Lecture note on

Structural design-II (TH-2)

5th Semester

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1.1. Common Steel Structures

In early society, human beings lived in caves and almost certainly rested in the shade of trees. Gradually, they learnt to use naturally occurring materials such as stone, timber, mud & biomass to construct houses.

The principal modern building materials are masonary, concrete (mass, reinforced, and prestressed), glass, plastic, timber and structural steel.

The main advantages of structural steel are strength, speed of erection, pre-fabrication & demountability. Structural steel is used in load-bearing frames in buildings & as members in trusses, bridges & span frames.

Common Steel Structures are :-

i) Roof trusses for factories, cinema halls, auditoriums etc.

ii) Trussed bents, crane girders, columns etc. in industrial structures.

3) Roof trusses and columns to cover platforms in railway stations and bus stands.

4) Single layer or double layer domes for auditoriums, exhibition halls, indoor stadiums etc.

5) Plate girder & truss bridges for railways & roads.

6) Transmission towers for microwave & electric power.

7) Water tanks.

8) Chimneys etc.

Advantages & Disadvantages of steel structures

The advantages of steel over other materials for construction are :-

1) It has high strength per unit mass. Hence, even for large structures, the size of steel structural element is small, saving space in construction & improving aesthetic view.

2) It has assured quality & high durability

3) Speed of construction is another important advantage of steel structure. Since standard

sections of steel are available which can be pre-fabricated in the workshop / site, they may be kept ready by the time the site is ready & the structure erected as soon as possible. Hence there is a lot of saving in construction time.

4) Steel structures can be strengthened at any later time, if necessary. It needs just welding ~~adding~~ additional sections.

5) By using bolted connections, steel structures can be easily dismantled & transported to other sites quickly.

6) If joints are taken care, it is the best water & gas resistant structure.

7) Material is reusable.

The disadvantages of steel structures are :-

1) It is susceptible to corrosion.

2) Maintainance cost is high, since it needs painting to prevent corrosion.

3) Steel members are costly.

1.2 Types of Steel

(a) Carbon Steel :- Steels is an alloy of carbon & iron. Apart from carbon, by adding small percentage of manganese, sulphur, phosphorous, chrome nickel & copper, special properties can be imparted to iron & a variety of steels can be produced, as follows:-

(i) Carbon Steel :- Increased quantity of carbon & manganese imparts higher tensile strength & yield properties but lower ductility, which is more difficult to weld. Their yield strength varies from 230 to 300 MPa.

(ii) High-Strength Carbon Steel :- Such steel has a high carbon content & hence shows reduced ductility, toughness & weldability. This steel is specified for structures such as transmission lines & microwave towers. Their yield strength varies from 350 to 400 MPa.

(iii) High-strength, quenched & tempered steel:- These steels are heat treated to develop high strength. Though they are tough & weldable, they require special welding techniques. Their yield strength varies from 550 to 700 MPa.

(iv) Weathering steels :- These are low-alloy atmospheric corrosion-resistant steels, which are often left unpainted. They have yield strength of about 350 MPa.

(v) Stainless steels :- These are essentially low-carbon steels to which chromium & nickel are added. It improves resistance to high temperature also.

(vi) Fire resistant steel :- These are also called TMT (Thermo-mechanically treated) steels. They perform better than ordinary steel under fire.

Structural steel may be mainly classified as mild steel and high tensile steel. Structural steel is also known as standard quality steel.

Structural steel other than those specified as mild steel & high tensile steel conforming to weldable quality may also be used provided the permissible stress & other design provisions are suitably modified.

Properties of Structural Steel

The properties of steel required for engg. design may be classified as :—

(i) Physical properties.

(ii) Mechanical properties.

(i) Physical properties :-

(a) Unit mass of steel, $\rho = 7850 \, kg/m^3$.

(b) Modulus of elasticity, $E = 2.0 \times 10^5 \, N/mm^2$

(c) Poisson's ratio, $\mu = 0.3$.
(d) Modulus of rigidity, $G = 0.769 \times 10^5$ N/mm^2.
(e) Coefficient of thermal expansion, $\alpha_t = 12 \times 10^{-6}$ /°c

(ii) Mechanical properties :—
(a) Yield stress (f_y)
(b) Ultimate stress (f_u).

1.3 Rolled Steel Sections :—

The largest categories of standard shapes of structural steel include those produced by hot rolling. In this process, molten steel is taken from furnace & poured into a continuous casting system where the steel solidifies, but it never allows to ~~cro~~ cool completely.

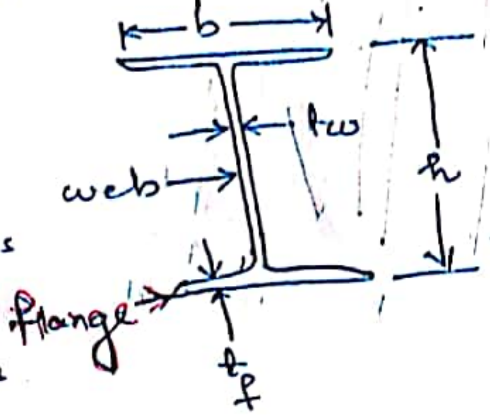
The hot steel passes through a series of rollers that squeeze the material into the desired cross-sectional shapes. Rolling the steel when it is still hot allows it to deform without any loss of ductility.

During ~~rolling~~ rolling process, the member increases in length & is cut to standard lengths, which are subsequently cut to the length required for a particular structure.

Cross-sections of some of more commonly used hot rolled shapes are listed below :—

(i) **Rolled Steel I-sections** (Beam sections)

The following five series of rolled steel I-sections are manufactured in India:—



(a) Indian Standard Junior Beams (ISJB).

(b) Indian Standard Light Beams (ISLB).

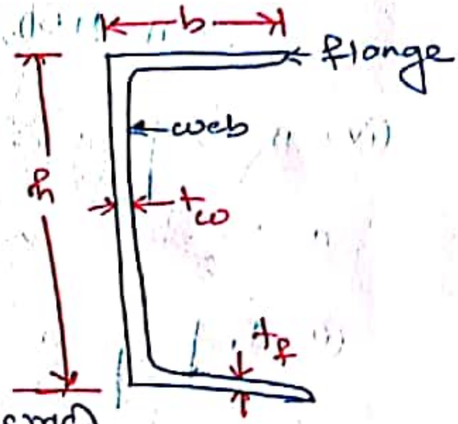
(c) Indian standard medium Beams (ISMB)

(d) Indian standard wide-flanged Beams (ISWB)

(e) Indian standard heavy beams (ISHB).

(ii) **Rolled steel Channel section**:

These are classified in following four series:—



(a) Indian standard Junior channel (ISJC)

(b) I.S. Light channel (ISLC)

(c) " Medium weight channel (ISMC)

(d) " Special channel (ISSC).

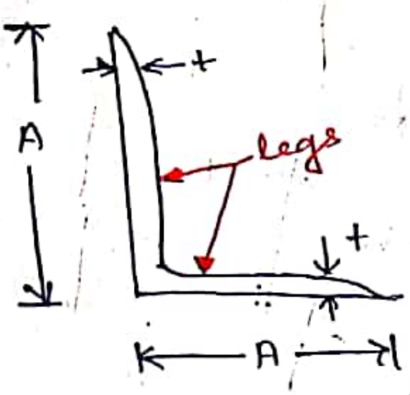
b = width
h = depth.

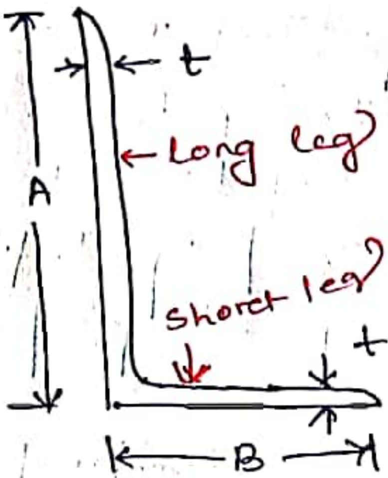
(iii) **Rolled Steel Angle Sections**

These are classified into following two series:—

(a) Indian Standard Equal Angle – ISA

(b) " Unequal " – ISA.





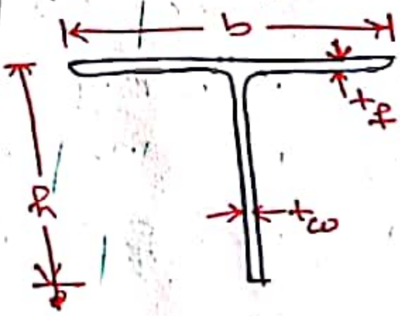
Thickness of legs of equal and unequal angles are same. They are designated by their series name ISA followed by length & thickness of legs. for eg:-

ISA 150 150 ; 12mm thick or ISA 150×150×12

ISA 150,115 , 10 mm thick or ISA 150×115×10

(iv) Rolled Steel Tee Sections :-

a. It is available in following five series :-



(a) Indian Standard normal Tee bars (ISNT)

(b) I.S. Heavy flanged Tee ~~beam~~ bars (ISHT)

(c) I.S. Special legged Tee bars (ISLT)

(d) " Light Tee bars (ISLT)

(e) " Junior Tee bars (ISJT).

(v) Rolled Steel Bars :-

These are classified into following 2 series :-

(a) Indian Standard Round bars - ISRO

(b) " " Square bars - ISSQ.

Special Considerations in steel design

Following special considerations are required in steel design.

1) **Size and Shape**:- Steel is manufactured in steel mills and is available in certain shapes and sizes. Hence, the member of a steel structure should be designed to consist of any of the available sections or a combination of them.

2) **Buckling Consideration**:- The permissible load per unit area in steel is much higher as compared to permissible value in concrete. Therefore, for the same load, the cross sectional area of a steel member is smaller. As the member in a steel structure are more slender, the compression members in steel structure are liable to buckling.

3) **Minimum Thickness**:- Corrosion needs special consideration in steel design. If very thin sections are used, a small amount of corrosion may result into a large percentage reduction in effective area. Hence, design practice specify minimum thickness to be used in structural members. For the members directly exposed to weather

following minimum thickness is to be used :-

(a) If fully accessible for cleaning and painting - 6 mm.

(b) If not accessible for cleaning and painting - 8 mm.

(c) The above limitations do not apply for rolled steel sections, tubes and cold formed light gauge sections.

However, IS 800-2007, has dropped the specification for minimum thickness.

4) <u>Need for design of connections</u> :- A steel design is not complete, if following connections are not designed :-

(a) Connections between various standard sections selected for a member.

(b) Connection between various members like beam, column, foundations etc. of the structure.

Commonly used connections are :-

(a) Rivetted connection
(b) Bolted connection
(c) Welded connection

1.4 Loads and Load Combinations

The forces that act on a structure are called loads. For the safe design of a structure it is essential to have knowledge of various types of loads & their worst combinations to which it may be subjected during it's life span.

Types of Loads

1) **Dead load** :- dead loads are examples of gravity loads. These are permanent loads & act vertically downward. For eg:- wt of structural elements like beams, columns, slab's etc.

2) **Live load** :- Live loads are those which may change in position and magnitude. For eg:- the furniture, equipments & occupants of the structure.

Some other examples of live loads are :- (a) impact load, (b) earth pressure, (c) Thermal loads, (d) water current load, (e) blast loads.

(a) Impact load :- When a live load is applied suddenly on a member, it experiences impact, like vibration of movable loads.

(b) Earth pressure :- In design of structures below ground level, eg:- basement, sheet piles, retaining walls etc., the pressure exerted by soil must be considered.

(c) <u>Water current load</u> :- The force exerted due to water current on the piers, abutments & other structures inside water must be taken into consideration.

(d) <u>Thermal forces</u> :- Due to fluctuation in temperature, the structural members expand or contract & produce some loading effects in the member, provided the ends are restrained.

(e) <u>Blast loads</u> :- It is caused by explosions and military weapons etc.

3) <u>Environmental loads</u> :- These are the loads caused by environment in which a particular structure is located. Forces due to wind, earthquake, snow, rain, temperature changes are the example of environmental loads.

(a) <u>Wind forces</u> :- All exposed structures irrespective of their heights are affected by wind forces. As wind blows against a structure, its surface experiences the effect of wind force. The wind pressure intensity of a structure depends upon velocity & density of air, shape & height of the structure, topography of the surrounding ground surface & the angle of wind attack.

(b) <u>Earthquake forces or Seismic forces</u> :- When a structure is subjected to ground motions in an earthquake, it responds in a vibratory fashion. Earthquake shocks cause movement of foundation structures. There are two methods used for computing seismic forces:—

(i) Seismic coefficient method

(ii) Response spectrum method.

(c) Snow & Rain loads :— Snow load is considered for buildings located in regions where snow is likely to fall. Snow load & rain load act on the roof vertically downwards & it is caused due to their accumulation of a structure.

(iv) Others

(a) Crane loads :— These loads include loads from cranes & other machines acting on the structure. The loads may be taken as per manufacturers or suppliers data.

(b) Dust load :— In areas prone to settlement of dust on roof (eg:- steel plants, cement plants) provision for dust load may be made.

(c) Erection load :— Prefabricated or precast members are subjected to different types of supports & different types of loads during erection compared to the types of supports & types of loads after erection.

(d) Accidental load :— Following accidental loads may be exerted on a structure :—

(i) Collision between vehicles, dropped objects from cranes, lifts, etc.

(ii) Explosion of gas or boilers or dynamite

(iii) Fire.

Load Combinations

A judicious combination of the loads is necessary to ensure the required safety & economy in the design keeping in view the probability of

(a) their acting together

(b) their disposition in relation to other loads and severity of stresses or deformation caused by the combination of various loads.

The recommended load combinations by IS 875 are 8 —

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

DL = Dead load
WL = wind load
TL = Temperature load
IL = Imposed load
EL = Earthquake load

1.5 Structural Analysis & Design Philosophy

Structural analysis is necessary to find the internal forces developed in the members of the structures. The required internal forces for design are axial forces & moments. IS code permits following methods of analysis:

(a) **Elastic Analysis** :- It is based on the assumption that no fibre of the member has yielded for the design load and stress is linearly proportional to strain. The analysis may be in two stages:-

stage-I :- First order analysis :- It is based on the loads acting on underformed geometry of the structure.

stage-II :- Second order analysis :- It is based on the deformed shape of the structure.

(b) **Plastic Analysis** :- In this method it is assumed that when every fibre at a section reaches yield stress, plastic hinge is formed. After plastic hinge is formed, it is assumed that the member rotates freely at the plastic hinge without resisting any additional moment. However, its resistance to moment remains constant.

(c) Advanced analysis :– For a frame with full lateral restraints, an advanced structure analysis may be carried out, provided the analysis can be shown to accurately model the actual behaviour of that class of frames. The analysis shall take followings into consideration :–

(i) Relevant material properties.
(ii) Residual stresses.
(iii) Geometric imperfections.
(iv) Reduction in stiffness due to axial compressions.
(v) Second order effects.
(vi) Erection procedures.
(vii) Interaction with foundations.

(d) Dynamic Analysis :– It is carried out for vibration & earthquake effect. The analysis is done in accordance with IS 1893 (part I).

Design Philosophy :–

The aim of design is to decide the shape, size & connection details of the members so that the structure being designed will perform satisfactorily during its intended life. With an appropriate degree of safety the structure should :–

(a) Sustain all loads expected on it.
(b) Sustain deformations during and after construction.
(c) Should have adequate durability.
(d) Should have adequate resistance to fire.
(e) Should be stable and have alternate load paths to prevent overall collapse under accidental loads.

The design philosophies used are listed below:—

(i) Working stress Method (WSM)
(ii) Ultimate load design (ULD)
(iii) Limit state Design. (LSD).

1.6 Brief Review Of Principles of Limit State design.

→ A structure may become unfit for use not only when it collapses but also when it violates the serviceability requirements of deflections, vibrations, cracks due to fatigue, corrosion and fire.

→ In LSM, various limits are fixed to consider a structure as fit.

→ This design is based on both probable load and probable strength.

→ Thus philosophy of LSM design is to see that structure remains fit for use throughout it's designed life by remaining within the acceptable limit of safety and serviceability requirements.

(1) Design Requirements :-

(a) The structure should remain fit with adequate reliability & be able to sustain all loads.

(b) Have adequate durability under normal maintance.

(c) Donot suffer overall damage or collapse under any accidental events like explosion, fire etc.

Limit States

⇒ Limit states are the states, beyond which the structure ~~has~~ no longer satisfies the specified performance requirements.

⇒ It is of two types:-

(a) Limit state of strength
(b) Limit state of serviceability.

(a) Limit State of Strength:-

↳ It prescribes to avoid collapse of the structure which may endanger safety of life and property.

↳ It includes:-

(i) Loss of equilibrium of whole or part of the structure
(ii) Loss of stability of structures as a whole or part of it.
(iii) Brittle fracture.
(iv) Fracture ~~by~~ due to fatigue.

(b) Limit State of Serviceability:-

It includes:-

(i) Deformation and deflections adversely affecting appearance or effective use of structure.

(ii) Vibrations in structures or any part of its component limiting its functional effectiveness.

(iii) Repairable damages or crack due to fatigue.

(iv) Corrosion

(v) Fire.

Q-10) Define ISA, ISMC, ISWB & ISMB. (04)

Q-1) what do you mean by rolled steel sections. - - - - - - - - (02)

Q-2) what are the types of rolled steel sections available in the market - - (02)

Q-3) Write down the advantages and disadvantages of steel structures - - - (05)

Q-4) why load combination is necessary in design - - - - - - - (02)

Q-5) what are the types of loads considered in design - - - - - (02)

Q-6) write the difference between LSM & WSM (05)

Q-7) Define characteristic strength of & design strength of material - - - (02)

Q-8) Define structural steel - - - - (02)

Q-9) what are the limit states available? (05)

1

STRUCTURAL STEEL FASTENERS & CONNECTIONS

=> The various elements of a steel structure like beams, columns etc. are connected by fasteners or connectors.

=> The forces exerted by one element on the other are transferred through these fasteners, which should therefore be adequate to transmit the forces safely.

=> (1) Different types of fasteners available in the design are :-

(a) Rivets
(b) Bolts
(c) Welds
(d) Pins.

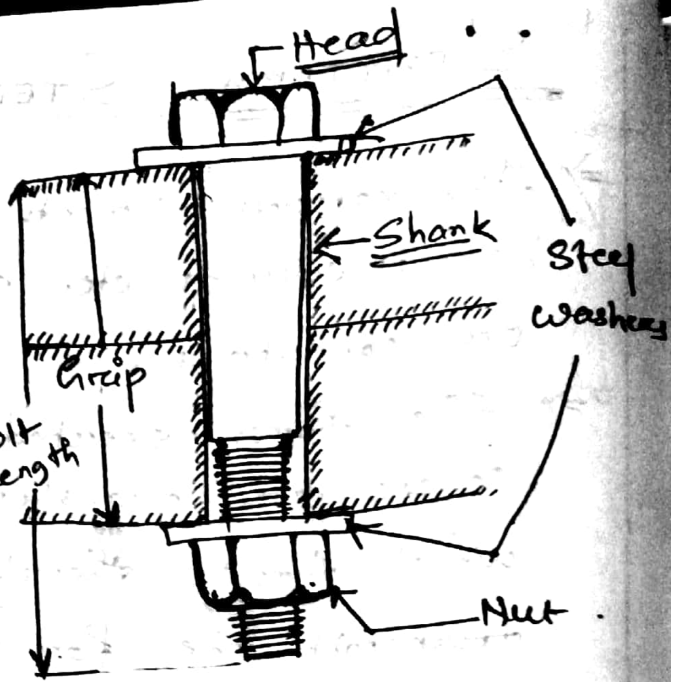
BOLTED CONNECTION

=> A bolt may be defined as a metal pin with a head at one end and a shank threaded portion at the other end to receive a nut.

=> Steel washers are usually provided under the bolts as well as under the nut to distribute the clamping pressure on the bolted member.

=> The washer also prevents ~~from~~ the threading to give large bearing pressure on the connecting members.

Advantages :-



(1) Making of joints using bolt is noiseless.

(2) Don't need skilled labourer.

(3) Needs less number of labourers for it's installation.

(4) These connections can be made quickly.

(5) Structure can be put to use immediately after connection.

(6) Alterations or changes in connection can be made easily if required.

(7) Area required for bolting is less.

Disadvantages :-

(1) Due to vibration, nuts are likely to loosen, endangering the safety of the structure.

(2) Gross area is reduced due to presence of bolt holes.

(3) Tensile strength is reduced considerably due to stress concentrations at the holes & due to reduction of area at the root of thread.

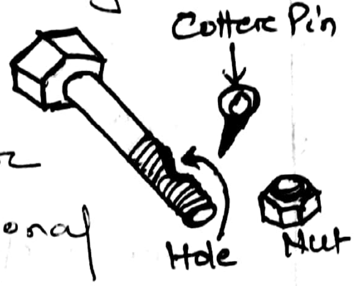
3
.1.1 Classification of Bolts.

⇒ There are several types of bolts used to connect the structural elements like :-

(a) Unfinished bolts
(b) Turned bolts
(c) Ribbed bolts
(d) High Strength bolts

(a) Unfinished Bolts :-

↳ These bolts are also called as ordinary, common, rough or black bolts. These are used for light structure & are not recommended for connection subjected to impact load, vibrations and fatigue.



↳ These bolts are made from low carbon, rolled steel, circular rod with square or hexagonal head, whereas ordinary bolts are made from mild steel.

↳ The bolt hole is punched 1.6mm more than bolt diameter.

↳ Some times a hole is drilled in the bolt & a cotter pin is used to prevent the nut from turning on the bolt.

↳ As bolt hole is more than bolt dia. & as the shank is unfinished, they may not establish contact with connecting members.

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(b) Turned Bolts.

→ These are similar to unfinished bolts with the difference that the shank of these bolts are formed from a hexagonal rod.

⇒ These bolts have high shear and bearing resistance as compared to unfinished bolts.

⇒ They are also called as Finished bolts.

(c) Ribbed Bolts.

→ The heads of these bolts are round as like rivets and the other end is provided with threads and nut.

→ From the shank core, longitudinal ribs project making the diameter of the shank more than the diameter of the hole.

⇒ These ribs ~~cut~~ cut grooves into the connected members while tightening and ensures a tight fit.

⇒ Hence these bolts have more resistance to vibrations as compared to ordinary bolts.

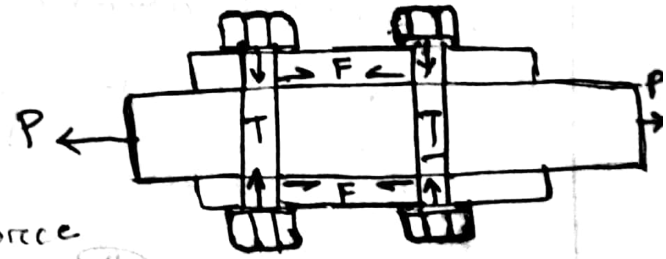
(d) High Strength Bolts / High Strength friction Grip bolts (HSFG Bolts):-

⇒ In normal bolts, the force is transferred through the interlocking and bearing of bolts.

⇒ However, for HSFG bolts, this force is accompanied with friction between the interface of ~~bolt~~ washer and connecting members. Hence these are also known as friction type bolts.

⇒ The shank of the bolt don't allow slippage in the joint and hence such bolts can be used to connect members subjected to dynamic loads also.

⇒ These bolts are made from bars of medium carbon steel.



⇒ In HSFG bolts, the nut is tightened to develop a clamping force on the plates which is indicated as the tensile force T.

⇒ Horizontal frictional force F, is induced in the joints which is equal to tensile force T multiplied with coefficient of friction μ i.e. $\boxed{F = \mu T}$

Advantages of HSFG bolts :-

(i) These provide a rigid joint & hence no slip takes place in the joint.

(ii) As load transfer is mainly by friction, the bolts are not subjected to shearing and bearing stresses.

(iii) Since nuts are prevented from loosening and stress concentration is avoided due to friction grip, they have high fatigue strength.

(iv) Smaller number of bolts are required.

Disadvantages of HSFG bolts :-

(i) Material Cost is high

(ii) Special attention is given to workmanship especially to give them right amount of tension.

BOLTS

- Bearing Type (Unfinished / black bolts, Finished / Turned bolts & Ribbed bolts.
- Friction Type (HSFG bolts)

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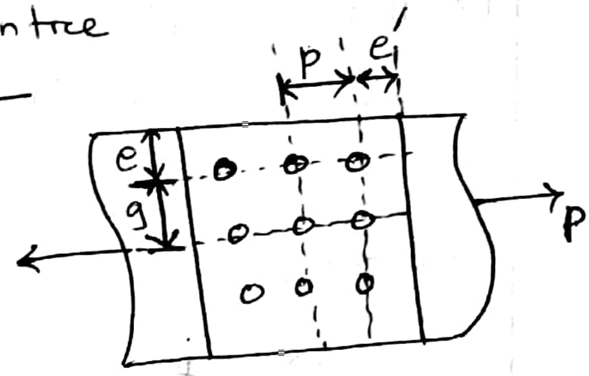
2.1.2 TERMINOLOGY

Pitch (P) :- It is the centre to centre spacing of the bolts in a row, measured along the direction of load.

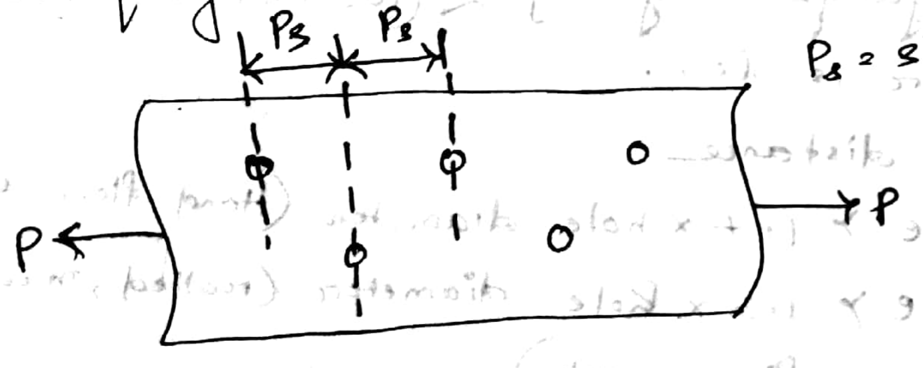


Gauge Distance (g) :- It is the distance between two consecutive bolts of adjacent rows and is measured at right angles to the direction of load.

Edge Distance (e) :- It is the distance of centre of bolt hole from adjacent edge of the plate measured at right angle to the direction of load.

End Distance (e') :- It is the distance of nearest bolt hole from end of the plate measured along the direction of load.

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



P_s = staggered pitch

IS Specifications a/c to IS 800 - 2007

1) Pitch shall not be less than 2.5d, where d is the nominal diameter of bolts

2) Pitch shall not be more than
(a) 16t or 200 mm whichever is less in case of tension member
(b) 12t or 200mm, whichever is less in case of compression member.

t = thickness of thinnest member.

3) In case of staggered pitch, pitch may be increased by 50% ~~spe~~ values ~~of~~ specified above, provided gauge distance is less than 75mm.

4) In case of butt joints maximum pitch is to be restricted to 4.5d for a distance of 1.5 times width of plate from butting surface.

5) The gauge length 'g' < (100 + 4t) or 200 whichever is less.

6) Edge distance
e > 1.7 × hole diameter (Hand flame cut)
e > 1.5 × hole diameter (rolled, machine flame cut).

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(7) $e < 12t\varepsilon$, where $\varepsilon = \sqrt{\frac{250}{f_y}}$

t = thickness of thinner outer plate.

$e < (40+4t)$; t = thickness of thinner connected plate, if exposed to corrosive influence.

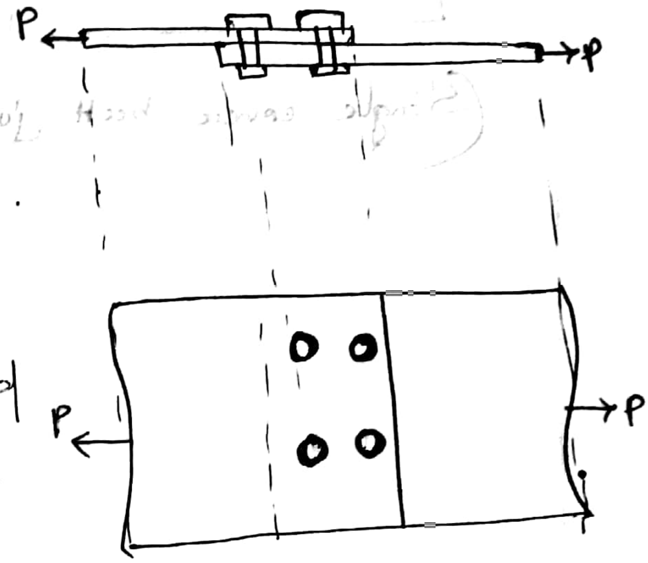
2.1.3 Types Of Bolted Connections

Bolted joints may be grouped into following types :—

(a) Lap joint

(b) Butt joint.

(a) Lap joint



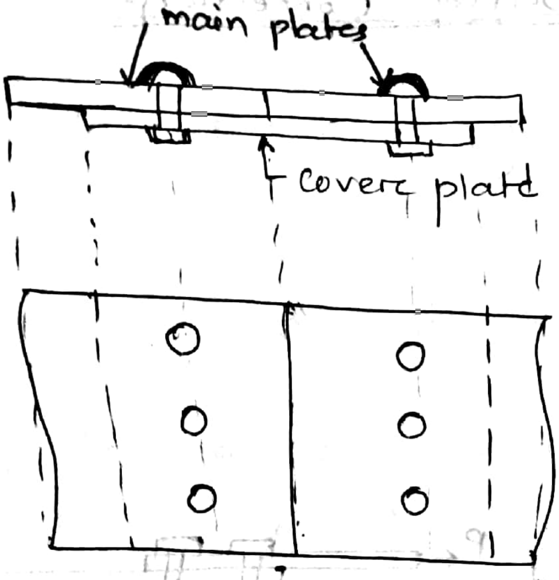
⇒ It is the simplest type of joint.

⇒ In this, the plates to be connected are overlapped with one another.

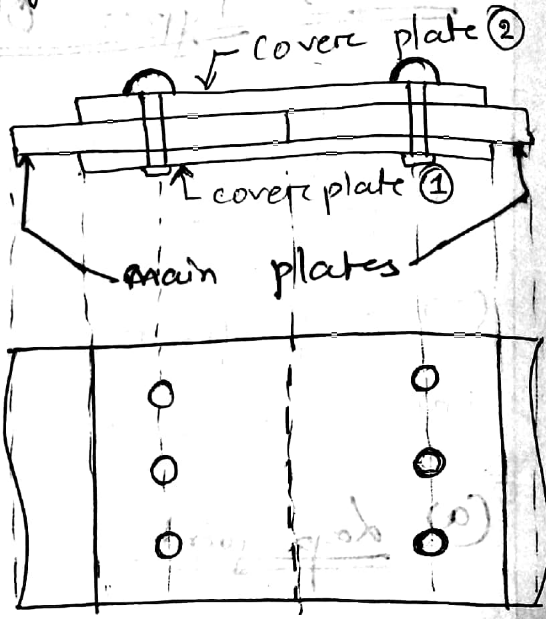
(b) Butt joint

⇒ In this type of connection, the two main plates to be connected are placed side by side & butting against each other..

→ The connection is made by providing a cover plate one one side or ~~may~~ ~~be~~ on both sides and connected to the main plate by bolting.



(Single cover butt joint)



(Double cover butt joint).

