitable due to low value of radius of gyration
5.	Circular hollow section	5. Tall building. Most efficent due to equal value of radius of gyration in every axis.
6.	Square & rectangular hollow section	6. Tall building. Easy fabrication and errection.
7.	I-Section	7. Suitable for columns. Difference in radius of gyration about two axes is smallest.
8.	Built-up section	8. Compression member subjected to heavy loads.



Buckling of Columns :

Buckling is defined as the sudden bending, warping or crumpling of the compression members under compression. Due to buckling developed in column occurs in a direction or plane normal to the direction of the loading. Buckling resistance depend on magnitude of applied load, bending stiffness of member, & length of member etc.



The mean compressive stress F_{cr} is given by,

$$F_{cr} = \frac{P_{cr}}{A} = \frac{\pi^2 E}{\lambda^2} = \frac{\pi^2 E}{L^2/r^2}$$

where A = Area of compression member

r = radius of gyration of compression member

L = effective length of compression member

λ = The slenderness ratio of column, l/r.

Effective length of a column and Appropiate radius of gyration :

- Effective length of a column is defined in terms of equivalent length hinged at both the ends for the various end conditions.
  - → Both the ends pin ended = $l_e = l$
  - → Both ends fixed = $l_e = l/2$
  - → one end fixed & other end pinned or hinged $l_e = \frac{l}{\sqrt{2}}$
  - → one end fixed and the other end free $l_e = 2l$.

  where l = actual length of column

- As per IS-800 2007, the effective length of KL, The actual length shall be taken as the length from centre to centre of its intersection with the supporting members in the plane of the buckling deformation.

1. A member carry compressive load from dead loads and imposed loads = 180
2. A member subjected to wind/earthquake action, provided the stress in structure = 250
3. compression flange beam against torsional buckling = 300
4. A member-tie in roof truss or bracing system subject to reversal of stress from action of wind or earthquake loads = 350

- The radius of gyration of member may be differ transverse axes (yy, zz). The radius of gyration of the compression member about the axis of buckling is known as appropiate radius of gyrations.

Slenderness ratio :- It is defined as the ratio of effective length to the corresponding radius of gyration.

Thus $\lambda = \frac{KL}{r} = \frac{L}{r}$.

Limiting thickness :- The compression member to satisfy the limiting width to the thickness ratio depending upon the class of sections.

Buckling class of cross sections :-

The minimum load, that causes collapse of the column section; IS-800 2007 recommends buckling of columns buckling curves a, b, c & d as. For ex :- If the ratio of overall height is to the overall width of the flange of rolled I section i.e. h/b greater than 1.2 and thickness of flange is less than 40 mm, the buckling class to axis y-y curve 'b'. It defines the buckling class with respect to limits of height to width ratio, thickness of flange and the axes about which the buckling takes of place.

Design compressive stress and strength of compression member :- Common rolled and built up section steel members used for carrying axial compression, usually fail by flextural buckling. The buckling strength of members affected by residual stresses, intial blow & accidental eccentricity of load.

- The design compressive strength of a member Pd is :-
  $P < P_d$ — where $P_d = A_e \cdot F_{cd}$ A_e = effective sectional area, F_{cd} = design compressive stress.
- Except class 4 slender section, the gross sectional area shall be as the effective sectional area for all compression members, fabricated by welding & bolting so as to long as the section is semi compact or better. Holes not fitted with rivets, bolts or pins shall be deducted from gross area to calculate effective sectional area.

Design compressive stress for columns :-

1. Initial imperfection or initial blow.
2. Eccentricity of application of loads.
3. Residual stress locked into the cross section.
4. Effect of a strain hardening and the absence of clearly defined yield point.
5. Effect of all features taken together.

• The design compressive strength of a member is given by

$$F_{cd} = \frac{f_y/\gamma_{m0}}{\phi + [\phi^2 - \lambda^2]^{0.5}} = \chi f_y/\gamma_{m0} \le f_y/\gamma_{m0}$$

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

λ = non dimensional effective slenderness ratio is given by $\sqrt{f_y/f_{cc}} = \sqrt{\left(\frac{Kl}{r}\right)^2 f_y/\pi^2 E}$

f_{cc} = Euler buckling stress $= \pi^2 E / (Kl/r)^2$

where F_{cd} = the design compressive stress

α = Imperfection factor

χ = stress reduction factor for different buckling class, slederness ratio and yield stress.

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

Imperfection Factor :

Buckling class	a	b	c	d
α	0.21	0.34	0.49	0.76

Design Compressive stress for angle struts :

• Single angle struts :

The compression in single angles may be transferred either concentrically to its centroid through end gusset or eccentrically by connecting one of its legs to a gusset or adjacent member.

• Double angle struts :

→ For double angle discontinuos struts, connected back to back, on opposite side of gusset or a section, by not less than two bolts in line along the angles at each end, by the equivalent in welding, the load may be regarded applied axially. The effective length KL, in the plane of end gusset shall be taken as between 0.7 and 0.85 times the distance between intersection.

→ Double angle discontinuos struts connected back to back to one side of gusset or section by one or more bolts in each angle, or by the equivalent in welding, the angles together satisfy requirements.

Continuos members :

- Double angle continuos struts such as those forming the flanges, chords or ties of trusses or trussed girders or the legs of towers shall be designed an axially loaded compression members.

Compression member in Trusses :

In the case of bolted or welded trusses and braced frames, the effective length KL, of the compression member shall be taken as 0.7 to 1.0 times the distance between the centres of connections, depending on the degree of end restraint provided. In case of members of trusses, buckling the plane perpendicular to the plane of truss. The effective length, KL shall be taken as the distance between the centres of intersection.

Compression members composed of two component back to back :

- Compression members composed of two angles, channels or tees back to back in contact or separated by small distance, shall be connected by together by bolting or welding so that the ratio of most unfavorable slenderness ratio of each member between the intermediate connection is not greater than 40 or 0.6 times the most unfavorable ratio of slenderness of the strut as whole, whichever is less.
- In no case shall the end of strut be connected together with less than two bolts or their equivalent in welding and there shall be not less than two additional connection in between, spaced equidistant along the length of strut.
- Where the legs of the connected angles or the connected tees are 125 mm wide or more, or where webs of channels are 150 mm wide or over not less than two bolts shall be used in each connection, one on line of each gauge mark.

- The bolts or welds in these connections shall be sufficent to carry the shear force and moments if any, specified for battened struts, and in no case the bolts be less than 16 mm diameter for members up to and including 10 mm thick, 20 mm diameter for member up to and including 16 mm thick.
- Compression members connected by such bolting or welding shall not be subjected to transverse loading in plane perpendicular to riveted, bolted or welded surface.
- where the components are in contact back to back, the spacing of bolts or intermittent welds shall not exceed the maximum spacing for compression members.

Tracking fasteners:-

- Tracking fastener shall have spacing in line not exceeding 32 times the thickness of the thinner outside plate or 300 mm, whichever is less.
- where the plates are exposed to weather, the spacing line shall not exceed 16 ~~time~~ of the thickness of the thinner plate or 200 mm, whichever is less.
- For compression members, tracking fastener in a line shall be spaced at a distance not exceeding 600 mm.

1. Calculate factored axial load on the column Section ISHB 400 @ 806.38 KN/m. The height of the Column is 3 m. and it is pin ended. use steel of Fe 410 grade.

Given data :

1. For steel grade Fe 410 :

$F_y = 250$ mpa, $\gamma_{m0} = 1.1$, $E = 2\times10^5$ N/mm^2

2. For ISHB 400 @ 806.38 N/m :

$a = 10466$ mm^2, $h = 400$ mm, $b_F = 250$ mm, $t_F = 12.7$ mm

$t_w = 10.6$ mm, $r_{zz} = 166.1$ mm $r_{yy} = 51.6$ mm

Radius of root $(r_1) = 14$ mm, Radius of toe $= 7$ mm

3. Buckling class :

- $\frac{h}{b_F} = \frac{400}{250} = 1.6 > 1.2$
- $t_F \le 40$ mm
$= 12.7 < 40$ mm

∴ Buckling class about zz-axis = class a
Buckling class about yy-axis = class b

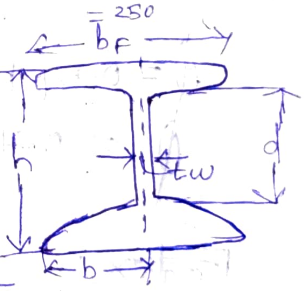
4. Effective Sectional area :

$A = 10466$ mm^2

5. Effective length of Columns :

For columns both end is pinned, $KL = 1.0L$

6. Check for limiting thickness :



Here root radius $(R_1) = 14$ mm,

$b = \frac{b_F}{2} = \frac{250}{2} = 125$ mm

$d = h - 2(t_F + R_1)$

$d = 400 - 2(12.7 + 14)$

$d = 346.6$ mm, $\epsilon = \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{250}} = 1$

- For rolled Section, outstand of Compression flange

$\frac{b}{t_f} = \frac{125}{12.7} = 9.84 < 10.5\times\overset{\epsilon}{1} = 10.5$ (class-2 compact)

~~$< 8.4\epsilon = 8.4\times1 = 8.4$ (class 1, plastic)~~

For web, Neutral axis at mid depth $\frac{d}{t_w} = \frac{346.6}{10.6}$

$= 32.7$

∴ so depth is plastic section. $= 32.7 < \overset{84}{\cancel{126}}\epsilon$

~~class 3~~ $= \overset{84}{\cancel{126}}\times1$

$= \overset{84}{\cancel{126}}$

∴ Hence the section is not slender and (Semi compact) full Section is available the design strength.

