C.V. RAMAN POLYTECHNIC, BHUBANESWAR

C.V.Raman Polytechnic

Quality Education for the New Millenium

LECTURE NOTE

STRUCTURAL DESIGN-II, (Th.2)

SEM-5TH

BRANCH- CIVIL ENGINEERING

Prepared by

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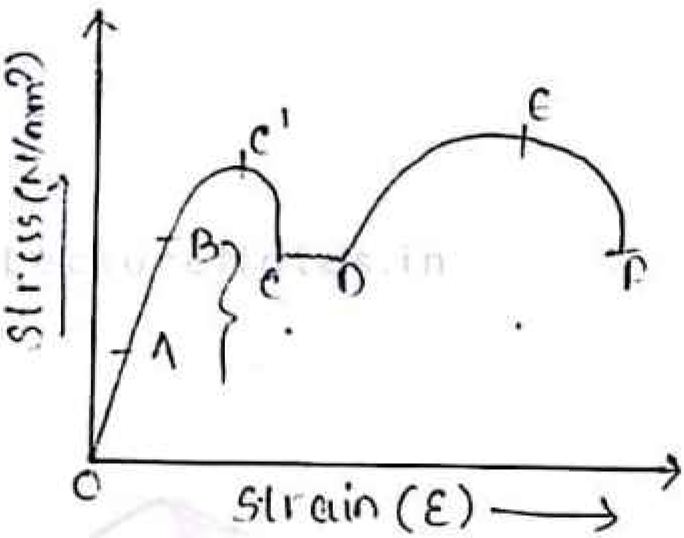
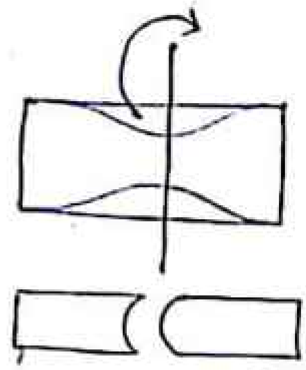
Internal forces are axial force, shear force, bending moment, torsion.





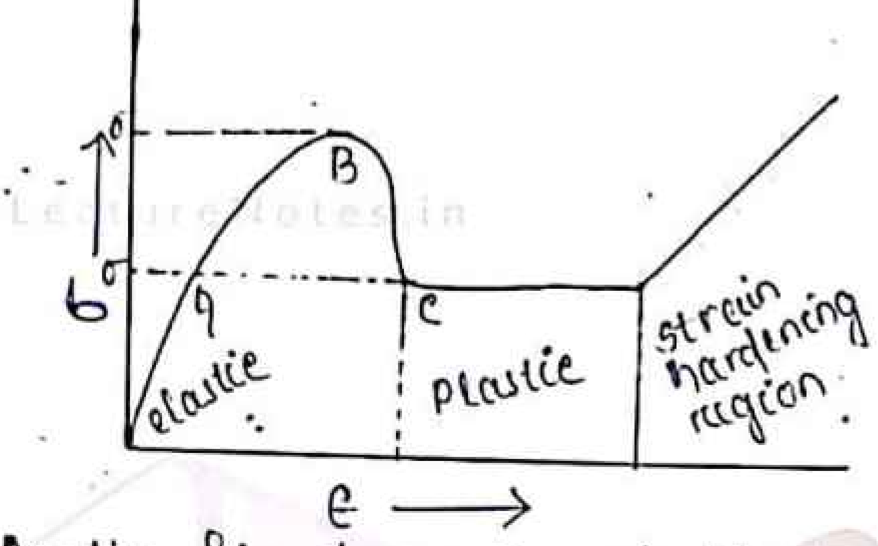
(Ductile failure)

A : Proportional limit
B : Elastic limit
C' : Upper yield point
C : Lower yield point
E : Ultimate strength point/stress corresponding to ultimate load.
F : Breaking stress corresponding to breaking load.
OAB : Elastic region
CD : Plastic yielding region
EF : Strain softening region
DE : Strain hardening region

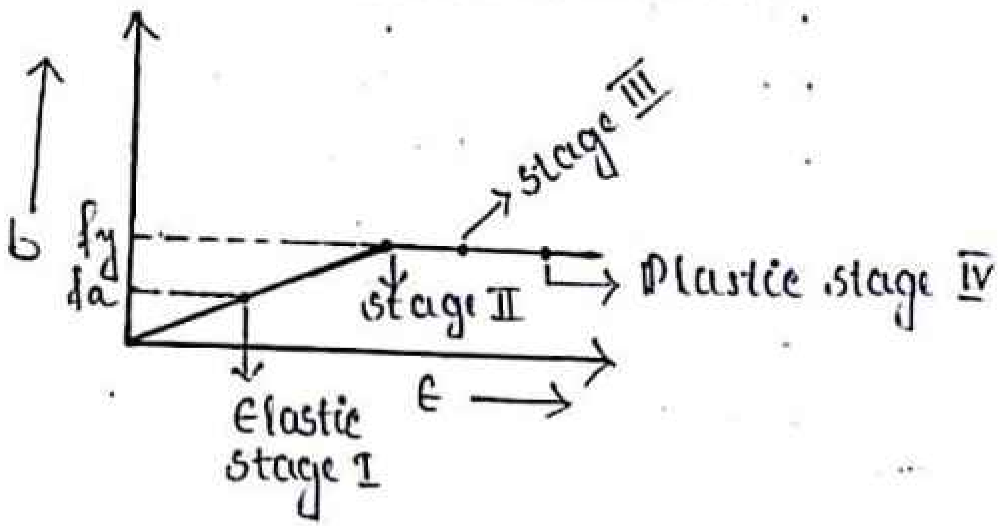
Strain increases fast with stress till ultimate load is reached.

Curve:-

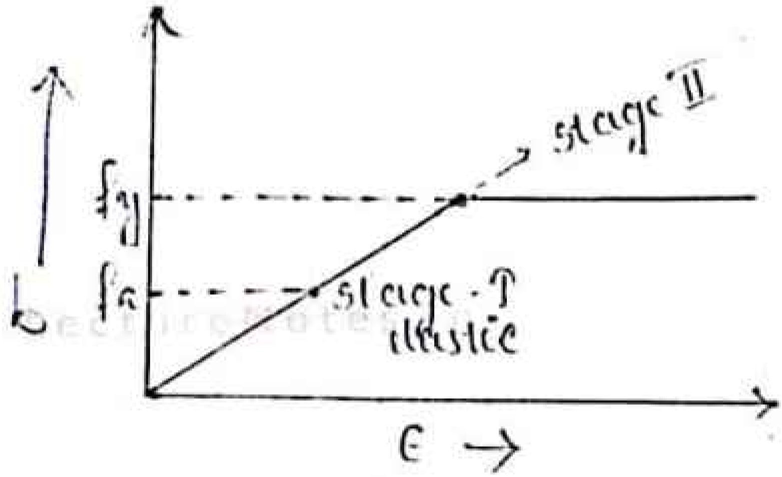
The yield range can be studied more conveniently by enlarging the strain scale considerably.

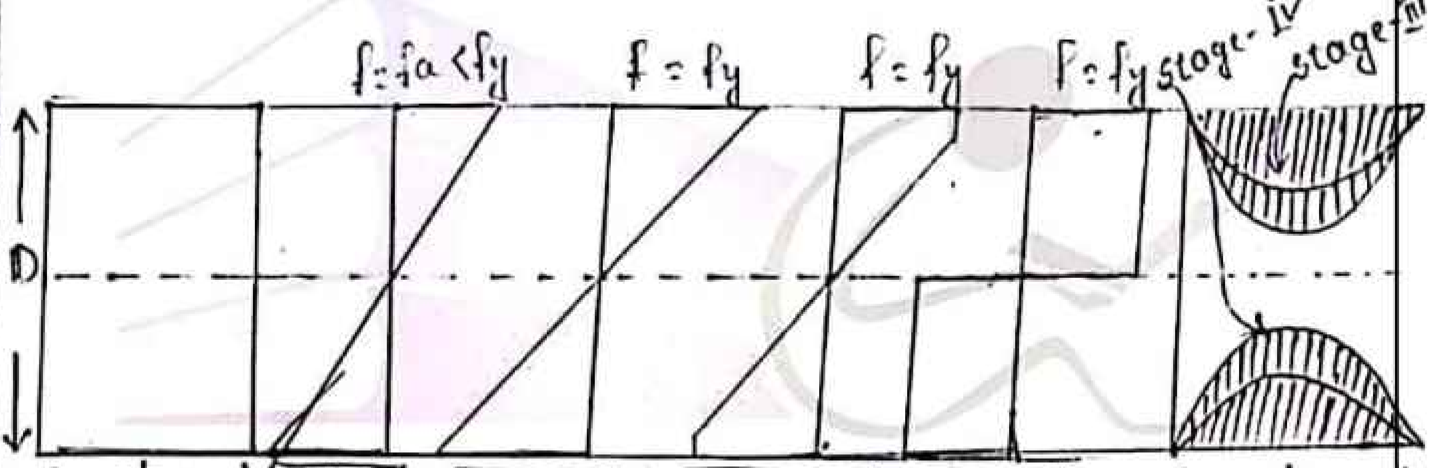


- As the fig. shows the plastic range is sufficiently large and it seems reasonable to extent it without limit that is to ignore the effect of strain hardening.
- So the extension of plastic range is supposed to be unlimited at the constant yield stress σ_y or f_y.
- So the idealised elastoplastic stress-strain curve is.



Bending Of Beam:-





$\boxed{m < m_y}$ Elastic range (Stage-I)

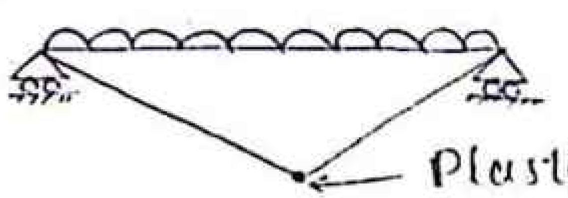
$\boxed{m = m_y}$ Strain is the yield strain. yield stress is 1st reached (Stage-II)

$\boxed{m_y < M < M_p}$ Stress is constant yield spread inward from the top layer partial plasticity (Stage-III)

$\boxed{m = m_p}$ whole section yields. yield grows from extreme fibres to neutral axis fully plastic stage. (Stage-IV)

when $m > m_p$, then rotation will occur.

Whole section yield = yield grows from extreme fibre to neutral axis (fully plastic stage). When $\underline{m > m_p}$ rotation will occur.



Introduction:—

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

The design of steel structure involves

1) Functional design
2) Structural design

Functional Design:—

The planning of the structure for specific purposes such as ventilations, lighting, aesthetic view (etc)

Structural Design:—

It consists of proportioning various elements of the building in the most economical manner so that the loads acting on it are transferred safely to the ground without using excess material.

The members are usually subjected to axial force bending or torsion or the combination of all loads. Axial force is either tension or compression. Members subjected to tensile force k/a tension members.

Ex:- Tie.

Members subjected to compressive force are compression members.

Ex:- Columns or strut

Members subjected to bending are flexural members.

Ex:- Beam.

Advantages & Disadvantages of steel structures:-

Advantages of steel structure over concrete structure:

- Steel members have high strength per unit weight.
- The high strength of steel results in smaller section should be used, able to resist heavy loads & use of fewer columns in building.
- It has a high ductility property due to which it doesn't fail suddenly, but gives visible evidence of failure by large deflection.
- Structural steel are tough i.e. they have both strength and durability. Thus during fabrication and errection steel member will not fracture easily.
- Due to light weight steel members can be conveniently handled & transported.
- Properly maintained steel structures have a long life.
- The properties of steel doesn't change with time; this makes steel the most suitable material for a structure.
- Addition & alternation be made easily with steel structure.
- They can be errected at a faster rate.
- Steel is ultimate recyclable material.

Disadvantages:-

- It is susceptable to corrosion therefore they required frequent painting & maintenance.
- For steel structure skilled labour is required.
- It has a high cost of construction as compared
- Maintenance cost is also high.

• Poor fire proofing as at 1000°F i.e. 538°C 65% at 1600°F 15% of strength remain. The strength is decrease with increase in temperature.

• Electricity may be required during irrection.

Note:-

• Composite construction of steel & concrete can also be used however the mainbody of present day structure consists of R.C.C or steel.

Ex:- of steel structure.

• The use of steel as a building material has been increased now-a-days.

Ex:- Bridges over a tank.
Highrise buildings.
Industrial buildings.
Transmittor towers.

Structural Steel :-

• It is an alloy of iron & carbon. In a standard structural steel carbon contains in between 0.2 to 0.35%.

• Structural steel has been classified by the BIS (Beuro of Indian Standard) based on ultimate or yield strength.

Physical Properties:-

Physical properties largely depends on chemical composition, rolling thickness, heat treatment & stress theory.