2.1.4 Types of actions on fasteners

Depending upon the types of connections and loads, bolts are subjected to following types of actions :-

(a) Only one plane subjected to shear (single shear)

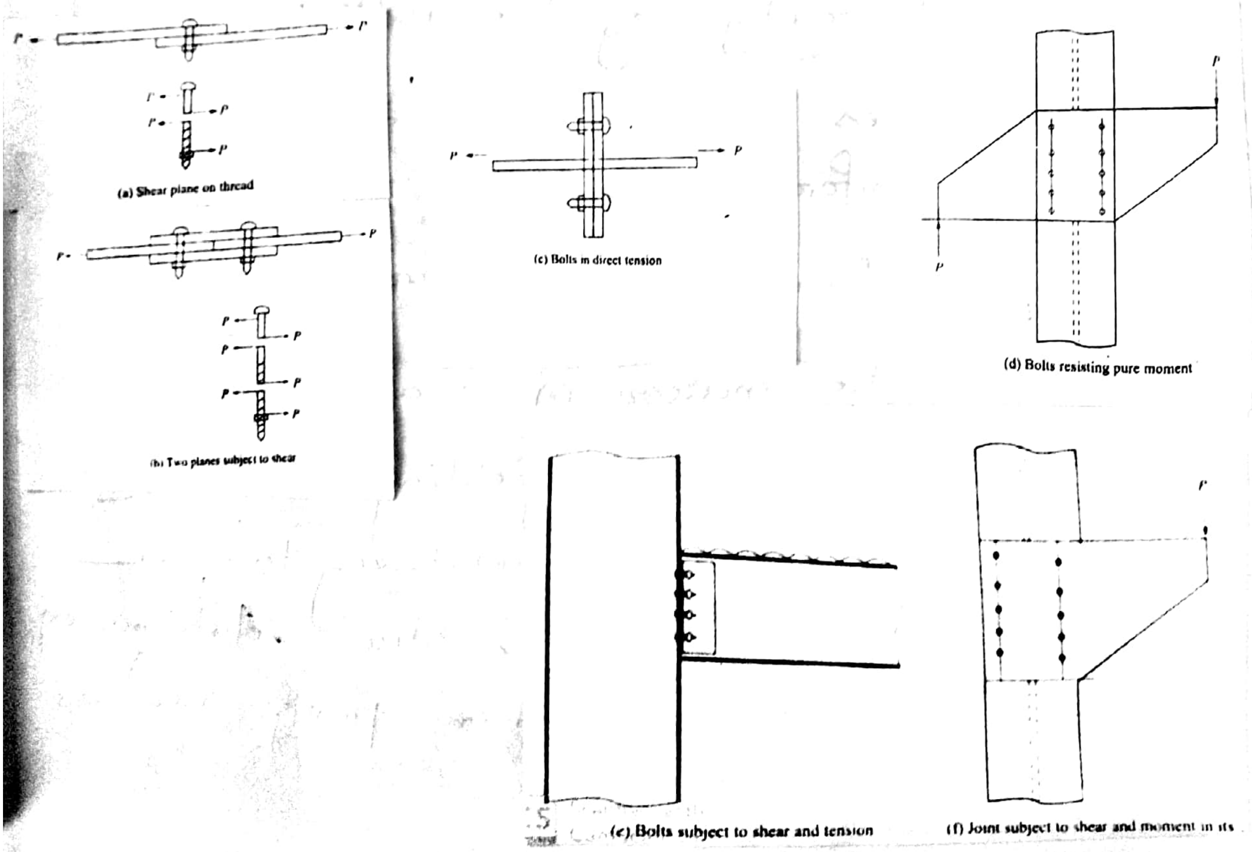
(b) Two planes subjected to shear (double shear)

(c) Pure tension

(d) Pure moment

(e) Shear & moments in the plane of connection

(f) Shear and tension.



(a) Shear plane on thread

(b) Two planes subject to shear

(c) Bolts in direct tension

(d) Bolts resisting pure moment

(e) Bolts subject to shear and tension

(f) Joint subject to shear and moment in its

2.1.4(b) ASSUMPTIONS IN DESIGN OF BEARING BOLTS

(1) Friction between the plates is negligible.
(2) The shear is uniform over the cross-section of the bolt.
(3) The distribution of stress on the plates between the bolt holes is uniform.
(4) Bolts in a group subjected to direct loads share the load equally.
(5) Bending stresses developed in the bolts are neglected.

Limitations :-

(1) Assumption-(1) is not correct, because the friction exists between the plates as they are held tightly by the bolts.

(2) Actual stress distribution in the plates are not uniform in working conditions. Stresses are very high near the bolt holes.

With the increase in load, the fibres near the holes start yielding & hence stresses ~~at are~~ start transferring to the whole member. At failure, stress distribution is uniform and all ~~members~~ part reaches to yield.

(3) The fourth assumption is questionable. Because the bolts far away from centre of gravity of bolt groups are subjected to more loads. But in a ultimate stage, when all bolts are about to fail, then bolts are found to b share load equally. Hence assumption-4 is not completely wrong.

(E).

2.1.4 (C) Principles of Designing bolted Connection

(1) The centre of gravity of bolts should coincide with the centre of gravity of the connected members.

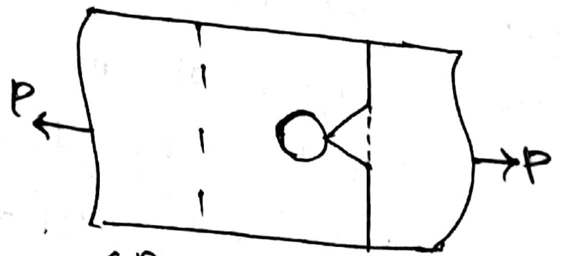
(2) The length of connection should be kept as small as possible.

2.1.5 (a) STRENGTH OF PLATES IN A JOINT

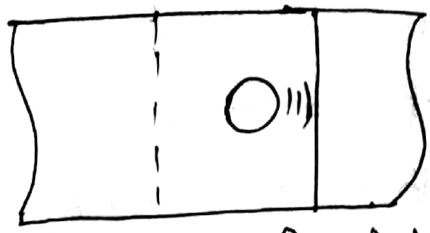
Plates in a joint made with bearing type bolts may fail under tensile force due to any of the following :—

1) Bursting or Shearing of edge
2) Crushing of Plates
3) Rupture of Plates.

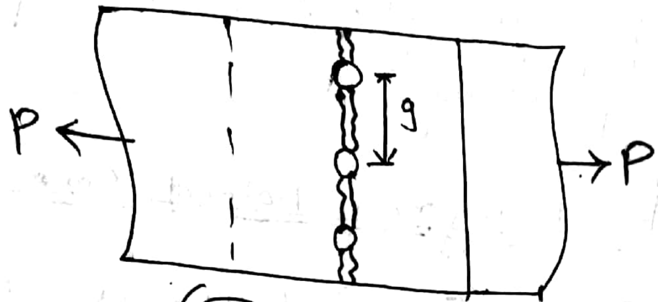
The ~~about~~ above failures are due to the disobeying of specifications like edge distance, end distance pitch & gauge.



(Bursting of plate)



(Crushing of plate)



(Rupture of plate)

⇒ Hence to avoid these failure minimum distances are provided.

⇒ If minimum distances are ensured in a joint, then the design tensile strength of plate is ~~the~~ taken as the strength of thinnest member :—

i.e. $$T_{dn} = \frac{0.9\, A_n f_u}{\gamma_{m1}}$$

T_{dn} = Strength of plate

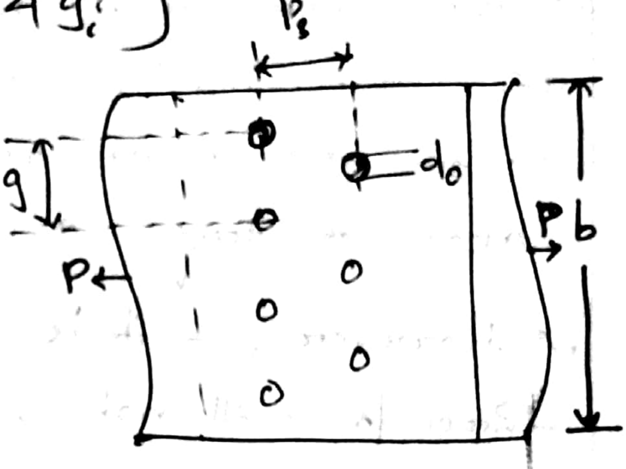
γ_{m1} = Partial safety factor for failure at ultimate stress = 1.25

f_u = ultimate stress of material.

A_n = net effective area of plates at

critical sections & is given by :-

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



b = width of plate

t = thickness of thinner plate

d_0 = diameter of bolt hole

g = gauge distance

p_s = staggered pitch
= 0 if staggering is not done.

n = number of bolt holes in critical section.

i = no. of legs connecting bolts obliquely.

when $p_{si} = 0$, $A_n = (b - n d_0) t$.

2.1.5 (b) Strength of Bearing type bolts

The design strength of bearing type of bolts are taken as the least of following :-

(i) Shear Capacity

(ii) Bearing Capacity.

(i) Shear Capacity :-

The failure of connections with bearing bolts in shear involves either bolt failure or the failure of connected plates.

⇒ The shearing of bolts can take place in the bolts and the threaded portion of the bolts. This so at the root area of threads. This area is taken as A_s i.e. shear area.

→ However, if it is ensured that the threads will not lie in the shear plane, then full area can be taken as shear area.

If V_{nsb} = nominal shear capacity of a bolt.

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

f_u = ultimate tensile strength of a bolt

n_n = no. of shear planes intersected by threads.

n_s = no. of shear plane without intersected by threads.

A_{sb} = nominal plain shank area of bolt $= \frac{\pi}{4} d^2$

A_{nb} = net tensile area at threads & it may be taken as area corresponding to root diameter. $= 0.78 A_{sb}$

If V_{sb} = factored shear force (external), then it should be $V_{sb} \leq \frac{V_{nsb}}{\gamma_{mb}}$

γ_{mb} = partial safety factor for bolt.

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6 (ii) **Bearing Capacity** :-

- If the strength of connected plates are more than that of bolts, then the failure of bolt can take place by bearing of plates on the bolts.
- If plate material is weaker than that of bolt, then failure will occur by bearing of bolt on the plate & the hole will elongate.

If V_{npb} = bearing strength of bolt,

$$V_{npb} = 2.5\, d\, t\, f_u \cdot K_b \quad \left[K_b = \text{smaller of } \frac{e}{3d_0}, \frac{p}{3d_0} - 0.25, \frac{f_{ub}}{f_u},\right]$$

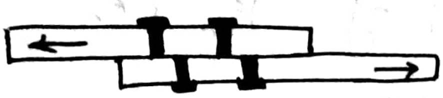
f_u = ultimate tensile stress of bolt

or " " " " plate

whichever is smaller

d = nominal diameter of the bolt.

t = summation of thicknesses of connected plates experiencing bearing stress.



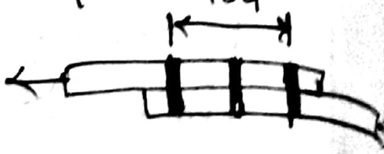
[Shearing failure of bolts]

[Bearing failure of plate.]

Reduction factors for shear capacity of Bolts :-

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

→ If the distance between first & last bolt in the joint measured in the direction of load exceeds 15 d, then the shear capacity V_{db} shall be reduced by a factor β_{lj} i.e.



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

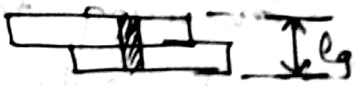
$$0.75 \leq \beta_{lj} \leq 1.0.$$

d = nominal diameter of bolt.

(ii) Reduction factor if grip length is large (β_{lg}).

→ If the total thickness of the connected plates exceeds 5d (d = nominal diameter of bolts), then the design shear capacity V_{db}, shall be reduced by β_{lg} i.e.

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



l_g = grip length.

= total thickness of connected plate.

$l_g < 8d$.

8 (iii) Reduction factor if packing plates are used (β_{pk})

If packing plates of thickness more than 6 mm are used in the joint, then the shear capacity shall be reduced by β_{pk} i.e.

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

t_{pk} : thickness of thicker packing plates.

So Now, ~~bear~~ shear capacity of ~~bolts~~ bolt can be written as :

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

~~Examples:~~

2.17 Efficiency of a joint

It is defined as the ratio of strength of joint to the strength of solid plate.

$$\eta = \frac{\text{strength of joint}}{\text{strength of solid plate}} \times 100$$

Bolt Grade

4.6 → 400 N/mm² ultimate & 60% of 400 N/mm² is yield strength

8.8 → 800 N/mm² " & 80% of 800 N/mm² " " "

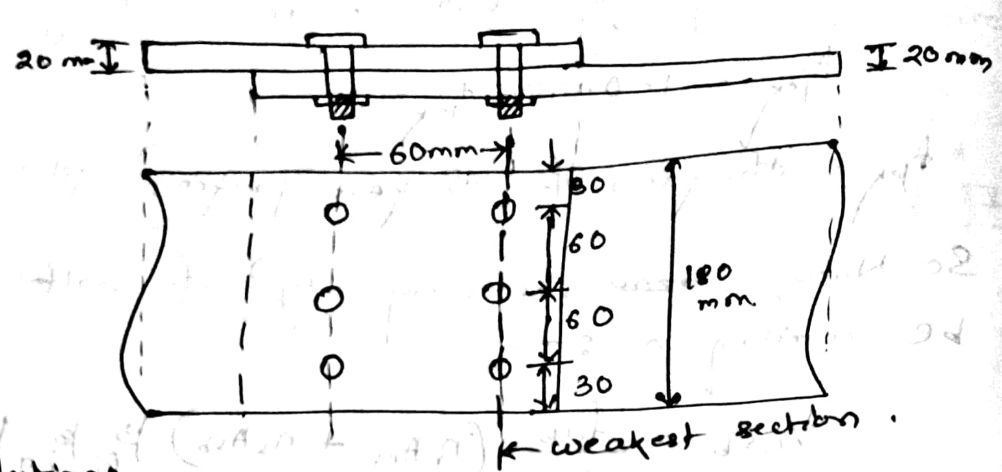
12.9 → 1200 N/mm² " & 90% of 1200 N/mm² " " "

Other grades are 4.8, 5.6, 5.8, 6.8, 8.8, 9.8, 10.9 & 12.9.

& HSFG.

Question :-

Find the efficiency of lap joint as given below. Given M20 bolts of grade 4.6 and Fe 410 (E 250) plates are used.



Solution:-

M20 grade 4.6 bolts & Fe 410 plate

M20 → dia of bolt.

4.6 → $f_{ub} = 400$ MPa ultimate tensile strength of bolt.

Fe 410 → $f_u = 410$ MPa ultimate ~~tensile~~ strength of plate

Given :- $d = 20mm$

so diameter of hole $\boxed{d_0 = d + 2mm}$

$d_0 = 22$ mm.

$f_{ub} = 400$ mm

$\gamma_{mb} = 1.25$

$f_u = 410$ MPa

$\gamma_{m_1} = 1.25$.

Available grades of plates are Fe 410 (E 250), Fe 410 (E 240), Fe 410 (E 230), Fe 290 (E 165), Fe 440 (E 300), Fe 490 (E 350), Fe ~~5~~540 (E 410), Fe 570 (E 450), Fe 590 (E

10

① Strength of plates in joint

Here thickness of thinner plate = 20mm = t.

width of plate b = 180 mm.

as no staggering is there, $\frac{p_{si}^2}{4g_i}$ = 0.

Number of bolts holes in the weakest section = 03.

Net area at the weakest section

$$A_n = (b - n d_0 + 0) t$$

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

$$= 2280 \ mm^2.$$

(i) Design strength of plates in joint

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

$$= \frac{0.9 \times 410 \times 2280}{1.25} = 673056 \ N$$

$$= 673.056 \ KN$$

② Strength of Bolts

(a) Design Shear Strength of bolt

Number of shear planes at thread n_n = 1 per bolt.

No. of shear plane at shank = 0 per bolt.

So total $n_n = 1 \times 6 = 6$ & $n_s = 0$.

$$\boxed{A_{nb} = 0.78 \frac{\pi}{4} d^2}$$

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$A_{nb} = 0.78 \times \frac{\pi}{4} \times 20^2 = 245\ mm^2$.

here, reduction factors could not be applied as ① distance between end bolts = 60 mm

& $15d = 15 \times 20 = 300$ mm.

② $l_g = 5d = 5 \times 20 = 100$ mm

& here grip = 20 + 20 = 40 mm

③ No packing plate has been used.

So $\beta_{lj} = \beta_{lg} = \beta_{pk} = 1$.

So nominal shear strength is

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

$$= \frac{400}{\sqrt{3}} (6 \times 245 + 0) = 339482\ N$$

$$= 339.482\ kN$$

Design shear strength $V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}}$

$$\Rightarrow V_{dsb} = \frac{339.482}{1.25} = 271.586\ kN.$$

(b) <u>Design strength in Bearing</u>

Nominal bearing strength, $V_{npb} = 2.5 k_b d t f_u$.

Here k_b = least of following

(a) $\frac{e}{3d_0} = \frac{30}{3 \times 22} = 0.4545$

(b) $\frac{p}{3d_0} - 0.25 = \boxed{\frac{60}{3 \times 22} - 0.25 = 0.659}$

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(c) $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.9756$

(d) 1.0.

$K_b = 0.4545$.

So $V_{npb} = 2.5 \times 0.4545 \times \overset{d}{20} \times \overset{t}{20} \times 400$

$=$ ~~186345~~ 144000 N per one bolt.

So for 6 nos. of bolts,

$V_{npb} = 6 \times 186345$.

Design bearing strength $= \frac{V_{npb}}{\gamma_{mb}}$

$= \frac{186345 \times 6}{1.25} = 894456.8$ N

$= 894.456$ KN.

Strength of joint = minimum of shearing strength & tensile & bearing strength of plate.

$= 271.586$ KN.

Efficiency of joint.

(Design strength of solid Plate $= \frac{f_y}{\gamma_{m0}} \times A_g$)

$A_g = 180 \times 20 = 3600$ mm^2.

(gross area of plate).

So strength of plate $= \frac{250}{1.1} \times 3600 = 818181.8$ N

$= 818.18$ KN.

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

$= \frac{271.586}{818.18} \times 100 = 33.19\%$.

Ques Find the efficiency of joint made by two ~~p~~ main plates of butt joint using two cover plates 20 mm thickness each & two cover plates of 12 mm thickness each. Connection was made using 6 nos. of ~~bolts of~~ M20 bolts of grade 4.6 and plates are of Fe 410 (E250), width of plate = 180mm.

Given

Thickness of main plates = 20 mm

" " Cover " = 12 mm

No. of bolts = 6 nos.

$d = 20$ mm (Nominal dia. of bolts).

$d_0 = 22$ mm

4.6 → $f_{ub} = 400$ N/mm²

→ $f_{yb} = 60\%$ of $f_{ub} = 240$ N/mm².

Fe410(E250) → $f_y = 250$ (yield strength of plate)

→ $f_u = 410$ (ultimate " " ").

$\gamma_{m1} = 1.25$ & $\gamma_{mb} = 1.25$.

(a) Strength of plate

Same as previous.

$\boxed{T_{dn} = 673.056 \text{ KN}}$, where $A_n = (b - nd_0)\,t$ & here t is taken as 20mm not 12mm as 12 mm is the thickness of cover plate & for two cover plates thickness = 12+12 = 24mm

14 (b) Strength of Plates,

(i) Design sheare strength of Bolt

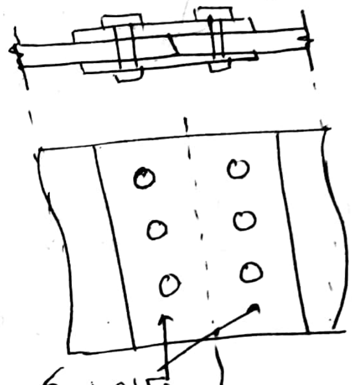
~~Asb = the~~ In butt joint, in each bolt a section remains in root & anothere in shank. Thus, here the total nos. of section resisting shear at shank, $n_s = 6\ (3+3)$ & $n_n = 6$.

So here,

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

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

$$= 314.16\ mm^2$$



6 bolts with 12 nos. of section crossing shear area. (6 - shank + 6 - root/thread).

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

$$= 0.78 \frac{\pi}{4} d^2 = 245\ mm^2$$

So Nominal Sheare strength

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

$$= \frac{400}{\sqrt{3}} (6 \times 245 + 6 \times 314.16) = 774795 N$$

$$= 774.795\ KN.$$

Design sheare strength $V_{dsb} = \dfrac{774.795}{1.25}$

$$\boxed{V_{dsb} = 619.836\ KN.}$$

(reduction factores are not applicable)

(ii) Design bearing strength of Bolt

Same as previous

$$\boxed{V_{dpb} = 894.456\ KN}$$

Strength of joint are the strength of connection is taken as minimum of T_{dn}, V_{dsb} & V_{dpb}.

So Strength of joint = V_{dsb} = 619.836 KN.

Design Strength of Solid plate

$$= \frac{f_y \times A_g}{\gamma_{m0}} = \frac{250}{\gamma_{m0}} \times (180 \times 20)$$

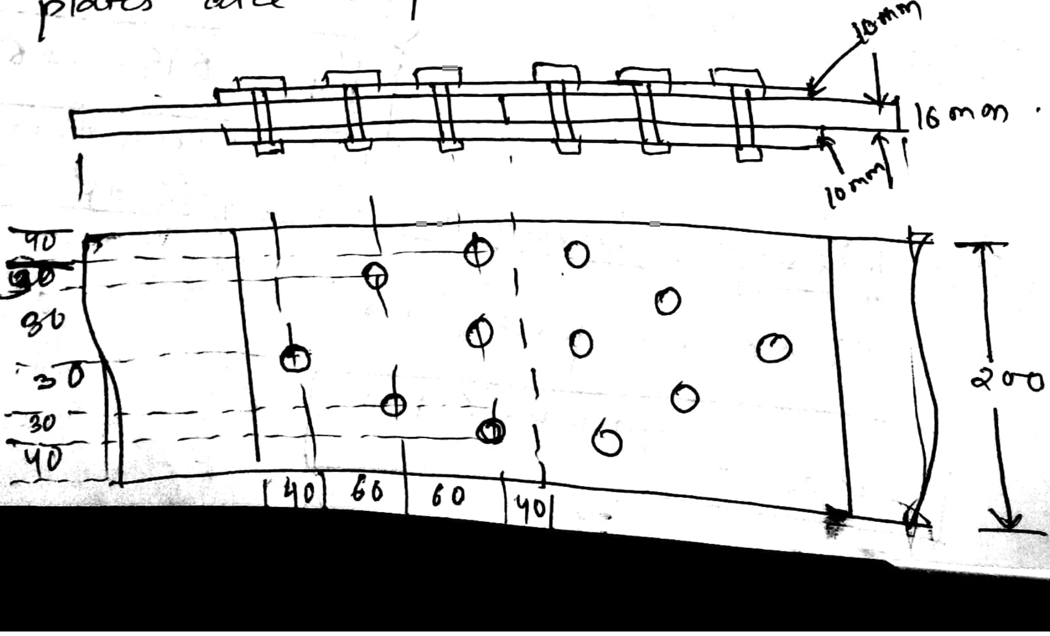
$$= \frac{250 \times 180 \times 20}{1.1} = 818182 N$$

$$= 818.182 KN.$$

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

$$= \frac{619.835}{818.182} \times 100 = 75.76\%.$$

Q-3 Find the maximum force, which can be transferred through the double covered butt joint in fig. Find also the efficiency of joint. Given M20 bolts of grade 4.6 & Fe 410 steel plates are used.



16 Given

$d = 20mm$, $d_0 = 22mm$

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

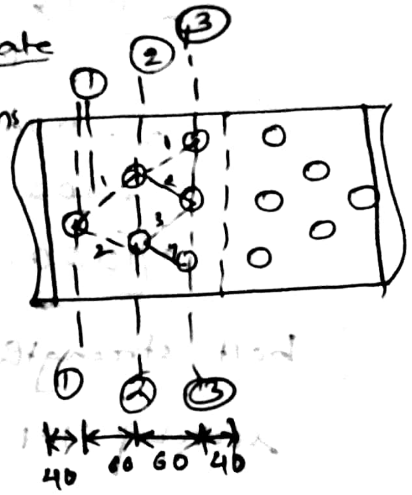
$f_u = 410 N/mm^2 \rightarrow$ Plate.

(a) Design tencile streength of plate

It is to be checked in 3 sections

Hence $t = 16 mm$.

(least thickness amgeing thickness of main plate & sum of thickness of both covere plates).



$f_u = 410 N/mm^2$.

(i) At Section 1 - 1

$$T_{dn_1} = \frac{0.9 A_n f_u}{\gamma_{m1}} = \frac{0.9 f_u}{\gamma_{m1}} \times A_n. \quad [A_n = (b - nd_0)t] \quad [\text{as } i = 0, \; \text{sop}_{si} = 0]$$

$$= \frac{0.9 \times 410}{1.25} \times \{(200 - 1 \times 22) \times 16\}$$

$$=$$

(ii) At Section 2 - 2

$$A_n = \left[b - nd_0 + \Sigma \frac{P_{si}^2}{4g_i}\right]t$$

$$= \left[\{200 - (2 \times 22)\} + \left\{\frac{P_{s1}^2}{4g_1} + \frac{P_{s2}^2}{4g_2}\right\}\right] \times t$$

$$= \left[(200 - 44) + \left\{\frac{60^2}{4 \times 30} + \frac{60^2}{4 \times 30}\right\}\right] \times 16.$$

$$= \left[(200 - 44) + \left(\frac{60^2}{4 \times 30} \times 2\right)\right] \times 16$$

$$= 3456 \; mm^2$$

$$T_{dn_2} = \frac{0.9 A_n f_u}{\gamma_{m1}} + \text{strength of bolt at section 1-1}$$

$$= \frac{0.9 \times 3456 \times 410}{1.25} + \text{bolt strength}$$

$$= 1020211.2 + \text{bolt strength}$$

[∵ bolt at 2-2 will fail only after failure of bolt at 1-1. So bolt at 1-1 is responsible in giving strength to section - 2 - 2]

$$= [1020.211 + \text{bolt strength of one bolt}] \, KN.$$

bolt strength of one bolt present at section 1-1 = minⁿ of shear strength V_{dsb} & bearing strength V_{dpb}.

at section 3 - 3

$$T_{dn_3} = \frac{0.9 A_n f_u}{\gamma_{m1}} + \text{bolt strength of bolt present before section 3-3}$$

$$= \frac{0.9 A_n f_u}{\gamma_{m1}} + 3 \times \text{bolt strength of one bolt,}$$

Hence
$$A_n = \left[(b - n d_0) + \left[\frac{P_{s1}^2}{4g_1} + \frac{P_{s2}^2}{4g_2} + \frac{P_{s3}^2}{4g_3} + \frac{P_{s4}^2}{4g_4}\right]\right] \times t$$

$$= \left[(200 - 3 \times 22) + \left(\frac{60^2}{4 \times 30} \times 4\right)\right] \times 16$$

$$= 4064$$

so
$$T_{dn_3} = \frac{0.9 \times 4064 \times 410}{1.25} + \text{bolt strength} \times 3$$

$$= 1199692.8 + 3 \times \text{bolt strength}$$

18 (b) **Strength of Bolt**

(i) **Design Shear Strength** :-

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

$$= \frac{400}{\sqrt{3}\times 1.25}\left(6\times\frac{\pi}{4}\times d^2 + 6\times\frac{\pi}{4}\times d^2\times 0.78\right)$$

$$= \frac{400}{\sqrt{3}\times 1.25}\left(6\times\frac{\pi}{4}\times 20^2\,(1+0.78)\right)$$

$$= \frac{400}{\sqrt{3}\times 1.25}\times 6\times\frac{\pi}{4}\times 20^2\times 1.78$$

$$= 619884.07\ N = 619.884\ kN.$$

(ii) **Design bearing strength**

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

at 3-3

$$K_b = \min^m \text{ of } \frac{e}{3d_0},\ \frac{p}{3d_0} - 0.25,\ \frac{f_{ub}}{f_u},\ 1.$$

$$= \min^m \text{ of } \frac{40}{3\times 22},\ \frac{60}{3\times 22} - 0.25,\ \frac{400}{410},\ 1$$

$$= \min^m \text{ of } 0.606,\ 0.659,\ 0.97,\ 1$$

$$= 0.606.$$

at 2-2

$$K_b = \min^m \text{ of } \frac{70}{3\times 22},\ 0.659,\ 0.97,\ 1$$

$$= \min^m \text{ of } 1.06,\ 0.859,\ 0.97,\ 1$$

$$= 0.659$$

at 1-1

$$K_b = \min^m \text{ of } \frac{100}{3\times 22} = 1.51,\ 0.659,\ 0.97,\ 1$$

$$= 0.659$$

so design bearing strength of bolt

$$V_{dpb} = \frac{1}{\gamma_{mb}}\left[\left(2.5 \times K_{b_1} \times d t f_u \times 3\right) + \left(2.5\, K_{b_2}\, d t f_u \times 2\right) + \left(2.5\, K_{b_3}\, d t f_u \times 1\right)\right]$$

$$= \frac{1}{1.25}\left[(2.5 \times 0.6069 \times 20 \times 16 \times 410) \times 3 + \left\{(2.5 \times 0.659 \times 20 \times 16 \times 410) \times 3\right\}\right]$$

$$= \frac{1}{1.25}\left[2.5 \times 20 \times 16 \times 410 \left(0.6066 \times 3 + (0.659 \times 3)\right)\right]$$

$= 995808$ N

$= 995.808$ KN

So bolt strength $= V_{dsb}$

$= 619.884$ KN.

Now $T_{dn_1} =$

$T_{dn_2} = 1020\cancel{21}.211 + 619.884$

$=$

$T_{dn_3} = 1199692.8 + 3 \times 619.884$

$=$

So, Strength of joint $= 619.884$ KN.

& Maximum force that can be transferred safely $= 619.884$ KN.

Now, Permissible load = $\frac{619.884}{1.5} = 413.257$ KN.

Design Strength of Solid Plate = $\frac{f_y}{\gamma_{m0}} \times A_g$

$$= \frac{250}{1.1} \times (200 \times 16) = 727272 \text{ N}$$

$$= 727.272 \text{ KN}.$$

Efficiency of the joint = $\frac{619.884}{727.272} \times 100$

$$= 85.23\%$$

Tension Capacity of Bolts

IS 800-2007, clause 10.3.5

⇒ Nominal tension Capacity of bolt.

$$T_{nb} = 0.9 A_n f_{ub} \leq f_{yb} A_{sb} \frac{\gamma_{mb}}{\gamma_{m0}}$$

⇒ Design tension Capacity $T_{db} = \frac{T_{nb}}{\gamma_{mb}}$

$$T_{db} = \frac{0.9 A_n f_{ub}}{\gamma_{mb}} \leq \frac{f_{yb} A_{sb}}{\gamma_{m0}}.$$

f_{ub} = ultimate tensile stress of bolt.

f_{yb} = yield stress of bolt.

A_n = Net area of rest of bolt = $0.78 \frac{\pi}{4} d^2$.

A_{sb} = shank area of bolt = $\frac{\pi}{4} d^2$

$\gamma_{mb} = 1.25$ $\gamma_{m0} = 1.1$

If T_b is external factored tensile force then

$$\boxed{T_b \leq T_{db}}$$

Bolt Subjected to Combined Shear and Tension

IS 800:2007, clause 10.3.6, a bolt required to resist both design shear force V_{sb} and design tensile force T_b at the same time shall satisfy the following :-

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

V_{db} = design shear strength
T_{db} = design Tensile strength.

Ques :- ① Design a lap joint between two plates each of width 120 mm, if the thickness of one plate is 16 mm & the other is 12 mm. The joint has to transfer a design load of 160 KN. Plates are of Fe410 grade. Use bearing type bolt.

Required - No. of bolts.

↓

$$\text{No. of bolts} = \frac{\text{total design load}}{\text{strength of one bolt}}$$

↓

find Strength of 1 bolt.