Design compressive strength :-

using formula :-

$$F_{cd} = \frac{F_y/\gamma_{mo}}{\phi - (\phi^2 - \lambda^2)^{0.5}} = \chi F_y/\gamma_{mo} \leq F_y/\gamma_{mo}$$

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

$$\lambda = \sqrt{F_y/F_{cc}} = \sqrt{\frac{F_y \times (KL/r)^2}{\pi^2 E}}$$

where $F_{cc} = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}$, α = Table-7, $\chi = \frac{1}{\phi + (\phi^2 - \lambda^2)^{0.5}}$

- about y-y axis :- For buckling class b, Imperfection factor $(\alpha) = 0.34$, $\lambda_y = \sqrt{\frac{F_y}{F_{cc}}}$, $F_{cc} = \frac{\pi^2 E}{\left(\frac{KL}{r_{yy}}\right)^2}$

$$F_{cc} = \frac{\pi^2 \times 2 \times 10^5}{\left(\frac{3000}{51.06}\right)^2}$$

$KL = 1.0L$
$= 1.0 \times 3$
$= 3$ mm. Given

$F_{cc} = \cancel{583.98}1$ N/mm^2

$$\lambda_y = \sqrt{\frac{250}{\cancel{185.884}\ 583.96}} = 0.654$$

$$\phi = 0.5[1 + 0.34(0.654 - 0.2) + 0.654^2]$$
$$\phi = 0.79$$

$$\cancel{F_{cd}}\ \chi = \frac{1}{0.79 + (0.79^2 - 0.654^2)^{0.5}}$$

$\chi = 0.810$

$$F_{cd} = \chi F_y/\gamma_{mo} = \frac{0.81 \times 250}{1.1} = 184.09$$

$F_y/\gamma_{mo} = 227.27$

$\chi F_y/\gamma_{mo} \leq F_y/\gamma_{mo}$

$\therefore F_{cd} = 184.09$

Factored axial load $= A_e \times F_{cd}$
For y-y axis $= 10466 \times 184.09$
$= 1926.6859$ KN

• about zz axis, For buckling class a, $\alpha = 0.21$, $r_{zz} = 166.1$ mm

$$F_{cc} = \frac{\pi^2 E}{\left(\frac{KL}{r_{zz}}\right)^2} = \frac{\pi^2 \times 2\times 10^5}{\left(\frac{3000}{166.1}\right)^2} = \cancel{60\ 90.89} \text{ KN} \quad 6050.99$$

$$\lambda_z = \sqrt{\frac{F_y}{F_{cc}}} = \sqrt{\frac{250}{\cancel{109.289}\ 6050.90}} = \cancel{1.512}\ 0.203$$

$$\phi = 0.5\left[1 + \alpha(\lambda_z - 0.2) + \lambda_{zz}^2\right]$$

$$= 0.5\left[1 + 0.21(0.203 - 0.2) + \cancel{1.512}\ 0.203^2\right]$$

$$\phi = \cancel{1.780}\ 0.52$$

$$x = \frac{1}{0.\cancel{5}8 + (0.\cancel{5}8^2 - 0.203^2)^{0.5}} = x = \cancel{0.3677}\ 1.00$$

$$F_{cd} = x F_y / \gamma_{mo} = \frac{\cancel{0.3677}\ 1.00 \times 250}{1.1} = \cancel{83.568}\ 227.27 \text{ KN}$$

∴ Hence $F_{cd} = 227.27$ KN

Factored axial load = $A_e \times F_{cd}$
For zz-axis $= 10466 \times 227.27$
$= 2378.60$ KN

using interpolation :-

about y-y axis of buckling class-b

$$\lambda_y = \frac{KL}{r_{yy}} = \frac{1\times 3000}{51.6} = 58.14 < \text{max}^m \text{ slenderness } (180)$$

$F_y = 250$ N/mm2

$$y_0 = y \mp \left[\frac{y - y_1}{x_1 - x} \times (x_0 - x)\right]$$

KL/r	Fcd
50 (x)	194 (y)
58.14 (x_0)	? (y_0)
60 (x_1)	181 (y_1)

$$F_{cd} = 194 - \left[\frac{(194 - 181)}{(60 - 50)} \times (58.14 - 50)\right]$$

$F_{cd} = 183.41$ N/mm2

Factored axial load $= F_y \times A_e$
$= 183.41 \times 10466$
$= 1919.56$ KN

about zz-axis buckling class a

$$\lambda_z = \frac{KL}{r_{zz}} = \frac{1\times 3000}{166.1} = 18.06 < \text{max}^m \text{ slenderness ratio } (180)$$

$F_y = 250$ N/mm2

KL/r	Fcd
10	227
18.06	?
20	226

$$F_{cd} = 227 - \left[\frac{(227 - 226)}{(20 - 10)} \times (18.06 - 10)\right]$$

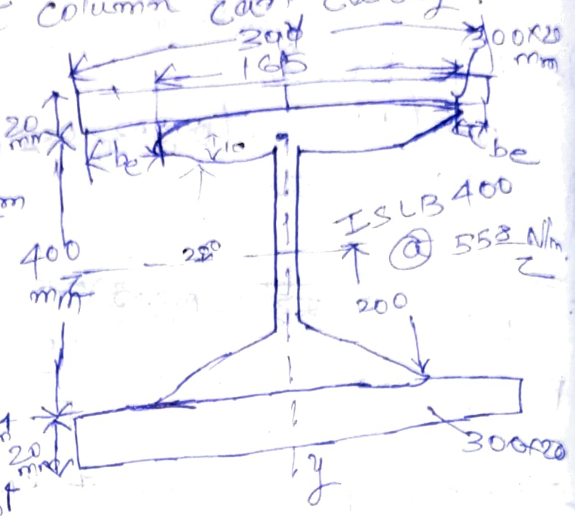
$F_{cd} = 226.194$ KN

Factored axial load
$P_d = 226.194 \times 10466$
$= 2367.34$ KN

∴ Design factored axial load $= 1919.56$ KN P_d

2. A compound column consists of ISLB 400 @ 558 N/m with one cover plate 300 mm × 20 mm in each flange. The actual length of column is 4.2 m and it is fixed at one end & other end hinged. Using the grade of steel (E250) (Fe 410W), B - Determine the working load that column can carry?

Given data:



1. For ISLB 400 @ 558 N/m

$h = 400\ mm$

$b_F = 165\ mm$

$t_F = 12.5$

$t_w = 8\ mm$

$I_{zz} = 19306.3 \times 10^4\ mm^4$

$I_{yy} = 716.4 \times 10^4\ mm^4$

$A = 7243\ mm^2$

2. Buckling class:

Any axis the buckling class - c.

3. Check for limiting thickness ratio:

$R_1 = 16\ mm$, $\epsilon = \sqrt{\frac{250}{F_y}} = \sqrt{\frac{250}{250}} = 1$

$d = h - 2(t_F + R_1)$

$d = 400 - 2(12.5 + 16)$

$d = 343\ mm$

Assuming welded joint between the cover plate and rolled sections.

So, external element width $(b_e) = \frac{300-165}{2}$

$(b_e) = 67.5\ mm$

Internal element width $(b_i) = 165\ mm$

- $\frac{b_e}{t_F} = \frac{67.5}{12.5} = 5.4 < 9.4\epsilon = 9.4$ — this is a plastic section

- $\frac{b_i}{t_F} = \frac{165}{12.5} = 13.2 < 42\epsilon$ — (this is a semi compact

- $\frac{d}{t_w} = \frac{343}{\cancel{12.5}\ 8} = 42.875 < 84\epsilon$

∴ this is not a slender section.