1) Modulus of Elasticity (E):-

$E = 2 \times 10^5\ N/mm^2$

2) Shear modulus :- (C or G)

$$G = 0.769 \times 10^2 \ N/mm^2.$$

3) Poisson's Ratio :- (μ)

$\mu \rightarrow$ Elastic range - 0.3
Plastic range - 0.5

$$\mu = \frac{\delta b / b}{\delta l / l}$$

4) Coefficient of thermal expansion (α):-

$$\alpha_s = 12 \times 10^{-6} / ^\circ C.$$

5) Unit mass of steel (ρ):-

$$\rho = 7850 \ kg/m^3.$$

Chemical composition :-

- Chemical composition of sum of the steels are carbon, sulphur, manganese & silicon out these carbon has maximum influence on the physical & mechanical properties of steel.

- Iron carbon alloys containing upto 2% carbon are called carbon steel & those having more than 2% carbon are called cast steel.

- With increase in carbon the tensile strength increase but the ductility falls & causing the steel to be more brittle.

- If the carbon contain is reduced the steel will be softer & more ductile but also weaker. However by alloying chromium, nickel, vanadium (etc); the tensile strength can be increase while retaining the desired ductility.

Rolled Steel Section:-

In the design process one of the main object is the selection of the appropriate cross-section for the individual member of structure. So it is more convenient to choose a standard cross-sectional shape i.e. widely available rather to choose a unit dimension that required a unique fabrication.

So different categories of standard shape of steel is turned by hot rolling and cold rolling.

Structural steel can be rolled into various shapes & sizes.

Sections having larger module of section in proportion to their cross-section are preferred.

$Z = I/y$

Steel sections are named according to their cross-section shape.

Rolled steel sections which are rapidly available in market due to its frequent demand higher called regular steel section.

Some commonly used rolled section are:-

1) Rolled Beams (I-section)

=> Junior Beams (ISJB)

=> Heavy weight Beams (ISHB)

=> Medium weight Beam (ISMB)

=> Light weight Beam (ISLB)

=> Wide flange Beam (ISWB)

2) Channel Section:-

=> Junior Channels (ISJC)

Light channels (ISLC)
Medium weight channel (ISMC)

3) Angle Section :-
Equal angle (ISA)
Unequal angle (ISA)
Bulb angle (ISA)

4) T- section :-
Junior T - section (ISJT)
Light T - section (ISLT)
Short flange T- section (ISST)
Heavy flange T- section (ISHT)
Normal T - section (ISNT)

5) Rolled steel bar :-
Square bar (ISSQ)
Round bar (ISRQ)

6) Rolled steel Tubular Section :-
Light weight tubular section
Medium weight tubular section
Heavy weight tubular section.

7) Rolled steel plate (IS)

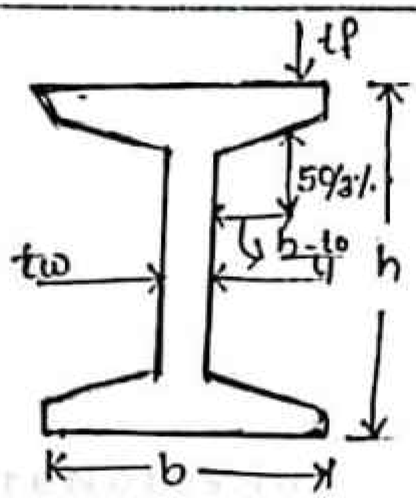
8) Rolled steel strips (ISST)

9) Rolled steel flats. (ISFI)

I- section :-

It is designated as its overall depth & weight

ISJB 150 @ 69.7 N/m.



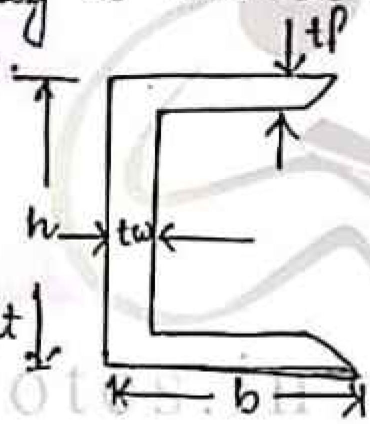
Uses Beams & Columns:—

ISLB, ISMB, ISWB & ISJB are used as beam section & ISHB is used as column.

Channel Section:—

It is designated by its overall depth & weight.

Ex:- ISJC 100 @ 56.9 N/m.



Uses:—

Are used as beams & columns.

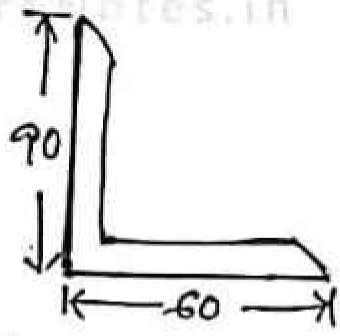
For heavy colums built up channels are used.

Angle section:—

It is designated by its length & thickness of leg.

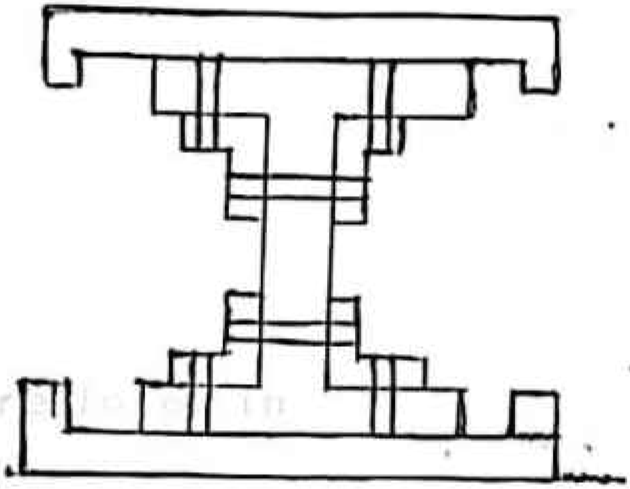
Ex:- ISA 90×60×6mm

Bulb sections are special sections and are used in ship buildings.



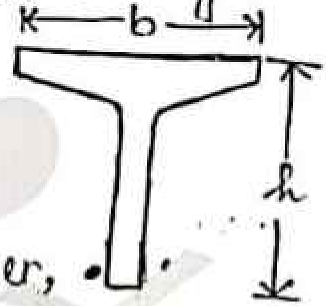
Uses:—

Compression members, tension members & component part of built up members.



I- Section:-

It is designated by overall depth & weight.



Ex:- ISLT @ 39·2 KN/m

ISLT 50 @ 39·2 KN/m

Uses:-

Compression member, Tension member, frames of doors & windows.

Rolled Steel Bars:-

A round bar is designated by its diameter where as a square bar is designated by its sides.

Ex:- ISRO 12

ISSQ 12.

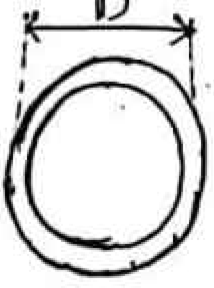
Rolled Steel Tubular Section:-

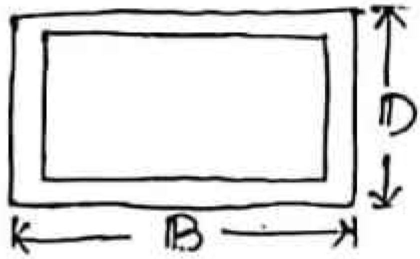
It is designated by its outside diameter and self weight.

Ex:- Circular hollow sections.

Square " "

Rectangular hollow sections





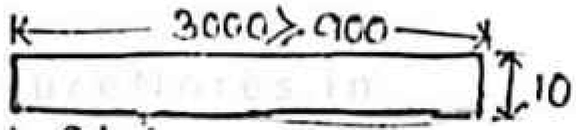
Uses:-

Compression member in roof trusses.

Rolled Steel plates:-

It is designated by length, width & thickness.

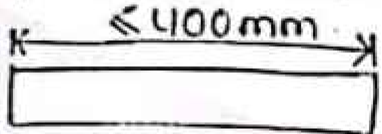
ex:- ISPL 3000 × 900 × 10



Rolled Steel Flats:-

These are designated by width & thickness.

ex:- ISFT 100 × 20 mm



This designation is same for strips.

Note:-

ISLB & ISMB are the only I-section been rolled in India.

All standard I-Beams & Channels has a slope on the inside face of flange of 50/3 %.

Loads:-

The force that act on a structure are called load.

For the safe design of a structure it is essential to have a knowledge of various materials or man-made loads or combinations of loads acting on it.

Design Philosophy:-

Design of steel structure consists of design of steel members & their connections.

So that they can safely and economically resist and transfer the applied load to the ground floor.

The design process begins with selection of trial section and checking its safety.

This is where different approaches to design come in the place.

The design of structural steel elements are based on attetment on initial yielding.

- Attendment of full yielding
- Tensile strength.
- Critical buckling
- Maxm. deflection permitted
- Stress Concentration
- Fatigue
- Brittle fracture.

The design philosophy are used & listed below inorder of their evolution :-

1) Working Stress Method.
2) Ultimate Load Method.
3) Limit State Method.

Ultimate Tensile Strength:-

It is the max^m stress that the material can withstand while being stretched or pulled before failure on breaking.

Yield Strength:-

It is the stress at which the stress-strain curve for axial loading deviates the strain of 0.2% from the linear elastic line on the stress-strain curve become non-linear.

Working Stress Method:-

It is the elastic method of design.

According to this method, the members are designed on basis of working stress & those will never exceed the permissible stress according to code.

- A permissible stress is defined as the ratio of yield stress according to factor of safety.

$$\boxed{\text{Permissible Stress} = \frac{\text{Yield Stress}}{\text{FOS}}}$$

Limitations:-

- According to this method, failure load is factor of safety times working load; which is not true.

$$\boxed{\text{Factor load} = \text{Working load} \times \text{FOS}}$$

- Actually it is more because a material can resist the load after yield apparatus at a fibre.
- In structures just formation of plastic hinge is not the collapse criteria. Since, it can resist load till some more hinges formed resulting into collapse mechanism.
- It gives uneconomical section.
- It deals only with elastic behaviour of member.
- The strength of the section at the working load is estimated from the yield strength of the section.

Advantages:-

The method is simple & reasonably reliable.

Ultimate Load Method:-

From the stress-strain curve it is observed that higher loads than elastic method can be applied in the structure.

This is due to the fact that a measure portion of the curve lies beyond the elastic limit.

• This strength is called deserved curve; & based upon this strength plastic design is made.

• This method is based on failure conditions rather than working load condition.

• The strength of the section is estimated from ultimate strength of the section.

• In plastic design method, the working loads are multiplied by a load factor get the collapse load and the members are designed on the basis of collapse strength.

• Since, the actual load should be less than the collapse load by a factor of safety, the members designed should be safe.

Advantages:—

Redistribution of internal forces is accounted & considered.

Disadvantages:—

It doesnot guarranty serviceability performances like deflection, instability, crack width & fatigues (etc).

So to take care of design requirements from strength & serviceability criteria limit state method is developed.

Limit State Method :—

• It is similar to plastic design which consider most critical limit state of strength & serviceability.

• The acceptable limit for the safety & serviceability requirements before failure occurs is called limit state.

• The section design should satisfy serviceability requirements such as limitations of deflection & vibration & should not collapse under accidental loads.

Limit State Of Strength :-

- For checking the strength & stability of structure the loads are multiplied by relevant load factor (γ_f) given in IS 800:2007, table no:-4.
- The modified loads are called factor loads account for the uncertainities involved in estimating the magnitude of dead and live loads.
- The design strength of members or its connections are determined by dividing ultimate strength w.r.t. partial safety factor (γ_m) for materials given in IS 800: 2007 -table 5.

Limit State Of Serviceability :-

It is the limit state beyond which the service criteria such as deflection, vibration, repairable damage due to fatigue, corrosion, fire existance are no longer made.

Load factor (γ_f) of one is used for all load to check serviceability requirements.

Codes for Loads :-

IS 875 : part 1 (dead load)
IS 875 : part 2 or 4 (live load)
IS 875 : part 3 (wind load).
IS 1893 (earthquake load)

Mechanical Properties Of Steel :-

1) Elasticity
2) Plasticity.
3) Ductility.
4) Brittleness.
5) Hardness.
6) Fatigue.
7) Creep.

Malleability :-

Property of material due to which it can rolled into thin sheet without toughness.

It can be stressed bend, stretch & twisted under a high stress before failure.

viii) Slow deformation

ix) Yield stress.

x) Ultimate stress

xi) Percentage elongation.