↓

Minimum of

shear strength & bearing strength

↓ V_{dsb} ↓ V_{nsb}

↓ V_{dpb} ↓ $V_{d\,npb}$.

(1) Shear Strength

Since it is a lap joint, bolt is in single shear.

Nominal Strength of bolt in shear $= \frac{f_{ub}}{\sqrt{3}} \left(1 \times 0 + 0.78 \times \frac{\pi}{4} \times d^2 \times 1\right)$

$$= \frac{400}{\sqrt{3}} \left(0 + 0.78 \times \frac{\pi}{4} \times 16^2\right)$$

$$= 36218 \text{ N}.$$

adopting 16 mm dia bolts of grade 4.6

$d = 16$ mm, $d_0 = 18$ mm & $f_{ub} = 400$ mm.

So Design Shear Strength of one bolt $= \frac{V_{nsb}}{\gamma_{mb}}$

$$= \frac{36218}{1.25} = 28974 \text{ N}.$$

(2) Bearing Strength

adopt minimum edge distance $e = 1.5\, d_0$
$= 1.5 \times 18$
$= 27$ mm.

& minimum pitch $= 2.5d = 2.5 \times 16 = 40$ mm

take $e = 30$ mm $p = 40$ mm.

K_b is least of $\frac{e}{3d_0}, \frac{p}{3d_0} - 0.25, \frac{f_{ub}}{f_u}, 1$.

$$= \text{least of} \left(\frac{30}{3 \times 18}, \frac{40}{3 \times 18} - 0.25, \frac{400}{410}, 1\right).$$

= least of (

= 0.4907.

Hence, Nominal bearing strength $= 2.5 K_b d t f_{ub}$

$= 2.5 \times 0.4907 \times 16 \times 12 \times 400$

$= 94222 N.$

Design bearing strength $= \frac{V_{npb}}{\gamma_{mb}}$

$= \frac{94222}{1.25} = 75378 N.$

③ Strength of One bolt

Design strength of one bolt

$=$ minimum of V_{dsb} & V_{dpb}

$= 28974$ N.

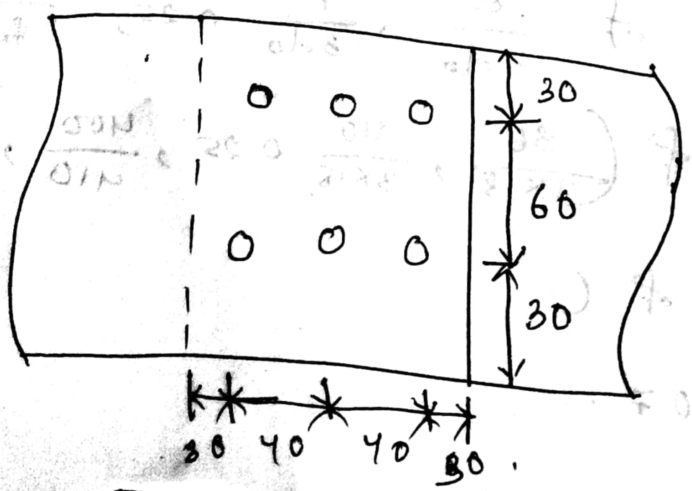
$= 28.974$ KN.

④ No. of bolts =

No. of bolts $= \frac{\text{total design load}}{\text{strength of one bolt}}$

$= \frac{160}{28.974} = 5.5.$

Provide 6 nos. of bolts in 2 rows with a pitch of 40mm.



(5) Check for tensile strength of plate

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

$$= \frac{0.9 \times \{(120 - 2\times18)\times12\} \times 410}{1.25}$$

$= 297562$ N $= 297.562$ KN > 160 KN

Strength of plate — external design load.

Solve - Example 3.8, Page - 65.

Shear Capacity Of HSFG Bolts.

⇒ The HSFG Bolts are made of high tensile steel which are pretensioned & provided with nuts. Hence shear force is mainly resisted by friction. & they slip at higher load.

⇒ IS 800: 2007 clause 10.4, recommends the shear capacity of HSFG bolts is

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

μ_f = Coefficient of friction or slip factor.

n_e = number of effective interfaces offering frictional resistance to slip.

[$n_e = 1$ for lap joint & $n_e = 2$ for double cover butt joint

$K_h = 1.0$ for fasteners in clearance holes.

$= 0.85$ for fasteners in oversized and short slotted holes loaded perpendicular to the slot.

K_h = 0.7 for fasteners in long slotted holes loaded parallel to the slot.

F_0 = minimum bolt tension at installation
= $A_{nb} f_0$.

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

f_0 = proof stress = 0.70 f_{ub}.

design shear capacity or the slip resistance

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

γ_{mf} = 1.1 for parallel shank HSFG & if slip resistance is designed at service load
= 1.25 for waisted shank HSFG & if slip resistance is designed at ultimate load.

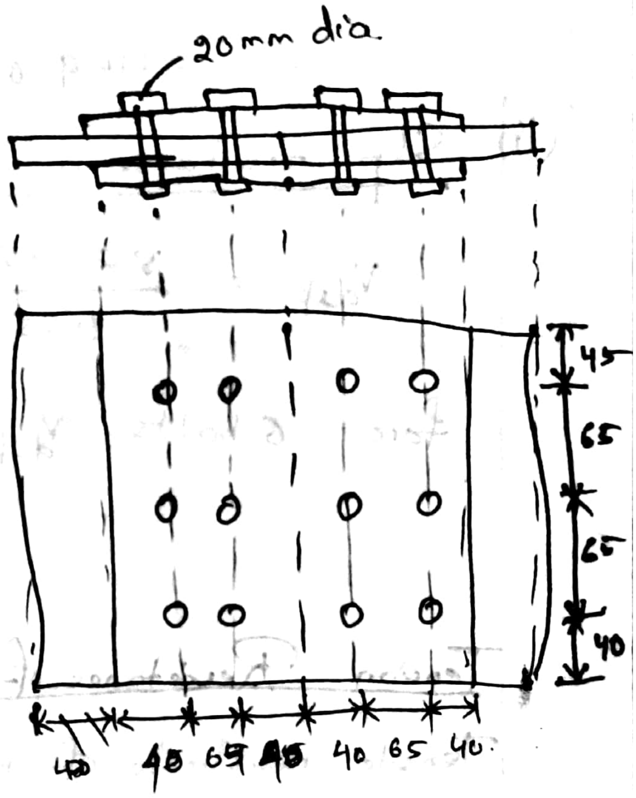
⇒ The reduction factors for shear strength of HSFG bolts are same as that of bearing bolts.

⇒ For commonly used HSFG bolts (grade 8.8)
yield stress = f_{yb} = 640 N/mm² & ultimate stress = 800 N/mm² = f_{ub}.

Determine the shear capacity of bolts used in connecting two plates as shown in fig., if

(i) slip resistance is designed at service load

(ii) " " " " " ultimate " .



Sol^n :-

Given

$e = 45\,mm$

$p = 65\,mm.$

Adopt $\mu_f = 0.3$.

$K_h = 1.0$.

HSFG Bolt of grade 8.8.

So $f_{ub} = 800\,N/mm^2$.

Nominal Shear Capacity of one bolt

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

$F_o = 0.7 f_{ub} A_{nb}$.

$= 0.7 \times 800 \times 0.78 \times \frac{\pi}{4} \times 20^2 = 137225\,N$

as it is a double cover butt joint, $n_e = 2$ for one bolt.

so $V_{nsf} = 0.3 \times 2 \times 1.0 \times 137225$

$= 82335\,N.$

(i) Slip is designated at service load

$V_{dsf} = \frac{V_{nsf}}{1.1} = 74850\,N.$

and design shear capacity (V_{dsf}) of 6 bolts is

$$V_{dsf} = (6 \times 74850)\,N$$

$$= 449099\ N$$

$$= 449.099\ kN.$$

(ii) Slip resistance is designated by at ultimate load

$$V_{dsf} = \frac{82335}{1.25} = 65868\ N$$

for 6 bolts, $V_{dsf} = 6 \times 65868$

$$= 395208\ N$$

$$= 395.208\ kN.$$

Tension Resistance of HSFG Bolts (10.4.5)

Tensile resistance of HSFG Bolts is same as that of bearing bolts.

$$T_{nf} = 0.9 f_{ub} A_n \leq f_{yb} A_{sb} \frac{\gamma_{mb}}{\gamma_{mo}}$$

$$\text{or } T_{df} = \frac{0.9 f_{ub} A_n}{\gamma_{mb}} \leq \frac{f_{yb} A_{sb}}{\gamma_{mo}}.$$

Interaction formula for combined shear and tension for HSFG Bolts. (10.4.6).

HSFG Bolts under both shear & tension must satisfy,

$$\left(\frac{V_{sf}}{V_{df}}\right)^2 + \left(\frac{T_f}{T_{df}}\right)^2 \leq 1.0.$$

Questions

1) List the assumptions made in the design of bearing bolts along with their limitations (4).

2) What do you mean by slip critical connections? Explain principle of HSFG bolts (4)

3) Find the maximum force, that can be transferred through double bolted chain lap joint consisting 6 bolts in two rows. Given M16 bolts of grade 4.6 and plates of Fe410 are used. Thickness of connected plates are 10mm & 12 mm. (8).

4) Write short note on:—
(i) Properties of structural steel. (3.5)
(ii) Types of joints in bolted connection (3.5)

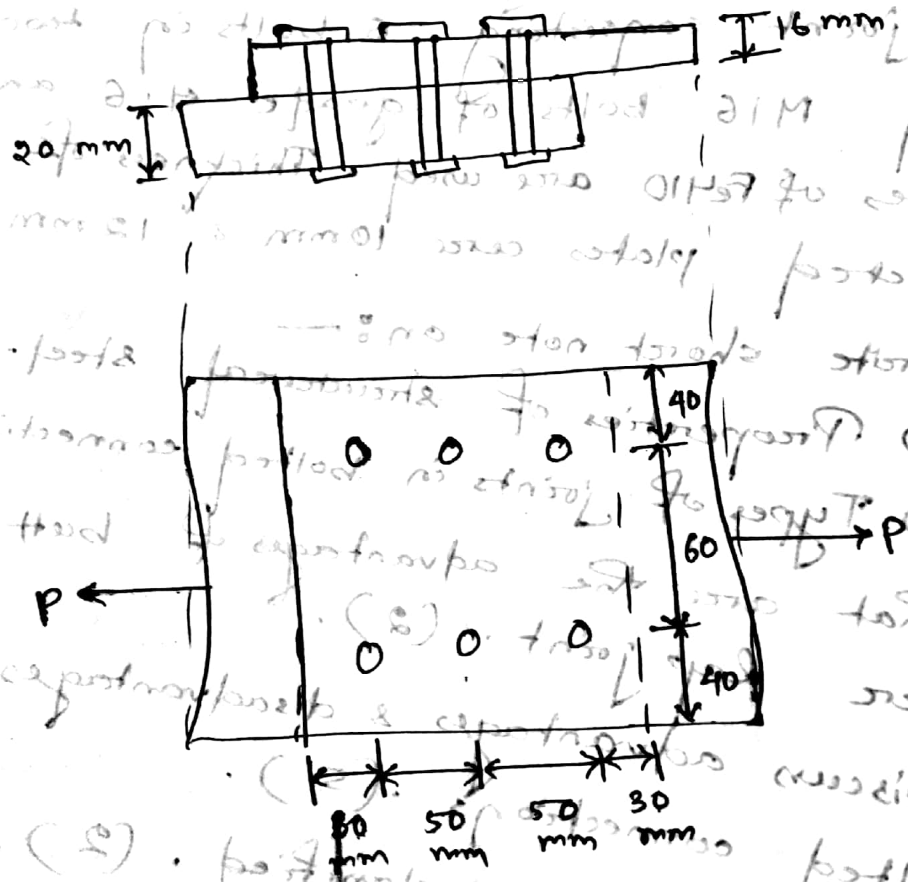
5) What are the advantages of butt joint over lap joint. (2).

6) Discuss advantages & disadvantages of bolted connections. (5).

7). How are bolts classified. (2).

8) Define following terms:— (5).
(i) Pitch
(ii) Gauge distance
(iii) Edge distance
(iv) End distance.

9) Find the maximum force that can be transmitted through a chain lap joint consisting of 6 bolts as shown in fig. Given M16 bolts of grade 4.6 and plates of Fe410 are used. Also find the efficiency of joint. (12)

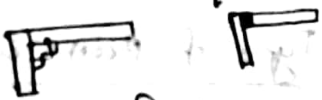


2.2 WELDED CONNECTION

Welding means joining two pieces of metal by establishing a metallurgical bond between them. The elements to be connected are brought closer and metal is melted by means of electric arc or oxy-acetylene flame. After cooling bond is established.

2.2.1 Advantages of welded connection

1) Due to absence of angles, welded structures are lighter.



2) Absence of making holes for fasteners, makes welding process quicker.
3) Circular tubes are easily connected by welding.
4) Noise production is lesser.
5) They provide good aesthetic appearance.
6) Welded connection is airtight and watertight.
7) No problem of mismatching of holes.

Disadvantages of welded connection

1) Greater possibility of brittle fracture is there.
2) It cannot take fatigue stress for a long time.
3) Inspection of welded joints are difficult and expensive as it requires non-destructive testing.
4) Highly skilled persons are required.
5) Difficult in field condition.
6) Welded parts cannot be dismantled and moved to other place.

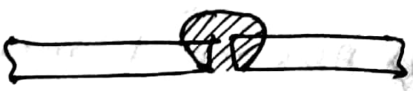
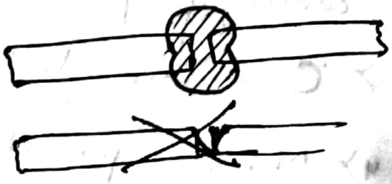
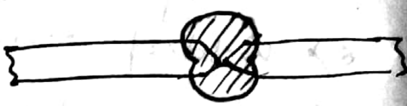
2.2.2 Types Of Welded Joints

(i) Butt welds

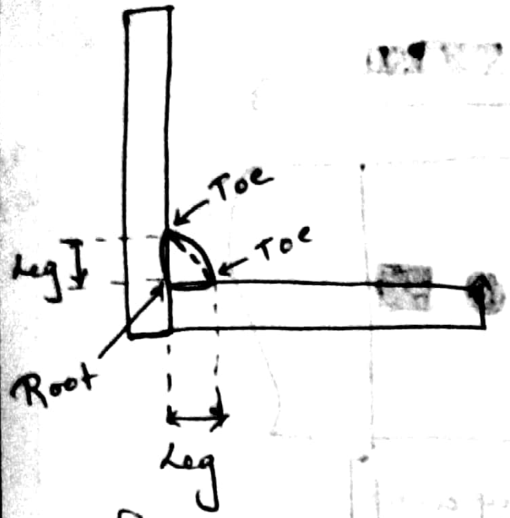
(ii) Fillet welds

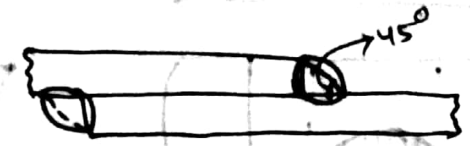
(iii) Slot weld and Plug weld

(i) **Butt weld** :- Butt weld is also known as groove weld. Depending on shape of groove made for welding, butt welds are of following types :-

Sl. No.	Type of Butt weld	Sketch
(a)	Square butt weld on one side	
(b)	Square butt weld on both sides.	
(c)	Single V butt joint	
(d)	Double V butt joint	
(e)	Single U butt joint	
(f)	Single J-butt joint.	

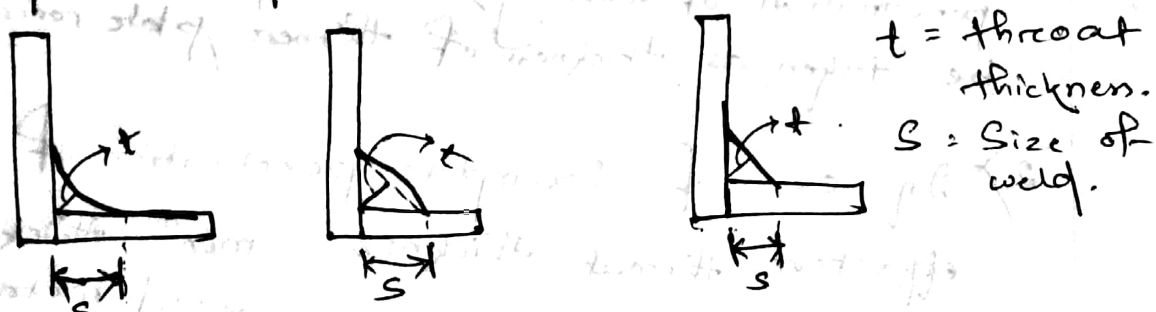
(ii) **Fillet Weld** :- It is of approximately triangular cross-section joining two surfaces approximately right angle to each other in lap joint, tee joint or corner joint.





=> When cross-section of fillet weld is isoceles triangle with face at 45°, it is known as standard fillet weld. In special circumstances 60° and 90° angles are also used.

=> Depending upon shape of weld face, fillet weld can be concave, convex or mitre.

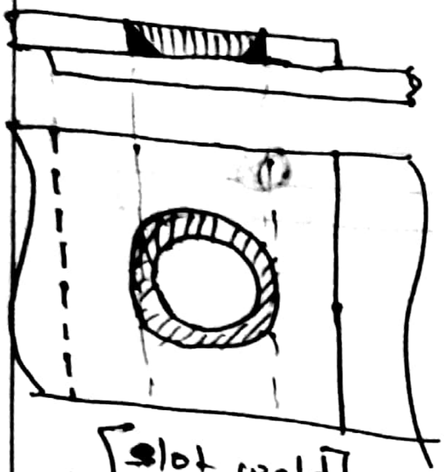


(a) Concave (b) Convex (c) Mitre.

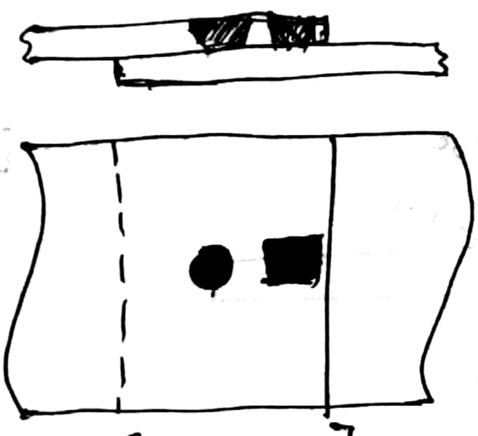
(iii) <u>Slot weld</u> and <u>Plug weld</u>

=> When fillet weld is made along the periphery of the hole, then it is known as slot weld.

=> When fillet weld is made to fill the entire hole with filler material, then it is known as plug weld.



[Slot weld]



[Plug weld]

Specifications for welding

(a) Butt weld

10.5.2.7.1 & 10.5.9.3 1) Size of butt weld shall be specified by the throat thickness. In double U, double V, double J ~~dou~~ butt welds, which gives complete penetration of welding, size of butt weld shall be taken as thickness of thinner plate connected.

2) In case of incomplete penetration of welding effective throat thickness = min^m thickness of weld metal.

In absence of appropriate data,

throat thickness = $\frac{5}{8}$ th of thickness of thinner material.

10.5.4.2 3) Effective length of butt weld = Length of full size weld.

4) Min^m ~~Size of~~ length of weld = 4 × Size of weld.

5) For Intermittent butt weld,

10.5.5.1 Effective length $\geq$ 4 x Size of weld

10.5.5.3. Space between two welds $\nleq$ 16 x thickness of thinner member joined.

(b) **Fillet Weld :-**

1) **Size :-**

(a) = minimum weld leg size

10.5.2.1 (b) ~~2.4 mm~~. for deep penetration weld with not less than 2.4 mm,

Size of weld = min^m leg size + 2.4 mm.

10.5.2.2 (c) For other penetration values :-

weld size = min^m leg size + actual penetrat^n.

2) Minimum size of weld = 3 mm.

10.5.3.2 (It is provided to avoid risk of cracking).

Table 21

Plate thickness	Min^m Size of weld
< 10 mm	3 mm
10 - 20 mm	5 mm
20 - 32 mm	6 mm
32 - 50 mm	8 mm.

3) Effective throat thickness

10.5.3.1 → > 3 mm & < 0.7 t or t.

t = thickness of thinner plate.

4) If faces of plates are inclined then,

effective throat thickness = K S.

10.5.3.2 S = Size of weld.

K = constant & it's values are given in IS 800: 2007

5) Effective length of weld is the length of weld for which specified size and throat thickness
10.5.4.1 exist. But welding length is made equal to effective length + 2 × S.

Effective length ≮ 4S.

6) In lap joint, minimum lap = 4 × t or 40 mm whichever is more.
10.5.1.2
t = thickness of thinner plate.

7) In Intermittent welds;
→ length ≮ 4s or 40 whichever is more
→ minimum clear spacing of intermittent welds
10.5.5.2 = 12 t for compression joints
= 16 t for tensile joints.
t = thickness of thinner plate.

8) Intermittent weld shall not be used in positions subjected to dynamic, repetitive and alternating stress.

(c) Plug Weld

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

2.2.3 (I) Design Stresses In Welds

10.5.7

(a) (I) Butt Weld

10.5.7 ↳ The thickness of butt weld = throat thickness.

↳ Stress in butt weld shall not exceed those permitted in parent metal.

(b) Fillet, Slot and Plug Weld

10.5.7.1.1 (I) Design Strength of weld $\boxed{f_{wd} = \frac{f_{wn}}{\gamma_{mw}}}$

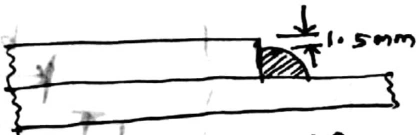
Nominal Strength $\boxed{f_{wn} = \frac{f_u}{\sqrt{3}}}$.

f_u = smaller of the ultimate stresses of the weld or parent metal.

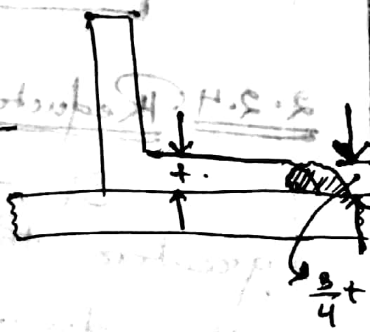
γ_{mw} = 1.25 for shop weld

= 1.5 for field weld.

(II) Provisions



10.5.8.1 1) If fillet weld is made to ~~plate~~ the square edge, then specified size of weld should be 1.5mm less than the edge thickness.

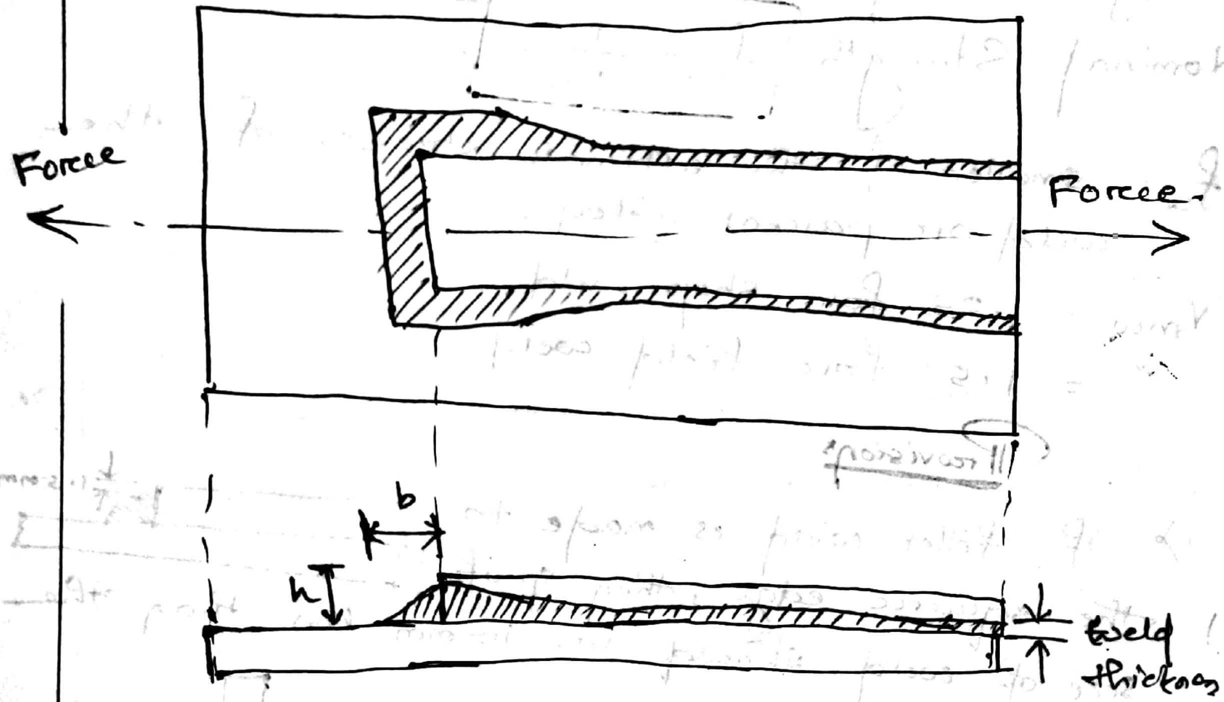


10.5.8.2 2) If fillet weld is made to the rounded toe, then specified size of weld should not exceed $\frac{3}{4}$th of thickness of toe.

10.5.8.4 3) In members subjected to dynamic loading, the fillet weld shall be of full size with it's leg length equal to thickness of plate.

Desirable Acceptable Not acceptable

10.5.8.5 4) End fillet weld, normal to the direction of force shall be of unequal size with throat thickness not less than $0.5t$.



2.2.4 Reduction In Design Stresses for long Joints

If the length of welded joint l_j is greater than $150t$, where t is throat thickness the design capacity of weld f_{wd} shall be
10.5.7.3 reduced by the factor

$$\beta_{lw} = 1.2 - \frac{0.2 l_j}{150t} \le 1.0 .$$

Solve Prob. from Page 101 - 108

S.S. Bhavikatti

1) A 18 mm thick plate is jointed to a 16 mm plate by 200mm long butt weld. Determine 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.

<u>Solution</u> :-

(i) <u>Double V butt weld</u>

In double V butt weld, complete penetration of welding occurs. Hence here

throat thickness = thickness of thinner plate

$t = 16mm$.

Effective length of weld $L_w = 200mm$.

$f_u = 410\ N/mm^2$

Since it is shop weld, $\gamma_{mw} = 1.25$

We know, design strength of weld $f_{wd} = \frac{f_{wn}}{\gamma_{mw}}$

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

So, $f_{wd} = \frac{f_u/\sqrt{3}}{\gamma_{mw}}$ in N/mm

$= \frac{f_u/\sqrt{3}}{\gamma_{mw}} \times$ area in N.

weld area = Length × throat thickness.

$= L_w \times t$

Now $f_{wd} = \frac{(f_u/\sqrt{3})}{\gamma_{mw}} \times L_w t = \frac{L_w t f_u}{\sqrt{3}\,\gamma_{mw}}$

$= \frac{200 \times 16 \times 410}{\sqrt{3} \times 1.25} = 605987$ N

$= 605.987$ KN.

(ii) <u>Single V butt joint</u>

Here, weld penetration is incomplete.

So throat thickness $t = \frac{5}{8} \times$ thickness of thinner plate

$\Rightarrow t = \frac{5}{8} \times 16 = 10$ mm.

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

$= \frac{200 \times 10 \times 410}{\sqrt{3} \times 1.25} = 378742$ N

$= 378.742$ KN.

2) A tie member of a roof truss consists of 2 ISA 10075, 8mm. The angles are connected to either of a 10 mm gusset plates and member is subjected to a working pull of 300 KN. Design the welded connection. Assume connections were made in workshop.

Given :- Working load = 300 KN.

factored load = $300 \times 1.5 = 450$ KN.

(11)

To find design strength of weld $= (L_w t)\frac{f_u}{\sqrt{3}} \times \frac{1}{\gamma_{mw}}$.

we have to find - t i.e. throat thickness.

& L_w - length of weld, which will be unknown.

~~so fo~~ In normal weld, throat thickness

$$t = 0.7 \times S.$$

So (i) <u>Finding Size of weld (S)</u>

(a) at rounded toe of angle section, size of weld should not exceed $\frac{3}{4} \times$ thickness (smaller)

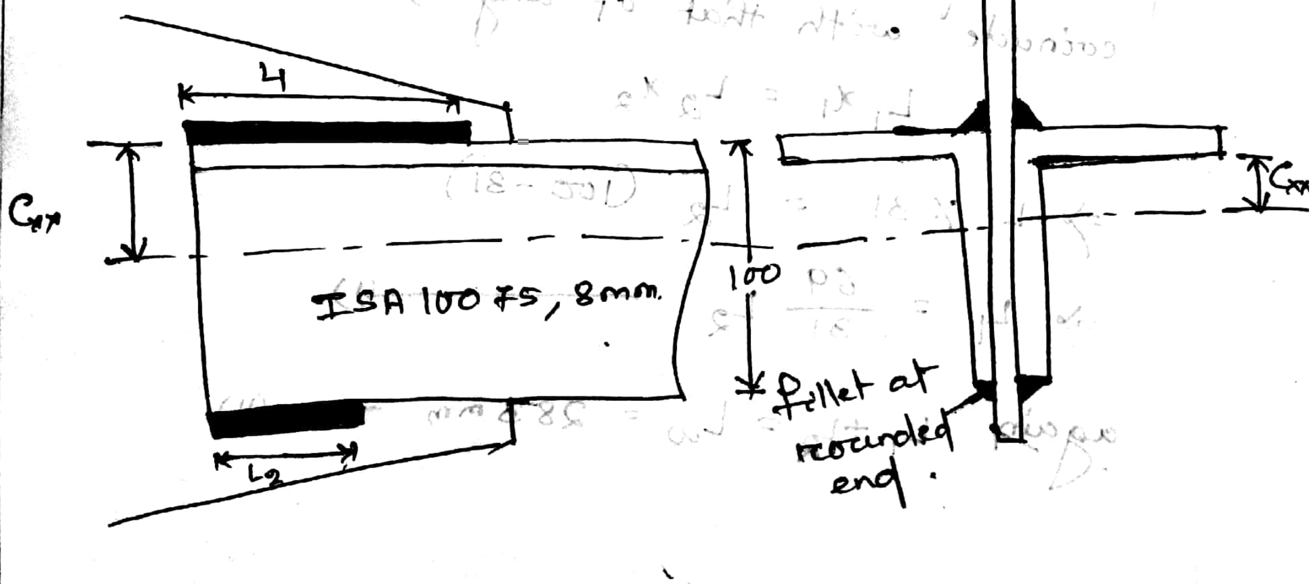
$S = \frac{3}{4} \times 8 = 6$ mm.

(b) at top, ~~thickness~~ / size should not exceed $t - 1.5$
$= 8 - 1.5 = 6.5$ mm.

So adopt $S = 6$ mm.

Hence throat thickness $t = 0.7 \times S = 0.7 \times 6 = 4.2$ mm.

So $f_{wd} = L_w \times t \times \frac{f_u}{\sqrt{3}\gamma_{mw}} = L_w \times 4.2 \times \frac{410}{\sqrt{3} \times 1.25}$



→ Strength of weld should be equal to external factored load at maximum.

→ Given, external factored load = 450 KN.

→ As there are two symmetric angle sections, total load will be equally on both sides.

→ Hence, Each angle carries a factored pull of

$$\frac{450}{2} = 225\ KN.$$

→ Now, according to statement ①

$$\therefore f_{wd} = 225\ KN \text{ at max.}$$

$$\Rightarrow L_w \times 9.2 \times \frac{410}{\sqrt{3} \times 1.25} = 225 \times 10^3\ N$$

$$\Rightarrow L_w = 283\text{mm}.$$

→ The centre of gravity of the section is at a distance 31 mm from top.