I_{zz} about I+ section $= 19306.3 \times 10^4 \text{ mm}^4$

Total $I_{zz} = 19306.3 \times 10^4 + 2 \times \left[\frac{300 \times 20^3}{12} + 210^2\right]$

$\Rightarrow \cancel{193555} \; 7226.62 \times 10^5 \text{ mm}^4$

$I_{zz} = 7226.62 \times 10^5 \text{ mm}^4$

$I_{yy} = 716.4 \times 10^4 + \left[\frac{20 \times 300^3}{12}\right] \times 2 + 2 \times (300 \times 20 \times 0^2)$

$= 9716.40 \times 10^5 \text{ mm}^4$

$I_{zz} < I_{yy}$

$r_{min} = \sqrt{\frac{I_{min}}{A}} = \sqrt{\frac{7226.62 \times 10^5}{A}} = \sqrt{\frac{7226.62 \times 10^5}{19243}}$

$A = 7243 + (2 \times 300 \times 20) = 19243 \text{ mm}^2$

$r_{min} = 193.79 \text{ mm}.$

For one end fixed and other end hinged

effective length, $KL = 0.8L$

$= 0.8 \times 4200$

$= 3360$

Slenderness ratio $(\lambda) = \frac{KL}{r} = \frac{3360}{193.79} = 17.338$

$\therefore 17.34 < 180$ — (OK)

Design strength using interpolation :

So $\frac{KL}{r} = 17.34$, $F_y = 250 \text{ N/mm}^2$

~~Fcd=~~

KL/r	Fcd
10	227
17.34	?
20	224

$227 - \left(\frac{227-224}{20-10}\right) \times (17.34 - 10)$

$F_{cd} = 224.798 \text{ N/mm}^2$

$F_{cd} = 224.80 \text{ N/mm}^2$

Design factored load $(P_d) = A_e \cdot F_{cd}$

$= 19243 \times 224.80$

$= 4325.83$ KN

$\therefore$ working load capacity of the column $= \dfrac{4325.83}{1.5}$

$= 2883.88$ KN (Ans.)

• In a truss, a Strut 3.3m. long consists of 2 angles ISA 110×110×10 mm. Find the factored Strength of the member if the angles are Connected on both side of 12 mm gusset plate by i. 1 bolt ii. 2 bolts iii. welding, which makes the Joint rigid. Assume that the angles are track bolted over their length :-

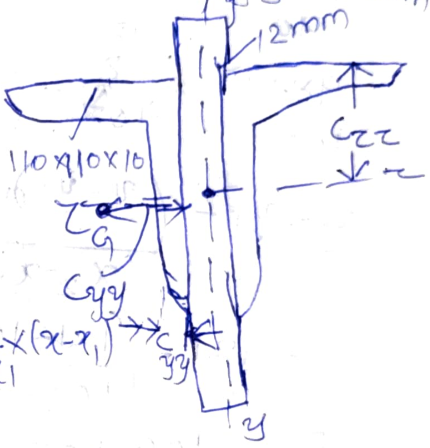
Given data :-

For ISA 110×110×10 mm :-

$a = 2106\ mm^2$, $C_{zz} = C_{yy} = 30.8\ mm$,

$I_{yy} = I_{zz} = 238.4 \times 10^4\ mm^4$, $r_{zz} = r_{yy} = 33.6\ mm$

Sectional property of member :-



$a = 4212\ mm^2$, $\cancel{I_{zz}} =$

$r_{zz} = 33.6\ mm$

$C_{zz} = 30.8\ mm$

$r_{yy} = ?$

$x_1 = 10mm - 49.1 = y_1$

$x_2 = 12 - 50.6 (y_2)$

$x_2 = 14mm -$

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

$y = 49.1 - \left[\left(\frac{49.1 - 50.6}{14 - 10}\right) \times (12 - 10)\right]$

$y = 49.85$

$r_{yy} = 49.85\ mm$, $r_{zz} = 33.6\ mm$

$r_{min} = 33.6\ mm$

Design Compressive strength about zz axis

• Buckling class – 'c'.

• check for limiting thickness :-

$b = d = 110\ mm$, $t = 10\ mm$

$\frac{d}{t} = \frac{110}{10} = 11\ mm$, $\epsilon = \sqrt{\frac{250}{250}} = 1$

$\frac{b}{t} = \frac{d}{t} = 11\ mm, < 15.7\epsilon$ ∴ ok

This is Semi compact Section.

Maximum slenderness ratio :-

$$\lambda = \frac{KL}{r_{min}} = \frac{KL}{r_{zz}}$$

i. when single bolt is used :-

$K = 1.0$, effective length $= KL = L = 3300$ mm

$$\lambda = \frac{KL}{r_{min}} = \frac{3300}{33.6} = 98.21 < 180 \therefore OK$$

For $F_y = 250$ mpa, & buckling class C

λ	F_{cd}
90	121
98.21	?
100	107

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

$$= 121 - \left(\frac{121 - 107}{100 - 90}\right) \times (98.21 - 90)$$

$F_{cd} = 109.506$ N/mm^2

$P_d = A_e \cdot F_{cd}$

$= 4212 \times 109.506$

$= 461.21$ KN

ii. when 2 bolts is used :-

$K = 0.85$, effective length $= KL$

$= 0.85 \times 3300$

$= 2805$ mm

$$\lambda = \frac{KL}{r_{min}} = \frac{2805}{33.6} = 83.482 \text{ mm}$$

$< 180 \therefore OK$

For $F_y = 250$ mpa & buckling class c

λ	F_{cd}
80	136
83.482	?
90	121

$$y = 136 - \left(\frac{136 - 121}{90 - 80}\right) \times (83.482 - 80)$$

$y = 130.777$

$F_{cd} = 130.77 \ N/mm^2$

$P_d = A_e . F_{cd}$

$= 4212 \times 130.77$

$= 550.803 KN$

iii. when welding is used

$K = 0.~~70~~$, effective length $= KL = 0.~~70~~ \times 3300$

$= 2310 mm$

$\lambda = \frac{KL}{r_{min}} = \frac{2310}{33.6} = 68.75 mm < 180$.: OK

For $F_y = 250 mpa$ & buckling class c.

λ	F_{cd}
60	~~183~~ 168
68.75	?
70	152

$168 - \left(\frac{168 - 152}{70 - 60}\right) \times (68.75 - 60)$

$y = 168 - \left(\frac{168 - 152}{70 - 60}\right) \times (68.75 - 60)$

$y = 154 \ N/mm^2$, $F_{cd} = 154 \ N/mm^2$

$P_d = A_e \cdot F_{cd}$

$= 4212 \times 154 = 648.648 KN$

—x—

Design problem : Design of Axially loaded in compression member

procedure :

The following procedure may be adopted in design

1. Assume slenderness ratio and determine the design compressive stress considering grade of steel and assuming buckling class. (The slenderness ratio may be considered as 70 to 90 for rolled steel beam, 110 to 130 for angle struts and 40 for member carrying large loading)

Alternatively the design stress in compression member may be directly assume in the range of
- 130 N/mm2 – 140 N/mm2 – for rolled steel I-sec
- 80 N/mm2 – 100 N/mm2 – For angle strut & channel
- 190 N/mm2 – 200 N/mm2 – Heavy load or built up sections.

2. Calculate effective section area required.

$A_e = \frac{P_d}{F_{cd}}$

- Choose a trial section from steel table
- Then find r_{min} of the section.

3. find effective length and maximum slenderness ratio $= \lambda = \frac{KL}{r_{min}}$, considering end condition and type of connection.

4. Determine permissible compressive stress F_{cd}. considering grade of steel and actual buckling class and compute the strength of the member $(P_d) = A_e F_{cd}$

5. Redesign if Pd differs considerable from the design load.

6. The Section may be checked for limiting thickness ratio.

1. Design a single angle strut connected to the gusset plate to carry 180 kN factored load. The length of strut between centre to centre connection is 3 m.