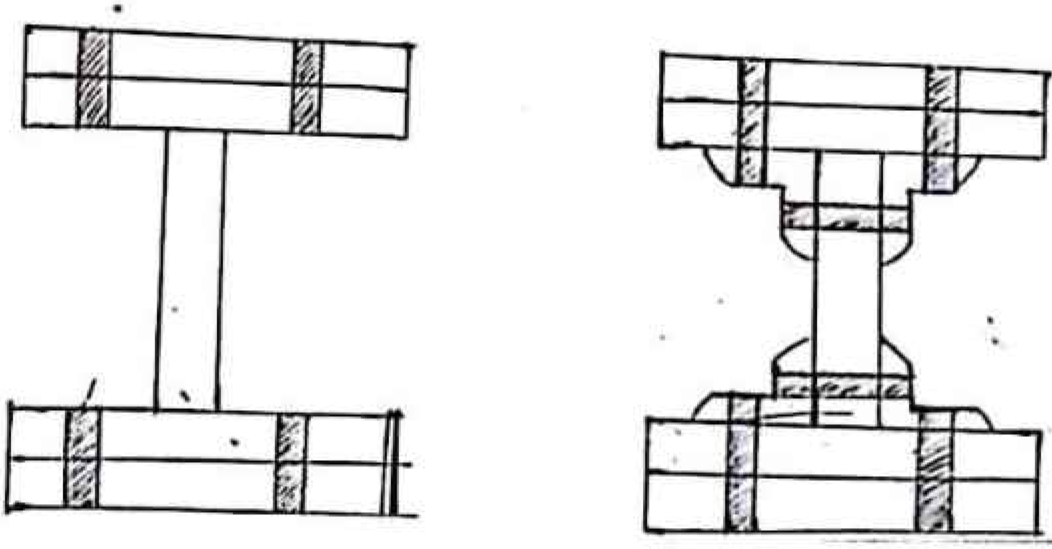
STRUCTURAL STEEL CONNECTIONS

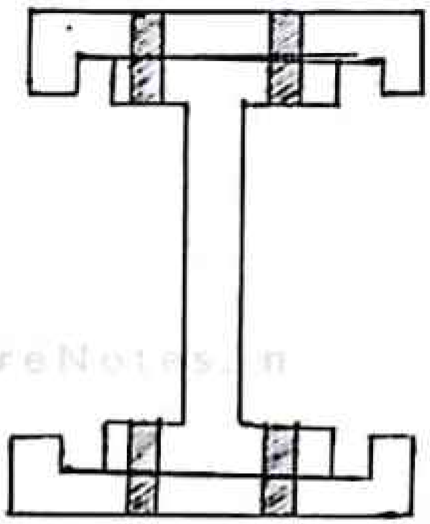
Various elements of a steel structure like tension, compression & flexural members are connected fasteners or connectors.

The need for designing connections are :-

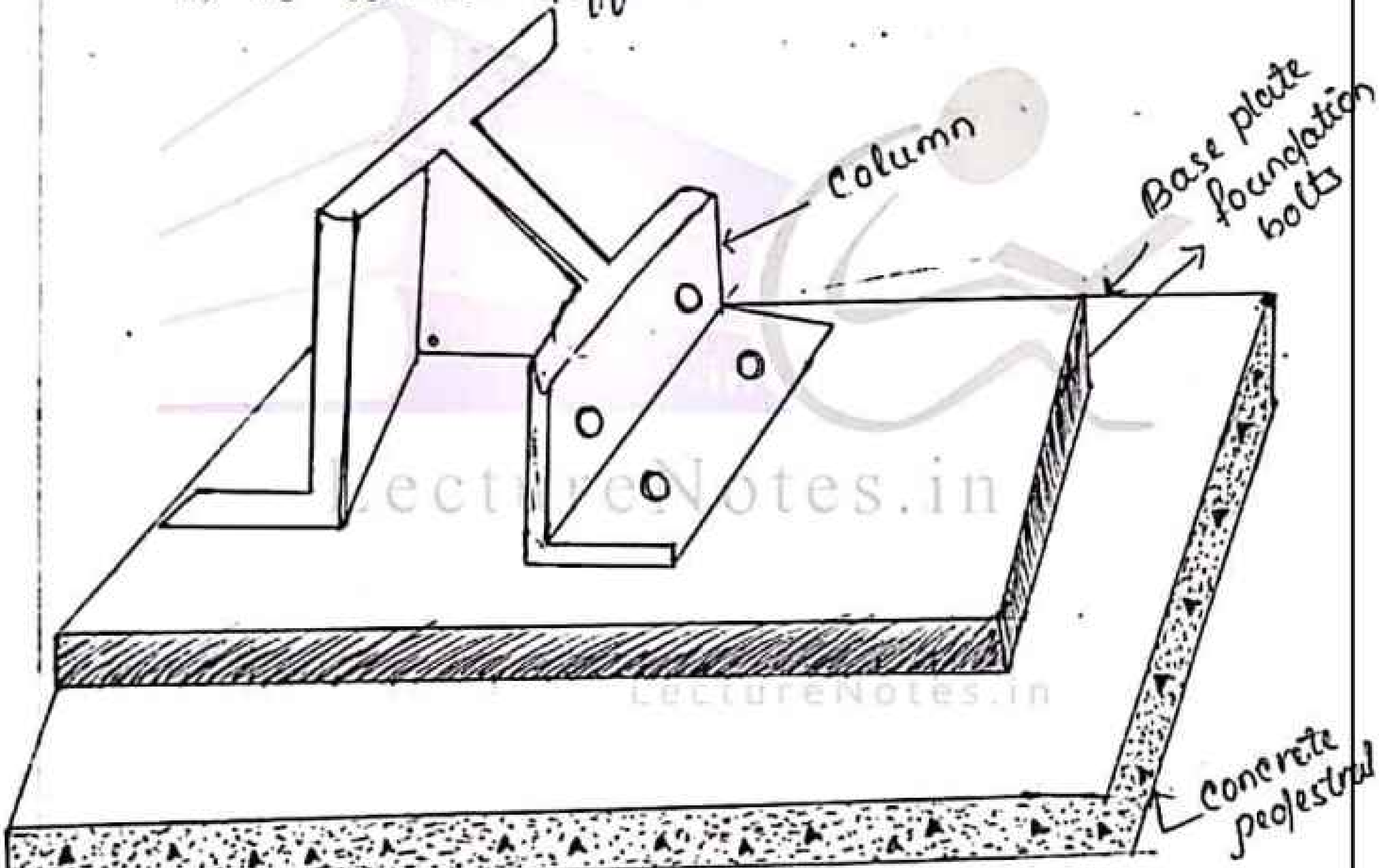
i) Different sections to form the required built up or composite section of a member.

ii) It connects plates, angles, channels I-section.





iii) To connect different members at the ends.



iv) Connections of two lengths of a member to make up a required length.

If the necessary connections are inadequate the result will be a poor structure inspite of the most efficiently designed members.

Therefore, design of connections is very important because the failure of joints is sudden and catastroph

The various types of connections used in steel structures are :-

1) Rivet connection
2) Bolt connection
3) Weld connection

Rivet connection :-

When members of a structure are connected using rivets the joints so far is known as rivet joint and the process of jointing is known as riveting.

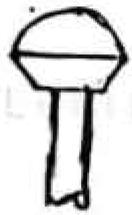
Rivet is made up of a round ductile steel bar or body called shank and a head at one end.

Note :-

Since, the analysis & design of a riveted connections are same as that for ordinary bolts, the design & details may be done similar to bolting.

Classification based on shape of rivet head :-

1) Snap head rivet 
2) Pan head rivet 
3) Flat counter shank 
4) Round counter shank



Classification based on method & placing of rivets :-

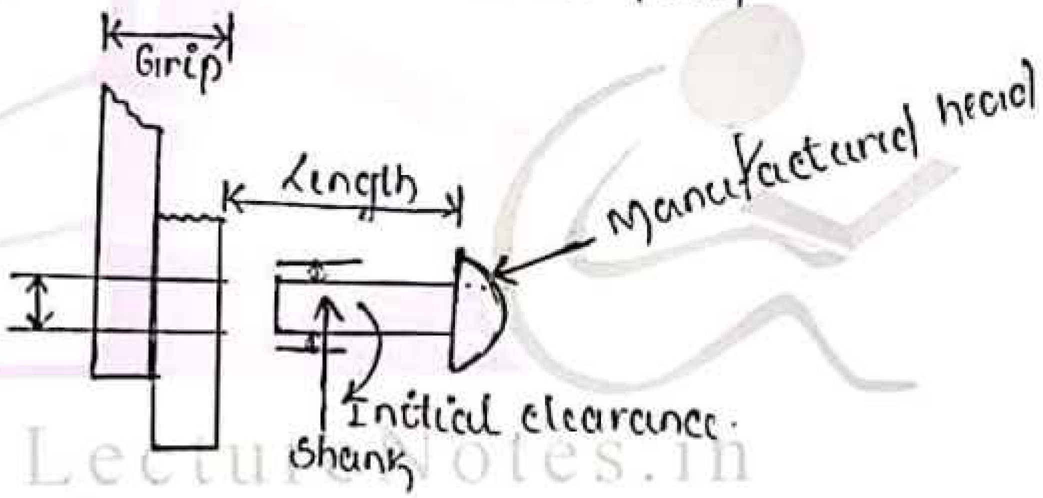
Power driven shop rivet :- The rivets which are driven by hydrocially in the shop under control condition.

Hand driven shop rivet :- The rivets which are driven by hand in the shop.

Field rivet :- The rivets which are driven at the place of work.

Hot driven rivet :- When the rivets are heated to red hot before driving they are known as hot driven rivet.

Cold driven rivet :- These are driven at room temperature and high pressure is required to form the head which is not visible to use in the field.



ϕ : nominal diameter.

d : Grip diameter.

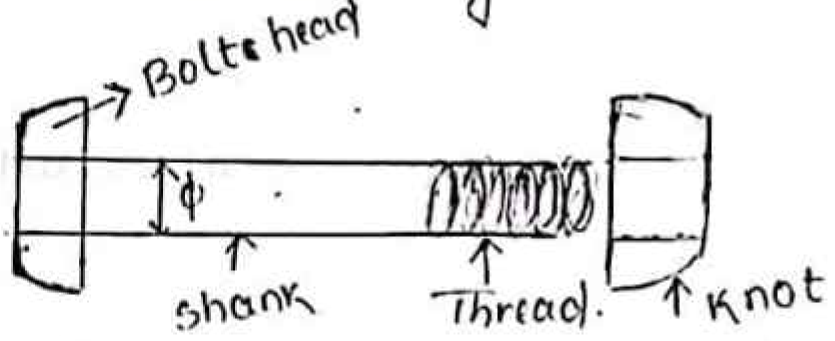
$\boxed{\text{Grip length} \not> 4 \times \text{diameter of rivet}}$

Disadvantages :-

- It is associated with high level of noise pollution.
- It needs heating the rivet to red hot.
- Inspection of connections required skill workers.
- Labour cost is high.

- Bolted Connection :—

A bolt may be defined as a metal pin with a head at one end, a shank threaded at the other end to receive a knot as shown in fig.



Bolts are used for joining together pieces of metals by inserting them (bolts) through hole in the metal & tightening the knot at the threaded ends.

Types:—

Bolts —
- Unfinished bolts /black bolt
- finished bolts / Turned bolt
- High strength friction grip (HSFG)

Unfinished bolt:—

It is made from mild steel rod with square or hexagonal head.

Nominal diameter are 12, 16, 20, 22, 24 ; 30 & 36 mm.

This was designated as M12, M16, M20, M22, M24, M30 & M36.

IS 1364 gives specification for such bolt

Yield strength is equal to 240 N/mm^2 & ultimate strength is 400 N/mm^2.

Uses:—

Light structures, temporary connections.

Finished bolts:-

It is made from mild steel but force for hexagonal rod & finished to a circular shape.

Actual dimension is larger than the nominal diameter 1.2 mm to 1.3 mm.

Bolt hole dia is 1.5mm larger than the nominal diameter of bolt.

IS 3640 covers the specification.

Uses:-

Special jobs like connecting machine parts subjected to dynamic loading.

HSFG:-

It is made from high strength steel rod & surface is finished.

The bolts are tightened by using calibrated wrenches and knots are provided by clamping devices.

In this bolts shearing load is resisted by frictional force betn the member & shank & washers.

IS 3747 covers the specification.

Nominal diameters are 16, 20, 24, 30 & 36.

Uses:-

Connect members subjected to dynamic loading.

Classification of bolts based on load transfer:-

1) Bearing type
2) Friction type

Bearing type:-

The force is transferred from member to member by bearing.

There are 2 types:-

1) Unfinished
2) Finished.

Friction Type:-

The force is transferred by friction in between member & bolt

Ex :- HSFG.

Classification of bolted Connection:-

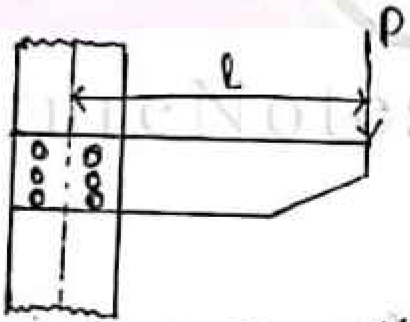
- On the basis of classification of resultant force transfer.

Concentric connection
Eccentric connection
Moment resisting

Examples:-

axially loaded, tension & compression member.

Bracket connection & seat connection.



Beam column connections in framed structure.

- On the basis of classification of types of forces:-

1) **Shear connection:-**

When the load is transferred to shear.

Ex:- Lap joint & butt connection.

2) **Tension connection:-**

In this, load is transferred through the friction. Ex:- Hanger connection.

Combined, Shear & Tension Connection :-

Ex :- Connection of bracings.

• On the basis of force mechanism :-

* Bearing type :-

• Bolts bear against the holes to transfer the force.

• Here force is transferred through interlocking & bearing of bolts.