→ To make the centre of gravity of weld to coincide with that of angle,

$$L_1 x_1 = L_2 x_2$$

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

$$\Rightarrow L_1 = \frac{69}{31} L_2 \quad \text{——— (i)}$$

again $L_1 + L_2 = L_w = 283\text{ mm}$ ——— (ii)

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from eqn (I) & (II)

$L_2 = 87\ mm$

$L_1 = 195\ mm.$

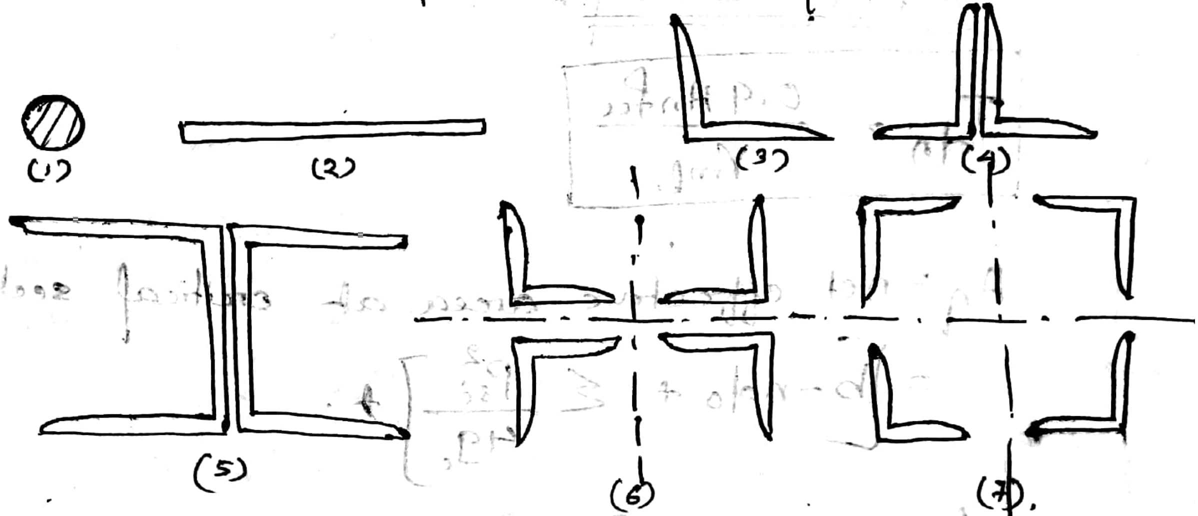
So provide 6 mm weld of 195 mm at top & 87 mm at bottom.

Prob:- 4.4, 4.5 Page - 107, S.S. Bhavikatti

3.0 Design Of Steel Tension Members.

⇒ Tension members are also known as tie members, which transfers tensile forces.

3.1 Common Shapes of tension members.



3.2 Design Strength of a Tension Member

Design Strength of a tension member is the lowest of the following :-

(a) Design Strength due to yielding of gross section (T_{dg})

(b) Rupture Strength of Critical Section (T_{dn})

(c) Block Shear Strength (T_{db}).

(a) Design Strength due to yielding of gross section

It is given by $\boxed{T_{dg} = \dfrac{A_g f_y}{\gamma_{mo}}}$

f_y = yield stress of material ②

A_g = gross area of the cross-section.

γ_{m0} = partial safety factor for failure in tension by yielding = 1.1

(b) (i) Design Strength due to rupture of Critical Section :-

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

A_n = net effective area at critical section

$$= \left[b - nd_0 + \sum \frac{P_{si}^2}{4g_i}\right] t.$$

f_u = Strength of plate

(ii) For threaded rods & bolts

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

A_n = net area at threaded section.

$$= \frac{\pi}{4}(d - 0.9382p)^2$$

$$= 0.78 \frac{\pi}{4} d^2. \quad p = \text{pitch}.$$

(iii) For Single Angle

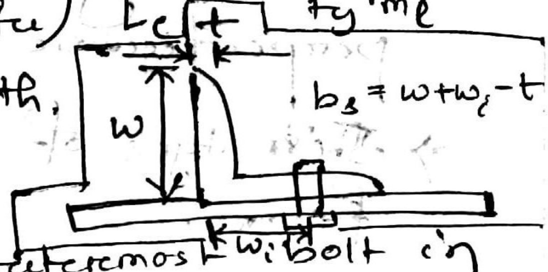
Angle sections have two legs. The leg connected to any plate is known as connected leg & the other is known as Outstanding leg.

For Single angles, $\boxed{T_{dn} = \frac{0.9 A_{nc} f_u}{\gamma_{m1}} + \frac{\beta A_{go} f_y}{\gamma_{m0}}}$ (3)

A_{nc} = net area of connected leg

A_{go} = gross area of outstanding leg.

& $\beta = 1.4 - 0.076 \left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\frac{b_s}{L_c} \le \frac{f_u \gamma_{m0}}{f_y \gamma_{m1}} \ge 0.7$



w = outstanding leg width.

b_s = shear leg width.

L_c = Distance between outermost bolt in the end joint, measured along load direction or length of weld along the load direction.

t = thickness of leg.

(iii) For Preliminary design IS Code recommends following formula :-

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

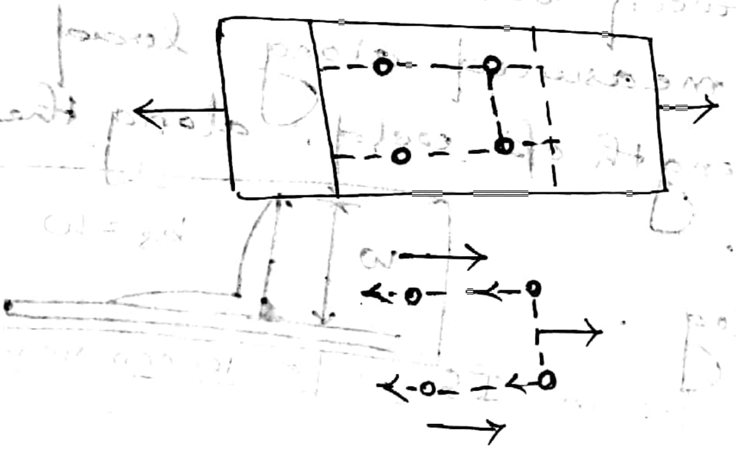
α = 0.6 for one or two bolts.
= 0.7 for three bolts.
= 0.8 for 4 or more bolts
& for equivalent weld length.

(7)

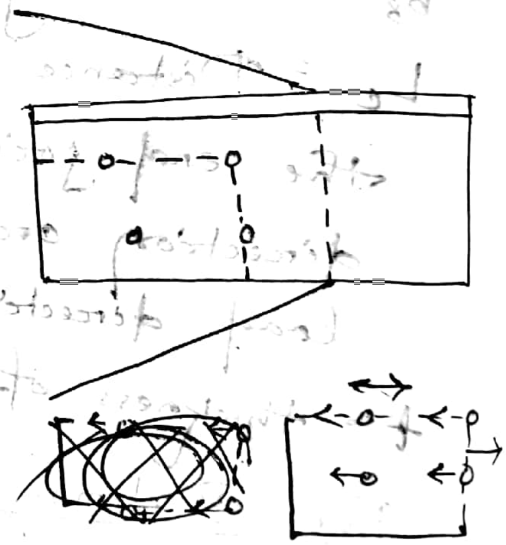
(c) Design Strength Due to Block Shear

→ At the connected end, failure of a tension members may occur along a path involving shear along one plane and tension on a perpendicular plane along the fasteners.

→ This type of failure is known as block shear failure.



(a)



(b)

IS 800-2007 recommends following formula:-

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

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

A_{vg} & A_{vn} = Min^m gross & net area in shear

A_{tg} & A_{tn} = Min^m " & " " " tension

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Design Procedure

1) Find the required gross area to carry factored load considering the strength in yielding i.e.

$$T_u = \frac{A_g f_y}{\gamma_{mo}}$$

T_u = factored tensile force (external).

$$\boxed{A_g = \frac{T_u}{(f_y / \gamma_{mo})}}$$

$$\boxed{A_g = \frac{1.1\, T_u}{f_y}}$$

2) Select suitable shape of section; depending upon the type of structure & location of member such that gross area is 25 to 40% more than calculated A_g.

3) Determine the no. of bolts or the welding required & arrange.

4) Find the strength considering:-

(a) Strength in yielding of gross area

(b) Strength in rupture of critical section.

(c) Strength in block shear,

5) Check if the strength is more than external factored tensile force.

6) Check for slenderness ratio, from Table - 3 IS 800 - 2007.

Q:- 11 Design a double angle tension member connected on each side of a 10 mm thick gusset plate, to carry an axial factored load of ~~33~~ 375 KN. Use 20 mm black bolts. Assume shop connection.

① Area required $(Ag)_{required} = \dfrac{1.1\, T_u}{f_y}$

$$(Ag)_{required} = \frac{1.1 \times 375 \times 1000}{250}$$

$$= 1650 \text{ mm}^2.$$

② Let's take 2 ISA ~~75~~ 7550, 8 mm thick whose gross area $= 2 \times 938$

$$= 1876 \text{ mm}^2.$$

③ Calculation of Number of bolts

$$\text{Number of bolts} = \frac{\text{External force}}{\text{Strength of one bolt.}}$$

So, to find no. of bolts, we have to calculate first the strength of one bolt.

3.1 Strength of one bolt

(a) Design shear strength

→ As the bolt will go through 3 plates i.e. one gusset plate & 2 connected legs of angles, the ~~sh.~~ no. of shear planes = 2.

→ So in double shear,

$$T_{dsb} = \frac{f_u}{\sqrt{3}\,\gamma_{mb}} \times [n_n A_{nb} + n_s A_{sb}]$$

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

[assuming M20 bolts of grade 4.6].

$\Rightarrow T_{dsb} = 103314$ N.

(b) Design bearing strength

Taking e = 40 mm & p = 60 mm.

K_b is smaller of $\frac{40}{3 \times 22}$, $\frac{60}{3 \times 22} - 0.25$, $\frac{400}{410}$, 1.0.

$\Rightarrow K_b = 0.606.$

$$V_{dpb} = \frac{1}{1.25} \times 2.5 \times 0.606 \times 20 \times 8 \times 400$$

$= 77568$ N.

So strength of one bolts are the

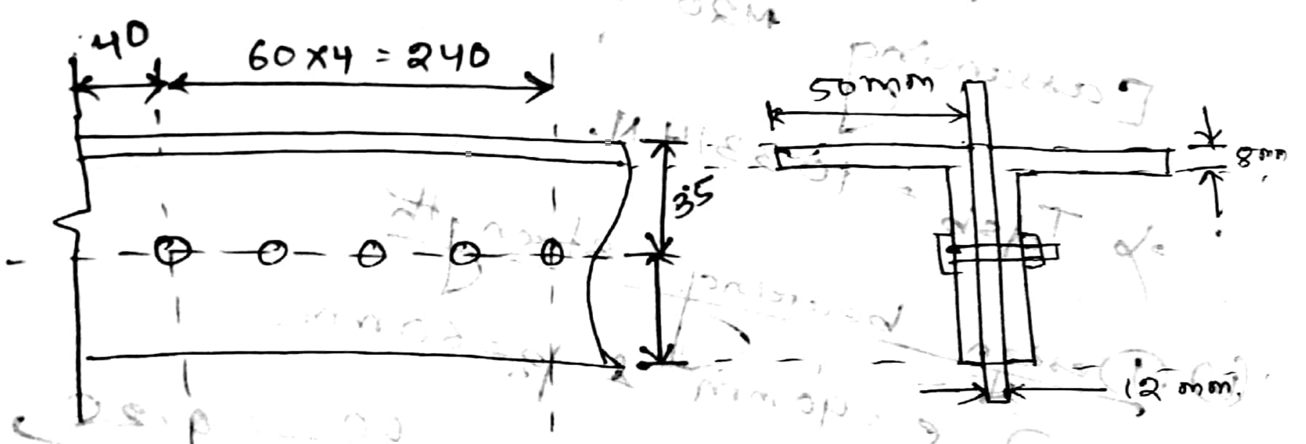
Bolt Value = Min^m of T_{dsb} & T_{dpb}

$\rightarrow$ Bolt Value = 77568 N

3.2 No. of bolts

$$\text{No. of bolts} = \frac{375 \times 10^3}{77568} = 4.83$$

Provides 5 bolts in a row.



Checks

4.1 Check for yield strength

$$T_{dg} = \frac{A_g f_y}{\gamma_{m0}} = \frac{1876 \times 250}{1.1} = 426364\,N > 375000$$ [OK]

4.2

4.2 Check for rupture strength

Area of connected leg, $A_{nc} = 2\left(75 - 22 - \frac{8}{2}\right) \times 8$

$\Rightarrow A_{nc} = 784 \text{ mm}^2$.

Area of outstanding leg, $A_{go} = 2 \times \left(50 - \frac{8}{2}\right) \times 8$

$\Rightarrow A_{go} = 736 \text{ mm}^2$

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

$w = 50$, $t = 8$.

$b_s = 50 + 35 - 8 = 77$ mm.

$L_c = 60 \times 4 = 240$ mm.

$$\text{so } \beta = 1.4 - 0.076 \times \frac{50}{8} \times \frac{250}{410} \times \frac{77}{240}$$

$= 1.307$.

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

$$= \frac{0.9 \times 410 \times 784}{1.25} + 1.307 \times \frac{736 \times 250}{1.1}$$

$= 450662 > 37500$ N [OK].

4.3 Check for Block Shear Strength

for angle section $\Rightarrow$

$A_{vg} = (40 + 60 \times 4) \times 8 = 2240 \text{ mm}^2$

$A_{vn} = (40 + (60 \times 4) - 4.5 \times 22) \times 8 = 1448 \text{ mm}^2$

$A_{tg} = (75-35) \times 8 = 320 \ mm^2$

or $40 \times 8 = 320 \ mm^2$

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

$$T_{dh_1} = \frac{A_{vg} f_y}{\sqrt{3} \gamma_{m0}} + \frac{0.9 A_{tn} f_u}{\gamma_{ml}}$$

$$= \frac{2240 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 232 \times 410}{1.25}$$

$= 362410 \ N.$

$$T_{dh_2} = \frac{0.9 A_{vn} f_u}{\sqrt{3} \gamma_{ml}} + \frac{A_{tg} f_y}{\gamma_{m0}}$$

$$= \frac{0.9 \times 1448 \times 410}{\sqrt{3} \times 1.25} + \frac{320 \times 250}{1.1}$$

$= 319575 \ N$

Block Shear strength T_{dh} = Smaller of T_{dh_1} & T_{dh}

$\rightarrow T_{dh} = 319575 \ N.$ for one angle

for both angles $T_{dh} = 2 \times 319575 > 37500$

[OK].

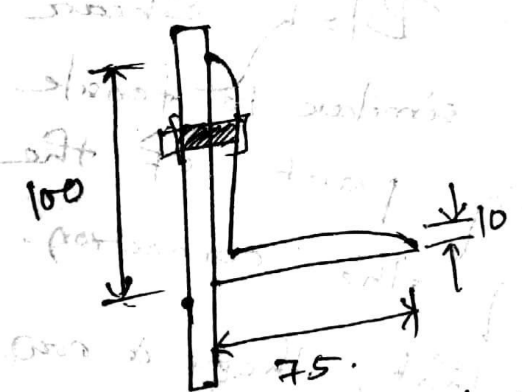
Ans
Hence Use 2 ISA 7550, 8 mm with 5 bolts of 20 mm dia.

Assignment :- 1) Design the previous examples by using welded connection.

2) Determine the tensile strength of a roof truss ~~diagonal~~ ∟100 × 75 × 10 mm ($f_y = 260\ N/mm^2$) connected to the gusset plate by :—

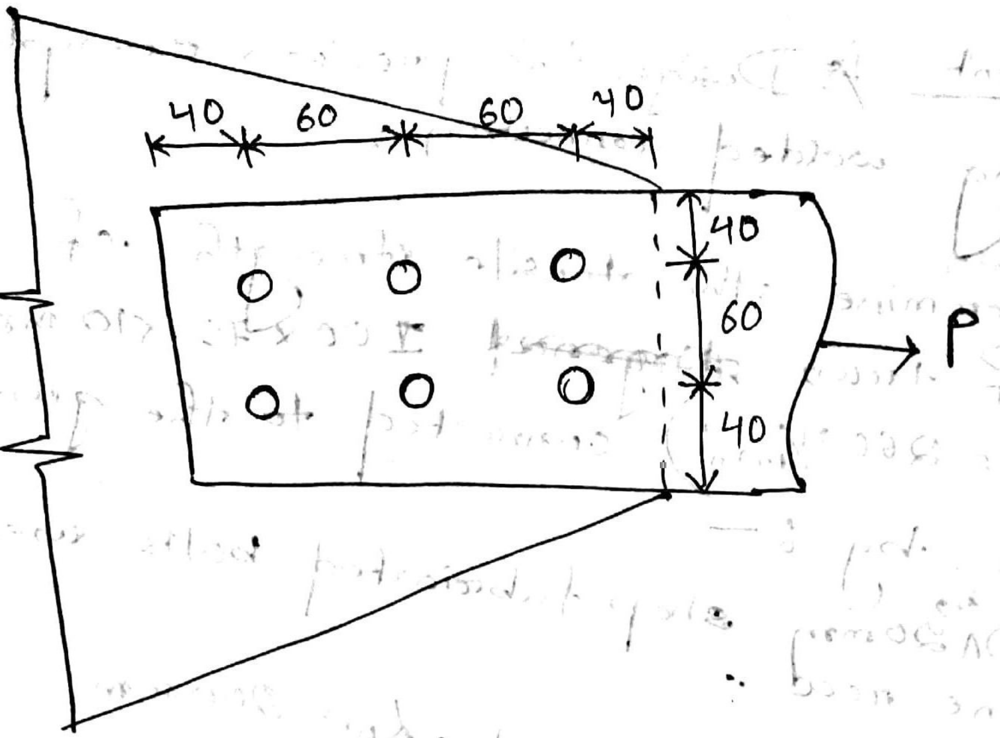
(a) 5, 20 mm shop fabricated bolts used in one row.

(b) 5 mm fillet weld, $L_w = 200$ mm.



3) Explain block shear failure with sketches for the case of bolted connection.

4) Determine the design tensile strength of the steel plate of size 200 mm × 10 mm of grade Fe410 connected with a 12 mm gusset plate if 24 mm dia shop bolts of property class 4.6 are used as shown in the figure :—

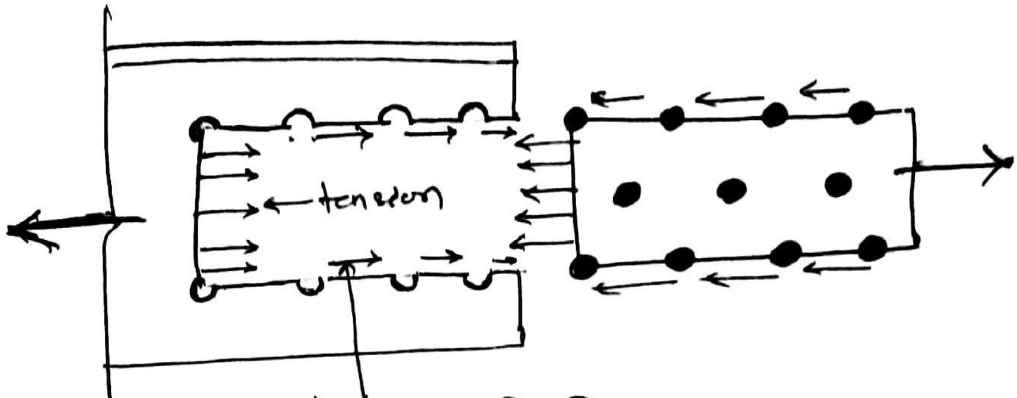


Solution to Q-4

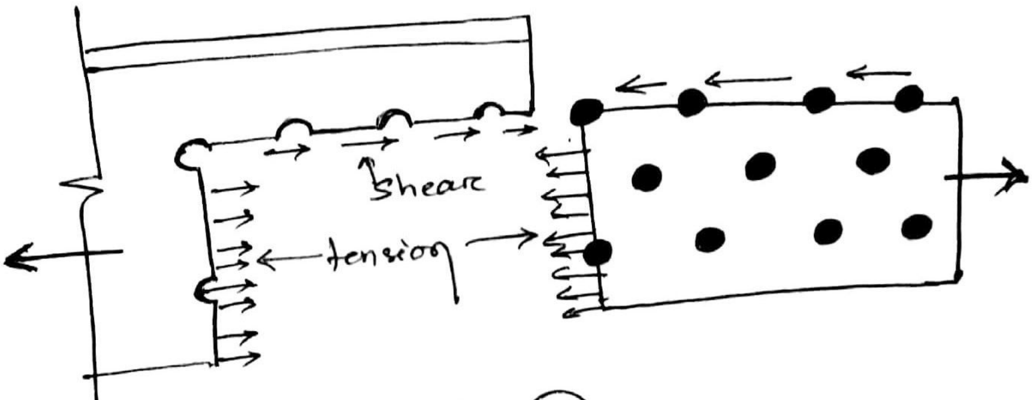
Block Shear Failure :- Block Shear failure in some ways is similar to tensile rupture, that is the main part of the member tears away from the connection.

The difference is that, there is now a combination of tension and shear on the failure path.

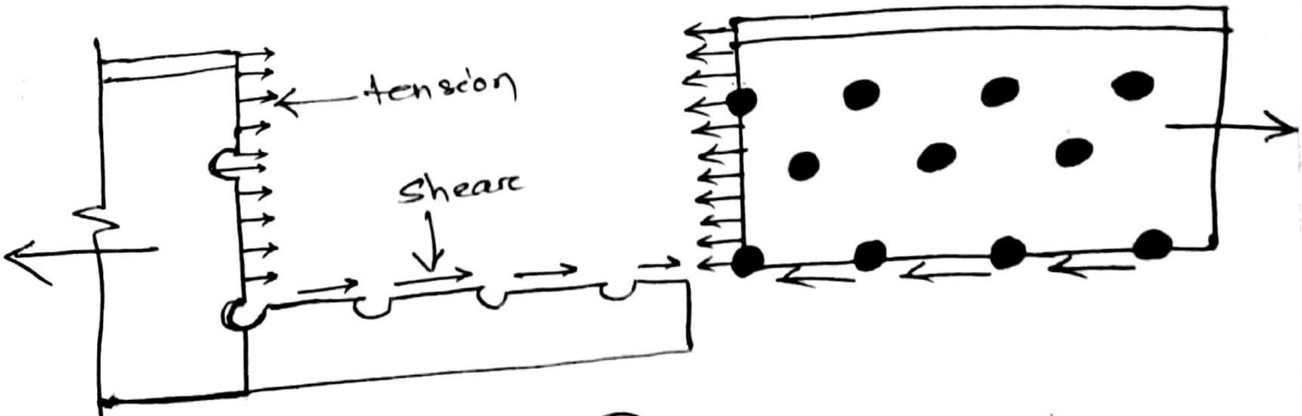
Like tensile rupture, it may have more than one failure path. Following diagram shows three different possible block shear failure paths :-



Shear path-1



Path-2

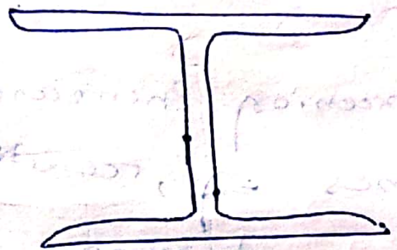


Path-3

DESIGN OF STEEL COMPRESSION MEMBERS

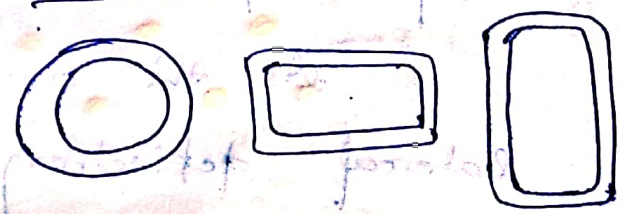
⇒ Many structural members are in compression. Vertical compression members in buildings are called columns & Compression members in trusses are called struts.

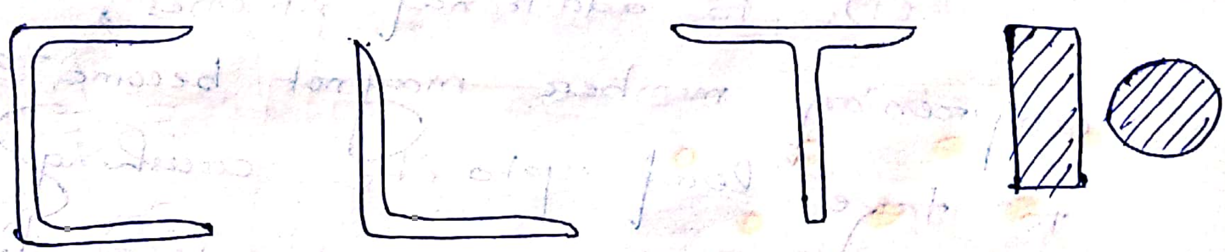
4.1 Common Shapes

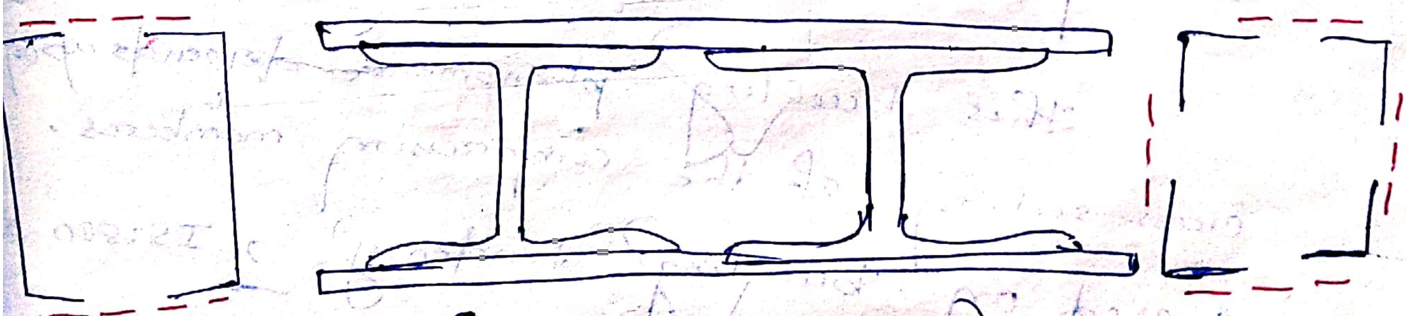


(Rolled I-Section)

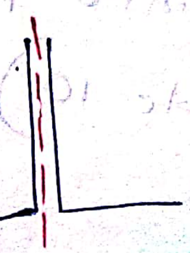
Hollow Sections:-







(Built-up members)



4.2) - Buckling class of Cross-Section

It is a common practice to transfer load axially through any member. But due to some imperfection, unexpected eccentricity may be imposed.

This eccentricity causes lateral bending moment, which results into bending compression.

As the axial compression increases, the lateral deflection increases in, resulting into additional bending stresses.

Due to additional stresses, the compression member may not become able to take load upto its crushing strength of column. This phenomena is called buckling.

This buckling phenomena depends upon cross-section of the compression members. Based on buckling tendency, IS:800-2007 divides various cross-sections into 4 buckling classes a, b, c, d. (Table 10).

4.2.1) Slenderness Ratio :-

It is defined as the ratio of effective length to the corresponding radius of gyration of the section.

Thus, Slenderness ratio $= \frac{l_e}{r} = \frac{KL}{r}$.

L = actual length of compression member.

$l_e = KL$ = effective length.

r = appropriate radius of gyration.

⇒ Effective length l_e is calculated by referring table 11 in IS 800:2007.

4.3 Design Compressive Stress & Strength

Design compressive stress f_{cd} of axially loaded compression member is :—

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

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

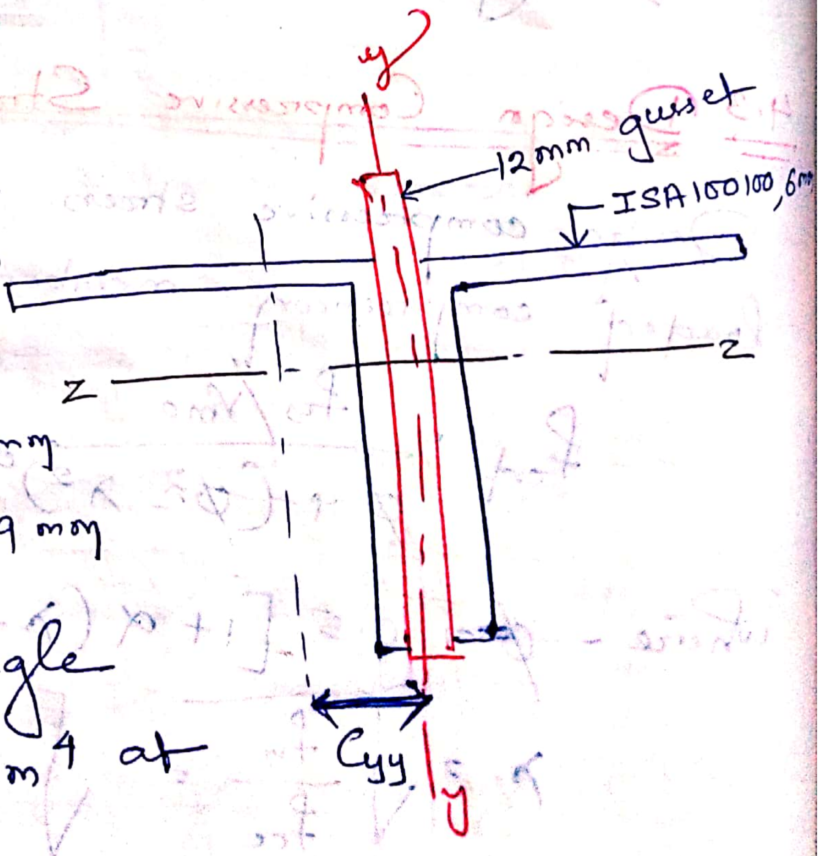
$$\lambda = \sqrt{\frac{f_y}{f_{cc}}} = \sqrt{\frac{f_y\left(\frac{KL}{r}\right)^2}{\pi^2 E}}.$$

$\gamma_{m0} = 1.1$ for Fe415.

Design compressive strength P_d of a member is $P_d = A_e f_{cd}$.

A_e = effective sectional area.

Q:- In a truss a strut 3m long consists of 2 angles ISA 100100, 6mm. Find factored strength of member if the angles are connected on both sides of 12mm gusset by
(i) One bolt
(ii) Two bolts
(iii) Welding?



From Steel table

For ISA100100, 6mm

area = 1167 mm^2

$C_{zz} = C_{yy} = 26.7$ mm

$r_{zz} = r_{yy} = 30.9$ mm

I_{yy} of one angle

$= 111.3 \times 10^4 \; mm^4$ at

it's own axis.

Scanned with CamScanner

I_{yy} of one angle at y-y of the member is I_{yy} = I_{yy} at it's own axis + $A x^2$.

Here $x = C_{yy} + \frac{\text{thickness of gusset plate}}{2}$.

so $I_{yy} = 111.3 \times 10^4 + 1167 \times \left(26.7 + \frac{12}{2}\right)^2$

For two angle sections :—

$I_{yy} = 2 \times I_{yy}$ of one angle section.

$= 2\left[111.3 \times 10^4 + 1167 \times (26.7 + 6)^2\right]$.

$= 4721723 \ mm^4$.

Now, $r_{yy} = \sqrt{\frac{I_{yy}}{A}} = \sqrt{\frac{4721723}{2 \times 1167}}$

$= 44.98 > 30.9$

so $r_{zz} = 30.9$ should be considered in designing compression member.

Case-(i) Single bolt is used

Here $KL = L = 3m = 3000mm$.

so $\frac{KL}{r} = \frac{3000}{30.9} = 97$.