Given data :-

Assume) ~~Fcd~~ = 110 N/mm², buckling class = c

$F_{cd} = 94.6$ N/mm² (view from steel code)

or either directly $F_{cd} = 80 - 110$ N/mm²

Assume F_{cd} = 90 N/mm²

∴ Hence $F_{cd} = 94.6$ N/mm²

∴ F_{cd} Assume = 90 N/mm²

$$A_e = \frac{P_d}{F_{cd}} = \frac{180 \times 10^3}{90} = 2000 \text{ mm}^2$$

∴ Area required 2000 mm²

Try 110 × 110 × 10 mm, where area = 2106 mm²

$r_{min} = r_{vv} = 21.4$ mm

∴ Assume welded joints, $KL = 0.7L$

$KL = 0.7 \times 3000$

$KL = 2100$ mm

$$\lambda = \frac{KL}{r_{min}} = \frac{2100}{21.4} = 98.130$$

KL/r	Fy
90	121
98.13	?
100	107

$$F_{cd} = 121 - \left(\frac{121-107}{100-90}\right) \times (98.13 - 90)$$

$= 109.618$ N/mm²

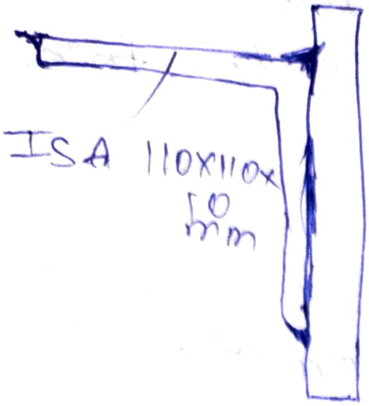
∴ $109.618 < 180$ ∴ Safe (~~Redesign~~)

$P_d = A_e \times F_{cd}$

$= 109.618 \times$ 2106

$= 230.85$ kN

~~KL = 0.7 × 3~~



• A column 4m. long has to support a factored load of 6000 KN. The column is effectively held at both ends and restrained the direction of the one of its ended. Design the column using beam sections & plates.

Solution :-

Assume $F_{cd} = 200\ N/mm^2$

Area required $= \frac{6000 \times 10^3}{200}$

$= 30,000\ mm^2$

Try ISHB 450 @ 855.4 of plate size 400×25

Area $= 31,114\ mm^2$,

$r_{min} = r_{yy} = 97.6\ mm$

$\therefore\ L_e = KL = 0.8 \times L$

$= 0.8 \times 4000 = 3200\ mm$

$\lambda = \frac{KL}{r_{min}} = \frac{3200}{97.6} = 32.78\ mm.$

For buckling class c

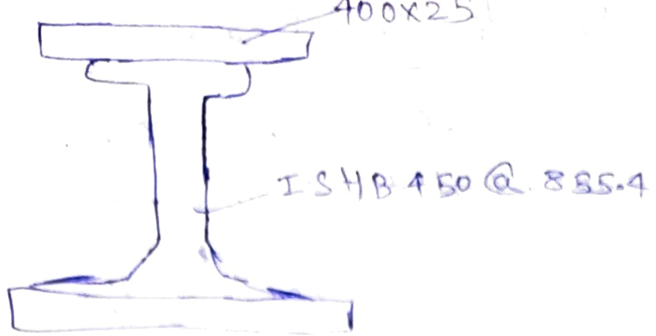
λ	F_{cd}
30	211
32.78	?
40	198

$F_{cd} = 211 - \left(\frac{211-198}{40-30}\right) \times (32.78-30)$

$F_{cd} = 207.386\ N/mm^2$

Factored load carried $= 31114 \times 207.386$

$\therefore$ Hence safe $= 6452.60\ KN$



Assignment

1. For bolts of property class 4.6 what do the no. 4 & 6 indicates?
2. what is the angle betn fusion faces for fillet weld?
3. what will be the buckling class of ISHB 400 @ 907 N/m about zz-axis & yy-axis?
4. Define bolt value?
5. Define pitch?
6. Define radius of gyration.
7. How are the connection classified?
8. what is the advantage of butt Joint over lap Joint?
9. Define staggered pitch?
10. what do you mean by partial safety factor in the limit state method of
11. Write 2 advantages of welding over bolting?
12. Write down the properties of structural steel.
13. 2 plates of 8 mm & 18 mm thickness are to be Jointed using longitudinal fillet weld. Suggest a suitable size of a weld?
14. Define net section area of tension member?
15. Mention the type of buckling in compression member.
16. what is recommended throat thickness for incomplete penetration butt welded welded from one side only.
17. what is the objective of providing ~~track~~ tack rivets in steel structural member?
18. Define & state the significance of slenderness ratio?
19. what is the value of maximum slenderness ratio for a member carrying compressive load resulting from dead load & super imposed load?
20. what are the type of structural steel?

Design of steel Beam

- A Beam is a structural member subjected to Bending. It subjected to transverse loads normal to its axis. Beam of light section that support floor construction at termed as Joists.
- Horizontal beam spanning between the adjacent trusses are known as purlins.
- Lintel is a beam that spans over openings in buildings.
- Header is a beam frame to two beams at right angles and support Joist and one side of it.

Beam that supports the header is termed as trimmer.

- The Beam supporting the stair steps is termed as stringer.

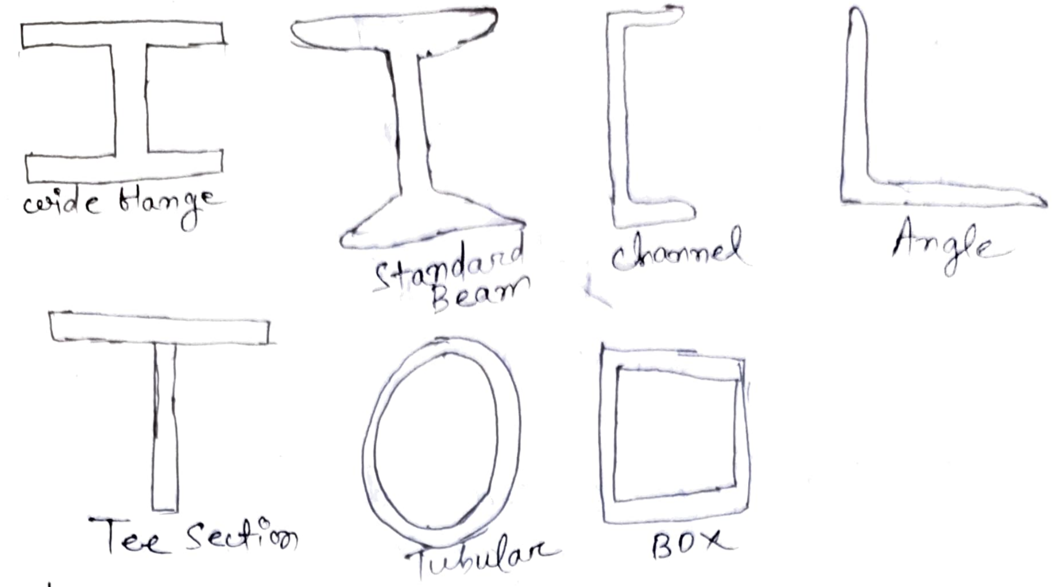
use of beam/steel section :

- Rolled I-sections with and without cover plates are normally used for floor beam.
- channel, T-section and angle section are used in roof trusses as purlins and common rafters.

Common cross sections and their classification

The common cross sections to be used as beam as shown in figure.

Depending upon the slenderness of the constituent plate element of the beam, their classified as slender, semi compact, compact and plastic.



plastic Cross Section :-

plastic cross section are those which can develop plastic hinges and have the rotation capacity required for failure of the structure by formation of plastic mechanisms.

Compact Cross Section :-

Compact cross section are those which can develop plastic moments of resistance but have an inadequate of plastic hinges rotation capacity for formation of plastic mechanism due to local buckling

Semi Compact Cross Section :-

- Semi compact cross sections are those in which the stress in the extreme fibres in compression should be limited to yield stress.
- These sections can't develop the plastic moment of resistant due to local buckling.

Slender cross Section :-

Slender cross section are those in which the element buckl locally even before reaching yield stress.

local buckling :

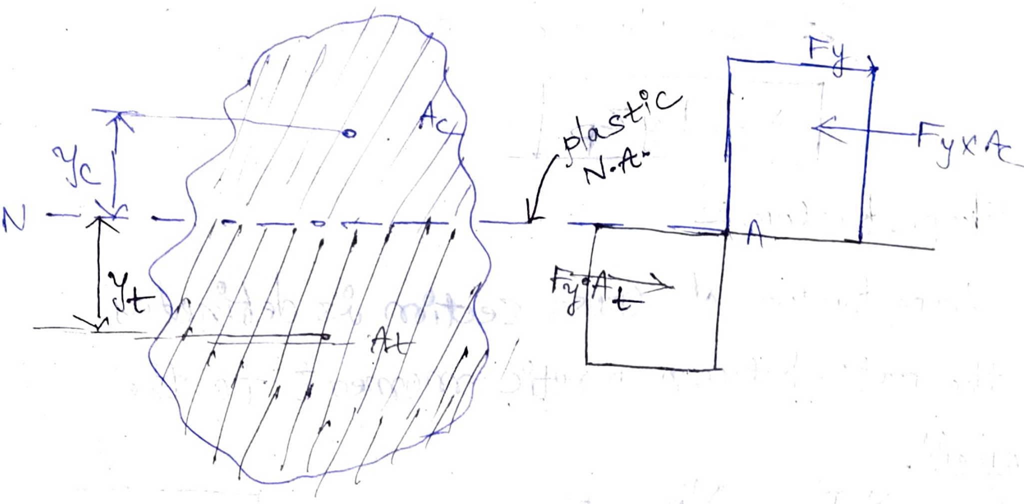
- Any plate element subjected to direct compression, Bending, or shear stress or combination of these stresses may buckle ~~prematurely~~. prematurely. plate elements may fail in buckling locally before overall column buckling or overall beam failure due to yielding or lateral buckling. This type of failure is called local buckling.
- This reduces stiffness and strength of locally buckled plate element. This also decreases load carrying capacity of columns and beams and ~~distorts~~/ distorts the cross section.
- Local buckling of any element doesn't lead always to overall failure of the structure.