* Friction type :-

When the load is transferred by friction betn the plates due to tensioning of the bolts.

Notes:-

The ratio of net tensile area at threads to nominal plain shank area of bolt is 0.78 (according to code IS 1367 part 1).

$$\boxed{A_n = 0.78 A_s}$$

As per IS 800 net tensile area is the area at root of the threads.

It is called stress area or proof area

Table 19 of IS code 800:2007 gives clearance for bolt holes.

The bolts of property class 4.6 & 8.8 are generally available.

Most common is black bolt of class 4.6.

The no. before decimal indicates 1/100th of the nominal ultimate tensile strength & the no. after decimal indicates the ratio of yield stress to ultimate stress expressed as %.

$4 = 1/100 \times n \times US$

$UTS = 400\ N/mm^2$.

$0.6 = YS/US \times 100$

$YS = \frac{0.6 \times 400}{100} = 2.4\ N/mm^2$

$8 = 1/100 \times n \times US$

$UTS = 800\ N/mm^2$.

$0.8 = YS/US$

$YS = \frac{0.8 \times 800}{100} = 6.4\ N/mm^2$.

Specification for spacing:-

'p' should not be less than 2.5d $\boxed{P \not< 2.5d}$

where, d = nominal dia. of bolt.

'p' is not more than 16t or 200mm; whichever is less.

$P \not> \left.\begin{matrix} 16t \\ 200mm \end{matrix}\right\}$ tension

$P \not> \left.\begin{matrix} 12t \\ 200mm \end{matrix}\right\}$ compression.

where,

t = thickness of inner plate.

In staggered pitch, pitch may increased by 50% of value specified value above the provided gauge distance is less than 75%.

Incase of butt joint

i) max^m pitch is to be restricted 4.5d

ii) for a ~~[illegible]~~ distance of 1.5 times the width of plate from the butting surface.

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

$\boxed{g \not> 100+4t \text{ or } 200mm}$

iv) Minm edge distance is, $e_{min} < 1.7 \times$ dia of hole incase of shear & hand plate cut edge.

$e_{min} < 1.5$ times $\times$ dia of hole incase of machine flange cut.

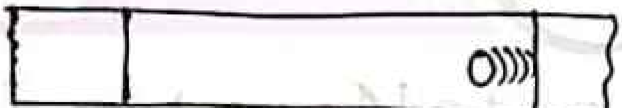
v) $e_{max} \not> 12t\epsilon$ where $\epsilon = \sqrt{250/f_y}$.

$\not> 40+4t$ where t = thickness of thinner connected plate.

Plates in a joint made with a bearing of bolts may fail under tension force due to these 3 causes:-

1) Shearing of edges;
2) Crushing of plate.
3) Rupture of plate.

Shearing



Crushing

Rupture

The shearing & crushing failure are provided if the minm edge & end distance as per IS 800 recommendation are provided.

Rupture failure :—

Tensile strength of plate of joint against rupture.

$$\boxed{T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}}} \quad (P\text{-}32,\ 6.3.1).$$

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

f_u : ultimate stress of the plate.
γ_{ml} : F.O.S of failure at ultimate stress.

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

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

where, b : width of plate.
n : no. of bolt hole.
t : thickness of inner plate.
d_0 : diameter of bolt hole.

Design Of Strength Of Bolt:—

1) Shearing capacity of bolt.
2) Bearing capacity of bolt.

Shearing Capacity Of bolt:—

Designing shearing strength of bolt.

$$\boxed{V_{dsb} = V_{nsb} / \gamma_{mb}}$$ (P-75, 10.3.3)

$$\boxed{V_{nsb} = f_{ub}/\sqrt{3}\ (n_n A_{nb} + n_s A_{sb})}$$ (P-13, T-1)

where, f_u : ultimate tensile strength of a bolt.
n_n : no. of shear planes with threads intercepting the shear planes.
n_s : no. of shear planes without threads intercepting the shear plane;
A_{sb} : nominal plane shank area of the bolt;&
A_{nb} : net shear area of the bolt at threads

Reduction Factor for Shearing Capacity Of Bolt:-

1) If the joint is too long.
2) If the grip length is large.
3) If the packing plate used.

Bearing Capacity Of Bolt :—

(P-75, 10.3.4)

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

γ_{mb} = FOS of bolt material

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

where, k_b is a factor depends on $\frac{e}{3d_0}, \frac{p}{3d_0} - 0.25, f_{ub}/f_u$.

where,

e = end distance.

p = pitch distance.

d_0 = dia of bolt hole. (P-73, T-19)

f_{ub} = ultimate strength of bolt. &

f_u = ultimate strength of plate. (P-14, T-1)

Specification Of Bolt :—

Nominal size of bolt - 12, 14, 16, 20, 22, 24, 30, 36.

Dia of hole - 13, 15, 18, 22, 24, 26, 32, 38.

Outer dia of washer - 30, 37, 44, 56, 60.

Gradation Of Bolt :—

Grade	f_y (N/mm²)	f_{ub} (N/mm²)
4.6	240	400
4.8	320	420
5.6	300	500
5.8	400	520

Efficiency Of Joint (η) :—

It is the ratio of strength of joint / designing strength of joint to the designing strength of plate.

It is always expressed in %.

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

Terminology :-

① Pitch :- (P)

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

② Gauge distance (g) :-

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

③ Edge distance (e) :-

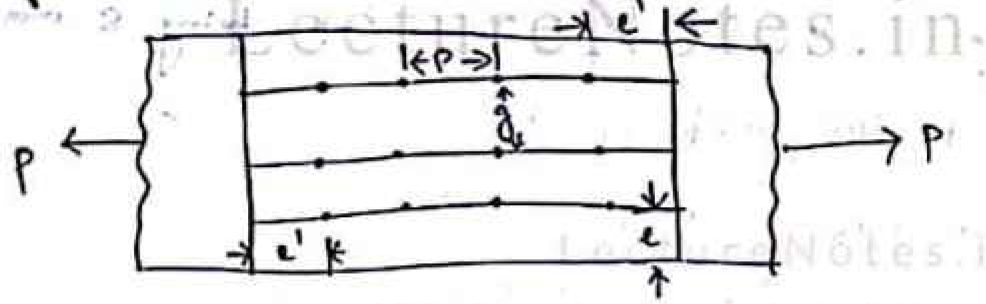
It is the distance of centre of bolt hole from the adjacent edge of plate.

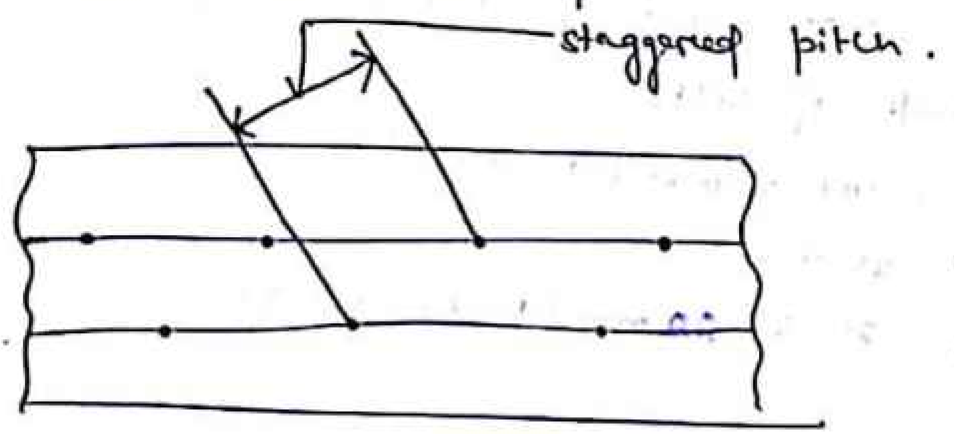
④ End distance (e') :-

It is the distance of the nearest bolt hole from the end of the plate.

⑤ Staggered distance :-

It is the centre to centre distance of staggered bolts measured obliquely on the member.





Specification

②

1) Pitch 'p' shall not less than $2.5d$, d = nominal dia of bolt.

2) Pitch 'p' shall not be more than.

a) $16t$ or 200 mm, whichever is less in case of tension member

b) $12t$ or 200 mm, " " " " " compression ".

3) In case of butt joint, maximum pitch will be $4.5d$

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

5) Minimum edge distance

a) $1.7 \times$ hole dia in case of sheared or hand flame cut edges.

b) $1.5 \times$ " " " " of rolled, machine flame cut, planed edges.

6) Max^{m} edge distance

a) $16t\epsilon$, $\epsilon = \sqrt{\frac{250}{f_y}}$ (t = thickness of thinner plate).

b) $40+4t$, (t = thickness of thinner plate)

Q. Calculate the strength of a 20 mm dia bolt of grade 4.6 for the following process. The main plates to be jointed are 12 mm thick.

a) Lap joint.

b) Single cover butt joint, the cover plate being 10 mm thick

c) Double cover " " each of cover plate being 8 mm thick.

Sol^{n}:- Assuming Fe 410 grade of steel,

$f_u = 410$ MPa (P-14, T-1)

$\gamma_{mb} = 1.25$ (P-30, T-5)

4.6 grade of bolt.

$f_{ub} = 400$ N/mm^2 (P-13, T-1)

$d = 20$ mm

$d_0 = 20+2 = 22$ mm (P-73, T-19)

For lap joint strength of bolt :- (2)

(1) Shearing strength of bolt (P-75, 10.3.3)

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

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

$n_s = 0$

$n_n = 1$

$A_{nb} = 0.78 \times \frac{\pi}{4} \times d^2 = 245 \ N/mm^2$

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

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

$= 56.59 \ kN$.

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{56.59}{1.25} = 45.264 \ kN.$$

(2) Bearing strength of bolt (P-75, 10.3.4)

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

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

K_b is smaller of the following

(i) $\frac{e}{3d_0} = \frac{33}{3 \times 22} = 0.5$

(ii) $\frac{p}{3d_0} - 0.25 = \frac{50}{3 \times 22} - 0.25 = 0.507$

(iii) $\frac{f_{ub}}{f_u} = 0.975$

iv) 1

Minimum edge distance

$e = 1.5 \times d_0$

$= 1.5 \times 22 = 33$ mm.

Minimum pitch

$p = 2.5 \times d$

$= 2.5 \times 20$

$= 50$ mm.

$\therefore K_b = 0.5$

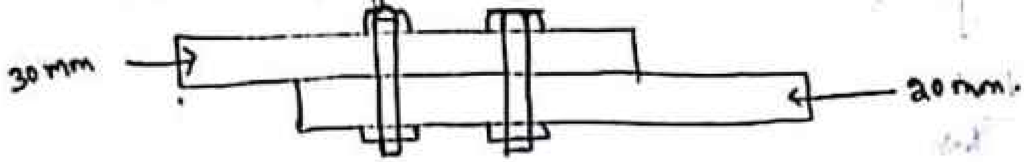
$V_{npb} = 2.5 \times 0.5 \times 20 \times 12 \times 410$

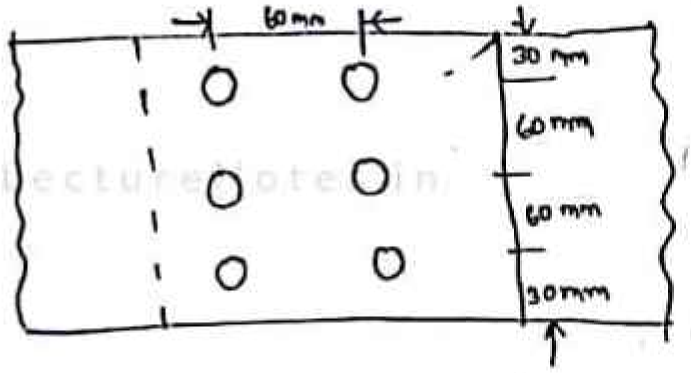
$= 123 \ kN$

$$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}} = \frac{123}{1.25} = 98.4 \ kN.$$

$\therefore$ Design strength of the bolt is equal to the least of the above two values i.e. 45.264 kN (Ans).

Q. Find the efficiency of lap joint shown in the figure. (4)
Given M_{20} bolts of grade 4.6 & Fe 410 plates are used.





Soln:- For M_{20} bolt and grade 4.6, we have

$d = 20$ mm
$d_0 = 22$ mm
$f_{ub} = 400$ N/mm^2
$\gamma_{mb} = 1.25$

$p = 60$ mm
$e = 30$ mm

For Fe 410 plate,
$f_u = 410$ N/mm^2
$t = 20$ mm
$\gamma_{m0} = 1.1$
$\gamma_{ml} = 1.25$

Design strength of solid plate (P-32, 6.3.1)

$$T_{dn} = \frac{0.9 \times A_n \times f_u}{\gamma_{mL}} = \frac{0.9 \times 2280 \times 410}{1.25}$$

$$= 673.056 \text{ KN}.$$

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

$$= (180 - 3 \times 22 + 0) \times 20$$

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

Design strength of bolts

① Shearing strength of bolt (P-75, 10.3.3)

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = 271.58\ kN$$

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

$\left(n_n \text{ for 6 bolts} = 6 \times 1 = 6 \right)$

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

$$= 339.481\ kN$$

② Bearing strength of bolt (P-75, 10.3.4)

$$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}} = \frac{186.345}{1.25} = 149.076\ kN.$$

$$V_{dpb} = 2.5 \times K_b \times d \times t \times f_u = 2.5 \times 0.4545 \times 20 \times 20 \times 410 = 186.345\ kN.$$

K_b is least of following

(i) $\frac{e}{3d_0} = \frac{30}{3 \times 22} = 0.4545$ (ii) $\frac{p}{3d_0} - 0.25 = \frac{60}{3 \times 22} - 0.25 = 0.65$

(iii) $\frac{f_{ub}}{f_u} = 0.975$ (iv) 1

$\therefore K_b = 0.4545$

Design bearing strength of 6 bolts $= 6 \times 149.076$
$= 894.457\ kN.$

∴ strength of joint is equal to least of above 3 values i.e. 271.58 kN.