Member belongs to buckling class c.

Now, for $\frac{KL}{r} = 97.2$ & $f_y = 250 MPa$

$f_{cd} = 121 - \frac{7}{10}(121-107)$

$= 111.2 \, N/mm^2$

$P_d = A_e f_{cd}$

$= (2 \times 1167) \times 111.2 = 259541 \, N$

$\therefore P_d = 259.541 \, KN$

Case-ii (two bolts are used)

Now $KL = 0.85 \times 3000 = 2550 \, mm.$

so $\frac{KL}{r} = \frac{2550}{30.9} = 82.5$

So Now, for $f_y = 250 \, N/mm^2$ & $\frac{KL}{r} = \cancel{80}\ 82.5$

$f_{cd} = 136 - \frac{2.5}{10} \times (136-121)$

$= 132.25 \, N/mm^2$

$P_d = 2 \times 1167 \times 132.25$

$= 308672 \, N = 308.672 \, KN$

Case-iii. Welding

Here $KL = 0.7 \times 3000$

$= 2100 \, mm.$

$\frac{KL}{r} = \frac{2100}{30.9} = 67.96$

So $f_{cd} = 168 - \frac{7.96}{10}(168 - 152)$

$= 155.26\ N/mm^2$

$P_d = 2 \times 1167 \times 155.26 = 362386\ N$.

$= 362.388\ KN$.

Q:- Determine the load carrying capacity of the column shown in fig, if it's actual length is 4.5 m. It's one end may be assumed fixed and other end hinged. Grade of steel is Fe415 (E250).

For ISMB 400,

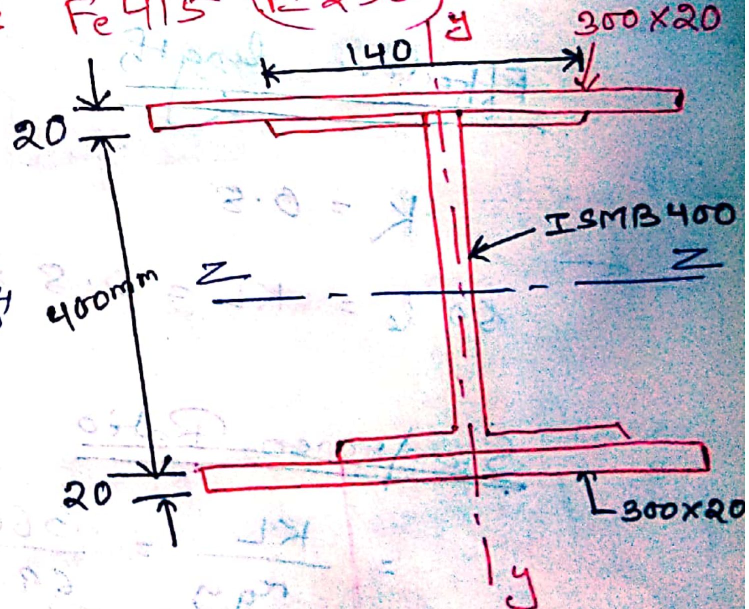
$h = 400mm,\ b_f = 140mm$

$t_f = 16\ mm$.

$I_{zz} = 20458.4 \times 10^4\ mm^4$

$I_{yy} = 622.1 \times 10^4\ mm^4$

Area $= 7846\ mm^2$



<u>Buckling class</u>:-

As it is a built-up section, it belongs to buckling class 'C'.

So, $I_{zz} = I_{zz}$ of I-Section $+ \frac{300 \times 20^3}{12} \times 2 + A x^2$ of both plate

$= 20458.4 \times 10^4 + \{2 \times (300 \times 20) \times (200 + 10)^2\}$

$= 733784000\ mm^4$.

$$I_{yy} = 622.1\times10^4 + \left(\frac{20\times300^3}{12}\times2\right)$$

$$= 96221000 \ mm^4$$

as $I_{yy} < I_{zz}$, buckling is about y-y axis

so take $r = r_{yy} = \sqrt{\frac{I_{yy}}{A}}$

Here $A = 7846 + 2\times300\times20$

$= 19846 \ mm^2$

so $r_{yy} = \sqrt{\frac{96221000}{19846}} = 69.63 \ mm$

Effective length

$K = 0.8$

so $l_e = KL = 0.8\times4500 = 3600 \ mm$

Slenderness Ratio

$$= \frac{KL}{r_{yy}} = \frac{3600}{69.63} = 51.70$$

from table of IS 800,

$$f_{cd} = 183 - \frac{1.7}{10}(183-168)$$

$$= 180.45 \ N/mm^2.$$

$P_d = Af_{cd}$

Scanned with CamScanner

$\Rightarrow P_d = 19846 \times 180.45$
$= 3581210 N$
$= 3581.210 KN.$

Load carrying capacity of the column

$= \frac{3581.210}{1.5} = 2387.474 KN.$

① Design Of Compression Members

1) Assume the design stress in compression.
i.e. assume value of f_{cd}.

2) Effective sectional area required is

$$A = \frac{P_d}{f_{cd}}$$

3) Select a section to give effective area required and calculate r_{min}.

4) Knowing end conditions & deciding the type of connection, determine the effective length.

5) Find the slenderness ratio and hence the design strength to be imposed f_{cd} and load carrying capacity P_d.

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

So, Design of compression member is a trial & error process.

Q:- Design a single angle strut connected the gusset plate to carry 180 KN factored load. The length of the strut between centre to centre connection is 3m.

(1) Assume $f_{cd} = 90 N/mm^2$.

(2) Effective sectional area required,

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

(3) Adopt a section ISA 9090, 12mm which has, $A = 2019 mm^2$.

$r_{min} = r_{yy} = 17.4 mm$.

(4) Assuming bolted connection with M20 bolts,

$V_{nsb} = 45 KN$,

so take 2 bolts, giving $V_{nsb} = 90 KN$

So effective length, $l_e = KL$
$= 0.85 \times 3000$
$= 2550 mm$.

(5) <u>Slenderness Ratio</u>

$$= \frac{KL}{r_{min}} = \frac{2550}{17.4} = 146.55$$

from table of IS800,

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

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

so for $\frac{KL}{r} = 146.55$,

$f_{cd} = 58.4 - \frac{6.55}{10}(58.4 - 52.6)$

$= 54.6 \ N/mm^2$.

So Load carrying capacity :-

$P_d = A f_{cd}$

$= 2019 \times 54.6 = 110239 < 180000 \ N$.

(6) So Section is to be revised.

Try ISA 130130, 8 mm.

Area $= 2022 \ mm^2$, $r_{min} = r_{yy} = 25.5$

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

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

& $P_d = 2022 \times 107 = 216354 > 180,000 \ N$

So, Provide ISA 130130, 8 mm with 2 bolts of M20.

Q:- A Column 4 m long has to support a factored load of 6000 KN. The column is effectively held at both ends and restrained in direction at one of the ends. Design the column using beam sections and plates.

(1) Assume $f_{cd} = 200$ KN

(2) Area required, $= \frac{6000 \times 10^3}{200} = 30,000\ mm^2$

(3) Use ISHB 450 @ 907 N/m

Area = $11789\ mm^2$, width of flange = 250 mm

(a) Area to be provided by plates

$= 30,000 - 11789$

$= 18211\ mm^2$

(b) So Select 20 mm plates with breadth

$2 \times (b \times 20) = 18211$

$\Rightarrow b = 455.3 \simeq 500\ mm$

(c) Provide 20 mm × 500 mm plates.

(d) Overhang $= \frac{500 - 250}{20}$

$= 12.5 < 12\varepsilon$ [clause 10.2.3.2]

Total Area provided,

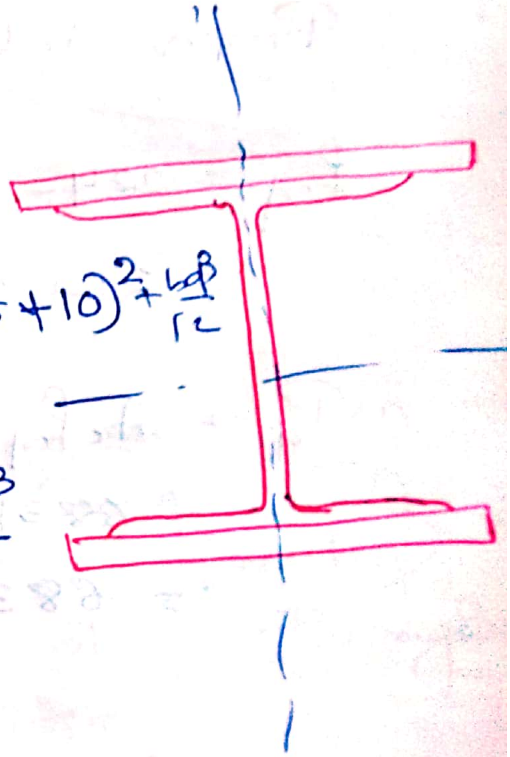
$A_e = 11789 + (500 \times 20 \times 2)$

$= 31789 \ mm^2 > A_{req} = 30,000.$

for ISHB450 @ 907 N/m,

$I_{zz} = 40349.9 \times 10^4 \ mm^4$

$I_{yy} = 3045 \times 10^4 \ mm^4.$



For total section selected

$I_{zz} = 40349.9 \times 10^4 + 2 \times 500 \times 20 \ (225+10)^2 + \frac{bd^3}{12}$

$= 1507.994 \times 10^4 \ mm^4$

$I_{yy} = 3045 \times 10^4 + 2 \times \frac{20 \times 500^3}{12}$

$= 447.1167 \times 10^6 \ mm^4$

$\therefore r = r_{yy} = \sqrt{\frac{I_{yy}}{A}}$

$= \sqrt{\frac{447.1167 \times 10^6}{31789}}$

$= 118.6 \ mm.$

Effective length $KL = 0.8L$

$= 0.8 \times 4000 = 3200 \ mm$

$\frac{KL}{r} = \frac{3200}{118.6} = 26.98.$

$t_f = t_f$ of I-Section $+ 20$

$= 13.7 + 20 = 31.7 < 40\ mm.$

It belongs to buckling class c for buckling about y-y axis,

From table :-

$$f_{cd} = 224 - \frac{6.98}{10}(224 - 211)$$

$$= 214.9\ N/mm^2$$

$$P_d = A_e f_{cd} = 31789 \times 214.9$$

$$= 6831456\ N$$

$$= 6831.456\ kN > \text{factored load}$$

[OK].

(1)

6.0. Design Of Steel Beams

Beams are those structural members, whose length is considerably larger than the cross sectional dimension.

6.1 Common cross sections

For beams, angles, I-sections, channels etc. are commonly used. For heavier loads I-sections, with additional plates connected on flanges are used.

Classification of cross section (Table-2 in IS 800)

During plastic analysis, it has been found that when all fibres of a beam cross section reach yield point, then plastic hinge is formed which donot allow the beam to take any extra load & beam fails due to rotation w.r.t. the plastic hinge.

But during this mechanism the beam should be capable of sufficient rotation capacity without local buckling.

Buckling in any small part of a member is called local buckling & buckling of whole beam is called global buckling.

If local buckling occurs before reaching the formation of plastic hinge then beam fails without developing full plastic moment or ~~no~~ full rotation about plastic hinge.

Hence it is necessary to see that plate elements of a cross-section do not buckle locally due to compressive stresses before plastic hinges are formed.

Local buckling can be avoided by providing proper width to thickness ratio. Based upon this criteria beam cross sections are divided into following 4 categories :-

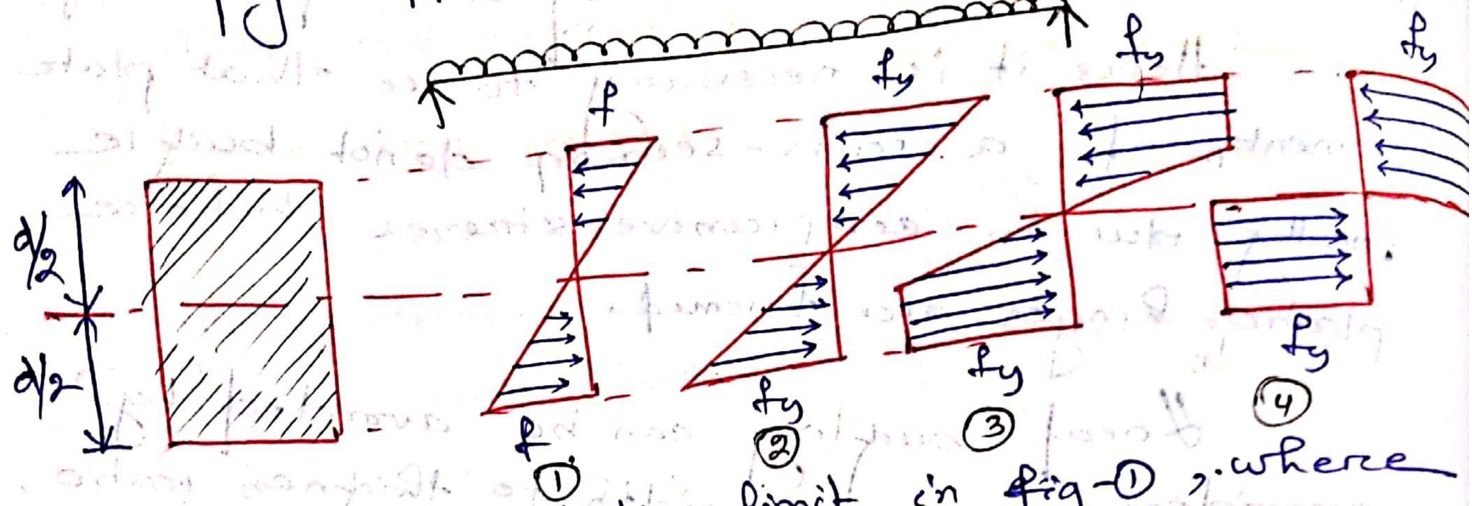
1) Class-1 (Plastic) Cross Section :- These are the sections that can develop plastic hinges and also have full rotation capacity for failure of the structure by plastic mechanism.

2) Class-2 (Compact) Cross-section :- Such sections can develop plastic moment & rotation capacity in inadequate amount due to local buckling.

3) Class-3 (Semi-compact) Cross-section :- These are the sections in which extreme ~~fibr~~ fibres in compression can reach yield stress, but cannot develop the plastic moment of resistance, due to local buckling.

4) Class-4 (Slender) Cross-section :- The cross-sections in which the elements buckle locally even before reaching yield stress belong to this category.

Consider the cross section of simply supported with UDL imposed on it.



Within elastic limit, in fig-1, where stress varies linearly from compression to tension.

As the load is gradually increased, stresses increases proportionately till extreme fibre is subjected to yield stress as shown in fig-2.

It is assumed that after yield point is reached fibre goes on yielding without resisting any additional load. But interior fibres are not yielded till now. Hence the additional loads are resisted by unyielded portion of the section.

As per fig-3 & 4; as the load is gradually increases, one by one fibre reach yield stress & stop resisting additional load. This condition, when all fibres at a section yield is called formation of plastic hinge.

After this stage, rotation at section will take place without resisting additional moment but moment corresponding to yield stress (f_y) is still resisted. This moment capacity is called plastic moment capacity of the section (M_p).

So M_p = (force due to f_y) × perpendicular distance

$\Rightarrow M_p = (f_y \times \text{Area}) \times (\text{distance})$.

Let the ~~distance~~ depth of beam = d

$\Rightarrow$ so distance of action of plastic moment = d/2.

Now, $M_p = (f_y \times \text{Area}) \times d/2$.

Area = Area of compression side (A_c)

or, Area of tension side (A_t)

= total area of beam. = $A_c + A_t$.

Let A = total area

as tension = compression for equilibrium

$\Rightarrow \cancel{f_y} \times A_t = \cancel{f_y} \times A_c$.

$\Rightarrow A_t = A_c = A/2$.

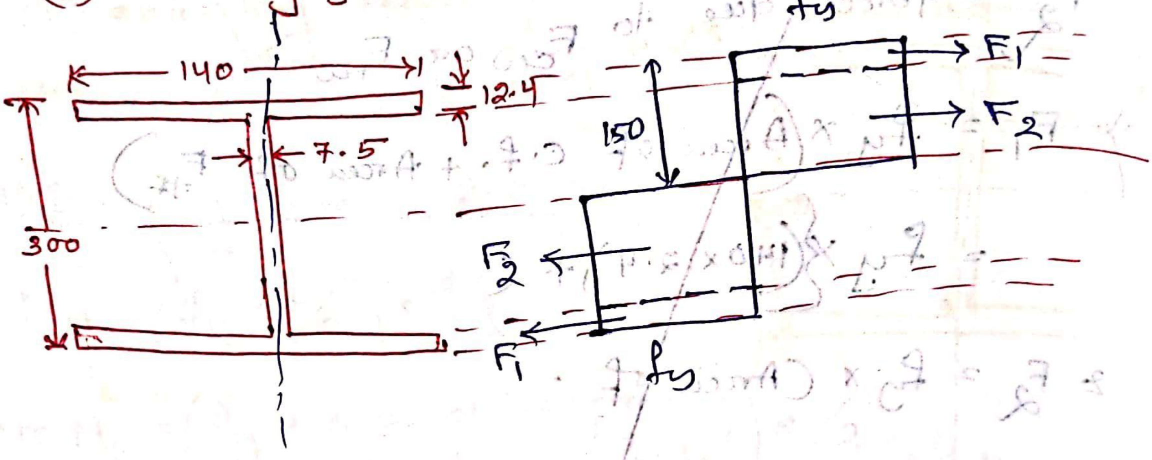
So $\boxed{M_p = \left(f_y \times \frac{A}{2}\right) \times \frac{d}{2}}$

5

Q:- Determine the plastic moment capacity and plastic section modulus of figure given below:-

(a) about z-z axis

(b) about y-y axis.



(a) <u>about - z-z axis</u>

force in flange $F_1 = f_y \times$ area of flange

$= f_y \times (140 \times 12.4)$

force in web, $F_2 = f_y \times$ area of web.

$= f_y \times \{(150 - 12.4) \times 7.5\}$

distance between F_1 forces $= 300 - \frac{12.4}{2} - \frac{12.4}{2}$

$= 287.6$.

distance between F_2 forces $= \frac{150 - 12.4}{2} \times 2$.

$= 137.6$ mm.

so $M_p = F_1 \times$ distance between F_1 forces $+ F_2 \times$ distance bet^n F_2 forces.

$= f_y \times (140 \times 12.4) \times 287.6 +$

$f_y \{(150 - 12.4) \times 7.5\} \times 137.6$.

$= 499273.6 f_y + 142003.2 f_y$.

$\Rightarrow M_p = 641276.8 f_y$

$= 160.31 \times 10^6$ Nmm

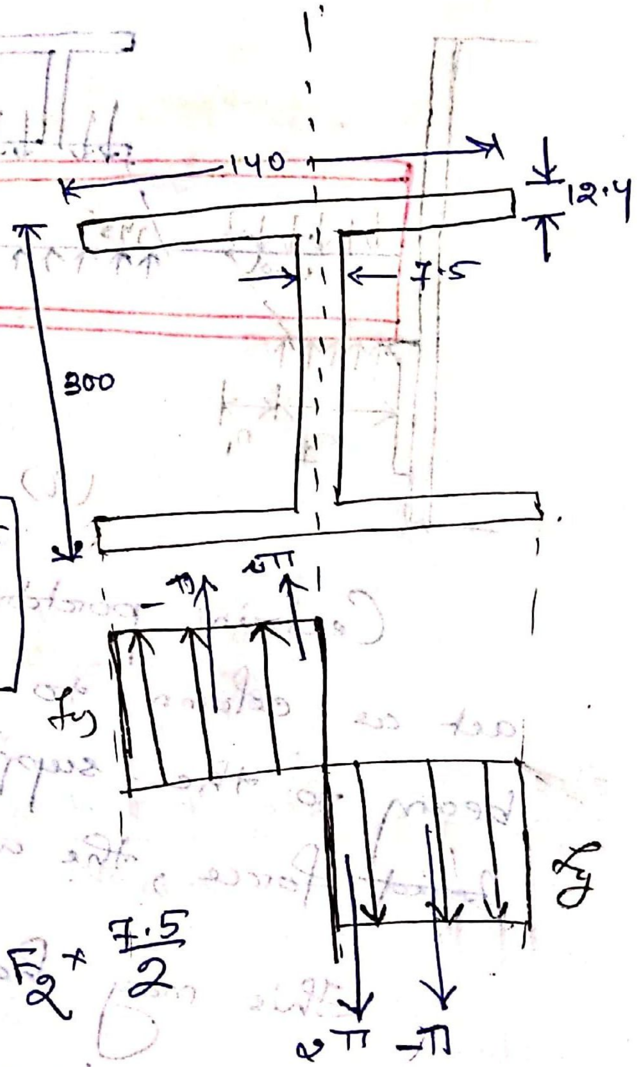
≈ 160.31 MPa.

<u>Section modulus</u>

$$Z_p = \frac{M_p}{f_y}$$

$$\Rightarrow Z_p = \frac{641276.8 f_y}{f_y} = 641276.8$$

$$= 641.27 \times 10^3 \text{ mm}^3$$



(b) <u>about - y-y-axis</u>

$F_1 = f_y \times$ Area

$$= f_y \times \left[\left(\frac{140 \times 12.4}{2}\right) \times 2\right.$$

$$F_2 = f_y \times \frac{\{300 - (2 \times 12.4)\} \times 7.5}{2}$$

$\Rightarrow F_1 = f_y \times (1736)\cancel{+1032}$

& $F_2 = \cancel{692000}\ f_y \times 1032.$

$$M_p = F_1 \times \left\{\left(\frac{140}{2}\right) \cancel{/2}\right) \times 2\right\} + F_2 \times \frac{7.5}{2}$$

$\Rightarrow M_p = f_y \times 1736 \times \frac{140}{2} + f_y \times 1032 \times \frac{7.5}{2}$

$= 31347500$

$= 31.34 \times 10^6$ Nmm.

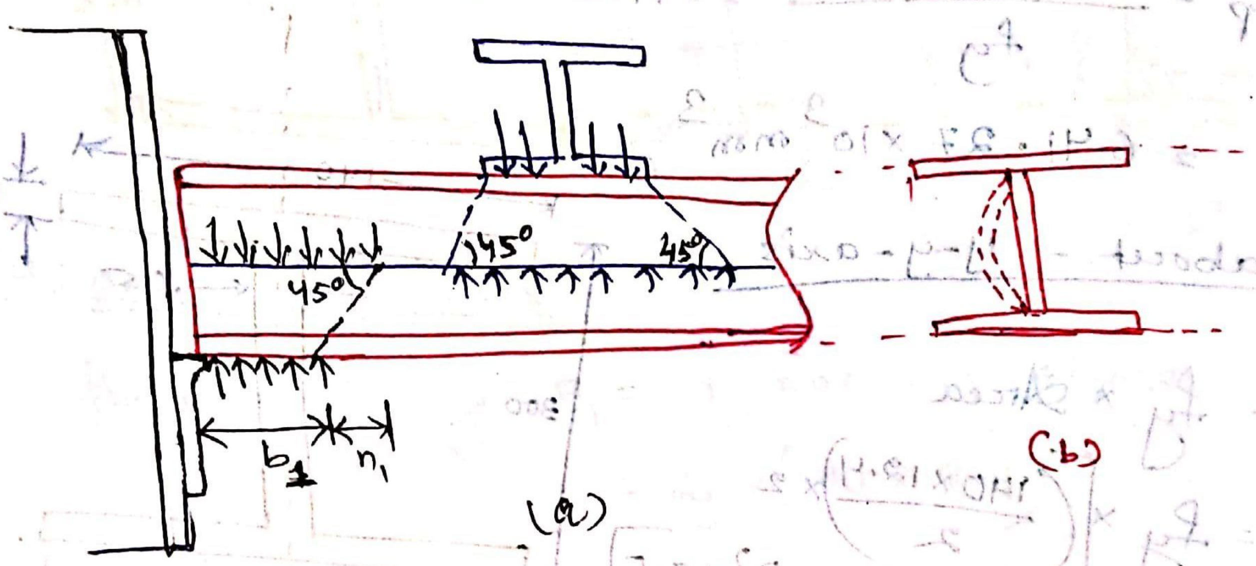
$= \cancel{692000 \times 140/2}$

$= \cancel{96880000/2}$

$= \cancel{\frac{96.88 \times 10^6 \text{ Nmm}}{2}}$

Deflection limits should be checked after designing a beam. Various deflection limits are provided in table – 6 of IS 800.

6.3.1 Web Buckling



Certain portion of beam at supports act as column to transfer the load from beam to the support. Hence under compressive ~~load~~ force, the web may buckle.

This may happen under a concentrated load on the beam also. The dispersion angle is taken as 45°.

However rolled steel sections are provided with sufficient thickness of web, to avoid web buckling. But in case of built-up section it is necessary to check web buckling.

As per IS-800-2007, effective web buckling strength is to be found based on the cross-section of web :—

$$= (b_1 + n_1)\, t_\omega$$

b_1 = width of stiff bearing on the flange

$n_1 = \frac{h}{2}$, where h is depth of section.

$$F_{cdw} = (b_1 + n_1)\, t_\omega f_c$$

F_{cdw} = web buckling strength.

f_c = allowable compressive stress.

Effective length = 0.7d of web column.

$$r_y = \sqrt{\frac{I_y}{A}} \text{ of web.}$$

$$= \sqrt{\frac{\frac{1}{12}(b_1 + n_1) t_\omega^3}{(b_1 + n_1) t_\omega}} = \frac{t_\omega}{2\sqrt{3}}$$

$$\text{Slenderness ratio} = \frac{\text{Effective length}}{r_y} \times 0.7d \cdot \frac{2\sqrt{3}}{t_\omega}$$

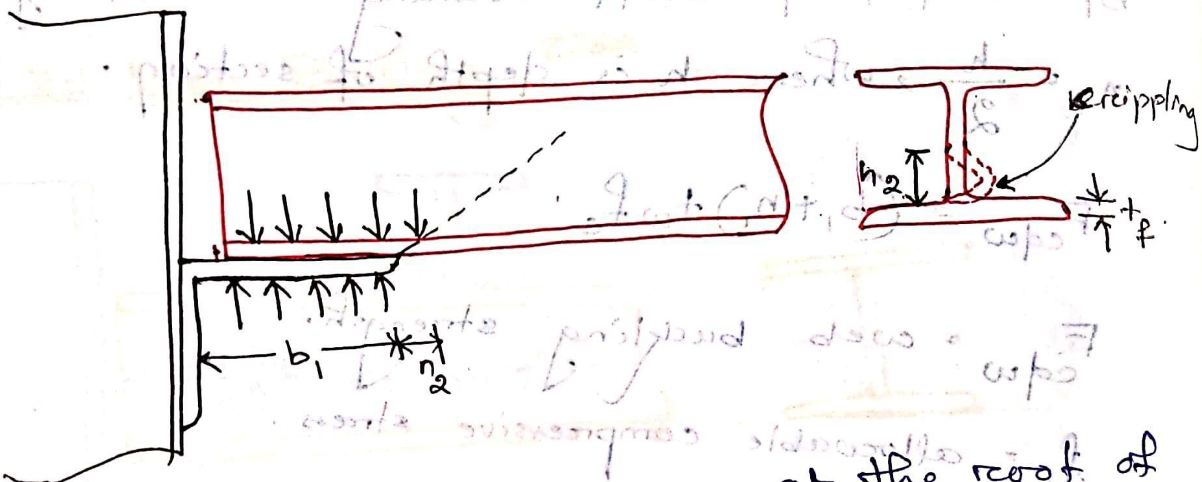
$$\simeq 2.5 \frac{d}{t_\omega}$$

Corresponding to this slenderness ratio from Table 9 of IS800-2007; buckling stress f_c can be found and hence,

$F_{cdw} = (b_1 + n_1) f_c \times t_\omega$ may be found.

6.3.2 Web Crippling

Near the support, web of the beam may cripple due to lack of bearing capacity.



The crippling occurs at the root of radius. The crippling strength can be found by :—

$$\cancel{F_{cdw} = (b_2 + n_1)\, t_w f}$$

$$F_{cdw} = (b_1 + n_2)\, t_w \frac{f_{yw}}{\gamma_{mo}}$$

b_1 = stiff bearing length

f_{yw} = yield stress of web.

If F_w > Load transferred by bearing [OK].

Care is taken in fixing web thickness of members to avoid this failure. If rolled steel section appropriate thickness is already given. Hence this check is very much necessary for built-up section.

Types Of Beams

Based on lateral supports to the compression flange, there are mainly 2 types of beams:-

~~(A) Latatto~~ (a) Laterally Supported beam
(b) Laterally Unsupported beam.

Compression flange of two adjacent beam are generally supported in lateral direction by flooring. ~~The~~ If not, then lateral buckling of compression flange occurs which ultimately reduces load carrying capacity of the beam.

6.4 Design Of Laterally Supported Beams

Beams mainly transfer bending moment & shear & Hence they are designed to transfer shear & bending.

STEPS

1) ~~A~~ A trial section is selected assuming it to be plastic section or class 1 section.
2) Then it is checked for the class it belongs to.
3) Check for bending strength.
4) Check for shear strength.
5) Check for deflection.
6) check for web buckling
7) check for web crippling.

If any check fails, then section is revised.

1) Bending Strength of Laterally supported Beam

(A) If $\frac{d}{t_w} \leq 67\varepsilon$; then two cases arise :-

(i) Design Shear Strength $\leq 0.6 V_d$

Then design bending strength

$$M_d = \beta_b Z_p f_y \times \frac{1}{\gamma_{m0}} \leq 1.2 Z_e f_y \times \frac{1}{\gamma_{m0}}$$

for simply supported beam.

& $\leq 1.5 Z_e f_y \times \frac{1}{\gamma_{m0}}$

for cantilever beam.

$\beta_b = 1$ for plastic & compact section.

$= \frac{Z_e}{Z_p}$ for semi-compact section.

(ii) Design Shear Strength (V) $> 0.6 V_d$

then, $M_d = M_{dv}$.

$$M_{dv} = M_d - \beta (M_d - M_{fd}) \leq 1.2 Z_e \times f_y \times \frac{1}{\gamma_{m0}}$$

for plastic or compact section.

& $M_{dv} = \frac{Z_e f_y}{\gamma_{m0}}$ for semi-compact section.

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2) Shear Strength of Laterally Supported Beam

Design shear strength,

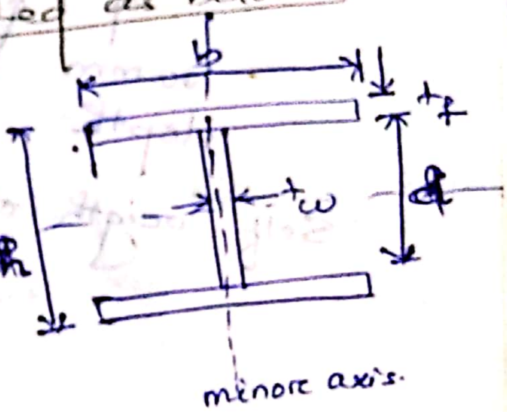
$$V_d = \frac{A_v f_{yw}}{\sqrt{3}} \times \frac{1}{\gamma_{mo}}$$

where A_v = shear area.

f_{yw} = yield strength of web.

Web is given priority in shear check as it transfers high shear stress in it's small cross sectional area.

Shear area may be calculated as below :—



(a) for I & channel section

(i) Major axis bending

Hot rolled → $A_v = h \times t_w$

welded → $A_v = d \times t_w$.

(ii) Minor axis bending

Hot rolled or welded, $A_v = 2 \times b \times t_f$.