— x —

Basic concept of plastic theory :

- To understand plastic theory, consider a rectangular shape, simply supported at both the ends subjected to a concentrated load at the centre.

1. when maximum bending moment (m) is less than yield moment (my), all the extreme fibres are stressed below the yield point.
2. With increasing load maximum bending moment is equal to yield moment, i.e. $M = M_y$. The stress in extreme fibre reaches the value of yield stress and begins to yield.
3. With further increasing load, the maximum bending moment lies between yield moment (M_y) and plastic moment (M_p)
4. Practically, all the fibres at the section reaches the yield stress and the section become fully plastic. The moment corresponding to this state is called the plastic moment of section.



• plastic moment (M_P) may also be defined as the magnitude of the bending moment (M) at which a plastic hinge is formed.

• Total force in compression = Total force in tension

$\cancel{F_y} \cdot A_c = \cancel{F_y} \cdot A_t$

$A_c = A_t$ (plastic neutral axis)

• The neutral axis that divides the cross section into 2 equal halves, is known as plastic neutral axis.

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

where A_t = Area of tension

A_c = Area in compression

A = Total Area

Considering equilibrium condition, $\Sigma M = 0$

$M_P = \{(F_y A_c \times y_c) + (F_y A_t \times y_t)\}$

$M_P = F_y (A_c y_c + A_t y_t)$

$A_c y_c + A_t y_t = \frac{M_P}{F_y} = Z_P$

$\boxed{Z_P = A_c y_c + A_t y_t}$

∴ Z_P = plastic modulus

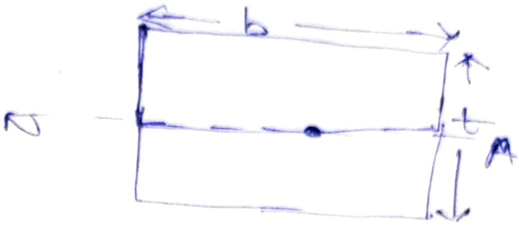
$\boxed{M_P = F_y Z_P}$

Shape factor :

Shape factor of cross section is defined as the ratio between plastic moment to the yield.

$S.F. = \frac{M_P}{M_y} = \frac{F_y Z_P}{F_y . Z_e} = \frac{Z_P}{Z_e}$ $\boxed{S.F. = \frac{Z_P}{Z_e}}$

1. Determine the plastic moment capacity and plastic section modulus of rectangular section size b×t about zz-axis



$A_c = b \times \frac{t}{2} = \frac{bt}{2}$

$A_t = b \times \frac{t}{2} = \frac{bt}{2}$

$y_c = \frac{t/2}{2} = \frac{t}{4}$

$y_t = \frac{\frac{t}{2}}{2} = \frac{t}{4}$

$Z_p = A_c y_c + A_t y_t$

$= \left(\frac{bt}{2} \times \frac{t}{4}\right) + \left(\frac{bt}{2} \times \frac{t}{4}\right)$

$Z_p = \frac{bt^2}{8} + \frac{bt^2}{8} = \frac{\cancel{2}bt^2}{\cancel{8}4} = \frac{bt^2}{4}$

$M_p = F_y \cdot Z_p = F_y \cdot \frac{bt^2}{4}$

2. Determine the plastic moment capacity & plastic section modulus of a symmetrical I section having depth 350mm, width of flange 140 mm. Thickness of flange = 14.2 mm, thickness of web = 8.1 mm. a. about zz-axis b. about yy-axis

Design strength of lateral Supported Beam in flexure :-

low shear

Factored design S.F. = $V < 0.6 V_d$

V_d = design Shear strength of cross Section.

high shear

$V > 0.6 V_d$

- Low Shear $V < 0.6 V_d$:-

$$\boxed{M_d \text{ (design bending strength)} = \beta_b Z_p f_y / \gamma_{mo}}$$

To avoid reversible formation under Serviciblity load :- $M_d < 1.2 Z_e f_y / \gamma_{mo}$ (simply supported beam)

$M_d < 1.5 Z_e f_y / \gamma_{mo}$ (Cantilever beams)

where β_b = 1.0 for plastic and compact section

$= Z_e / Z_p$ for semicompact Section.

- High shear $V > 0.6 V_d$:-

design bending strength (M_d) = M_{dv}

where M_{dv} = design bending strength under high shear.

a. plastic or Compact Section :- (cl. 9.2.2)

$M_{dv} = M_d - \beta(M_d - M_{fd}) \leq 1.2 Z_e \cdot f_y / \gamma_{mo}$

where $\beta = (2V/V_d - 1)^2$

M_d = plastic design moment.

V = Factored applied shear force

V_d = design shear strength

M_{fd} = plastic design strength of area of the cross Section excluding shear area, Considering partial safety factor γ_{mo}.

Z_e = elastic Section modulus of whole section

b. For Semicompact Section :-

$M_{dV} = Z_e f_y / \gamma_{mo}$

Design strength of Lateral Supported Beam in shear :-

$V \leq V_d$

V_d = design strength $= \frac{V_n}{\gamma_{m0}}$

- $V_n = V_p$

V_p = nominal plastic shear

where $V_p = \frac{A_v F_{yw}}{\sqrt{3}}$

A_v = shear area

F_{yw} = yield strength of web

Rolled Section	Shear Area
I & channel section	
• major axis Bending	— $h t_w$ (hot rolled)
	— $d t_w$ (welded)
• Minor axis Bending	— $2 b t_f$ (Hot rolled or welded)
• Rectangular hollow sec^n of uniform thickness	— $\frac{Ah}{b+h}$ (Loaded parallel to depth)
	— $\frac{Ab}{b+h}$ (Loaded parallel to width)
• circular hollow tubes of uniform thickness	— $\frac{2A}{\pi}$
• plates and solid bars	— A

where A = cross sectional area

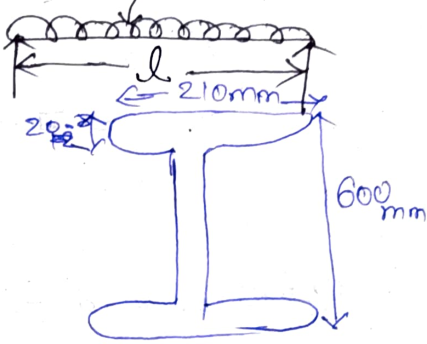
b = overall breadth of tubular section, breadth of I-section of flanges

d = clear depth of the web between flanges

h = overall depth of the section

t_f = thick

1. A laterally supported Beam ISMB 600 @ 1202.71 N/m is placed between two supports. Determine the safe uniformly distributed load per metre length which can be placed over the beam for an effective span of 12 m.