Efficiency of the joint :- ~~(21-22)~~

$$\eta = \frac{\text{strength of the joint}}{\text{strength of solid plate}} = \frac{271.58}{785.45} = 0.34$$

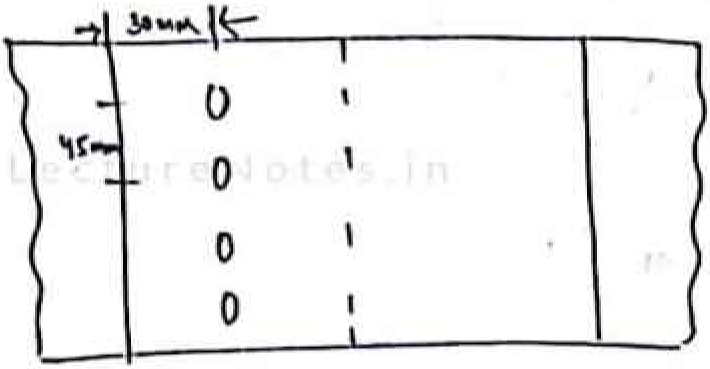
$= 34\%$ (Ans)

Strength of solid plate (P-32)(6.2)

$$T_{dg} = \frac{A_g \times f_y}{\gamma_{m0}} = \frac{120 \times 20 \times 250}{1.1} = 785.45$$

Q. A single bolted double cover butt joint is used to connect two plates which are 8 mm thick. Assuming 16 mm dia bolts of grade 4.6 and cover plates to be 6 mm thick. Calculate the strength and efficiency of the joint, if 4 bolts are provided in the bolt line at pitch of 45 mm as shown in fig.

(6)



Sol^n:- Let's assume Fe 410 grade of steel.

$f_u = 410 \ N/mm^2$

$\gamma_{m1} = 1.25$

$\gamma_{m0} = 1.1$

For 4.6 grade of bolt,

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

$\gamma_{mb} = 1.25$

$d = 16 \ mm$

$d_0 = 16 + 2$

$= 18 \ mm.$

$p = 45 \ mm.$

$e = 30 \ mm.$

thickness of thinner plate = 8 mm.

Design strength of solid plate (P-32, 6.3.1)
(per pitch width)

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

$$= \frac{0.9 \times (P - d_0) \times t \times f_u}{\gamma_{m1}}$$

$$= \frac{0.9 (45 - 18) \times 8 \times 410}{1.25}$$

$$= 63.76 kN.$$

Design strength of bolt

① **Shearing strength of bolt**

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{82.65}{1.25} = 66.12\ kN.$$

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

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

$$= 82.65\ kN$$

② **Bearing strength of bolt**

$$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}} = \frac{\cancel{87.180}\ 72.160}{1.25} = 57.728\ kN.$$

$$V_{npb} = 2.5 \times K_b \times d \times t \times f_u = 2.5 \times 0.55 \times 16 \times 8 \times 410 = \cancel{87.180\,kN}\ 72.160\ kN.$$

K_b least of the following

(i) $\frac{e}{3d_0} = \frac{30}{3 \times 18} = 0.55$ (iii) $\frac{f_{ub}}{f_u} = 0.975$

(ii) $\frac{p}{3d_0} - 0.25 = \frac{45}{3 \times 18} - 0.25 = 0.583$ (iv) 1

$\therefore K_b = 0.55$

$\therefore$ strength of joint is equal to least of above 3 values i.e. 57.725 kN.

Efficiency of the joint :-

$$\eta = \frac{\text{strength of joint}}{\text{strength of solid plate}} = \frac{57.725}{\cancel{81.818}\ 106.27} \times 100 = \cancel{70.55\%}\ 54.31\ \%\ (\text{Ans})$$

strength of solid plate

$$\cancel{T_{dg} = \frac{A_g \times f_y}{\gamma_{m0}} = \frac{p \times t \times f_y}{\gamma_{m0}} = \frac{45 \times 8 \times 250}{1.1} = 81.818\ kN.}$$

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

$$= \frac{0.9 \times 45 \times 8 \times 410}{1.25}$$

$$= 106.27\ kN.$$

5 Two plate (Fe 410 grade steel) each 210 mm x 8 mm are to be joined using 20 mm dia, 4.6 grade bolts to form a lap joint. The joint is supposed to transfer in factored load of 250 kN. Design the joint and determine the suitable pitch for bolts. (8)

Solⁿ:- For Fe 410 grade of steel,

$f_u = 410 \ N/mm^2$

$\gamma_{m1} = 1.25$

$b = 210$ mm

$t = 8$ mm.

For 4.6 grade of bolt

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

$\gamma_{mb} = 1.25$

$d = 20$ mm

$d_0 = 20 + 2 = 22$ mm.

$P = 250$ kN.

$$\text{No. of bolts} = \frac{\text{load transmitted}}{\text{shear strength of one bolt}}$$

Strength of bolt :-

① Shearing strength of bolt.

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = 45.27 \text{ kN}.$$

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

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

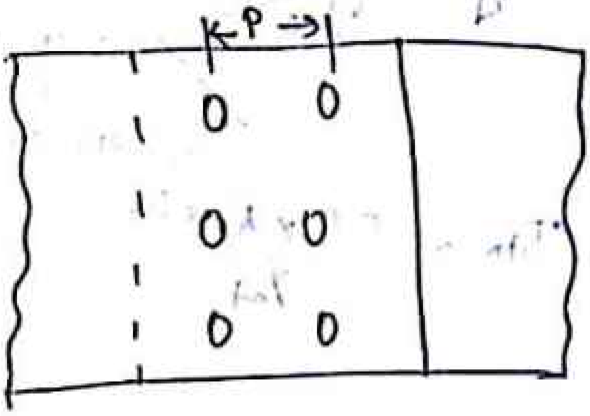
$$= 56.59 \text{ kN}.$$

* In the bearing pitch, end distance are not given. Therefore value of K_b is not known. That's why bearing strength (can't be determined).

$$\text{No. of bolts} = \frac{250}{45.27} = 5.52 \approx 6 \text{ Nos}.$$

Let's arrange the bolts in two columns.

Determination of pitch (P)



Shear strength of two bolts per pitch width $= 2 \times 45.27$

$= 90.54$ kN.

Strength of the plate per pitch width

$0.9 \times A_n \times f_u = 0.9 \times P \times t \times f_u$

$$\frac{0.9 \times p \times t \times f_u}{\gamma_{m1}} = 90.52$$

$$\Rightarrow p = \frac{90.52 \times \gamma_{m1}}{0.9 \times t \times f_u} = \frac{90.52 \times 1.25}{0.9 \times 8 \times 410}$$

$$= 38.32 \text{ mm}$$

Min^m. pitch $= 2.5d$

$= 2.5 \times d$

$= 50$ mm.

Let's provide 65 mm pitch.

~~eccentricity~~ edge distance (e) $= \frac{210 - 2 \times 65}{2}$

$= 40$ mm.

Check against bearing strength

K_b is least of the following.

i) $\frac{e}{3d_0} = \frac{40}{3 \times 22} = 0.606$

ii) $\frac{p}{3d_0} - 0.25 = \frac{65}{3 \times 22} - 0.25 = 0.734$

iii) $\frac{f_{ub}}{f_u} = 0.975$

iv) 1

$\therefore K_b = 0.606$

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

$$= \frac{2.5 \times 0.606 \times 20 \times 8 \times 410}{1.25}$$

$= 79.507$ kN > 48.6.

So design is OK.

Q. Two plate 10 mm & 18 mm thick are to be jointed by double cover butt joint. Design the joint for the following data.

(10)

Factored designed load = 750 kN.

Bolt dia = 20 mm

Grade of steel = Fe 410

Grade of bolt = 4.6

Cover plate 2 = 8 mm thick.
(one on each side)

Soln :- Data given

For a 4.6 grade of bolt

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

$\gamma_{mb} = 1.25$

$d = 20$ mm

$d_0 = 22$ mm

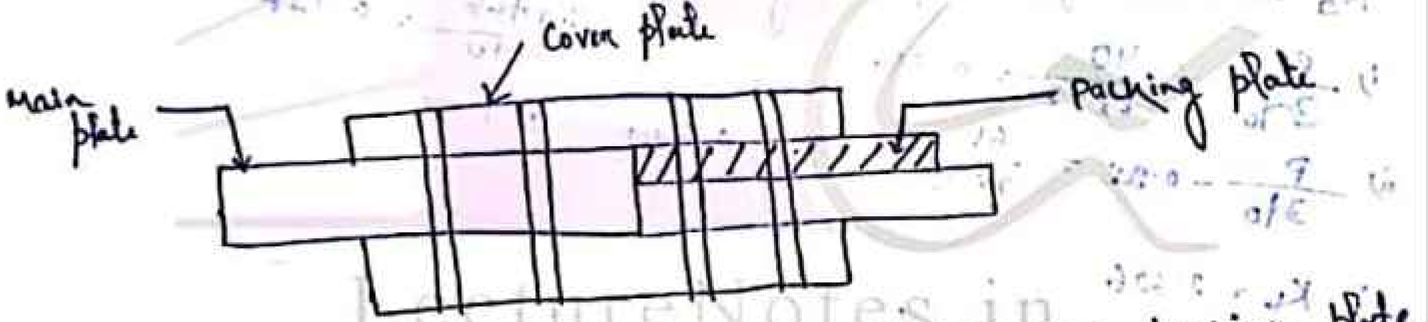
For Fe 410 grade of steel

$f_u = 410\ N/mm^2$

$\gamma_{m1} = 1.25$

Thickness of main plate = 10 mm & 18 mm.

" " cover " = 8 + 8 = 16 mm.



Since thickness of main plates are 18 mm and 10 mm, packing plate of thickness (18 − 10) = 8 mm is used.
Since the thickness of packing plate is more than 6 mm, it will reduce by a factor. (P-75, 10.3.3.2)

$$\beta_{pk} = (1 - 0.0125) \times t_{pk}$$
$$= (1 - 0.0125) \times 8$$
$$= 0.9$$

No. of bolt :-

Factored load = 750 kN.

Assuming bearing strength of the bolt is more than that of shearing strength of bolt, no. of bolts is to ~~the~~ determine on the basis of shear strength.

- Shear strength of bolt

(11)

$$V_{dsb} = \frac{f_{ub} V_{nsb}}{\gamma_{mb}} = \frac{116.628}{1.25} = 92.98 \text{ kN}.$$

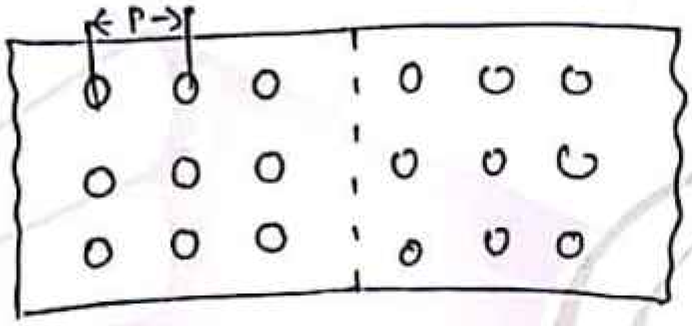
$$V_{nsb} = \left\{\frac{f_{ub}}{\sqrt{3}}\left(n_n A_{nb} + n_s A_{sb}\right)\right\} \times \beta_{pk}$$

$$= \left\{\frac{400}{\sqrt{3}}\left(1 \times 0.78 \times \frac{\pi}{4} \times d^2 + 1 \times \frac{\pi}{4} \times d^2\right)\right\} \times 0.9$$

$$= 116.628 \text{ kN}$$

$$\text{No. of bolts} = \frac{P}{V_{dsb}}$$

$$= \frac{750}{92.98} = 8.06$$
$$\approx 9 \text{ nos.}$$



Let $P \rightarrow$ be the pitch length.

Strength of the plate per pitch width

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

$$= \frac{0.9 \times (P - d_0) \times t \times f_u}{\gamma_{m1}}. \quad (t = 10 \text{ mm})$$

$$= 2.952(P-22) \text{ kN} \text{ ——— (1)}.$$

strength of two bolts per pitch width ——— (11).

$$= 2 \times 92.98 = 185.96 \text{ kN}.$$

$\Rightarrow$ Equating eqn (1) & (11).

$$2.952(P-22) = 185.96$$

$\Rightarrow p = \cancel{436}\ 85$ mm.