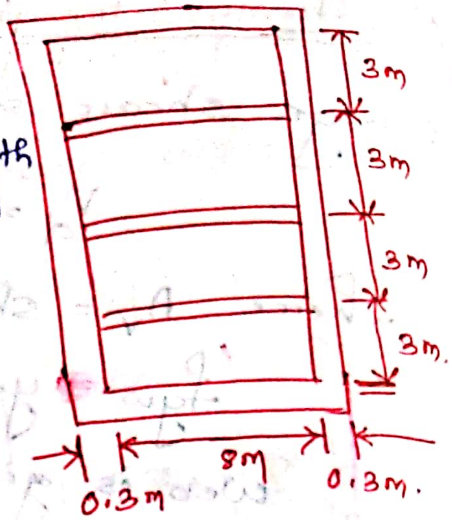
~~(b) Rectangular~~

Q :- A roof hall measuring 8m × 12 m consisting of 100mm thick RCC slab supported on steel I-beams spaced 3 m apart as shown in fig. The finishing load may be taken as 1.5 KN/m². and live load as 1.5 KN/m². Design the steel beam, if the stiff bearing at ends is in 75mm.

13

13

Each beam has a cleare span of 8m & ~~tak~~ width of 3m. Hence load per unit length of the beam is :-



total finishing load = 1.5 × 3
= 4.5 KN/m.

∴ Self weight of slabs
= 0.1 × 1 × 3 × 25 = 7.5 KN/m.

100 mm depth, 1m width, 3m length.

∴ Self weigth of beam = 0.8 KN/m (assumed).

Total dead load = 12.8 KN/m.

Live Load = 3 × 1.5 = 4.5 KN/m.

Total Load = 17.3

Total factored load = 17.3 × 1.5 = 25.95 KN/m

Effective Span = centre to centre distance from supports

$= 8m + \frac{0.3}{2} + \frac{0.3}{2}$ [assume 300mm supports.

= 8.3m.

So External moment $= \frac{wl^2}{8}$

$= \frac{25.95 \times 8.3^2}{8} = 223.46$ KN-m.

External Shear (V) $= \frac{wl}{2}$

$= \frac{25.95 \times 8.3}{2} = 107.69$ KN.

So Section modulus required $= \frac{M}{f_y} \times \gamma_{mo}$.

$\Rightarrow (Z_p)_{req} = \frac{223.46 \times 10^6 \times 1.1}{250}$

$= 983224\ mm^3$.

choose a beam section with Z_p value more than the required value.

<u>Try ISMB 400</u>.

whose $Z_p = 1176.163 \times 10^3$

$h = 400mm$

$b_f = 140mm$

$A = 7845.58\ mm^2$

$t_f = 16.00\ mm$

$t_w = 8.9\ mm$

depth of web $= d = 400 - (2 \times 16) = 400 - 32$

$= 368$

$I_{zz} = 20458.4 \times 10^4\ mm^4$

$Z_e = 1022.7 \times 10^3\ mm^3$

. b = outstanding leg of compression flange

$= \frac{140}{2} = 70mm$

3m

3m

3m

3m.

3m.

95 KN/m

mm

15

Section Classification

$$\varepsilon = \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{250}} = 1.$$

$$b = \frac{b_f}{2} = \frac{140}{2} = 70$$

$$\frac{b}{t_f} = \frac{70}{16} = 4.36 < 9.4\varepsilon$$

$$\frac{d}{t_w} = \frac{368}{8.9} = 41.34 < 84\varepsilon$$

Hence section is classified as plastic section.

① Check for Shear Strength

External Design Shear force $V = 107.69\,KN.$

Design Shear Strength:

$$V_d = \frac{f_y}{\sqrt{3} \times \gamma_{m0}} \times \text{shear area}$$

$$= \frac{250}{\sqrt{3} \times 1.1} \times h \times t_w$$

$$= \frac{250}{\sqrt{3} \times 1.1} \times 400 \times 8.9$$

$$= 467128N = 467.128KN.$$

$$0.6 V_d = 0.6 \times 467.128$$

$$= 280.277 > 107.61\,KN.$$

As $V < 0.6V_d$, it is not a high shear case.

② Check for moment capacity

as $V < 0.6 V_d$.

$$M_d = \beta_b Z_p \frac{f_y}{\gamma_{m0}} \leq 1.2 \frac{Z_e f_y}{\gamma_{m0}}.$$

$\beta_b = 1.0 \rightarrow$ for plastic section.

$$\text{so } M_d = 1.0 \times 1176.163 \times 10^3 \times \frac{250}{1.1} \leq 1.2 \times 1022.7 \times 10^3 \times \frac{250}{1.1}$$

$$= 267.310 \times 10^6 \leq 278.918 \times 10^6$$

$$\Rightarrow M_d = 267.310 \times 10^6 \text{ N-mm}.$$

$M_d > M$. [OK].

③ Check for Deflection

Maximum deflection $\delta = \frac{5}{384} \times \frac{wl^4}{EI}$

$$\Rightarrow \delta = \frac{5}{384} \times \frac{17.3 \times (8300)^4}{(2 \times 10^5) \times (20458.4 \times 10^4)}$$

$$= 26.127 \text{ mm}$$

Permissible deflection $= \frac{l_e}{300} = \frac{8300}{300}$

$$= 27.67 \text{ mm}.$$

as $\delta <$ Permissible deflection. [OK].

④ Check for web buckling

$b_1 = 75$ mm (given in question).

$F_{cdw} = (b_1 + n_1)\, t_w \times f_c$

$b_1 = 75\,mm$.

$n_1 = \frac{400}{2} = 200 = \frac{h}{2}$,

$t_w = 8.9\,mm$.

f_c is to be found from table of for which design compressive strength slenderness ratio and buckling class is required

slenderness ratio $\lambda = 2.5 \times \frac{h_1}{t_w}$ (calculate from $\frac{KL}{r}$ formula)

$= 2.5 \times \frac{334.2}{8.9} = 93.88$.

Since cross-section of web is rectangle, it falls under ~~btr~~ buckling class C.

Hence from table-9(C) of IS 800

$f_c = 121 - \frac{3.88}{10}(121 - 107) = 115.568\ N/mm^2$

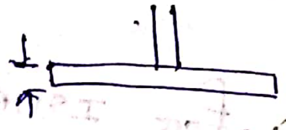
So $F_{cdw} = (75 + 200) \times 8.9 \times 115.568$

$= 282.852 \times 10^3\ N$

$= 282.852\ KN > 107.61\ KN$ [OK].

(5) Check for web crippling

$F_w = (b_1 + n_2) \times t_w \times f_{yw} \times \frac{1}{\gamma_{m0}}$



$b_1 = 75\,mm$

$h_2 = 2.5 \times t_f = 2.5 \times \cancel{32.80}\ \cancel{282\,mm}$ 16 18

$= 40$

$t_w = 8.9\,mm$

$f_{yw} = 250$

$\gamma_{m0} = 1.1$

so $F_w = (75 + \cancel{82}^{40}) \times 8.9 \times 250 \times \frac{1}{1.1}$

$= \cancel{232.518} \times 10^3\,N = [232.613\,KN]$

$= \cancel{232.518}\,KN$ [OK].

> 107.61 ①

Q:- Design a simply supported beam of effective span 1.5m carrying a factored concentrated load of 360 KN at midspan.

Ans ① Externally imposed maximum moment

$M = \frac{wl}{4}$ [as it is condition of point load at mid of s/s beam.

$\Rightarrow M = \frac{(360 \times 1.5)}{4}$

$= 135\,KN\text{-}m = 135 \times 10^6\,N\text{-}mm$

② Zp Required $= \frac{135 \times 10^6}{250} \times 1.1$

$\frac{M}{f_y} \times \gamma_{m0} = Z_{preq}$

$= 594.0 \times 10^3\,mm^3$

$\Rightarrow Z_{preq}$

③ Select a section

Try ISMB 300,

for ISMB 300, $Z_p = 651.731 \times 10^3\ mm^3$.

$h = 300mm$ $\quad t_f = 12.4mm$

$b_f = 140mm$, d = depth of web $= 300 - 2\times12.4$
$= 275.2\ mm$.

$t_w = 7.5mm$.

$I_{zz} = 8603 \times 10^4\ mm^4$

$Z_e = 573.6 \times 10^3\ mm^3$

$Z_p = 651.73 \times 10^3\ mm^3$

Self weight of beam $= 0.4336\ KN/m$.

factored selfweight $= 0.4336 \times 1.5$

Additional ~~mod~~ moment due to self weight

$$= \frac{(0.4336\times1.5)\times1.5^2}{8} = 0.183\ KNm.$$

So total factored moment $= 135+0.183$

$$\Rightarrow \boxed{M = 135.183\ KNm}$$

Factored total shear force

= SF. due to load + SF due to S.W.

$$= \frac{360}{2} + \frac{(1.5\times0.4336)\times1.5}{2} \quad \frac{wl}{2}$$

$= 180+0.488 = 180.448\ KN$.

④ Section Classification

$$\varepsilon = \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{250}} = 1.$$

$$b = \frac{b_f}{2} = \frac{140}{2} = 70$$

$\frac{b}{t_f} = \frac{70}{12.4} = 5.64 < 9.4\varepsilon$ or 9.4.

$\frac{d}{t_w} = \frac{300 - (2 \times 12.4)}{7.5} = \frac{247.2}{7.5} = 32.96 < 84\varepsilon$

Hence this section is classified as plastic (class I)

(5) <u>Shear Capacity</u>

$V_d = \frac{f_y}{\sqrt{3}} \times \frac{1}{\gamma_{mo}} (h \times t_w)$

$= \frac{250}{\sqrt{3}} \times \frac{1}{1.1} (300 \times 7.5)$

$= 295.235 \times 10^3 N$

$= 295.235 KN$.

$= 295.235 KN$

$0.6 V_d = 0.6 \times 295.235 = 177.145 KN$.

$\therefore V > 0.6 V_d$

(6) <u>Moment capacity of the section</u>:

$V > 0.6 V_d$ & section belongs to plastic category :-

$M_{dv} = M_d - \beta (M_d - M_{fd}) \leq 1.2 Z_e f_y \times \frac{1}{\gamma_{mo}}$

$M_d = Z_p f_y \times \frac{1}{\gamma_{mo}} \leq 1.2 Z_e f_y \times \frac{1}{\gamma_{mo}}$.

$= \frac{651.73 \times 10^3 \times 250}{1.1} \leq 1.2 \frac{\times 573.57 \times 10^3 \times 250}{\cancel{Z_e f_y \times 1} \; 1.1}$

$\leq 156.428 \times 10^6 Nmm$

$= 148.120 \times 10^6 Nmm$

$= 148.120 \times 10^6 N mm$.

$\Rightarrow M_d = 148.120$

(4)

$$\beta = \left(\frac{2V}{V_d} - 1\right)^2 = \left(\frac{2\times 180.448}{295.235} - 1\right)^2 = 0.05$$

$$\frac{KL}{r} = \frac{1500}{28.4} = 52.8$$

from table 13(a). as $\alpha_{LT} = 0.21$ for rolled steel section

f_{cd} can be found by knowing $f_{cr,b}$.

$f_{cr,b}$ is found from table-14:—

$\frac{KL}{r} = 52.8$ & $\frac{h}{t_f} = \frac{300}{12.4} = 24.19$

(i) $\frac{KL}{r} = 50 \rightarrow \frac{h}{t_f} = 20 \rightarrow f_{cr,b} = 995.3\ N/mm^2$

$\frac{KL}{r} = 50 \rightarrow \frac{h}{t_f} = 25 \rightarrow f_{cr,b} = 951.7\ N/mm^2$

$\frac{KL}{r} = 50 \rightarrow \frac{h}{t_f} = 24.19 \rightarrow f_{cr,b} = ?$

$$\frac{25-20}{24.19-20} = \frac{951.7 - 995.3}{f_{cr,b} - 995.3}$$

$\Rightarrow f_{cr,b} = 958.7632\ N/mm^2$.

(ii) $\frac{KL}{r} = 60 \rightarrow \frac{h}{t_f} = 20 \rightarrow f_{cr,b} = 726.4$

$\frac{KL}{r} = 60 \rightarrow \frac{h}{t_f} = 25 \rightarrow f_{cr,b} = 684.6$

$\frac{KL}{r} = 60 \rightarrow \frac{h}{t_f} = 24.19 \rightarrow f_{cr,b} = ?$

$$\frac{25-20}{24.19-20} = \frac{684.6-726.4}{f_{cr,b}-726.4}$$ (5)

$\Rightarrow f_{cr,b} = 691.3716 \ N/mm^2$.

<u>Now</u> $\frac{KL}{r} = 50$, $\frac{h}{t_f} = 24.19$, $f_{cr,b} = 958.7632$

$\frac{KL}{r} = 60$, $\frac{h}{t_f} = 24.19$, $f_{cr,b} = 691.3716$

$\frac{KL}{r} = 52.8$, $\frac{h}{t_f} = 24.19$, $f_{cr,b} = ?$

$$\frac{60-50}{52.8-50} = \frac{691.3716-958.7632}{f_{cr,b}-958.7632}$$

$\Rightarrow f_{cr,b} = 883.8935 \ N/mm^2$.

<u>again</u> from table - 13(a)

$f_{cr,b} = 800 \rightarrow f_y = 250$, $f_{bd} = 206.8$

$f_{cr,b} = 900 \rightarrow f_y = 250$, $f_{bd} = 204.5$

$f_{cr,b} = 883.8935$, $f_y = 250$, $f_{bd} = ?$

$$\frac{900-800}{883.8935-800} = \frac{204.5-206.8}{f_{bd}-206.8}$$

$\Rightarrow f_{bd} = 204.87044$

So $M_{fd} = f_{bd} \times A = 204.87044 \times 5626.38$

$= 1152679 \ N\,mm$.

$= 1.1526 \times 10^6 \ kNm$.

$M_{dv} = M_d - \beta(M_d - M_{fd}) \leq 1.2 Z_e f_y \times \frac{1}{\gamma_{mo}}$

$= 148.120 \times 10^6 - 0.05(148.24 \times 10^6 - 1.15476 \times 10^6)$

$\leq \frac{1.2 \times 573.57 \times 10^3 \times 250}{1.1}$

$= 140.77 \times 10^6$ N-mm $< 156.428 \times 10^6$ N-mm

$= 140.77$ KNm $> M = 135.190$ [OK].

CHECKS

① Deflection Check

Maximum deflection due to working load

$= \frac{WL^3}{48EI} = \frac{360 \times 10^3 \times 1500^3}{48 \times 2 \times 10^5 \times 8603 \times 10^4}$

$= 1.68$ mm.

Permissible deflection $= \frac{l}{360}$

$= \frac{1500}{300} = 5$

as Max. deflection by working load < Permissible Deflection.

[OK].

Hence section adopted is OK.

② Check for web buckling :-

Slenderness ratio, $\lambda = 52.8$

Buckling class →

$h/b_f = \frac{300}{140} = 2.14 > 1.2$

$t_f = 12.4 \leq 40$ mm

$r_{yy} = 2.84$ cm. $\quad r_{yy} < r_{zz}$ (7)

$r_{zz} = 12.37$ cm

r_{yy} is minm. So buckling is about yy-axis

& buckling class = b.

$\frac{KL}{r} = 50 \rightarrow f_{cd} = 194$

$\frac{KL}{r} = 60 \rightarrow f_{cd} = 181$

$$\frac{60-50}{52.8-50} = \frac{181-194}{f_{cd}-194}$$

$f_{cd} = 190.36\ N/mm^2$

$n_1 = \frac{h}{2} = \frac{300}{2} = 150$.

(take $b_1 = 100$).

$F_{cdw} = (b_1+n_1) t_w f_c$

$= (100+150)\ 7.5 \times 190.36 = 356925$ N [OK]

$= \cancel{287824.32\ N}$

$= 356.92$ KN > 180.448 KN (i.e. end reaction).

(3) Check for web crippling

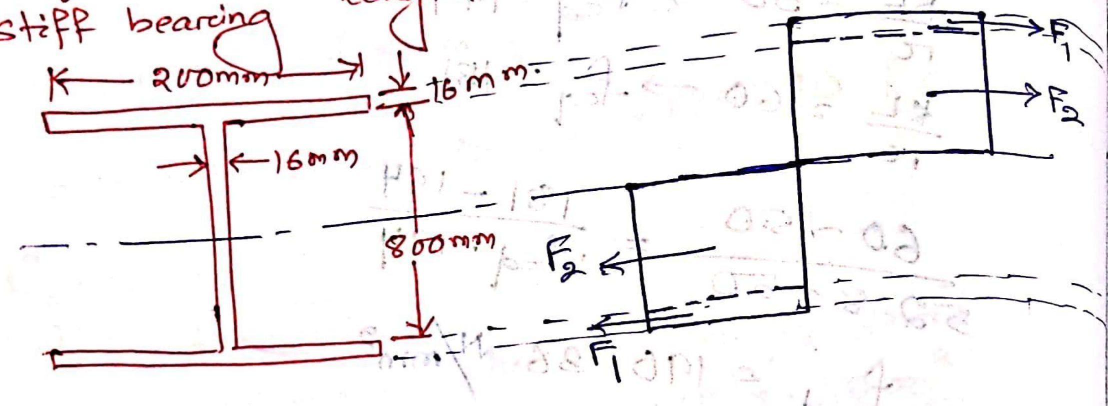
$n_2 = 2.5 \times t_f = 2.5 \times 12.4 = 31$

$F_w = (b_1+n_2) t_w f_{yw} \times \frac{1}{\gamma_{mo}}$

$= (100+31) \times 7.5 \times 250 \times \frac{1}{1.1} = 223295$ $\cancel{138068.18}$

$= 223.29$ KN > 180 KN [OK].

Q:- Determine the UDL carrying capacity of the welded plate girder as shown in fig. when it is used as a cantilever beam of 4m effective span & check for shear, deflection, web buckling & web crippling. Assume stiff bearing length as 100mm. (8)



Solution :-

$$(Z_p)_{req.} = \frac{M_p}{f_y} \qquad M = \frac{wl^2}{2} \Rightarrow w = \frac{M \times 2}{l^2}$$

(i) Calculation of M

Assume $M = M_d$.

(ii) Calculation of M_d

Assuming $V < 0.6 V_d$

$$M_d = \beta_b Z_p f_y \times \frac{1}{\gamma_{m0}}$$

β_b depends on class of section.

(iii) finding class of section

$\varepsilon = 1$

$$\frac{b}{t_f} = \frac{200}{16} = 12.5 < 15.7\varepsilon$$

a $\frac{d}{t_w} = \frac{800}{16} = 50 < 84\varepsilon$

So it belongs to semi-compact section.

So now, $M_d = \left(\frac{Z_e}{Z_p}\right) \times \beta Z_p f_y \times \frac{1}{\gamma_{m0}}$ $\left[\text{as, } \beta_b = \frac{Z_e}{Z_p}\right.$

$\rightarrow M_d = Z_e f_y \times \frac{1}{\gamma_{m0}}$

(iv) Calculation of Z_e

$Z_e = \frac{I_{zz}}{y}$

$I_{zz} = \frac{200 \times 832^3}{12} - \left[2 \times \left\{\frac{\left(\frac{200-16}{2}\right) \times 800^3}{12}\right\}\right]$

$= 1748.173 \times 10^6 \text{ mm}^4$

& $y = \frac{832}{2} = \cancel{4202.338 \times 10^3 \text{ mm}^3}$

So $Z_e = \frac{1748.173 \times 10^6}{4202.338 \times 10^3} = 4202.338 \times 10^3 \text{ mm}^3$.

So $M_d = 4202 \times 10^3 \times 250 \times \frac{1}{1.1} = 955.0768 \times 10^6 \text{ Nm}$

$= 955.0768 \text{ KNm}.$

Hence taking $M = M_d \Rightarrow M_d = \frac{wl^2}{8}$

$\Rightarrow w = \frac{8 M_d}{l^2} = \frac{8 \times 955.0768 \times 10^6}{4^2}$

$\boxed{w = 119.385 \text{ KN/m}}$

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10 (v) Check if asscemption $V < 0.6 V_d$ is correct

$V = w \times l = 119.385 \times 4$ [for cantilever]

$= 477.538$ KN

$$V_d = \frac{f_y A_v}{\sqrt{3}} \times \frac{1}{\gamma_{m0}}$$

$$= (h \times t_w) \times \frac{f_y}{\sqrt{3}} \times \frac{1}{\gamma_{m0}} \quad \left[\because A_v = h \times t_w\right.$$

$= 1746.747 \times 10^3 > V$ [OK]

So M_d is correct.

(vi) check for deflection

deflection due to load $= \frac{wl^4}{8EI} = \delta$

$w = \frac{119.38 \times 10^3 N}{1.5 \times 10^3 mm}$ ∵ working load

$= 79.587$ N/mm.

so $\delta = \frac{79.587 \times (4000)^4}{8 \times 2 \times 10^5 \times 1748.173 \times 10^6}$

$= 7.28$ mm. $< \frac{l}{300} = \frac{4000}{300}$

$= 13.33$ [OK]

(vii) Check for web buckling

Slenderness ratio, $\lambda = 2.5 \frac{h}{t_w}$

$\Rightarrow \lambda = 2.5 \times \frac{800}{16}$

$= 125$

Hence from table - 9(c).

11

$f_{cd} = 79.00\ N/mm^2$

$n = \frac{832}{2} = 416$.

So $F_{cdw} = (b_1 + n_1)\, t_w f_c$. [stiff bearing length $b_1 = 100$

$= (100 + 416) \times 16 \times 79$

$= 652.224 \times 10^3\ N$

$= 652.224\ KN. > V$ [OK]

(viii) Check for web crippling

$n_2 = 2.5 t_f = 2.5 \times 16$

$F_w = (b_1 + 2.5 t_f)\, f_y \times \frac{1}{\gamma_{m0}}\, t_w$.

$= (100 + 2.5 \times 16) \times 250 \times \frac{1}{1.1} \times 16$

$= 509.09 \times 10^3\ N$.

$= 509.09\ KN. > V$ [OK].

$\boxed{\text{So } w = 119.385\ KN/m.}$ (Ans)

Design of Built-up section

When moment to be resisted is heavy & available rolled sections may not be sufficient. In such cases built-up beams are used.

Q:- Design a simply supported beam of 10m ~~efficient~~ effective span carrying a total factored load of 60 KN/m. The depth of beam should not exceed 500mm. The compression flange of beam is laterally supported by floor construction. Assume stiff bearing.

Given $L = 10m = 10000\,mm$

$w = 60\ KN/m$

Maximum BM, $M = \frac{wL^2}{8} = \frac{60 \times 10^2}{8} = 750\ KN\text{-}m$

$= 750 \times 10^6\ N\text{-}mm.$

$(Z_p)_{req} = \frac{M}{f_y} \times \gamma_{mo} = \frac{750 \times 10^6 \times 1.1}{250}$

$= 3300 \times 10^3\ mm^3.$

Since depth is restricted to 500mm, we can select ISMB 450 with flange plates.

Z_p of ISMB 450 $= 1553.4 \times 10^3$

Z_p to be provided by cover plates

$= (3300 - 1553.4) \times 10^3$

$= 1746.6 \times 10^3\ mm^3$.

$Z_p = \dfrac{M_p}{f_y}$.

M_p = force × distance

F = force in plates

F = f_y × area of plate (A_p).

& distance = $d_1 = 450 + \frac{t}{2} + \frac{t}{2} \simeq 450$ mm (neglecting thickness).

Now $(Z_p)_{\text{of plates}} = \dfrac{M_p}{f_y}$

$\Rightarrow 1746.6 \times 10^3 = \dfrac{(\cancel{f_y} \times A_p) \times d_1}{\cancel{250}}$

$\Rightarrow A_p = \dfrac{1746.6 \times 10^3}{d_1}$

$= \dfrac{1746.6 \times 10^3}{450} = \cancel{1746.6 \times 10^3}\ 4269.5\ mm^2$

Let thickness of plate = 20 mm

~~width~~ $= \left(\dfrac{\cancel{1746.6 \times 10^3}}{20}\right) \times \cancel{\dfrac{1}{20}} =$

width $= \dfrac{4269.5}{20} = 213.45$

provide width = 220 mm.

Provide 220×20 mm plates on either side.

(i) Check for shear

$$V_d = \frac{f_y}{\sqrt{3}} \times \frac{1}{\gamma_{m0}} \times h \times t_w = \frac{250}{\sqrt{3}} \times \frac{1}{1.1} \times 450 \times 9.4$$

$$= 555.044 \times 10^3 N = 559.044 \ KN.$$

$$V = \frac{wl}{2} = \frac{60 \times 10}{2} = 300 \ KN.$$

$$V < V_d \quad \boxed{OK}$$

(ii) Check for bending strength

(a) Section Classification

$$\frac{b}{t_f} = \frac{(150/2)}{17.4} = 4.3 < 9.4\varepsilon$$

$$\frac{d}{t_w} = \frac{450 - 2(17.4 + 15)}{9.4} = 40.9 < 84\varepsilon$$

Hence it is a plastic section.

(b) $0.6V_d = 0.6 \times 559.044 = 333.026$

$V < 0.6V_d$.

$$M_d = \beta_b \times \frac{Z_p f_y}{\gamma_{m0}} < 1.2 Z_e f_y \times \frac{1}{\gamma_{m0}}$$

$\beta_b = 1.0$ for plastic sect^n

$\cancel{Z_p \frac{f_y}{\gamma_{m0}}} =$

$Z_p = [Z_p$ of I-section $+ Z_p$ of plates$]$

$$Z_p = \left[1553.4\times10^3 + \{(220\times10)\times f_y\}\times(450+20)\right].$$

$$= 260.0534\times10^6\ mm^3.$$

$$Z_e = \frac{I}{y}$$

I_{zz} = I_{zz} of ISMB 450 + I_{zz} due to plate $\times 2$

$$= 30390.8\times10^4 + 2\left(\frac{bd^3}{12} + Ax^2\right).$$

$$= 30390.8\times10^4 + \left\{2\times\left(\frac{220\times20^3}{12} + 220\times20\times\left(\frac{450}{2}+\frac{20}{2}\right)^2\right)\right\}$$

$$= 30390.8\times10^4 + \{\cancel{2}\times 48612.66\times10^4\}$$

$$= 789.88\times10^6\ mm^4.$$

$$y = \frac{450+20}{2} = 255+20$$

$$Z_e = \frac{789.88\times10^6}{(255+20)} = 3224033\ mm^4.$$

Hence $M_d = 1.0\times\frac{260.0534\times10^6\times250}{1.1} < 1.2\times\frac{3224033\times250}{1.1}$

$$= 823.045\times10^6 < 879.28\times10^6\ N.mm$$

[OK]

again $M_d > M \longrightarrow$ so [OK].

(iii) <u>Check for deflection</u>

Working load $= \frac{60}{1.5} = 40\ KN/m.$

for cantilever with UDL, deflection $\delta = \frac{5wL^4}{384EI}$

$I \to I_{zz}$

$$\Rightarrow \delta = \frac{5 \times 40 \times (10{,}000)^4}{384 \times 2\times10^5 \times 789.884\times10^6}$$

$$= 32.97 \text{ mm}.$$

Permissible deflection $= \frac{L}{240} = \frac{10{,}000}{240} = 41.67$ mm

as $\delta <$ Permissible deflection [OK]

(iv) Check for web buckling

$$\lambda = 2.5 \frac{h_1}{t_w}.$$

$$h_1 = (450 - 2\times t_f) = 450 - 2\times 17.4 = \cancel{365.2}$$

$$= 415.2$$

$$\lambda = 2.5 \frac{h_1}{t_w} = 2.5 \times \frac{415.2}{9.4}$$

$$= 110.425$$

So $f_{cd} = 105.929 \text{ N/mm}^2$

$$n_1 = \frac{450+2\times20}{2} = (490/2)$$

$$F_{cdw} = (b_1+n_1) \times t_w f_{cd}$$

$$= \left(75 + \frac{490}{2}\right) \times 9.4 \times 84.605$$

$$= 318.634\times10^3 \text{ N} > 300 \text{ kN [OK]}$$

17

(v) Check for web crippling

$$F_w = (b_1 + n_2) f_y \times \frac{1}{\gamma_{m0}} \times t_w$$

$$= \{75 + (2.5 \times t_f)\} \times 250 \times \frac{1}{1.1} \times 9.4$$

$$= 573.614 \times 10^3 N > 300 KN \ [OK].$$

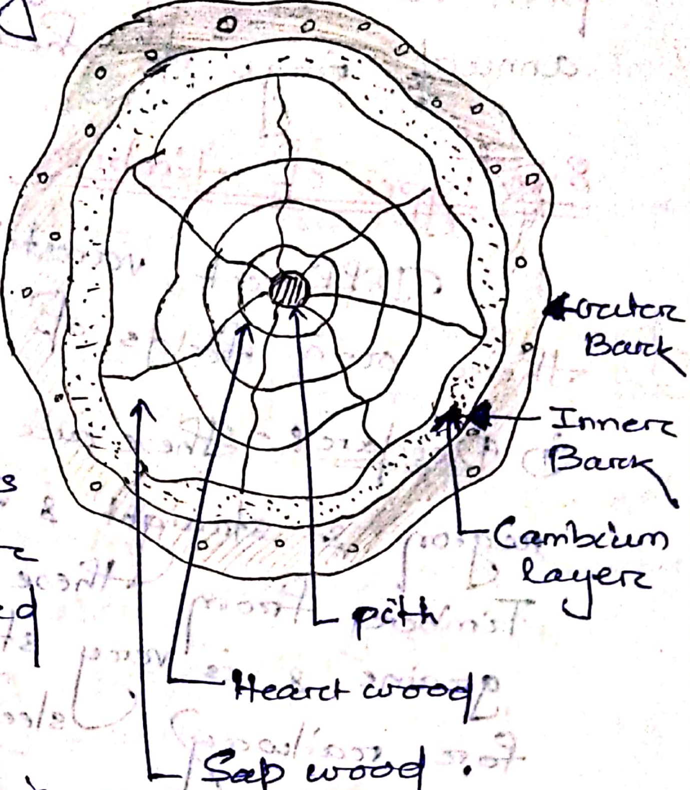
Design of connection between plate & flange

Bolts/welds joining the plates & flanges are to be designed.

Timber Structures

Timber is one of the earliest building material used for beams, columns, roofs, doors, windows, bridges etc.

Section of a timber



The **outer bark** is the outermost layer of tree & consists of fissures and cracks.

The **inner bark** is just inside the outer bark & get's protected by outer bark.

Cambium layer is the ~~in~~layer between innerbark & sapwood. This part represents the sap which has not yet been converted to sap wood.

Sap wood is the region between the cambium layer & heart wood. It is light in weight & also light in colour & is formed by sap.

Heart wood is the zone of inner rings surrounding the pith. It is the dead wood providing rigidity to the tree. It produces strong timber meant for construction.

Pith is the innermost region of tree.
Medullary rays are radiating fibres from pith to cambium layers, which hold the annual rings of heart wood & sap wood.

8.4 Types of timber

Different varieties of timber have their own fields of utility.

(i) Deodar :- These are found in Himalayan region & straight & tall with short branches. Timber from these trees has well defined grains & it's very strong. These are used for railway sleepers, bridges & piles.

(ii) Sal :- It is found in the foothills of Himalayas. It provides a very good variety of timber. It is hard, coarse grained, heavy & durable. It is used for bridges, railway sleepers. Since it's very hard & difficult to work, it is not suited for ornamental work.

(iii) Teak :- It provides a strong, durable timber with a dark brown colour. It is light weight & can easily be worked. It takes a fine polish. It is used for furnitures, cabinets, decorative pieces etc.

(iv) Mango :- It is found almost everywhere in India. It has a lot of sap & moisture. So it is of inferior quality & cheap. It is used for making low quality doors and windows.

(v) Shisham :- It is generally found in central India. These are heavy and strong. It is very satisfactory for making ~~timber~~ furniture.

(vi) Kail :- It is seen in Himalayas. It is hard, durable and coarse grained. It is used for furnitures and railway sleepers.