Sectional properties :

For ISMB 600 1202.7 N/m

Sectional area $(a) = \cancel{2167}$ 15621 mm^2

$h = 600$ mm

$b = 210$ mm

$t_f = 20.8$ mm

$t_w = 12.0$ mm

Moment of inertia $I_{zz} = 91813 \times 10^4$ mm^4

$Z_e = 3060.4 \times 10^3$ mm^3, $Z_p = 3510630$ mm^3

Section classification :

For Rolled Section, $\epsilon = \sqrt{\frac{250}{250}} = 1$

$\frac{b}{t_F} = $ $b = \frac{b}{2} = \frac{210}{2} = 105$ mm

$\frac{b}{t_F} = \frac{105}{20.8} = \cancel{8.75}$ $5.04 < 9.4\epsilon$

so flange is

$\therefore$ plastic section

web N.A. at mid depth, $\frac{d}{t_w} = $ $d = h - 2(t_F + R_1)$

$= 600 - 2(20.8 + 20)$

$= 518.4$ mm

$\frac{518.4}{12} = 43.2\epsilon$

$\therefore$ Hence web is plastic $\epsilon 84\epsilon$

Design bending strength :

Assume $V \leq 0.6 V_d$

$M_d = \beta_b \cdot Z_p \cdot F_y / \gamma_{mo}$

$\beta_b = 1.0$ for plastic section

$M_d = \frac{1 \times 3510630 \times 250}{1.1}$

$M_d = 797.870$ KN-M

M_d will be less than $1.2 Z_e F_y / \gamma_{mo}$ for simple ~~conti~~

$= \frac{1.2 \times 3060.4 \times 10^3 \times 250}{1.1} = \cancel{1043.31}$ KN-M

$834.6545 > M_d$

$\therefore$ OK

Design moment and load carrying Capacity:

Maxium bending moment, for simply supported Beam, $\frac{WL^2}{8}$

$$\frac{W\times12^2}{8} = \frac{W\times144}{8} = 18W$$

$M = M_d$

$18W = 797.8704$

$$W = \frac{797.8704}{18} = 44.326\ KN/m.$$

$$\text{working load} = \frac{44.326}{1.5} = 29.550\ KN/m$$

∴ Safe working load that the beam can carry (29.551 − self wt. of beam)

Self wt. of beam (unit weight) = 1.202 KN/m.

$29.551 - 1.202$

$\boxed{\text{load} = 28.349\ KN/m}$ ∴ OK.

Check for shear:

$$\text{Factored shear force} = \frac{WL}{2}$$

$$= \frac{44.326\times12}{2}$$

$$V = 265.956\ KN/m$$

V_d = design Beari shearing force

$$V_d = \frac{A_v F_y}{\sqrt{3}\,\gamma_{m0}}$$

For Rolled section, $A_v = h t_w$

$= 600\times12 = 7200$

$$V_d = \frac{600\times12\times250\times10}{\sqrt{3}\times1.10} = 944.754\ KN$$

$V \le 0.6\,V_d$ = 265.956 = 0.6×944.754 = 566.852 KN

$V < 0.6\,V_d$ ∴ OK

Load carrying capacity for deflection :

$$\delta_{max} = \frac{5WL^4}{384EI}$$

$$= \frac{5 \times w \times (12000)^4}{384 \times 2\times10^5 \times 91813\times10^4}$$

$$= 1.470W$$

Assume simple span, live load and elastic cladding as per table-6

Maximum deflection $= \frac{Span}{240} = \frac{12000}{240} = 50\,mm$

$1.47W = 50$

$W = \frac{50}{1.47} = 34.01$

Working load $= \frac{34.01}{1.5} = 22.6757$ KN/m

$= 22.676$ KN/m.

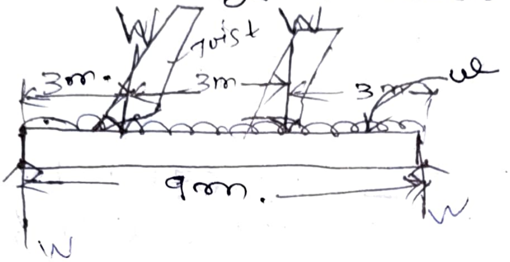
So load carrying capacity $= 22.676 - 1.0202$

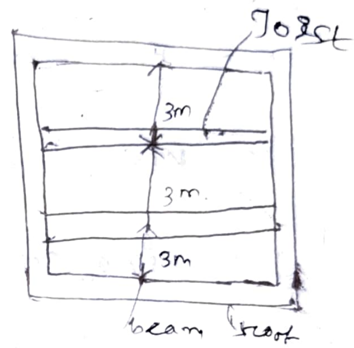
$= 21.4737$ KN/m

21.4737 KN/m > 28.349 KN/m.

Hence the safe working load that the beam can carry is minimum of bending & deflection, Hence the beam can carry 21.473 KN/m

2. A laterally supported beam ISWB 550 @ 110.63 N/m. has an effective span of 9m. two floor joist transmit the floor load at a distance of 3m. from each end. Determine the safe working load which 2 floor joist can transmit on the beam if the beam is effectively restrained laterally by the floor joist. (fixed both end) Take $F_y = 280\ N/mm^2$. Neglect web buckling web crippling & deflection criteria.





Section properties :-

ISWB 550 @ 110.63 N/m.

$Z_p = 3066290\ mm^3$,
$h = 550\ mm$, $b_F = 250\ mm$, $t_F = 17.6\ mm$
$a = 14334\ mm^2$, $t_w = 10.5\ mm$
$I_{zz} = 74906.1 \times 10^4\ mm^4$, $R_1 = 16\ mm$

$d = h - 2(t_F + R_1)$
$= 550 - 2(17.6 + 16)$
$d = 482.8\ mm$, $Z_e = 2723.9 \times 10^3\ mm^3$

Section classification

$\epsilon = \sqrt{\frac{250}{F_y}} = \sqrt{\frac{250}{280}} = 0.9449$
$= 0.945$

Hence $b = \frac{b_F}{2} = \frac{250}{2} = 125\ mm$

$\therefore \frac{b}{t_F} = \frac{125}{17.6} = 7.10 < 9.4\epsilon$
$= 7.10 < 8.883$

$\therefore$ this section is plastic section.

$\frac{d}{t_w} = \frac{482.8}{10.5} = 45.98 < 84\epsilon$

$= 45.98 < 79.38$

$\therefore$ Hence the web is a plastic sections.

design bending strength:

as per 8.2.1.1, $\frac{d}{t_w} < 67\epsilon$

$= 45.98 < 63.315$ — (OK)

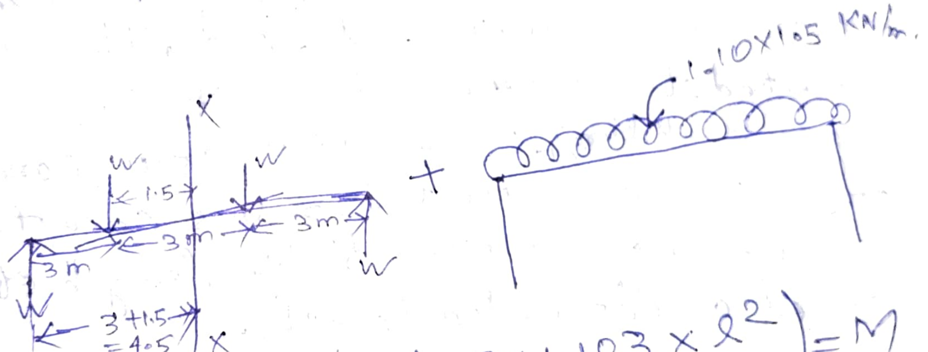
Assume $V \le 0.6 V_d$

$$M_d = \frac{\beta_b Z_p F_y}{\gamma_{mo}}$$

$\beta_b = 1.0$ for plastic section.

$$M_d = \frac{1 \times 3066290 \times 280}{1.1}$$

$M_d = 780.510$ KN/M.



$$\left(W \times 4.5 - W \times 1.5\right) + \left(1.5 \times 1.103 \times \frac{l^2}{8}\right) = M$$

$M = M_d$

$$W \times 4.5 - W \times 1.5 + \left(1.5 \times 1.03 \times \frac{9^2}{8}\right) = 780.51$$

$\Rightarrow 4.5W - 1.5W + 15.643 = 780.51$

$\Rightarrow 3W = 780.51 - 15.643$

$3W = 764.867$

$W_u = 254.95$ KN

Safe working load that the beam can carry $= \frac{254.95}{1.5} = 169.72$ KN.