Min^m pitch $= 2.5 \times d$

$= 50$ mm.

Max^m pitch $= \cancel{32t}$ or $16t$ or 200 mm.

Let's provide a pitch 90 mm.

Check against bearing strength

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

K_b is least of the following.

i) $\frac{p}{3d_0} - 0.25 = 0.81$

ii) $\frac{e}{3d_0} = 0.5$

iii) 0.978

iv) 1

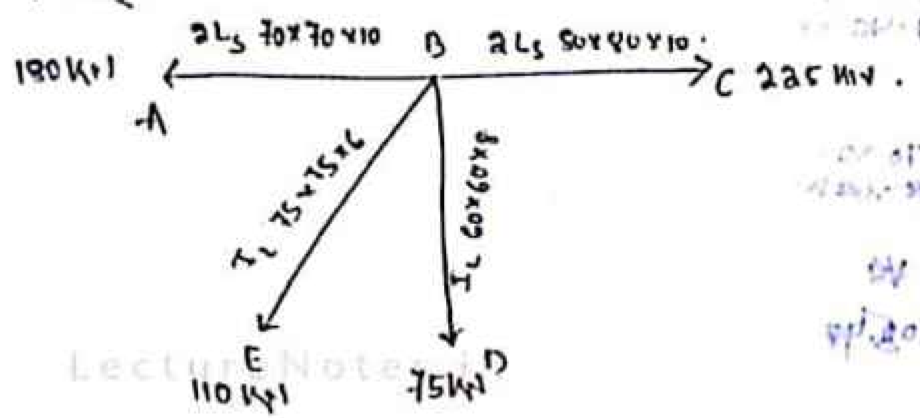
$\therefore K_b = 0.5$

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

$$= \frac{2.5 \times 0.5 \times 20 \times 10 \times 410}{1.25}$$

$= \; > 92.98$ kN.

Design joint beam of a roof truss as shown in fig. The members are connected in 16 mm dia bolt of 4.6 grade for the gusset plate 12 mm thick.



Sol^n :- Data given :-

For 4.6 grade of bolt,

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

$\gamma_{mb} = 1.25$

$d = 16$ mm

$d_0 = 16+2 = 18$ mm.

Fe ~~Fe~~ 410 grade of ~~bolt~~ plate.

$f_u = 410\ N/mm^2$

$\gamma_{m1} = 1.25$

Strength of the bolt in single shear

$$V_{dsb} = \frac{\frac{f_{ub}}{\sqrt{3}}(n_n A_{nb} + n_s A_{sb})}{\gamma_{mb}} = \frac{\frac{400}{\sqrt{3}}\left(1 \times 0.78 \times \frac{\pi}{4} \times 16^2\right)}{1.25}$$

$= 29$ kN

So strength of the bolt in double shear $= 29 \times 2$

$= 58$ kN

Strength of bolt in bearing

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

k_b least of the following (Assuming $e = 40$ mm, $P = 50$ mm)

i) $\frac{e}{3d_0} = \frac{40}{3 \times 18} = 0.74$

ii) $\frac{P}{3d_0} - 0.25 = \frac{50}{3 \times 18} - 0.25 = 0.67$

iii) $\frac{f_{ub}}{f_u} = 0.975$

iv) 1

$\therefore K_b = 0.67$.

Strength of the bolt in bearing on

i) t = 6m.

V_{dpb} = ~~57.46 kN~~ 52.74 kN.

ii) t = 8 mm,

V_{dpb} = ~~68.60 kN~~ 70.32 kN.

(iii) t = 10 mm,

V_{dpb} = 87.90 kN.

iv) t = 12 mm,

V_{dpb} = 105.48 kN.

Member AB :-

Factored load = 180 kN.

The member is composed of double angle section ISA 70 x 70 X 10 mm and is connected on the opposite sides of a 12 mm thick gusset plate. The bolts will be in double shear and will bear against the 12 mm thick i.e. (least of 12 mm and 2x10 = 20 mm) gusset plate.

Hence, strength of the bolt will be least of 58 kN and 105.48 kN i.e. 58 kN.

No. of bolts required = $\frac{180}{58}$ = 3.10 ≈ 4 Nos.

Member BC :-

Factored load = 225 kN.

The member is composed of double angle section ISA 80 x 80 x 10 mm. and is connected on the opposite sides of a 12 mm thick gusset plate. The bolts will bear against 12 mm thick (least of 12 mm and 2x10 = 20mm) gusset plate.

Hence strength of the bolt will be least of 58 kN and 105.48 kN i.e. 58 kN.

No. of bolts required = $\frac{225}{58}$ = 3.87 ≈ 4 Nos.

Member BD :-

Factored load = ~~110~~ 75 KN.

The member is single angle section ISA 60 × 60 × 8 mm and is connected to a 12 mm thick gusset plate. The bolt will be in single shear and bearing against 8 mm thick (least of 8 mm and 12 mm)

∴ Strength of bolt will be least of 29 KN and 70.32 KN. i.e. 29 KN

$$\text{No. of bolts required} = \cancel{\frac{110}{29}} = \frac{75}{29} = 2.58 \approx 3 \text{ nos.}$$

Member BE :-

Factored load = 110 KN.

The member is single angle section ISA 75 × 75 × 6 mm and is connected to a 12 mm thick gusset plate. The bolt will be in single shear and bearing against 6 mm thick (least of 6 mm and 12 mm).

∴ Strength of bolt will be least of 29 KN and 52.74 KN i.e. 29 KN.

$$\text{No. of bolts} = \frac{110}{29} = 3.79 \approx 4 \text{ Nos.}$$

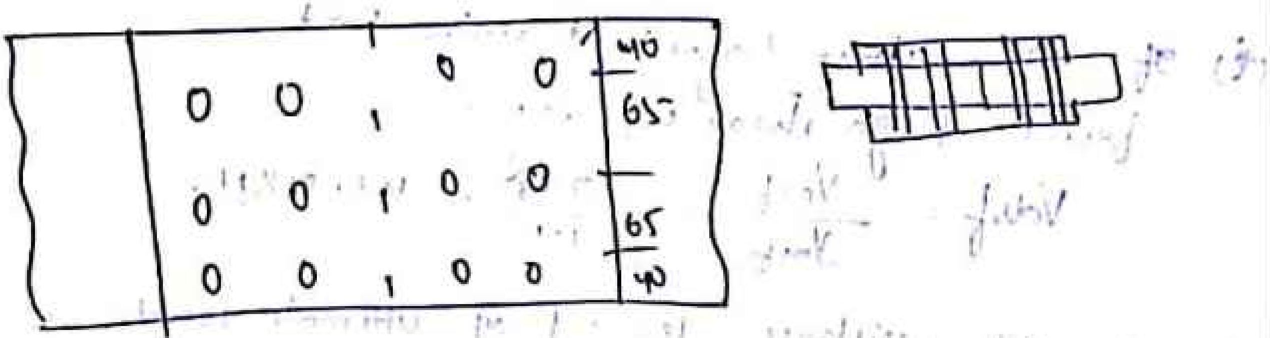
E Determine the shear capacity of bolts used in connecting two plates as shown in fig. if

i) Slip resistance is designated at service load

ii) " " " " ultimate load.

Given: HSFG of bolts of grade 8.8 are used in clearance holes if coefficient of friction 0.3.

S:-



Data given :-

for 8.8 grade of bolt

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

Assuming $d = 16$ mm.

$d_0 = 18$ mm.

For HSFG bolt (P-76, 10.4)

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

$V_{nsf} = \mu_f \times n_e \times K_h \times F_0$.

μ_f = coefficient of friction = 0.3

n_e (for double cover butt joint) = 2

$K_h = 1$ (clearance hole)

$F_0 = A_{nb} \times f_0$

$= 0.78 \times \frac{\pi}{4} \times d^2 \times 0.70 \times f_{ub}$

$= 87.8$ kN.

$\gamma_{mf} = 1.10$ (If slip resistance is designed at service load)

$= 1.25$ (" " " at ultimate load)

$V_{nsf} = 0.3 \times 2 \times 1 \times 87.8$

$= 52.68$ kN.

i) If slip resistance designed at service load,

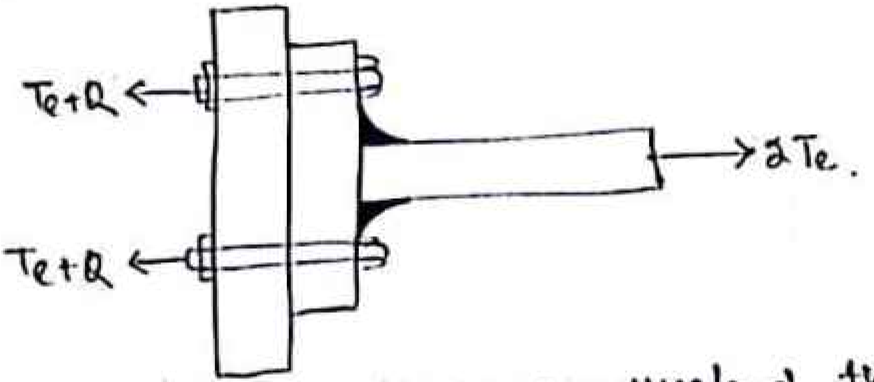
factor design shear capacity

$$V_{dsf} = \frac{V_{nsf}}{\gamma_{mf}} = \frac{52.68}{1.1} = 47.89 \, kN.$$

ii) If slip resistance designed at ultimate load,

$$V_{dsf} = \frac{V_{nsf}}{\gamma_{mf}} = \frac{52.68}{1.25} = 42.14 \, kN. \text{ (Ans)}$$

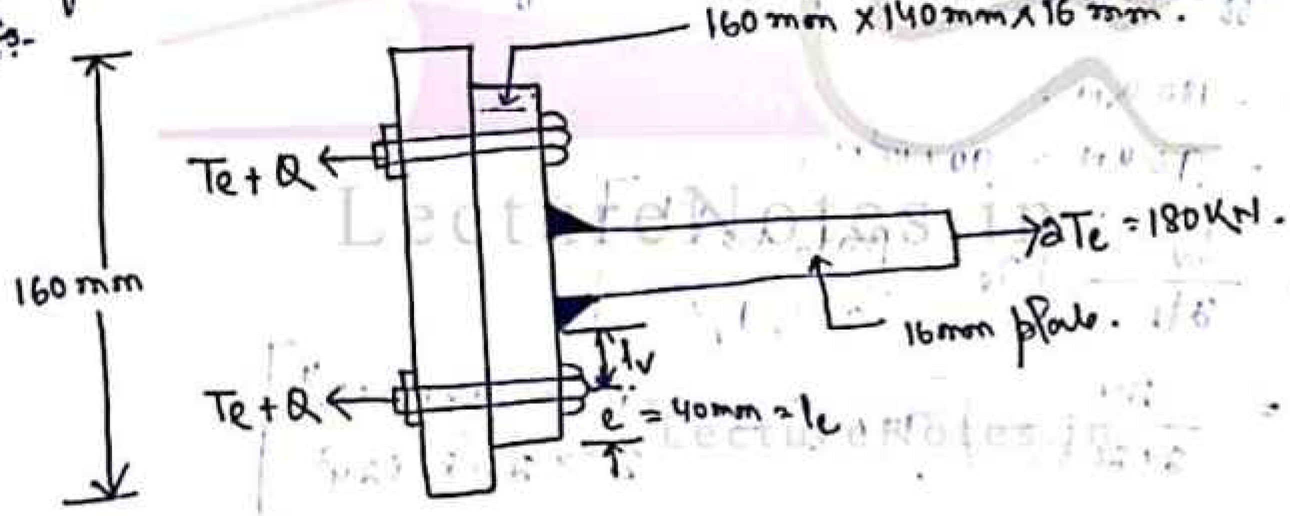
Combined Prying force and Tension : (P-77, 10.4.7)



If at connection point fails to withstand the load of 2Te. There will be an additional load will be provided to balance it. The additional tension is called prying force. Generally it occurs in HSFG bolts.

Q: The joint shown in the figure has to carry a factored load of 180 kN. End plate used each of size 160 mm × 140 mm × 16 mm. The bolts used are M_{20}. HSFG of grade 8.8. Check weather the design is safe or not.

Sol:-



Soln:- Let's assume size of the fillet weld is 8 mm.

$$\therefore l_v = \frac{160}{2} - \frac{16}{2} - 8 - 40 = 24 \text{ mm}.$$

Prying force

$$Q = \frac{l_v}{2l_e}\left[T_e - \frac{\beta \eta f_o b_e t^4}{27 l_e l_v^2}\right] \quad (P-77, 10.4.7)$$

$l_e = 1.1t\sqrt{\frac{\beta f_o}{f_y}} = 1.1 \times 16 \times \sqrt{\frac{1 \times 581}{250}} = 26.83$ mm.

$\beta = 1$ (Because HSFG are always pretensioned bolt). (18)

$\eta = 1.5$

$f_o = 0.70 \times f_{ub}$

$= 0.70 \times 830 = 581\ N/mm^2$

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

$t = 16$ mm.