(vii) Sissoo :- It is found in Maharashtra, Bengal, Punjab & UP. It is also strong and durable & used for making furniture. It is reddish brown in colour.

8.1.2 Defects in timbers ~~structure~~

The common defects shown in timber are as follows :-

(i) Shakes :- The discontinuities or separation between annual rings ~~due to~~ is known as shakes. It affects the shear strength.

It may be of two types :-

(a) Heart shake :- It is a split which originates at the centre and runs from pith towards the sap wood.

(b) Star shake :- These are radial splits wider in outer region & narrow towards the centre.

(v) Cup Shake :- These are curved splits seen between the annual rings.

(ii) Ringgall :- A ringgall is characteristic swelling due to fresh growth of sapwood over cutting. When breach is cut off in an irregular manner, the freshly developed layers donot joint properly with old one. These result into cavity from which decay starts.

(iii) Knot :- It is an assembly of roots of small breaches which breaks the continuity of fibres causing reduction of strength.

(iv) Rupture :- It is an injury caused due to crushing of fibre.

(8) ~~Wane~~ :-

8.1.3. Grading of timber :-

The cut sizes of timber stock for structural purpose are graded after seasoning. It is of following 3 grades :-

a) Select grade
b) Grade I
c) Grade II.

	Select grade	Grade I	Grade II
Slope of grain wane	< 1 in 20 < $\frac{1}{8}$ x width	< 1 in 15 < $\frac{1}{6}$ x width	< 1 in 12. < $\frac{1}{4}$ x width
Worm holes	Except those powder post ~~beetles~~ are permissible	do	do

8.1.4 Permissible Stresses

It is checked for various groups of timbers. Those groups are :—

(a) Group-A — E above $12.6 \times 10^3\ N/mm^2$
f_b above $18\ N/mm^2$.

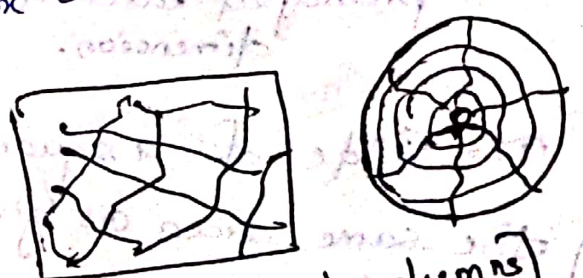
(b) Group-B — E above $9.8 \times 10^3\ N/mm^2$
& upto $12.6 \times 10^3\ N/mm^2$
f_b above $12 \times 10^3\ N/mm^2$ & upto $18 \times 10^3\ N/mm^2$.

(c) Group-C — E above $5.6 \times 10^3\ N/mm^2$ &
upto $9.8 \times 10^3\ N/mm^2$.
f_b above $8.5 \times 10^3\ N/mm^2$ &
upto $12\ N/mm^2$.

Their permissible stresses has been given in table-1 of IS 883:1994.

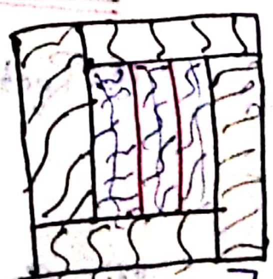
8.2 Axially Loaded Timber columns (Solid Box & Built up Sections)

Timber columns may be solid columns, box columns or built-up columns.



[Solid columns]

[Box column]



[Built-up column]

→ A solid column means column which consists of a single piece of wood. They may be rectangular or circular.

→ Columns are again divided into 3 categories based on S/D ratio :- (Clause: 7.6.1).

L = unsupported length of column

D = least lateral dimension of the column section.

(a) Short column :- $S/D \not> 11$.

(b) Intermediate column :- $11 < \frac{S}{D} < K$

$$K = 0.702\sqrt{\frac{E}{\sigma_{cp}}}.$$

E = modulus of elasticity

σ_{cp} = allowable compressive stress.

(c) Long column :- $\frac{S}{D} > K$.

(d) Solid column :- $S/D < 50$.

→ The least dimension D shall be taken as follows :-

(i) Rectangular Column : D = lesser of two principal lateral dimension.

(ii) Circular Column : D = Side of a square having the same area as given in column.

(iii) Tapered solid Column : $D = d_2 + \frac{d_1 - d_2}{3}$

d_2 = least lateral dimension of ~~largest~~ smaller end.

→ A solid column means column which consists of a single piece of wood. They may be rectangular or circular.

→ Columns are again divided into 3 categories based on S/D ratio :- (clause : 7.6.1).

L = unsupported length of column

D = least lateral dimension of the column section.

(a) Short column :- $S/D \ngtr 11$.

(b) Intermediate column :- $11 < \frac{S}{D} < K$

$$K = 0.702\sqrt{\frac{E}{\sigma_{cp}}}.$$

E = modulus of elasticity

σ_{cp} = allowable compressive stress.

(c) Long column :- $\frac{S}{D} > K$.

(d) Solid column :- $S/D < 50$.

→ The least dimension D shall be taken as follows :-

(i) Rectangular Column : D = lesser of two principal lateral dimension.

(ii) Circular Column : D = Side of a square having the same area as given in column.

(iii) Tapered solid Column : $D = d_2 + \frac{d_1 - d_2}{3}$

d_2 = least lateral dimension ~~for~~ of ~~[illegible]~~ smaller end.

& d_1 = least lateral dimension of larger end.
However D shall not be greater than $1.5 d_2$.

→ Safe permissible stress in column :-

(i) For short column : $f_c' = f_{cp}$ (7.6.1.1)

f_{cb} can be found from table-1.

(7.6.1.2)

(ii) For Intermediate column :-

$$f_c = f_{cb}\left[1 - \frac{1}{3}\left(\frac{s}{K_8 d}\right)^4\right].$$

s = unsupported overall length of column in mm.

K_8 = constant $= \frac{\pi}{2}\sqrt{\frac{U \times E}{5 f_{cp}}}$

(iii) For long columns :- (7.6.1.3).

$$f_c = \frac{0.329 E}{(s/d)^2}.$$

⇒ For Box & Built-up column :- [7.6.2].

(i) For short column, where $\frac{s}{\sqrt{d_1^2 + d_2^2}} < 8$.

$$\boxed{f_c = q f_{cp}}$$

(ii) For intermediate columns, where

$$8 \leq \frac{L}{\sqrt{d_1^2 + d_2^2}} < K_9,$$

$$\boxed{f_c = q f_{cb}\left[1 - \frac{1}{3}\left\{\frac{L}{K'\sqrt{d_1^2 + d_2^2}}\right\}^4\right]}$$

$K_9 = \frac{\pi}{2}\sqrt{\frac{UE}{5 f_{cb}}}$, $U = 0.8$ for 25 mm thick planks
= 0.6 for 50 " "

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(iii) For long column: $\frac{L}{\sqrt{d_1^2+d_2^2}} > K_q$

$$f_c = \frac{0.329\,UE}{\left(\frac{S}{\sqrt{d_1^2+d_2^2}}\right)^2}$$

Value of U & q is given in 7.6.2.5

Q:- A timber column is 200mm × 200mm in section having an unsupported length of 2m. Assume column to be of Sal wood. Find the safe axial load.

(4) to find f_c
↑
(3) Check $\frac{S}{D}$
(2) find S
(1) find D

(1) Least lateral dimension = 200mm
(as both dimensions are same)

(2) Unsupported length of column S = 2m
= 2000mm.

(3) $\frac{S}{D} = \frac{2000}{200} = 10 < 11$

(4) For $\frac{S}{D} < 11$, column is solid column &
for solid column :-

$f_c = f_{cp}$
$= 11\ N/mm^2$ for sal wood.

So, Safe axial load = 11 × Area = 11 × 200 × 200
= 440000 N.

Permissible axial load for

Sal wood = 11 N/mm2

Deodar wood = 7 N/mm2.

for Box or Built-up columns, safe axial load = 80% of the safe stress value of corresponding solid column.

Q:- A timber column is 200mm x 200mm in section having an unsupported length of 3.5m. Find safe axial load for column assuming it to be sal wood.

Unsupported length of column $S = 3500mm$

Least lateral dimension $D = 200mm$.

$$\frac{S}{D} = \frac{3500}{200} = 17.5 > 11.$$

So find $K_8 = 0.702\sqrt{\frac{E}{f_{cp}}}$ — $f_{cp} = 11\ N/mm^2$ for sal solid column

$$\Rightarrow K_8 = 0.702\sqrt{\frac{1.08\times10^4}{11}} = 22.$$

$$\boxed{E = 1.08\times10^4\ N/mm^2}$$

So $11 < \frac{S}{D} < 22$, → intermediate solid column.

Hence, $f_c = f_{cp}\left[1 - \frac{1}{3}\left(\frac{S}{K_8 d}\right)^4\right]$

$$= 11\left[1 - \frac{1}{3}\left(\frac{3500}{22\times200}\right)^4\right]$$

$$= 9.53\ N/mm^2.$$

So safe axial load = f_{cp} × Area

$= 9.53 \times 200 \times 200$

$= 381200\,N$.

Q:- A timber column is 200mm × 200mm in section having an unsupported length of 4.75m. Find the safe axial load assuming column to be of sal wood.

Unsupported length of column = S = 4750 mm.

D = 200mm.

$$\frac{S}{D} = \frac{4750}{200} = 23.75 > 11$$

$$K_8 = 0.702\sqrt{\frac{E}{f_{cp}}} = 0.702\sqrt{\frac{1.08 \times 10^4}{11}}$$

$= 22$.

So $\frac{S}{D} > K$ also. ⟶ Long column.

Here, $$f_c = \frac{0.329E}{\left(S/D\right)^2}$$

$$= \frac{0.329 \times 1.08 \times 10^4}{(23.75)^2} = 6.3\,N/mm^2$$

Safe axial load = $63 \times 200 \times 200$

$= 252\,000\,N$.

Q:- A timber column is made of deodar wood and is 200mm in diameter. The effective length of column is 1.25m

(ii)

Find the safe axial load. The safe axial load for deodar wood for a short column may be taken as 7 N/mm²

Least lateral dimension :—

D = side of square of same area.

i.e. $D \times D = \frac{\pi}{4} \times 200^2$

$\Rightarrow D = \sqrt{\frac{\pi}{4} \times 200^2} = 177.2$ mm.

Effective length = 1250 mm.

so $\frac{\ell}{D} = \frac{1250}{177.2} = 7.05 < 11 \longrightarrow$ short column

so $f_c = 7$ N/mm²

Safe load = 7 × area

$= 7 \times \frac{\pi}{4} \times 200^2 = 219911$ N.

Q :- Design a solid wood column to following requirements :—

Load on column = 450 kN

Column Material = Deodar

Location = Outside

Effective length = 3 m.

→ Assume column is short column.

i.e. $\frac{\ell}{D} < 11$

$\Rightarrow \frac{3000}{D} < 11 \rightarrow D > \frac{3000}{11} = 272.7$ mm

take D = 300 mm.

For deodar wood $f_c = 7 \, N/mm^2$ ①

So Area of column required $= \dfrac{\text{Load in N}}{7 \, N/mm^2}$

$= \dfrac{450 \times 1000}{7} = 64286 \, mm^2$.

So other dimension $= \dfrac{64286}{300} = 215 \, mm.$

$\simeq 220 \, mm.$

So take column dimension $= 220 \times 300$.

Check for $\frac{S}{D}$ ratio

$\dfrac{S}{D} = \dfrac{3000}{220} = 13.63 > 11$

So assumption is wrong.

$K_8 = 0.702 \sqrt{\dfrac{E}{f_c}} = 0.702 \sqrt{\dfrac{1.08 \times 10^4}{7}}$

$= 27.57$.

So $11 < \dfrac{S}{D} < K \longrightarrow$ Intermediate Column.

So Safe stress $f_c = f_{cp} \left[1 - \frac{1}{3} \left(\dfrac{S}{K_8 d} \right)^4 \right]$

$\Rightarrow f_c = 7 \left[1 - \frac{1}{3} \left(\dfrac{13.63}{27.57} \right)^4 \right]$

$= 6.86 \, N/mm^2$.

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Now, Load carrying capacity of column = 6.86 × area

$= 6.86 \times 220 \times 300$

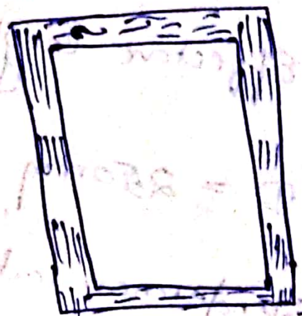
$= 452760\ N.$

So Load carrying capacity = 452.76 KN > 450 KN

[OK].

So provide section of 220mm × 300mm

Box and Built-up Columns :- These columns consists of timber components connected to each other by screws, bolts, nails etc.



[Box Column]

→ These are classified into :-

(a) Short Column

$$\frac{s}{\sqrt{d_1^2+d_2^2}} < 8$$

$$f_c = q f_{cb}$$

(b) Intermediate Columns

$$8 < \frac{s}{\sqrt{d_1^2+d_2^2}} < K_1 \quad \text{where } K = \frac{\pi}{2}\sqrt{\frac{UE}{5 q f_s}}$$

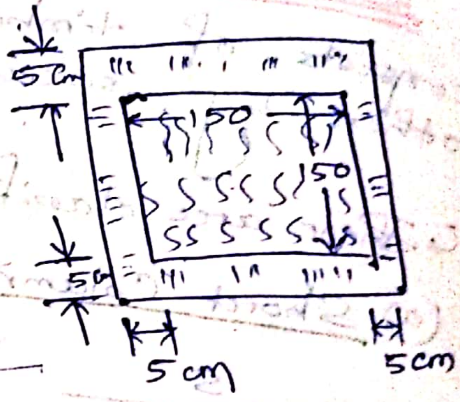
$$f_c = q f_{cb}\left[1 - \frac{1}{3}\left\{\frac{s}{K_1\sqrt{d_1^2+d_2^2}}\right\}^4\right]$$

(c) Long Columns

$$\frac{s}{\sqrt{d_1^2+d_2^2}} > K \quad ; \quad f_c = \frac{0.329\, UE}{\left(\frac{s}{\sqrt{d_1^2+d_2^2}}\right)^2}$$

Built-up Columns are designed as solid columns, provided stresses actually taken by column donot exceed 80% of specified permissible stress.

Q:- A timber column is a built up section consisting of a central solid core of 150mm×150mm with 4 supporting planks as shown in figure. The effective length of column is 2m. Find safe load for the column assuming it to be sal wood



Effective length $s = 2m$

$= 2000 mm$

$D = 250 mm$

Slenderness ratio $= \frac{2000}{250}$

$= 8.$

So, it is a short column.

f_c = 80% of that taken by solid column

$= \frac{80}{100} \times 11 \ N/mm^2$ [∵ for sal wood safe stress = 11%]

$= 8.8 \ N/mm^2$

Safe load = Safe stress × area

$= (8.8 \times 250 \times 250) \ N$

$= 550000 \ N.$

$= 550 \ KN.$

Q:- Determine the safe axial load on a circular column of sal wood of diameter 20 cm and length 3.5 m

Q:- Design a 5 m long rectangular box column built up by 5 cm thick Deodar planks to carry an axial load of 400 kN.

Q:- A timber column supports an axial load of 800 kN. The effective length of the column is 3 m. Taking the column to be of sal wood design the column as a built up section.

$s = 3m$

Assuming the column to be short column.

$$\frac{s}{d} < 11 \qquad d > \frac{3000}{11}$$

$$\Rightarrow \frac{3000}{d} < 11 \Rightarrow$$

$\Rightarrow d > 272.7$ mm.

take $d = 300$ mm.

Safe stress = 0.8 × safe stress for the solid column.

$= 0.8 \times 11$

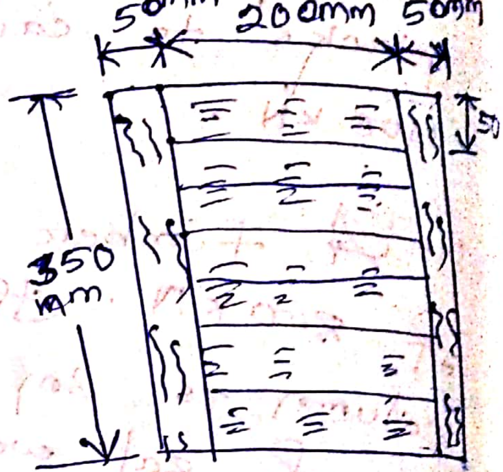
$= 8.8 \ N/mm^2$.

Area required for column section

$$= \frac{800 \times 1000}{8.8} = 90909 \, mm^2$$

So othere dimension $= \frac{90909}{300} = 350$

So provide $(300 \times 350) \, mm^2$ column.



↳ Provide planks of 50mm

↳ provide planks of 50mm, 2 nos. on both side &

↳ length of middle plank

$= 300 - (2 \times 50)$

$= 200 \, mm$

↳ No. of middle planks $= \frac{350}{50} = 7$ nos.

Actual area of column $= 300 \times 350$

$= 105000 \, mm^2$

1) Check for assumption

$\frac{S}{D} = \frac{3000}{300} = 10 < 11$ [OK].

2) Check for load carrying capacity

load carrying capacity = Permissible Stress × Area

$= (11 \times 0.8) \, N/mm^2 \times 105000$

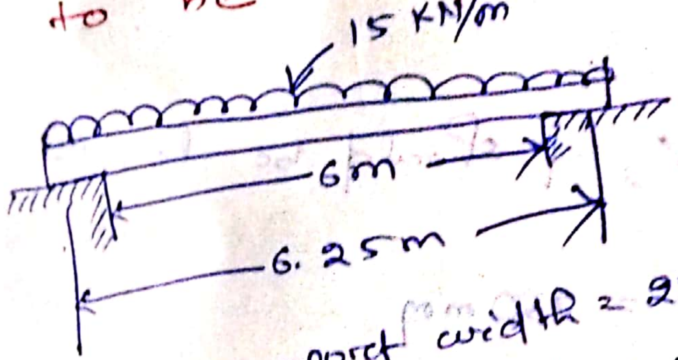
$= 8.8 \times 105000$

$= 924000N = 924KN > 800$

8.3 Timber Structure in flexure or Timber Beams

IS code specifies various specification of beams which are given in clause 7.5 (Page-11 to 13) in IS 883:1994.

Q:- A timber beam having a clear span of 6m carries a UDL of 15 KN/m including S.W. of beam. Assuming beam to be made of deodar wood & design



Cross Section can be found from b & d

$z = \frac{bd^2}{6}$

find z from $z = \frac{M}{\sigma}$

find b from $\frac{span}{50}$ or 50

σ from table-1

find $M = \frac{wl^2}{8}$

support width = 250mm.

Assume support width = 6 + 0.25

So effective span = 6.25m. = (6250 mm)

So bending moment $M = \frac{wl^2}{8}$

$\rightarrow M = \frac{15 \times 6.25^2}{8} = 73.24 KNm$

Permissible bending stress for deodar wood = 11.6

So $Z = \frac{M}{\sigma} = \frac{73.24 \times 10^6}{11.6} = 7324000 mm^3$

$b = 50\,mm$ or $\frac{Span}{50}$ whichever is greater

$= 50\,mm$ or $\frac{6250}{50}$

$= 250\,mm.$

so $Z = \frac{bd^2}{6} \Rightarrow d^2 = \frac{Z \times 6}{b}$

$\Rightarrow d = \sqrt{\frac{7324000 \times 6}{250}} = 419\,mm$

take $d = 420\,mm.$

(1) <u>Check for depth</u> :-

According to code, d should be less than 3b.

$3b = 3 \times 250 = 750\,mm$

so $d = 420 < 3b$ [OK]

(2) <u>Check for Shear</u>

Maximum Shear force $= \frac{wl}{2}$

$= \frac{15 \times 6.25}{2} = 46.875\,KN.$

Max. Shear stress for rectangular beam

$= \frac{3V}{2bd} = \frac{3}{2} \times \frac{46.875 \times 10^3}{250 \times 420}$

$= 0.64\,N/mm^2$

From table - 1,

Permissible sheare stress = 3 N/mm^2.

as $0.64 < 3$ [OK].

(3) Check for deflection

Max. deflection $= \frac{5}{384} \frac{wl^3}{EI}$

$= \frac{5}{384} \times \frac{15 \times 10^3 \times (6.25 \times 10^3)^3}{0.95 \times 10^4 \times 1.543 \times 10^9}$.

$I = \frac{bd^3}{12} = \frac{250 \times 420^3}{12}$

$= 1543 \times 10^9$ mm^4.

So Max. deflection $= \cancel{\frac{5}{384}} \times 20.33$ mm.

Permissible deflection $= \frac{l}{240}$

$= \frac{6250}{240} = 26.04$ mm.

Max. deflection < 26.04 [OK].

(4) Check for bearing :-

Reaction at bearing $= \frac{wl}{2} = \frac{15 \times 6.25}{2}$

$= 46.875$

Bearing stress $= \frac{46.875 \times 10^3}{(250 \times 250)} = 0.75$ N/mm^2 ← bearing area

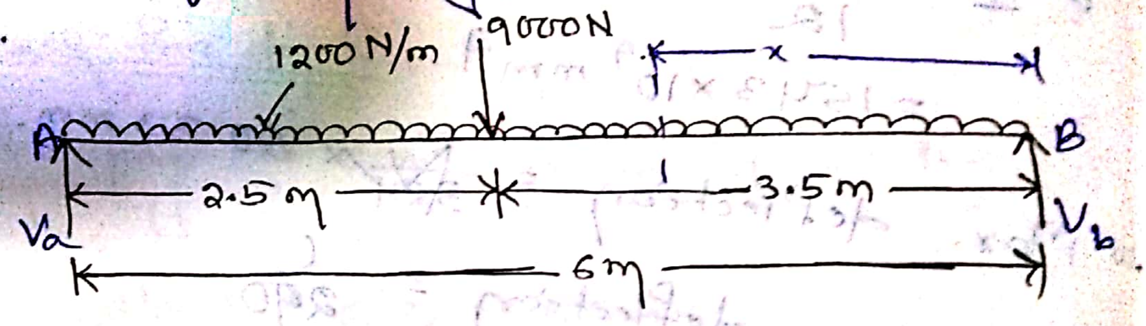
Permissible bearing stress = 7.50 N/mm^2 > 0.75 N/mm^2

[OK]

4

Q:- A timber beam is freely supported, supports 6m apart. It carries a u of 12 KN/m & a concentrated load of 9 KN at 2.5m from left support. If Stress in timber is not to exceed 8 N/m then design a suitable section making the depth twice the width.

→ Let the V_a and V_b be the reactions at left and right supports.



Calculation of V_a and V_b

$\Sigma M_a = 0$

$\Rightarrow V_b \times 6 = 1200 \times 6 \times \frac{6}{2} + 9000 \times 2.5$

$\Rightarrow V_b = 39750 N$

$V_a = (\text{total load}) - V_b$

$= (1200 \times 6 + 9000) - 39750$

$= 41250 N.$

So max. shear force $= 41250 N.$

$= 41.25 KN.$

Calculation of max. bending moment

Bending moment is maximum at the point of zero shear.

Let at x from B, shear force is 0.

so $V_b - 1200 \times x = 0$

$\Rightarrow 39750 - 1200x = 0 \Rightarrow x = \frac{39750}{1200}$

$\Rightarrow x = 3.3125$ m.

So Max. B.M. $= (39750 \times 3.3125) - \left(1200 \times 3.3125 \times \frac{3.3125}{2}\right)$

$\Rightarrow M_{max} = 65836$ Nm

$= 65836 \times 10^3$ Nmm.

$= 65.836 \times 10^6$ Nmm

Finding Dimensions of beam

Given, $d = 2b$.

We know $\frac{M}{\sigma} = z$

$\Rightarrow$ where $z = \frac{bd^2}{6} = \frac{b \times (2b)^2}{6} = \frac{4b^3}{6}$

So $\frac{M}{\sigma} = \frac{4b^3}{6} \Rightarrow b^3 = \frac{6 \times M}{4\sigma}$

$\Rightarrow b = \frac{6 \times 65.836 \times 10^6}{4 \times 8}$

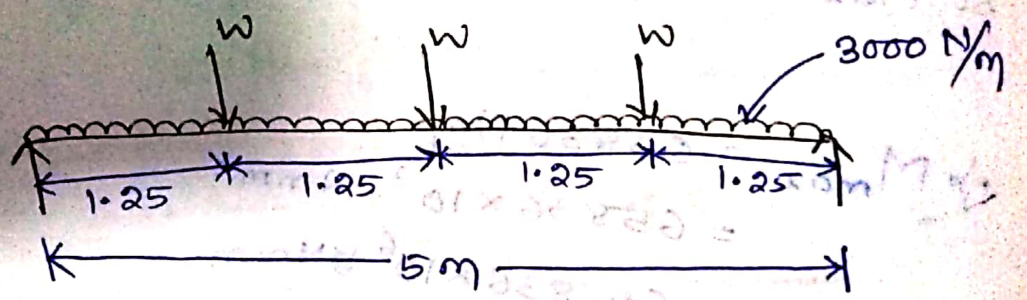
$= 231$ mm

$d = 2b = 462$ m

Provide beam section of (231×462) mm²

6

Q:- A timber beam is 160mm wide and 300mm deep and is simply supported on a span of 5m. It carries a UDL of 3000 N/m over whole span & ~~having~~ three equal concentrated loads W N each placed at mid span and quarter span points. If stress in timber is not to exceed 8 N/mm², find the maximum value of W.



total load = $(3000 \times 5) + 3W$.

So reaction at each support = $\frac{3000 \times 5 + 3W}{2}$

$= (7500 + 1.5W)$ N.

Max. BM will occur at mid span.

Max. BM = $(7500 + 1.5W)\frac{5}{2} - 3000 \times \left(\frac{5}{2}\right) \times \left(\frac{5/2}{2}\right)^2$

Vertical reaction

$- W \times 1.25$

$= 18750 + 3.75W - 9375 - 1.25W$

$= (9375 + 2.5W)$ Nm.

Moment of resistance of the section is $\frac{M}{\sigma} = Z$

$\Rightarrow M = \sigma \times Z = \sigma \times \frac{bd^2}{6}$

$= 8 \times \frac{160 \times 300^2}{6}$

So for equilibrium, Max. BM is equal to MOR.

$\Rightarrow (9375 + 2.5W) \times 10^3 \text{ N mm} = 8 \times \frac{160 \times 300^2}{6}$

$\Rightarrow 9375 + 2.5W = 19200$

$\Rightarrow W = 3930 \text{ N}$.

Q:- A timber beam 100 mm wide and 150 mm deep supports a UDL over a span of 2m. If safe stresses are 28 N/mm² in bending and 2 N/mm² in shear, calculate the maximum load which can be supported by the beam.

Let the maximum UDL on beam is (w) N.

(1) Max. Bending stress = 28 N/mm²

So $\frac{M_R}{\sigma} = Z \Rightarrow M_R = Z \times \sigma$

$\Rightarrow M_R = \frac{bd^2}{6} \times 28 = \frac{100 \times 150^2}{6} \times 28$

Maximum bending moment $M = \frac{wl^2}{8}$

$M_{max} = \frac{w \times 2^2}{8} \text{ Nm}$

8

For equilibrium $M_{max} = M_R$

$\Rightarrow \frac{W \times 2}{8}$ Nm $= \frac{100 \times 150^2}{6} \times 28$

$\Rightarrow \frac{W \times 2}{8} \times 1000$ Nmm $= \frac{100 \times 150^2}{6} \times 28$

$\Rightarrow W = 42000 N = 42 KN$

(2) From shear stress consideration,

Max. shear stress = Permissible shear stress.

$\Rightarrow$ Max^m shear stress = ~~Max shear Rectangle~~

$= \frac{3}{2} \times \frac{V}{bd} = \frac{3}{2} \times \frac{W}{2bd}$

$\left[V = W/2\right]$ $\quad = \frac{3}{4} \times \frac{W}{100 \times 150}$

Permissible shear stress $= 2 N/mm^2$.

Equating these :—

$\frac{3}{4} \times \frac{W}{100 \times 150} = 2 \Rightarrow W = 40{,}000 N$

$\Rightarrow W = 40 KN.$

So from bending moment consideration & shear stress consideration,

$W = 40 KN$ (min^m value)

& UDL $= \frac{W}{l} = \frac{40}{2} = 20 KN/m.$

9

Q:- A simply supported timber beam 100 mm wide and 200 mm deep carries a point load W at the mid point of the span. The Permissible stresses in flexure and shear are 10 N/mm² and 1.5 N/mm² respectively. Ignoring self wt. of the beam, calculate the span length below which shear stress will govern the safe load and above which bending stress will govern safe load.

Let W = safe point load at mid span.
& l = span in m.

(i) Bending moment consideration

Max. B.M. $M_{max} = \frac{Wl}{4}$ N m.

$\Rightarrow M_{max} = \frac{Wl}{4} \times 10^3$ N mm.

(ii) Shear force consideration

Max. shear force $= \frac{W}{2}$ N.

Max. shear stress $= \frac{3}{2} \times \frac{V}{bd}$.

$= \frac{3}{2} \times \frac{W/2}{100 \times 200}$

Max. shear stress = Permissible shear stress

$\Rightarrow \frac{3}{2} \times \frac{W/2}{100 \times 200} = 1.5$

$\Rightarrow (W/2) = 20 \times 10^3$ N

$\Rightarrow W = 40 \times 10^3$ KN

Now, from bending moment consideration,

$$M_{max} = M_{Resistance}.$$

$$\Rightarrow \frac{wl}{4} \times 10^3 \text{ Nmm} = Z \times \sigma$$

$$\Rightarrow \frac{40 \times 10^3 \times l}{4} \times 10^3 = \frac{100 \times 200^2}{6} \times (10)$$

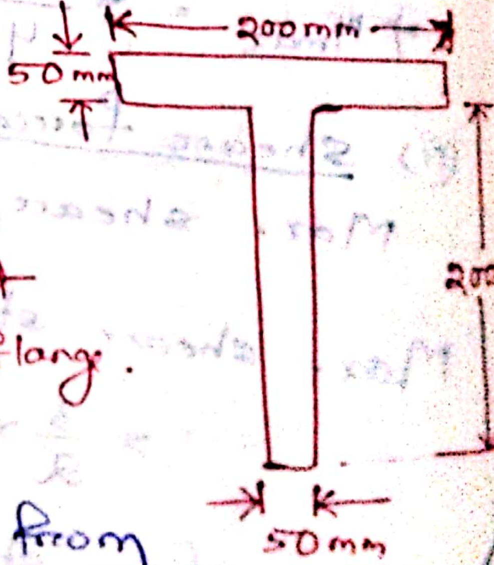
$$\Rightarrow l = \frac{2}{3} \text{ m}.$$

If span $> \frac{2}{3}$ m, then bending stress will govern safe load.

& if span $< \frac{2}{3}$, the shear " " " "

Q: — The T-shaped cross-section of a beam shown in fig. is subjected to a vertical shear force of 100 KN. Calculate shear stress at the neutral axis and at the junction of web & flange.

$I = 1.134 \times 10^8 \text{ mm}^4.$



Distance of neutral axis from top edge $= \bar{y} = \dfrac{\left(200 \times 50 \times \frac{50}{2}\right) + \left(50 \times 200 \left(\dots\right.\right)}{(200 \times 50) + (50 \times 200)}$

$= 87.50$ mm.