Check for shear :-

$W + \frac{1.5 \times 1.03 \times 9}{2} = W + \frac{WL}{2}$

$W + 6.9525$

$= 254.59 + \frac{1.5 \times 1.03 \times 9}{2}$

$\Rightarrow 262.035$ KN

Shear Strength $V_d = \frac{F_{yw} \times A_V}{\sqrt{3}\, \gamma_{m0}}$ $\quad A_V = h t_w$

$= \frac{280 \times 550 \times 10.5}{\sqrt{3} \times 1.1}$

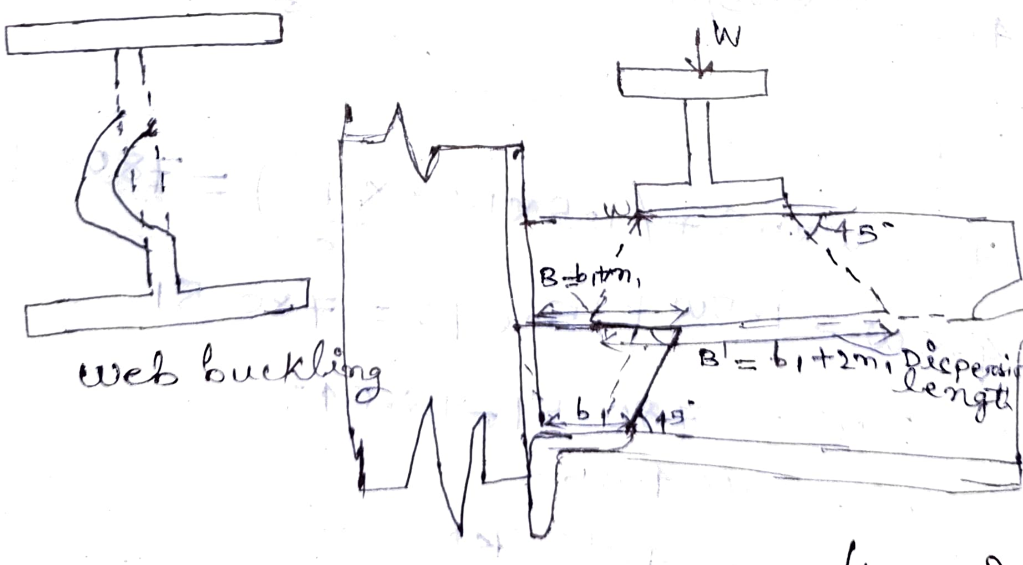
$= 848.704$ KN

$0.6 V_d = 0.6 \times 848.704$
$= 509.229$ KN.

$\therefore$ Hence $V \leq 0.6 V_d$ $\therefore$ OK

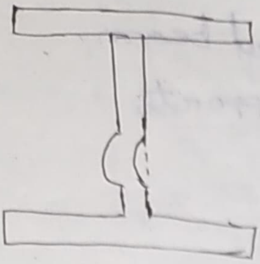
web buckling :-

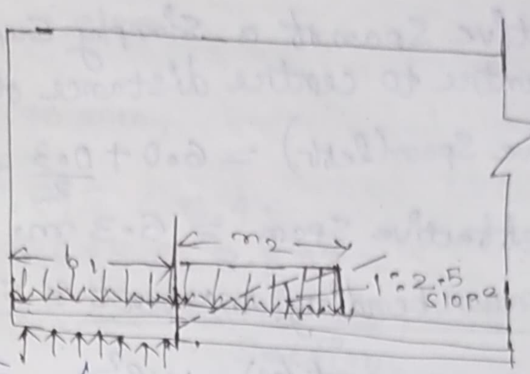
web buckling is the failure of web under the action of concentrated load. the web is Subjected to column action during buckling. Buckling may occur below concentrated load or above a Support. In this case the load is Spread out over a finite length of the web called as dispersion lengths.



$\lambda = 2.5 \frac{d}{t_w}$, $P_{wb} = (b_1 + n_1) t_w f_{cd}$
(P_{wb} Buckling Strength)

Web crippling :





$P_{crip} = (b_1 + n_2) t_w f_{yw}/\gamma_{m0}$ (at support)

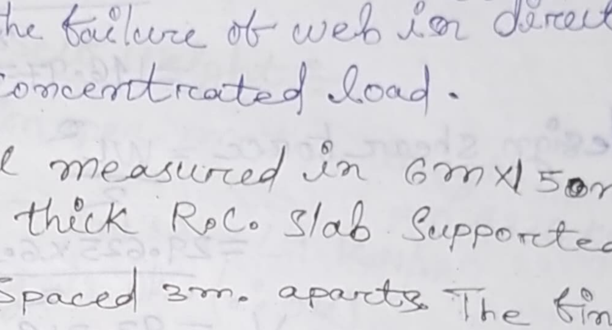
$P_{cr} = (b_1 + 2n_2) t_w f_{yw}/\gamma_{m0}$ (under concentrated load)

Web crippling is the failure of web in direct crossing under concentrated load.

3. A roof of a hall measured in 6m × 15m consists of 130 mm thick R.C. slab supported on steel I-beams spaced 3m. aparts. The finishing load may be taken as 1 KN/m² and live load as 2 KN/m². Design the steel beam. thickness of wall may be taken as 300 mm. Consider $f_y = 300$ N/mm²

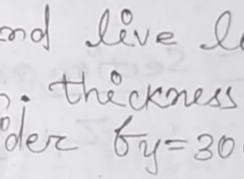
- Given data :

1. Load calculation :



Weight of R.C. Slab
= density × thick
= 25 × 0.13
= 3.25 KN/m²

Finishing load = 1 KN/m²
live load = 2 KN/m²



Total load = 6.25 KN/m²

Load transfered from slab to beam per metre length = 6.25 KN/m² × 6 × 3 × m²

$= \frac{112.5 \text{ KN}}{6} = 18.75$ KN/m

Self wt Beam = 1 KN/m

Total = 19.75 KN/m, Factored load = 1.5 × 19.75
= 29.625 KN

2. Determine effective Span :-

effective Span of a simply supported beam = centre to centre distance of supports

effective Span (l_{eff}) = $6.0 + \frac{0.3}{2} + \frac{0.3}{2}$

effective Span = 6.3 m.

3. Maximum bending moment & shear force :-

design moment (M) = $\frac{w l_{eff}^2}{8}$

$= \frac{29.625 \times 6.3^2}{8}$

$= 146.977 \simeq 146.98$ KN-m

design shear force = $\frac{WL}{2}$

$= \frac{29.625 \times 6.3}{2}$

$V = 93.319$ KN

plastic Section modulus :-

$Z_p = \frac{M}{F_y} \times \gamma_{m0}$

$= \frac{146.98}{300} \times 1.1$

$= 538.926 \times 10^3$ mm^3

Sectional properties of trial Section :-

Try ISMB 300 @ 433.6 N having

$Z_p = 65174 \times 10$, $a = 5626$ mm^2,

$= 651740$ mm^3, $R_1 = 14$ mm

$h = 300$ mm; $b_F = 140$ mm, $t_F = 12.4$ mm,

$t_w = 7.5$ mm, $d = h - 2(t_F + R_1)$

$= 300 - 2(12.4 + 14)$

$= 247.2$ mm

$I_{zz} = 8603.6 \times 10^4$ mm^4

$Z_e = 573.6 \times 10^3$ mm^3

Section classification :

$\epsilon = \sqrt{\frac{250}{F_y}} = \sqrt{\frac{250}{300}} = 0.9128 \simeq 0.913$

here. $b = \frac{b_F}{2} = \frac{140}{2} = 70$ mm,

$\frac{b}{t_F} = \frac{70}{12.4} = 5.645 < 9.4\epsilon$
$= 5.645 < 8.5082$

∴ Hence this is plastic Section.

$\frac{d}{t_w} = \frac{247.2}{7.5} = 32.96 < 84\epsilon$
$= 32.96 < 76.692$ — (plastic)

check for assuming Self weight :

weight of Section per metre length
$= 0.4336$ KN/m

$0.4336 < 1$ KN/m

∴ Hence Assuming is ok. (∴ OK)

check for shear strength :

Design shear $V = 93.319$ KN

Design shear strength $(V_d) = \frac{f_y h t_w}{\sqrt{3}\,\gamma_{mo}}$

$= \frac{300 \times 300 \times 7.5}{\sqrt{3} \times 1.1}$

$= 354.28$ KN.
$> V$

∴ OK

Design of Tubular Structure

The steel tubes or tubular steel section are commonly being used as a structural components and a large number of such structures having constructed in the past. Their used in truss members, mill building aeroplane hangers, truss bracings and beams.

Advantages & Disadvantages of tubular Steel Structure :

Advantages :

1. The tubular steel structure can be used economically where long span coloumn areas are required such as single storey industrial building, ware houses and shopping centre etc.

2. There use to advantage in structures designed for material handling equipment (Ex: Bridge, derrick and tower cranes) where weight saving may b

3. The tubular section have as much as 30-40% less surface area then that of an equivalent rolled steel sections and hence the cost of maintenance and other protective measures reduces considerabl.

4. particularly the circular and round section occur less resistance to wind, compact to their rolled section. The moisture and dirt also do not collect on the smooth external surfaces of tubes there by reducing the possiblity of corrosion.

As the ends of structural steel tube are generally sealed, the interior surface is not subjected to corrosion and also do not need any protective treatment.

5. Round tubular section are more efficient in compression as well as in bending having maximum value of radius of gyration.

6. Also they have more torsional resistance then the other sections of the equal weights.

7. The tube sections have higher frequency of vibrations under dynamic or earthquake loadings.

8. The uses of round tubular members have become popular and their special interest also to architects from aesthetic point of view and to engineers for their structural effectiveness.