~~l_e should be less than end~~

l_e is least of the following

(i) $1.1 \times t\sqrt{\frac{\beta f_o}{f_y}} = 26.83$ mm.

(ii) $e = 40$ mm.

$\therefore l_e = 26.83$ mm.

b_e = effective width of flange per pair of bolts = 140 mm.

$2T_e = 180$ kN.

$T_e = 90$ kN $= 90 \times 10^3$ N.

$$Q = \frac{l_v}{2l_e}\left[T_e - \frac{\beta \eta f_o b_e t^4}{27 l_e l_v^2}\right]$$

$$= \frac{24}{2 \times 26.83}\left[90 \times 10^3 - \frac{1 \times 1.5 \times 581 \times 140 \times 16^4}{27 \times 26.83 \times 24^2}\right]$$

$= 31.68$ kN.

Design tension strength of bolts

$$T_{dn} = \frac{0.9 \times A_n f_{ub}}{\gamma_{mf}}.$$

$$= \frac{0.9 \times 0.78 \times \frac{\pi}{4} \times d^2 \times 830}{1.25} = 146.43\ kN.$$

Total load on the bolt $= T_e + Q = 90 + 31.68$.

$= 121.68$ kN $< T_{dn}$.

So design is safe.

Welded Connection :-

When two structural members are jointed by means of weld, the connection is called welded connection which develops metalurgical bond between them. (19)

- The members to be connected are brought closer and the metal is melted by means of electric arch or by oxi-acetalin flame along with the weld rod, which adds melted to the joint.

Advantages of welded joint :-

① welded designs offer the opportunity to achieve a more efficient use of materials. Welding is the only process that produces a one piece construction.

② The speed of fabrication helps compress production schedules.

③ welding saves weight and consequently cuts cost.

④ No deductions are there for holes; thus the gross section is effective in carrying loads.

⑤ welded joints are better for impact loads and vibration.

Assumption in the analysis of welded joints :-

① The welds connecting the various parts are homogeneous isotropic and elastic elements.

② The parts connected by the weld are rigid and their deformations are therefore neglected.

③ Only stresses due to external loads are considered. Effects of residual stresses, stress concentration and shape of the welds are neglected.

Types of welded joint

① Butt weld or groove weld

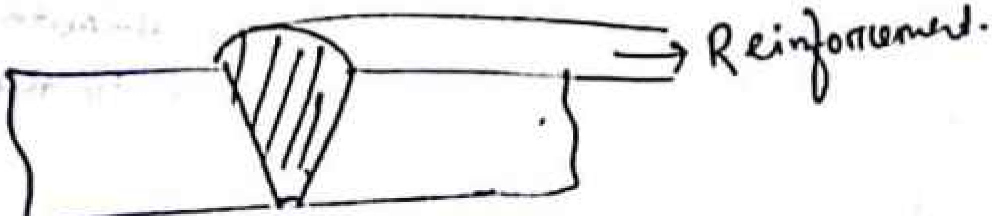
② Fillet weld

③ Slot weld

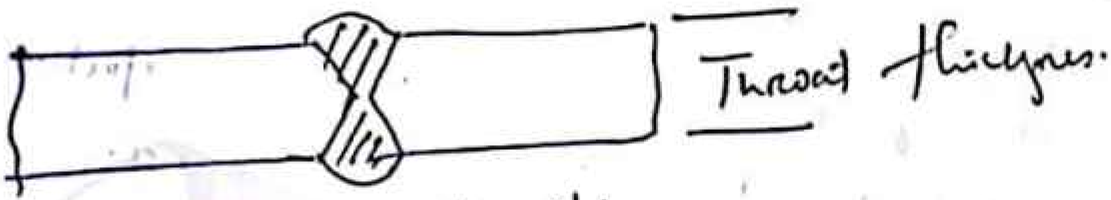
④ Plug weld.

① Butt weld :-

Butt weld is also known as groove weld depending upon the shape of the groove made for welding. Butt weld are classified as single butt weld & double butt weld. (20)

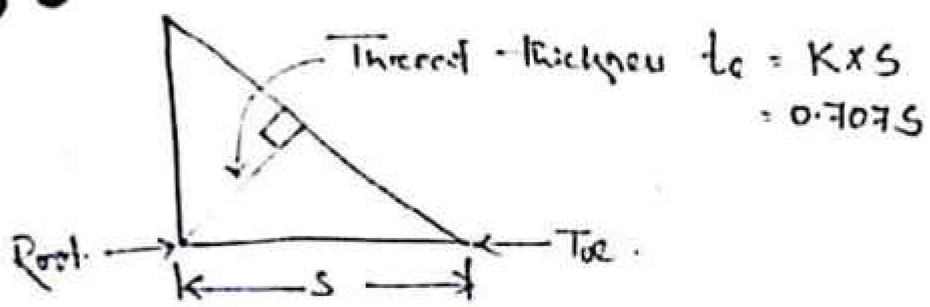


Single butt weld.



Double butt weld.

Specification of welding.



Size of weld (S) :- (P-78, 10.5.2)

* Size of the fillet weld is taken as the minimum weld leg size.
* The distance between the toe and root is called leg size of the weld.
* Max^m size of weld :-

The max^m size of a fillet weld is obtained by substracting 1.5 mm from the thickness of the thinner member to be jointed.

* Minimum size of weld :-

Thickness of thicker member. Over (mm)	upto and including (mm)	Minimum size (mm).
0	10	3
10	20	5
20	32	6
32	50	8 first run 10.

* Effective throat thickness (P-78, 10.5.3)

It is the shortest distance from the root of the fillet weld to the face of weld.

* The effective throat thickness should not be less than 3 mm. And it should not exceed 0.7 t or 1t.

Effective throat thickness = $K \times S$

Effective length

It is the length of the fillet weld for which the specified size and throat thickness of weld exist. It is taken equal to its overall length minus twice the weld size.

(53)

Design strength of groove weld :-

The design strength of groove weld in tension or compression.

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

$L_w \rightarrow$ effective length of the weld in mm.

$t_e \rightarrow$ effective throat thickness of weld.

$\gamma_{mw} \rightarrow$ partial safety factor.

= 1.25 for shop welding.

= 1.5 for site welding.

Design procedure :— (groove weld)

① In case of complete penetration of the groove weld, design calculations are not required as the weld strength at the joint is equal to the strength of the member connected.

② In case of incomplete penetration of the butt weld, the effective throat thickness is computed and the required effective length is determined to furnish the strength equal to the strength of the members connected.

Design ~~procedure~~ strength of fillet weld :-

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

f_{wn} = nominal strength of fillet weld = $f_u/\sqrt{3}$

The design strength of a fillet weld is based on its throat area is given by $P_{dw} = L_w \times t_t \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}}$

minimum effective length of four times the size of the weld with a minimum of 40 mm, except for plate girder.

(3) The clear spacing between an intermittent fillet weld should not exceed 12t for compression and 16t for tension and should in no case be more than 200 mm.

(4) The longitudinal intermittent fillet weld should be of a length not less than the width of the member or else transverse welds should also be provided.

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Stresses due to individual forces :-

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

$f_a \rightarrow$ calculated normal stress.

$q \rightarrow$ shear stress

$P \rightarrow$ force transmitted

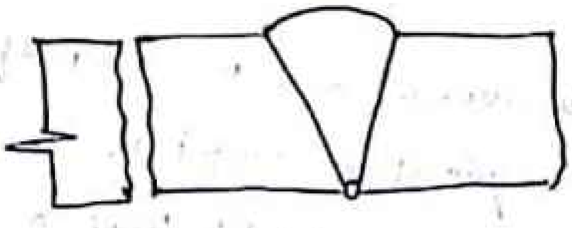
$t_t \rightarrow$ effective throat thick

$l_w \rightarrow$ effective length of weld.

Failure of welds :-

(1) Butt weld :-

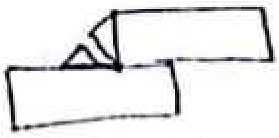
When the butt weld is reinforced on both the sides of the plate, the section through the weld is increased to such an extent that it is unlikely for failure to occur in the weld, and the fracture normally occurs some distance away.



अवतल fillet weld :-

The plane of the fracture in a normal proffile convex fillet weld is along the diagonal from the root of the fillet.

(25)



~~(I) concave fillet welds :-~~

Welded joint Vs Bolted and riveted joints :-

1. welded joints are economical
2. welded structures are more rigid as compared to bolted/riveted joint. In bolted and riveted joints, cover plates, connecting angles etc deflect along with the member during load transfer and make the joint more flexible.
3. Due to fusion of two metal pieces jointed, a continuous structure is obtained, which gives a better architectural appearance than bolted and riveted joints
4. Alterations can be done with less expences in case of welding as compared to bolting.
5. The process of welding is quicker in comparison to bolting
6. The process of welding is silent, whereas in the case of riveting a lot of noise is produced.
7. In welding less safety precautions are required for the public in the vicinity, whereas a hot rivet may toss and injure the persons working.
8. As splice plates, bolts and rivets etc. are not used, the details and drawing of welded str. are easier and less time consuming.

(9) The efficiency of welded joint is more than that of a bolted or riveted joint. In fact, a proper welded joint may have 100% efficiency.

(10) Members to be jointed may distort due to the heat during the welding process, whereas there is no such possibility in bolted and riveted joint.

(11) The possibility of a brittle fracture is more in the case of welded joints as compared to bolted and riveted joints.

(12) The inspection of welded joints is difficult and expensive, whereas the bolted/riveted joints can be inspected simply by tapping the joint with a hammer.

(13) A more skilled person is required to make a welded joint as compared to a bolted/riveted joint.

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

i) single V groove weld is provided

ii) Double V " " "

Assume the plate will be shop welded.

Soln:- Data given,

Let's assume Fe 410 grade of steel.

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

length of weld $= l_w = 175$ mm.

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

Case-I

Single V- grooved weld

Here incomple penetration of weld takes place.

∴ Throat thickness $t_t = 0.707 \times t$

$= 0.707 \times 14$

$= 9.89$ mm.

Strength of weld $T_{dw} = L_w \times t_t \times \frac{f_y}{\gamma_{mw}}$.

$= 175 \times 9.89 \times \frac{250}{1.25}$

$= 346$ KN $\not> 430$ KN.

So it is unsafe.

Case-II

Single double V- grooved weld

Here comple penetration of weld takes place.

∴ $t_t = 14$ mm.

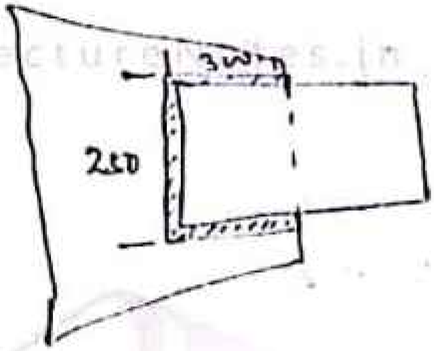
Strength of weld

$P_{dw} = l_w \times t_t \times \frac{f_y}{\gamma_{mw}}$.

$= 175 \times 14 \times \frac{250}{1.25} = 490$ KN > 430 KN.

∴ So it is safe.

Q. A tie member in a truss girder is 250 mm × 14 mm in size. It is welded to a 10 mm-thick gusset plate by a fillet weld. The overlap of the member is 300 mm and the weld size is 6 mm. Determine the design strength of the joint, if the welding is done as shown in fig. What is the increase in strength of the joint, if welding is done all around. Assume shop welding. (28)



Soln:- For Fe 410 grade of steel,

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

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

γ_{mw} for shop weld $= 1.25$.

Effective length of weld $L_w = 2 \times 300 + 250$.

$= 850$ mm.

Effective throat thickness $t_t = K \times S$

$= 0.707 \times 6$

$= 4.24$ mm.

Design strength of weld, $P_{dw} = L_w \times t_t \times \dfrac{f_u}{\sqrt{3} \times \gamma_{mw}}$

$= 850 \times 4.24 \times \dfrac{410}{\sqrt{3} \times 1.25} = 682$ KN.

When the welding is done all around.