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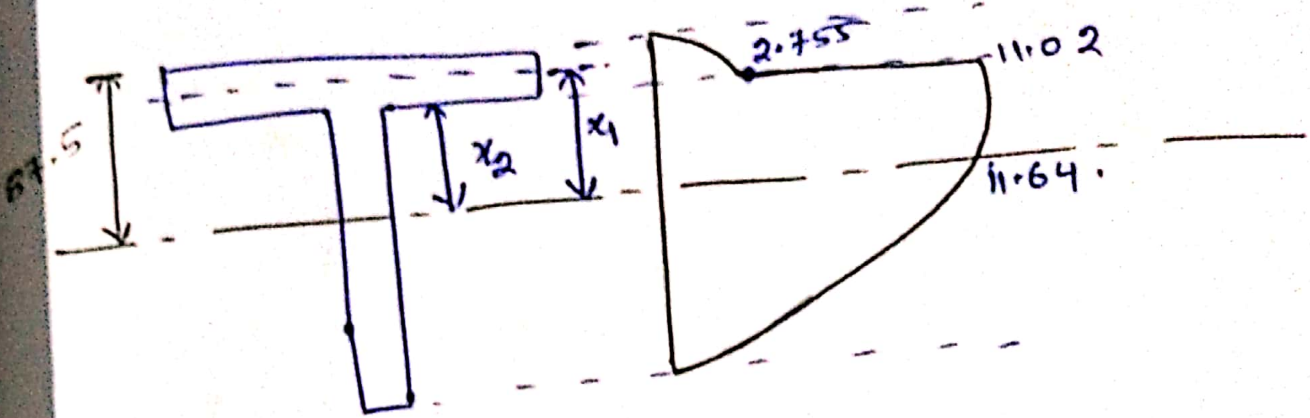
Shear stress at neutral axis

$$= \frac{Va\bar{y}}{Ib}.$$

where, $Va = (100\times10^3 N) \times \left[(200\times50)\times62.5 + (37.5\times50)\times\frac{37.5}{2}\right]$



$x_1 = 87.5-25$
$= 62.5$ mm.

$x_2 = 87.5-50 = 37.5$ mm

So shear stress $= \frac{(100\times10^3)\left[200\times50\times62.5+37.5\times50\times\frac{37.5}{2}\right]}{1.134\times10^8\times50}$

$= 11.64\ N/mm^2.$

Shear stress in junction of web & flange

$= \frac{Va\bar{y}}{Ib} = \frac{(100\times10^3)\times(50\times200\times25)}{1.134\times10^8\times50}$

$= 11.02\ N/mm^2.$

Shear stress at flange :-

$200 \longrightarrow 11.02$

50 mm $\longrightarrow \frac{11.02}{200}\times50 = 2.755\ N/mm^2$

5.0 Steel Column Bases & Foundation

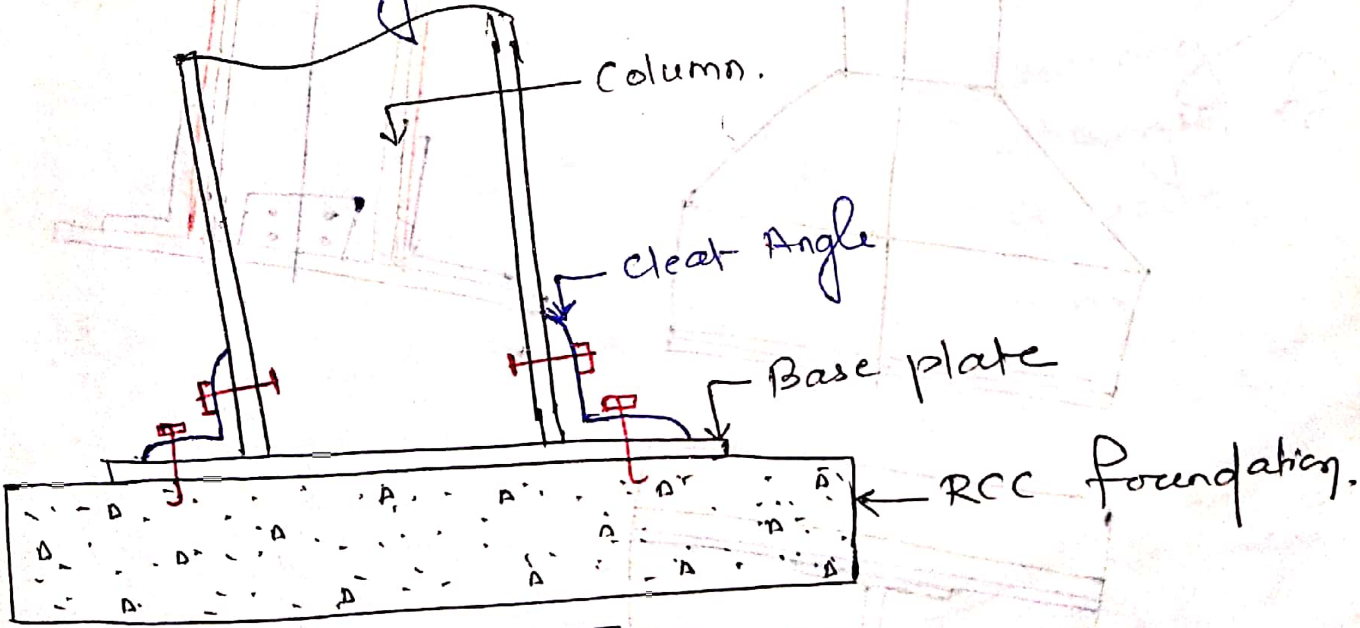
Column bases transmit the column load to the concrete or masonary foundation blocks. The column base spreads the load on wider area, so that intensity of bearing pressure on the foundation block is within the bearing strength.

5.1 Types of Column Base

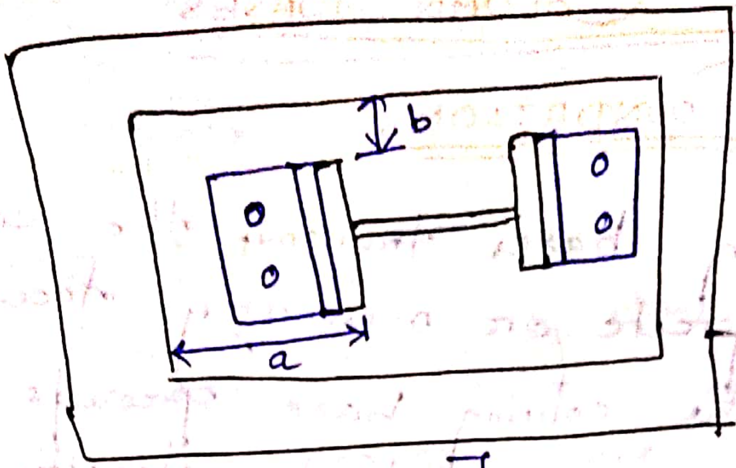
These are of two types:-

1) Slab Base
2) Gusseted Base

1) **Slab Base**:- These are used in columns carrying small loads. In this type, the column is directly connected to the base plate through cleat angles.



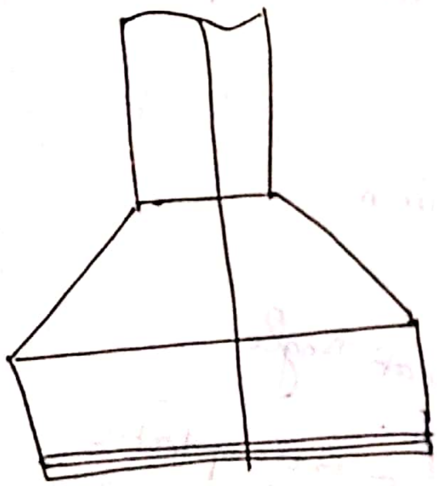
[Elevation of Slab Base]

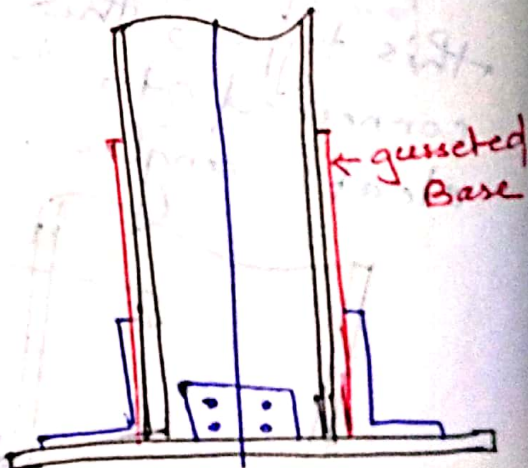


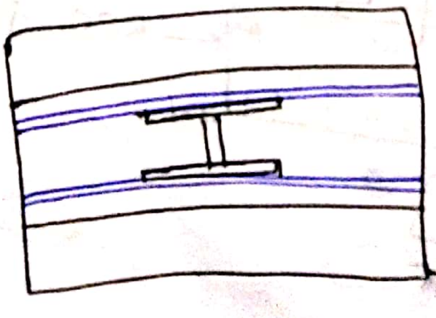
[Plan of Slab base].

2) <u>Gusseted Base</u> :- For columns carrying heavy loads gusseted bases are used where column is connected to base plate through gussets.

The load is transferred to the base partly through bearing and partly through gussets.







5.2 Design of Slab Base

The design of slab base consists in finding size and thickness of slab base. In designing, it is assumed that the pressure is distributed uniformly under the slab base.

PROCEDURE :-

(1) Find bearing strength of concrete $= 0.45 f_{ck}$

(2) Area of base plate required

$$= \frac{\text{External load in N}}{\text{Strength of concrete in N/mm}^2}$$

$$= \frac{P_u}{0.45 f_{ck}}, \quad P_u = \text{factored load.}$$

(3) Select the length and width of base plate by leaving projections behind column section.

(4) Check for the intensity of pressure due to external load :-

$$w = \frac{P_u}{\text{Area of base plate}} < \text{Bearing strength of concrete.}$$

(5) Minimum of thickness required.

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

should be more than flange thickness.

t_f = flange thickness.

t_s = thickness of base plate

(6) Design the connection by using bolts or welding.

for bolting use
4 20mm bolts and
2, ISA 6565, 6mm

for welding find L_w for fillet weld
~~which should be more than~~

Q:- Design a slab base for column ISHB 300 @ 577 N/m carrying an axial factored load of 1000 KN. M20 concrete is used for the foundation.

① find thickness. length & width.

⑤ $t_s = \sqrt{\dfrac{2.5w(a^2 - 0.3b^2)\gamma_{m0}}{f_y}} > t_f$

④ find area of base plate

③ Area reqd = $\dfrac{P_u}{0.45 f_{ck}}$

② P_u = external factored load

① $0.45 f_{ck}$ is bearing strength of concrete

② Check bearing pressure = $\dfrac{P_u}{\text{Area of base plate provided}}$ ⑥

⑦ Bearing pressure $\ngtr$ < Bearing strength $(0.45 f_{ck})$

[OK]

3 Design the connection using bolts.

↓

11 find no. of bolts & choose angle section or cleat angle

↓

10 no. of bolts = $\frac{\text{External load } (P_u)}{\text{Bolt value}}$

↓

9 Bolt value = minm of shearing strength and bearing

↓

8 find V_{dsb} & V_{dpb}.

4 Design the connection using welding.

↓

15 find available length & check length required.

↓

14 available length = L − end returns

13 length required = L_w. length can be found by equating weld strength = External load

↓

12 $L_w \times t \times \frac{f_u}{\sqrt{3}} = P_u$.

↓

find L_w.

① Bearing Strength of concrete $= 0.45 f_{ck}$

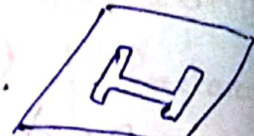
$= 0.45 \times 20 = 9 \ N/mm^2$

② factored external load $P_u = 1000 \ KN$

③ Area required for base plate $= \dfrac{P_u}{0.45 f_{ck}}$

$$= \frac{1000 \times 10^3 \ N}{9 \ N/mm^2} = 111111 \ mm^2$$

④ Provide $360 \times 310 \ mm^2$ size plate.



So area provided $= 111600 \ mm^2$

Check

⑤ Now, bearing pressure $= \dfrac{P_u}{\text{Area provided}}$

$$= \frac{1000 \times 10^3}{111600} = 8.96 \ N/mm^2$$

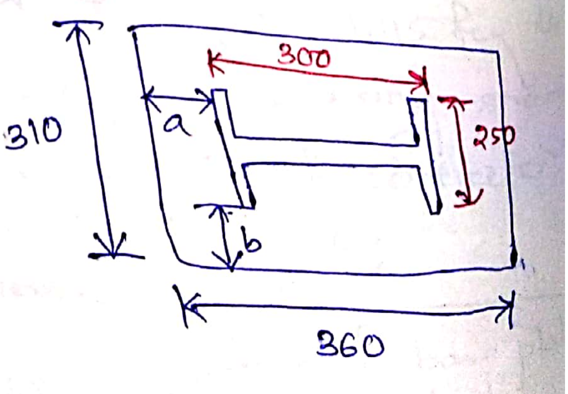
as bearing pressure $= 8.96 \ N/mm^2 <$ bearing Strength of foundation $= 9 \ N/mm^2$,

So it is $\boxed{OK}$.

⑥ Projections are

$$a = \frac{360 - 300}{2} = 30 \ mm$$

$$b = \frac{310 - 250}{2} = 30 \ mm.$$



⑦ Thickness of base plate :-

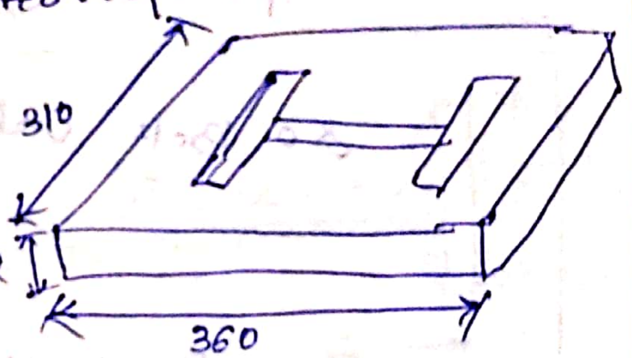
$$t_s = \sqrt{\frac{2.5\,w\,[a^2 - 0.3\,b^2]\,\gamma_{mo}}{f_y}} > t_f$$

$$= \sqrt{\frac{2.5 \times 8.96\,(30^2 - 0.3 \times 30^2) \times 1.1}{250}}$$

$= 7.88$ mm.

$t_f = 10.6$ mm.

as $t_s < t_f$, So provide 12 mm thick base plate.



Design of bolted connection

⑧ Choose M20 bolts of grade 4.6.

Finding V_{dsb}

$$V_{dsb} = \frac{1}{1.25} \times \frac{f_u}{\sqrt{3}}\left(0.78 \times \frac{\pi}{4} \times 20^2\right)$$

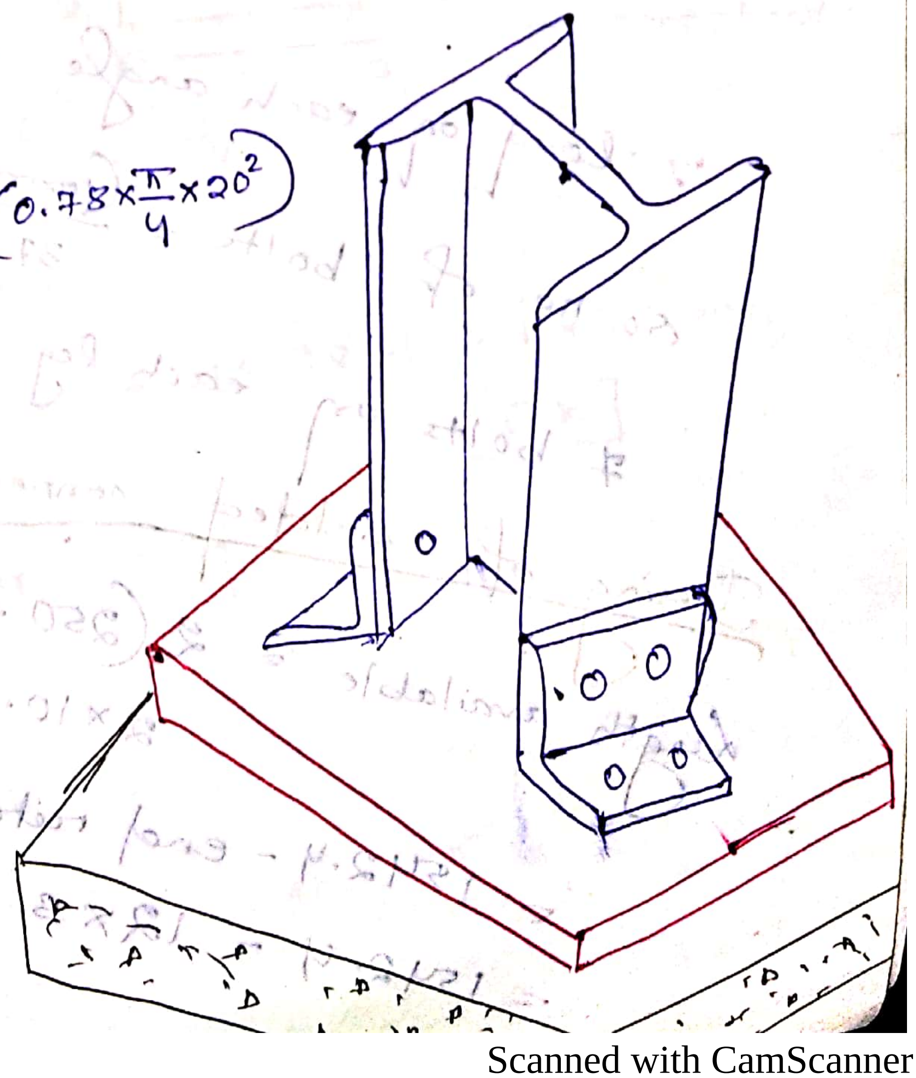
$= 45272$ N.

Finding V_{dpb}

take $e = 90$
$p = 80$

so K_b = least of

$\frac{90}{3\times22}$, $\frac{80}{3\times22} - 0.25$, $\frac{400}{410}$, 1.0,



$K_b = 0.96212$

So $V_{dpb} = \frac{1}{1.25} k_b d t f_u$

$= \frac{1}{1.25} \times 0.96212 \times 20 \times \cancel{10.6}\,6 \times 410$

(6 ie smaller of 10.6 and 6mm)

So $V_{dpb} = 37869.0\ N < 45272.$

So Bolt value $= 37869\ N.$

$= 37.86\ KN.$

No. of bolts $= \frac{Pu/2}{\text{Bolt value}}$

$\because$ load on each angle will be $Pu/2$

So no. of bolts $= \frac{(1000\ KN/2)}{37.8\ KN} = 13.2$

7 bolts on each leg of angle.

Desing of welded connection

Length available $= 2(250 + (250 - 7.6) + (300 - 2 \times 10.6)) -$ end returns

$= 1542.4 -$ end returns

$= 1542.4 - 12 \times S$

S = 6 mm

So Length available = ~~18~~ 1542.4 − 6×12

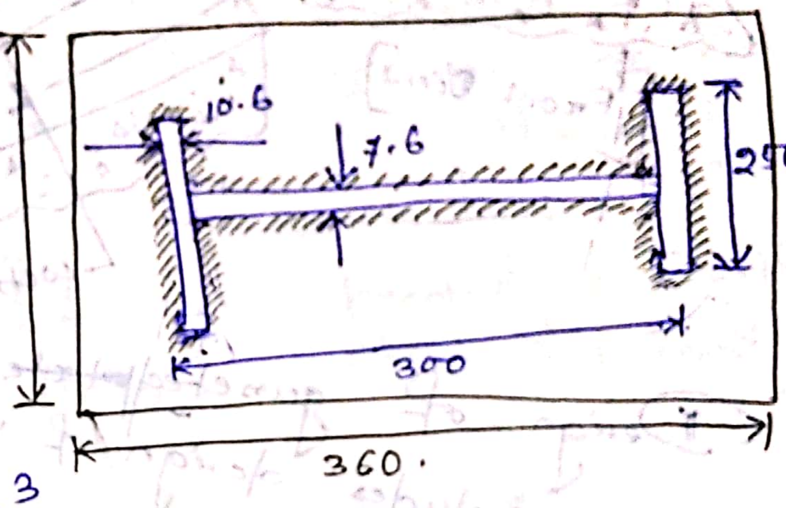
= 1398.4

Length required can be calculated from the following formula :—

Strength of weld = external load

$\Rightarrow L_w t \times \frac{f_u}{\gamma_{mw} \times \sqrt{3}} = 1000 \times 10^3$

$\Rightarrow L_w = \frac{1000 \times 10^3}{t \times f_u} \times \sqrt{3}$



$t = 0.7 S$

$t = 0.7 \times 6$.

So $L_w = \frac{1000 \times 10^3}{0.7 \times 6 \times 410} \times \sqrt{3}$.

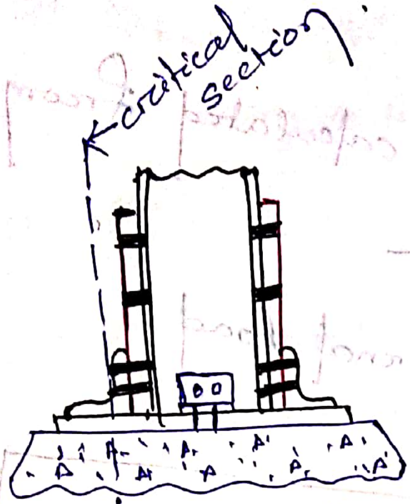
$= \underline{1257 \text{ mm}}$.

Length available = 1398.4 > 1257 mm

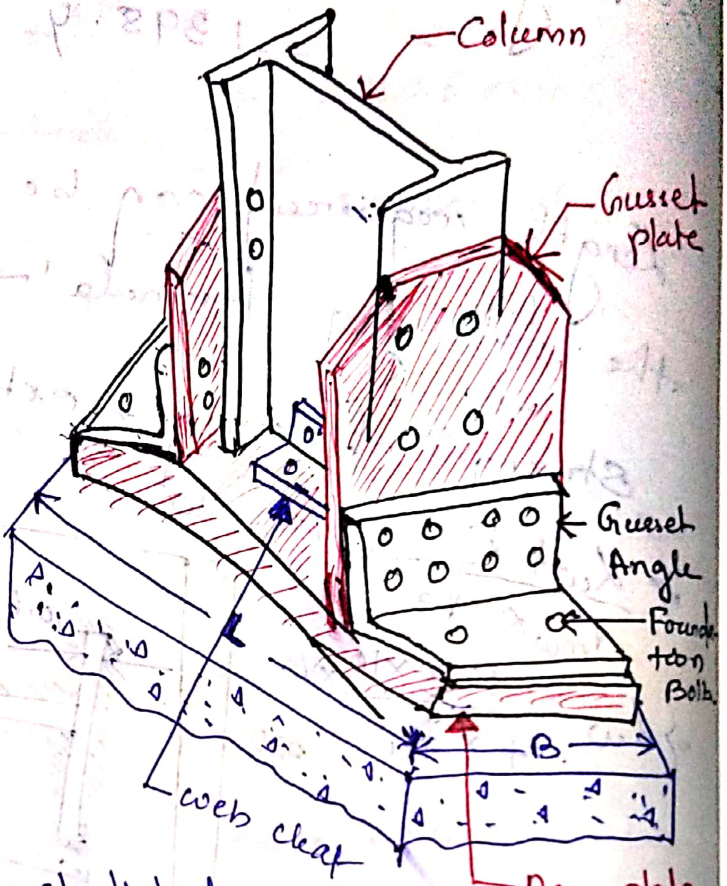
[OK]

So Provide 6 mm weld.

Design Of Gusseted Base



[Front View]



Design of gusseted ~~plate~~.

Base includes design of :—

(i) gusset plate

(ii) Gusset angle

(iii) Angle cleats

(iv) Base plate

(v) fasteners.

↳ Thickness of gusset plate is assumed as 16 mm and its size is so selected that atleast 2 bolts can be accomodated in one vertical line.

↳ The gusset angle is selected from steel table & it's thickness is kept approximately equal to the thickness of gusset plate.

↳ So after adopting different dates; it requires only design of Base plate or the gusset base and connection.

Design of Base plate

Find $(Area)_{req} = \frac{P_u}{0.45 f_{ck}}$

↓

$L \times B = Area_{req}$

↓

Adopt length from length available and find width $B = \frac{Area}{L}$

↓

Provide L & B.

Thickness.

↓

It can be found from flexural strength or bending strength at critical section.

Q:- Design a gusseted base for a column ISHB 350 @ 710 N/m with 2 plates 450mm x 20mm. The column carrying a factored load of 3600 KN is to be supported on concrete pedestal built with M20 concrete.

(1) Adopt gusset plate of thickness = 16 mm.

(2) Length of gusset plate is kept more than width of flange of ISHB 300.

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(3) Gusset angle = ISA 150115, 15 mm

thickness of angle is 15 mm as gusset plate has a thickness of 16 mm.

(4) Design of Base plate

$f_{ck} = 20\ N/mm^2$

$$A_{req} = \frac{P_u}{0.45 f_{ck}}$$

$$= \frac{3600 \times 10^3}{0.45 \times 20} = 4,00,000\ mm^2$$

Minimum width of Base plate ~~to be~~ $= d = 652$ mm

So provide B = 700 mm

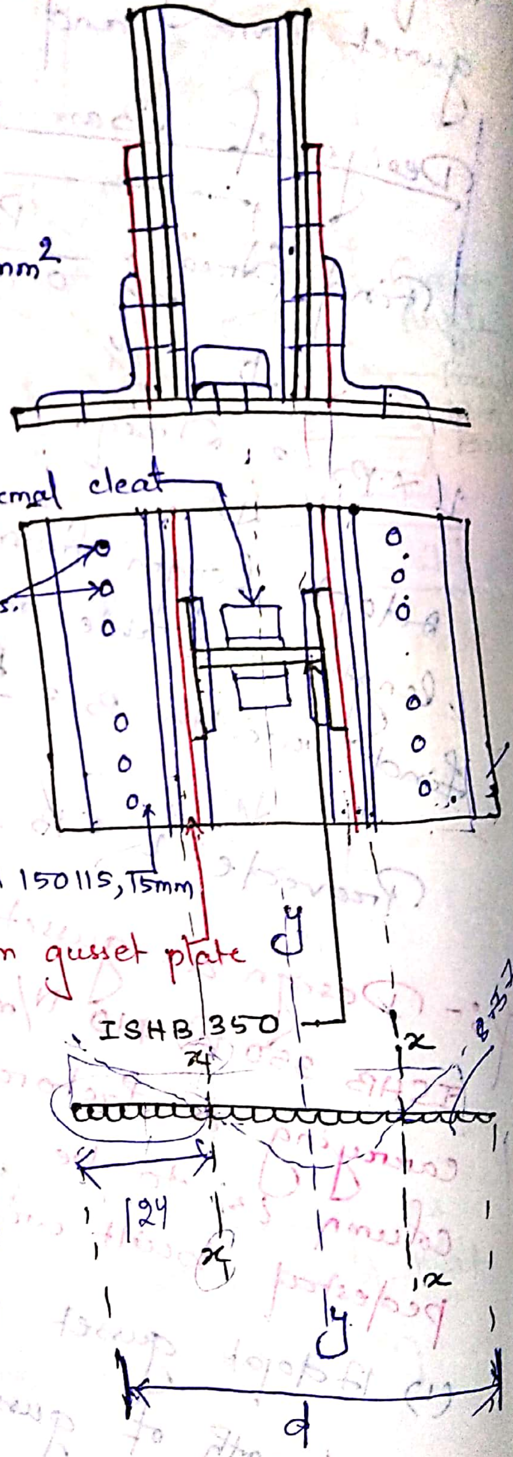
$$L = \frac{A_{req}}{B} \quad \boxed{A = L \times B}$$

$$\Rightarrow L = \frac{4,00,000}{700}$$

$$= 571\ mm$$

$$\Rightarrow L = 600\ mm$$

So provide Base plate of 700 × 600 mm.



$d = 350 + 2\times20 + 2\times16 + 2\times115 \approx 652$ mm

Check for bearing pressure

Bearing pressure on the base plate $= \frac{P_u}{A_{provided}}$

$$= \frac{3600 \times 10^3}{(700 \times 600)} = 8.57 \ N/mm^2.$$

$$0.45 f_{ck} = 0.45 \times 20 = 9 \ N/mm^2.$$

as bearing pressure on base plate < bearing strength of RCC in foundation $\longrightarrow$ [OK].

Projections

$$a = \frac{700 - (350 + 20 \times 2 + 16 \times 2 + 2 \times 15)}{2}$$

$= 124 \ mm.$

Finding Thickness of Base Plate

Thickness is found from flexural Strength at critical section $x-x$.

Bending moment due to external force at the critical section $M = \frac{w \cdot l^2}{2}$

Moment of resistance provided by the plate at critical section $M_R = Z \sigma$.

[from bending equation $Z = \frac{M}{\sigma}$].

$\Rightarrow M_R = \frac{bd^2}{6} \times \sigma_{bs}$.

Take σ_{bs} = permissible bending stress of slab base = 185 MPa.

& $Z = \frac{bd^2}{6}$

$= \frac{1 \times t^2}{6}$. ($b = 1$ & d = thickness t).

For equilibrium, the moment should be equal to moment of resistance at critical section.

i.e. $M = M_R$

$\Rightarrow \frac{wl^2}{2} = \frac{1 \times t^2}{6} \times \sigma_{bs}$.

$\Rightarrow \frac{wl^2}{2} = \frac{t^2}{6} \times \sigma_{bs}$

$\Rightarrow t^2 = \frac{wl^2}{\cancel{2}} \times \frac{\cancel{6}^{3}}{\sigma_{bs}}$

$$\boxed{\Rightarrow t = \sqrt{3wl^2 / \sigma_{bs}}}$$

l = length from outer fibre to the critical section:

= projection 'a'

= 124 mm.

w = bearing pressure on the base plate
$= 8.57\ N/mm^2$

so $t = \sqrt{\frac{3 \times 8.57 \times 124^2}{185}} = 46.22$.

$\boxed{\text{Use 56 mm base plate of size } 700 \times 600\ mm}$

Design of Bolted Connection

Load on the fastener & gusset plate $= Pu/2 = \frac{3600}{2} = 1800\ KN$.

[When ends of column & gusset plate are faced for complete bearings, then column base will be designed for 50 % of axial load].

[If ends of column & gusset plate are not faced for complete bearing, then column base will be designed for full load].

Load on each side $= \frac{1800}{2} = 900\ KN$.

Adopt 24 mm shop bolts.

No. of bolts $= \frac{900 \times 10^3}{\text{Bolt value}}$.

So Shear strength & Bearing strength is to be calculated for finding Bolt value.

Adopt bolt of M24 & grade 4.6

Sheare strength of one bolt

$$= \frac{f_u}{\sqrt{3}} \left(n_n \times A_{nb} + n_s A_{sb} \right) \times \frac{1}{\gamma_{mb}}$$

$$= \frac{400}{\sqrt{3}} \times \frac{1}{1.25} \left(1 \times \frac{\pi}{4} d^2 + 1 \times 0.78 \times \frac{\pi}{4} \times d^2 \right)$$

$$= \frac{400}{\sqrt{3} \times 1.25} \times 1.78 \times \frac{\pi}{4} \times 24^2$$

$= 148772.177$ N.

$= 148.77$ KN

Bearing strength of one bolt

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

Minm edge distance $e = 1.5 \times d_0$

$= 1.5 \times 26 = 39$ mm $\simeq 40$

Minm pitch $= 2.5 \times d_0$

$= 2.5 \times 26 = 65$ mm.

K_b is the least of $\frac{e}{3d_0}, \frac{p}{3d_0} - 0.25, \frac{f_{ub}}{f_u}, 1$

$$= \frac{40}{3 \times 26}, \frac{65}{3 \times 26} - 0.25, \frac{400}{410}, 1$$

$= 0.51, 0.58, 0.97, 1$

$= 0.51$

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So $V_{dpb} = \frac{2.5 \times 0.51 \times 24 \times 15 \times 400}{1.25}$

$= 146880$ N

$= 146.88$ KN.

Bolt value $= 146.88$.

So No. of Bolts $= \frac{900 \times 10^3}{146.88 \times 10^3}$.

$= 6.12 \simeq 7$.

Provide 7 bolts on each side to connect the gusset angle, gusset plate & the column.