Disadvantages :-

1. The pose difficulty in connections among themselves or to any plate element due to their shape problems.

2. Bolting and riveting on those sections are not convenient.

3. They required special devices for effective bearing on supports.

4. Their light weight sometimes become responsible for the structural instability.

5. Highly skilled man power and special welding technics are required for their connections.

• Depending upon the manufacturing process, the structural steel tubes may be categorized as,

1. Hot finished Seamless (H.F.S)
2. Cold finished Seamless (CFS)
3. Hot finished welded (HFW)
4. electric resistance welded (ERW) or Highly frequence induction welded (HFIW)

1. A tubular steel column 4.8 m. length is hinged at both the ends. It has nominal diameter of 225 mm. and confirmed to Yst 25 grade. Determine safe load carrying capacity of column.

Given data :-

$L = 4.8$ m.

both end hinged, $= 1L$

$L_{eff} = 4.8 \times 1 = 4.8$ m.

$d = 225$ mm ; $A = 4420\ mm^2$

$Y_{st} = 25$, Radius of gyration $= 84.4$ mm
from steel table
D = 225 mm

Slenderness ratio $(\lambda) = \frac{L_{eff}}{r}$

$$\lambda = \frac{4.8 \times 10^3}{84.4} = 56.87$$

λ/r	Fcd
50	125.5 N/mm2
56.87	?
60	120.7 N/mm2

$$F_{cd} = 125.5 - \left(\frac{125.5 - 120.7}{60 - 50}\right) \times (56.87 - 50)$$

$$F_c = 122.20\ N/mm^2$$

So, load carrying capacity $F = F_c \times A$

$= 122.20 \times 4420$

$= 540.124$ KN

2. A tension member, carrying a force of 40 KN meets the ~~mod~~ principle ~~sp~~ tie of tubular truss at an angle 45°. If the force in the principal tie is 100 KN, Design the members and the welded Joint between the two. use Yst 22 grade of steel and assume inside location (not exposed ~~to~~ to weather)

<u>Given data</u> :-

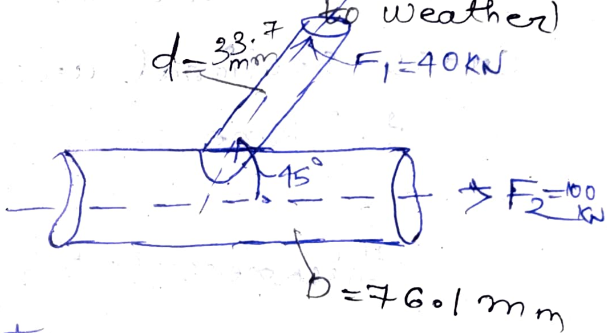
$F_1 = 40$ KN (tension)

$F_2 = 100$ KN (tension)

$\theta = 45°$

grade of steel Yst-22,

connection - welded Joint



- Design of tension member branch

a. For Yst 22 grade, the permissible stress $(F_t) = 125$ N/mm²

Area required for the tension member

$$(A_1) = \frac{F_t}{F_t}$$

$$= \frac{40 \times 10^3}{125}$$

$= 320$ mm²

Let us provide a heavy steel tube of Nominal bore $= 25$ mm, having cross sectional area $= 373$ mm² and outside dia $= 33.7$ mm,

thickness of member $= 4$ mm > 3.2 mm

$\therefore$ OK

b. Design of ~~tension member~~ principal tie main pipe,

$F_t = 125\ N/mm^2$

Area required for tension member

$(A_2) = \frac{F_2}{F_t}$

$= \frac{100 \times 10^3}{125}$

$A_2 = 800\ mm^2$

Let us provide a medium steel tube of nominal bore of = 65 mm, having a cross sectional area = 820 mm²

thickness = 3.6 mm.

thickness of member = 3.6 > 3.2 mm

∴ ok.

<u>Length of Connection</u> :

$L = a + b + 3\sqrt{a^2+b^2}$

$a = \frac{d}{2} \operatorname{cosec}\theta = \frac{33.7}{2} \times \operatorname{cosec}(45^\circ)$

$= 23.829$

$b = \frac{d}{3} \times \frac{3-\left(\frac{d}{D}\right)^2}{2-\left(\frac{d}{D}\right)^2}$

$b = \frac{33.7}{3} \times \frac{3-\left(\frac{33.7}{76.1}\right)^2}{2-\left(\frac{33.7}{76.1}\right)^2}$

$= 17.46\ mm$

$L = a + b + 3\sqrt{a^2+b^2}$

$= 23.829 + 17.46 + 3\sqrt{23.829^2 + 17.46^2}$

$L = \cancel{23.083} \ 129.913\ mm$

permissible shear stress in the weld,

Assuming fillet weld $= (F_s) = 90\ N/mm^2$

$90\ N/mm^2$ = permissible shear stress in parent metal.

Required throat thick, (t)

$$\text{Area} = t \times L$$

$$= \frac{P(\text{load})}{\text{Length} \times \text{stress}}$$

$$t = \frac{40 \times 10^3}{129.91 \times 90}$$

$$t = 3.42\ mm$$

throat thickness $(t) = 0.7 \times S$

$$\text{size of weld}, (S) = \frac{t}{0.7} = \frac{3.42}{0.7} = 4.88\ mm \simeq 5\ mm$$

Let us provide 5mm ~~the~~ of size of weld

Design of Masonry Structure

1. A ground floor masonry work is of 4m. clear height upto the bottom of the roof slab. Height of plinth above foundation footing = 0.8m.
If wall thickness is 30 c.m. Calculate the effective height and slenderness ratio for the following support condition. (Assume concrete D.P.C properly bonded with masonary)
1. full restrained at top and bottom
2. full restrained at one end and only lateral restrained at the other end.
3. partially restrained on both ends.
4. Full restrained at the bottom but have no restrained at top.

Given data :

Clear height = 4m.
Height of plinth above foundation footing = 0.8m.

$H = 4 + 0.8\,m. = 4.8\,m.$

wall thickness = 30 c.m. = 0.30 m.

i. full restrained at top and bottom :

- effective height (h) $= 0.75H$

$= 0.75 \times 4.8\,m.$

$= 3.6\,m.$

- Slenderness ratio (λ) $= \dfrac{\text{effective height}}{\text{thickness of wall}}$

$= \dfrac{3.6 \times \cancel{10}3}{\cancel{0}0.30} = 12$

ii. full restrained at one end & only lateral restrained at other end

• effective height $(h) = 0.85H$

$= 0.85 \times 4.8$

$= 4.08$ m.

• Slenderness ratio $(\lambda) = \dfrac{\text{effective length}}{\text{thickness of walls}}$

$= \dfrac{4.08}{0.30} = 13.6$

iii. partially restrained on both ends.

• effective height $(h) = 1.00H$

$= 1.00 \times 4.8$

$= 4.8$ m.

• Slenderness ratio $(\lambda) = \dfrac{\text{effective length}}{\text{thickness of walls}}$

$= \dfrac{4.8}{0.30} = 16$

iv. full restrained at the bottom but have no restrained at top.

• effective height $(h) = 1.50H$

$= 1.50 \times 4.8$

$= 7.2$ m.

• Slenderness ratio $(\lambda) = \dfrac{\text{effective length}}{\text{thickness of walls}}$

Slenderness ratio $= \dfrac{\cancel{4.8}\ 7.2}{0.30} = 24$

... masonry wall is of 4m. height and
6m. length. Calculate the effective
length of the wall for the following
support conditions.

1. wall is continues and supported by
a cross walls and there is no opening
within a distance of 0.5m. from the
face of the cross walls.

2. wall is supported by a cross wall at one
end and continues with cross wall at the
other ends.

3. wall is supported at each end
by cross walls.

4. wall is free at one end and is continues
with a cross wall at the other end.

5. wall is free at one end and supported
by a cross wall at the other end.

Ans :-

Given data :-

→ clear height = 4m.

→ length = 6m.

i. wall is continues and supported by
a cross walls and there is no opening
with a distance of $\frac{H}{8} = \frac{4}{8} = 0.5m.$
from the face wall

- effective length = 0.8L

$= 0.8 \times 6$

$= 4.8$ m.

ii. wall is supported by a cross wall at one
end and continues with cross wall at
other end.

- effective ~~len~~ length $= 0.9L = 0.9 \times 6 = 5.4$ m.

iii. wall is supported at each end by cross walls

effective length = 1.0 L
= 1.0 × 6 = 6 m.

iv. wall is free at one end and supported at other end by a cross walls.

effective length = 1.5 L
= 1.5 × 6 = 9 m.

v. where wall is free at one end and supported by a cross wall at other end

effective length = 2.0 L
= 2.0 × 6 = 12 m.

(Storage device - 3)