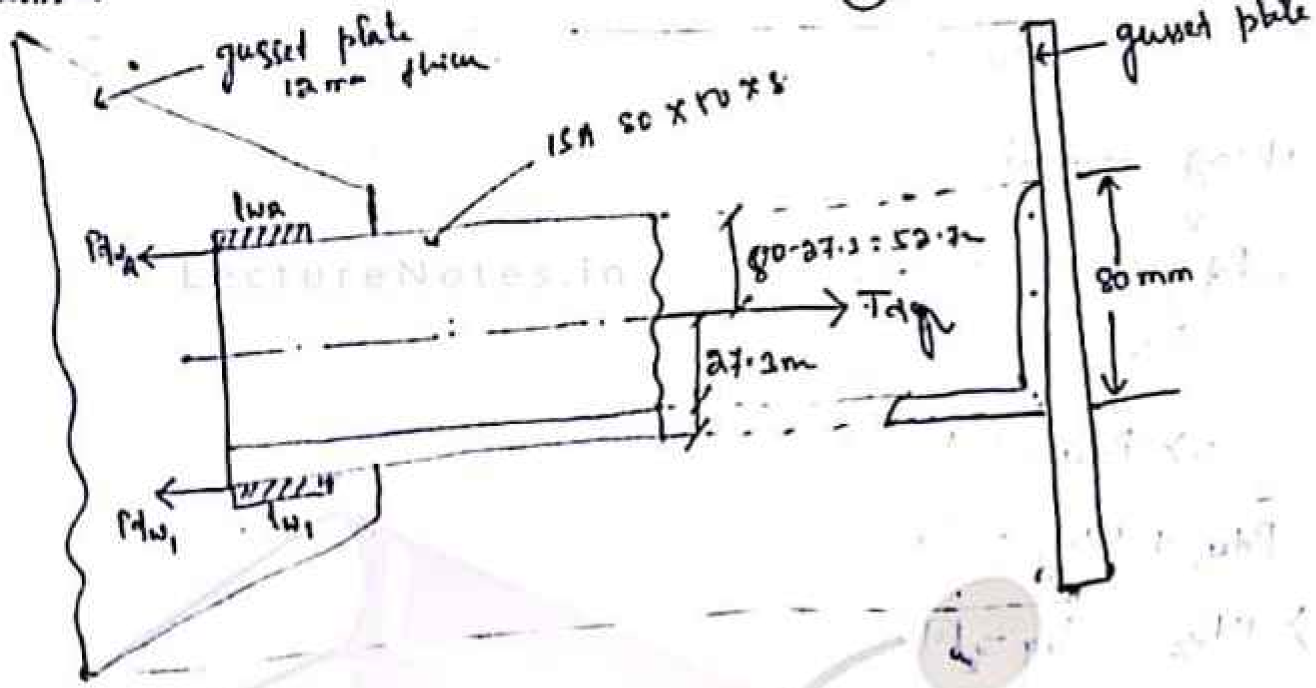
$L_{w_1} = 2(300 + 250) = 1100$ mm.

$P_{dw_1} = 1100 \times 4.24 \times \dfrac{410}{\sqrt{3} \times 1.25} = 883$ KN.

∴ Increase in the strength of the joint $= 883 - 682 = 201$ KN Ans

Q. A tie member (tension member) consisting of ISA 80 mm × 50 mm × 8 mm (Fe ●0 grade steel) is welded to a 12 mm thick gusset plate at site. Design welds to transmit load equal to the design strength of member.

(29)



Sol^n:-

Data given :-

For Fe 410 grade steel,

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

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

Partial safety factor against yielding $\gamma_{m0} = 1.1$

For site weld, $\gamma_{mw} = \cancel{\ }\,1.5$

From Steel table for ISA 80 × 50 × 8 (P-6)

$A_g = 978\ mm^2$, $C_{xx} = 27.3\ mm$.

The design strength of the member governed by yield of gross section.

$$T_{dg} = \frac{A_g \times f_y}{\gamma_{m0}} \quad (P\text{-}32)$$

$$= \frac{978 \times 250}{1.1} = 222.27\ kN.$$

∴ The weld will be designed to transmit a force of 222.27 kN.

Let, P_{dw_1} = Strength of the fillet weld or tensile force resisted by weld of effective length l_{w_1}

P_{dw_2} = Strength of the fillet weld or tensile force resisted by weld of effective length l_{w_2}.

30

Taking moment about line of action of P_{dw_2}.

~~$P_{dw_1} \times 80$~~

$$P_{dw_1} \times 80 = T_{dn} \times (80 - 27.3)$$

$$\Rightarrow P_{dw_1} = 146.42 \text{ KN}.$$

$$P_{dw_1} + P_{dw_2} = T_{dg}$$

$$\Rightarrow P_{dw_2} = T_{dg} - P_{dw_1}$$

$$= 75.85 \text{ KN}$$

- **Size of fillet weld (S) :–**

Min^m size of the fillet weld = 5 mm.

Max^m size of the " " $= 8 - 1.5$

$= 6.5$ mm

Let's provide 6 mm size of fillet weld.

Effective throat thickness $= t_t = 0.7 \times 6$

$= 4.2$ mm.

design strength of weld

Since $P_{dw_1} = \dfrac{l_{w_1} \times t_t \times f_u}{\sqrt{3} \times \gamma_{mw}}$

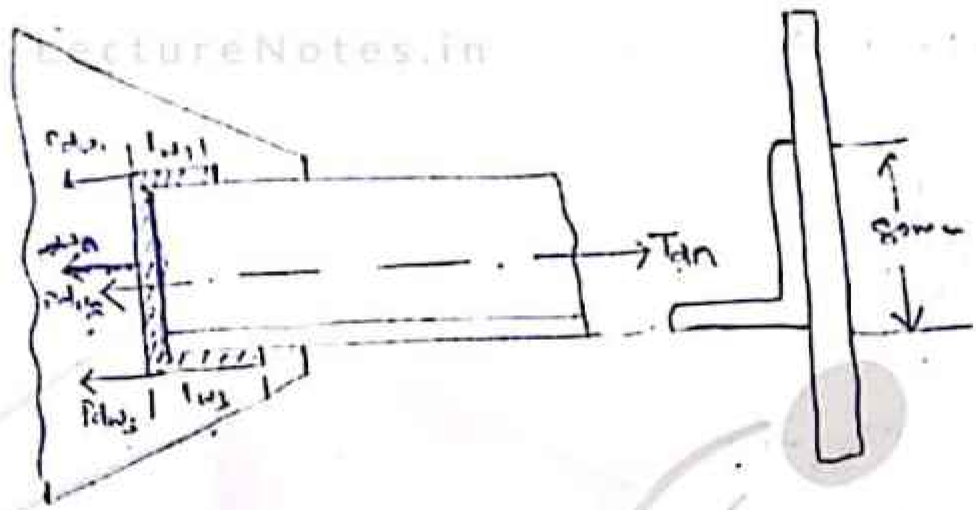
$$\Rightarrow l_{w_1} = \frac{P_{dw_1} \times \sqrt{3} \times \gamma_{mw}}{t_t \times f_u}$$

$$= \frac{146.42 \times \sqrt{3} \times 1.5}{4.2 \times 410} = 218 \text{ mm}$$

$$P_{dw_2} = \frac{l_{w_2} \times t_t \times f_u}{\sqrt{3} \times \gamma_{mw}}$$

$$\Rightarrow l_{w_2} = 113 \text{ mm}.$$

Q. Design the fillet weld for the angle section of ISA 80×50×8 mm (Fe 410 grade of steel) is welded to a 12 mm thick gusset plate at site. The weld is to be done on its three sides.



Solⁿ:- For steel of grade Fe 410,

$f_u = 410$ MPa.

for site weld, $\gamma_{mw} = 1.5$

Total weld length $= l_{w_1} + 80 + l_{w_3}$

Tensile strength of weld

$$T_{dn} = \frac{A_g \times f_y}{\gamma_{m0}} = \frac{978 \times 250}{1.1}$$ ($A_g = 978\ mm^2$, P-6)

$C_{xx} = 27.3$ mm.

$= 222.27$ kN.

$$P_{dw_2} = \frac{l_{w_2} \times t_t \times f_u}{\sqrt{3} \times \gamma_{mw}}$$

$$= \frac{80 \times 4.2 \times 410}{\sqrt{3} \times 1.5} = 53\ kN.$$

(minⁿ $S = 5$ mm

Maxⁿ $S = 8 - 1.5$
$= 6.5$ mm.

$S = 6$ mm

$t_t = 0.7 \times 6$
$= 4.2$ mm.)

Taking moment about bottom fiber.

$$P_{dw_1} \times 80 + P_{dw_2} \times 40 = T_{dn} \times 27.3.$$

$$\Rightarrow P_{dw_1} = \frac{222 \times 27.3 - 53 \times 40}{80}$$

$$= 49.34 \text{ kN}.$$

$$P_{dw_1} + P_{dw_2} + P_{dw_3} = T_{dn}.$$

$$\Rightarrow P_{dw_3} = 119.912 \text{ kN}.$$

$$P_{dw_1} = \frac{l_{w_1} \times t_t \times f_u}{\sqrt{3} \times \gamma_{mw}}$$

$$P_{dw_3} = \frac{l_{w_3} \times t_t \times f_u}{\sqrt{3} \times \gamma_{mw}}$$

$$\Rightarrow l_{w_1} = \frac{P_{dw_1} \times \sqrt{3} \times \gamma_{mw}}{t_t \times f_u}$$

$$= \frac{49.34 \times \cancel{10^3} \sqrt{3} \times 1.5}{4.2 \times 410}$$

$$= 74.42 \text{ mm}$$

$$= 75 \text{ mm}.$$

$$\Rightarrow l_{w_3} = 180.91 \text{ m}.$$

$$\simeq 181 \text{ mm}.$$

Q: An ISLC 300 @ 324.7 N/m (Fe 410 grade of steel) is to carry a factored tensile force of 900 kN. The channel section is to be welded at the site to a gusset plate 12 mm thick. Design a fillet weld, if the overlap is limited to 310 mm.

Soln:- For Fe 410 grade of steel.

$f_u = 410 \text{ N/mm}^2$.

for site weld, $\gamma_{mw} = 1.5$.

For ISLC 300 @ 324.7 N/m. (P-16)

from steel table. $A_g = 4211 \text{ mm}^2$, $t_f = 11.6 \text{ mm}$

$t_w = 6.7 \text{ mm}$.

Min^m size of fillet weld = 5 mm (12 mm thick plate).

Max^m " " " " = 6.7 - 1.5

= 5.2 mm.

Let's provide size of fillet weld = 5 mm.

$t_t = 0.7 \times 5$

$= 3.5$ mm.

Strength of weld per unit length

$$P_{dw} = \frac{l_w \times t_t \times f_u}{\sqrt{3} \times \gamma_{mw}}$$

$$= \frac{1 \times 3.5 \times 410}{\sqrt{3} \times 1.5} = 552.33 \text{ N/mm}.$$

Length of weld required $L_w = \frac{P}{P_{dw}}$

$$= \frac{900 \times 10^3}{552.33}$$

$$= 1629.46 \simeq 1630 \text{ mm}.$$

Because of the restriction of 350 mm overlap length of weld that can be provided in usual way

$= 2 \times 350 + 300 = 1000 \text{ mm} < 1630$ mm.

Hence, let us provide slot welds.

width of slot should be less than 3t or 25 mm, whichever is greater.

$3 \times 6.7 = 20.1$ mm or 25 mm.

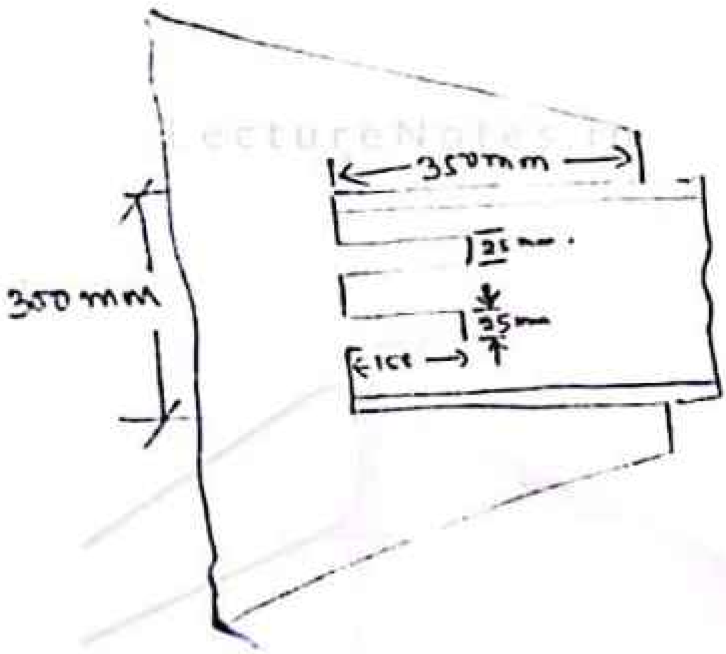
∴ Let's provide two slots and let the length of slots be l_1

$1630 = 2 \times 350 + 350 + 4 \times l_1$

$\Rightarrow l_1 = 157.5$

≈ 158 mm.

Provide 158 mm x 25 mm slots, two in nos.



Note:-

A/c to IS 816:1969

i) width or dia of the slot weld should be less than 3 times the thickness or 25 mm which ever is greater.

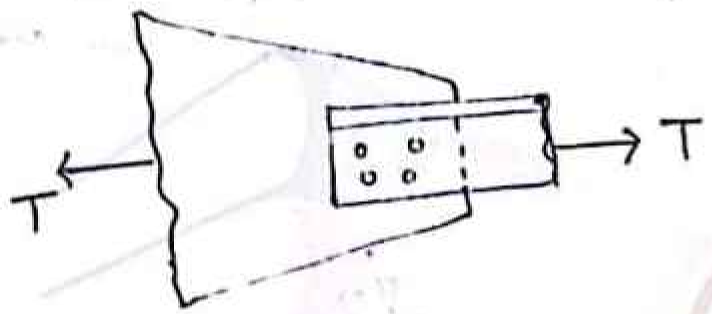
(ii) Slots should be rounded with radius not less than 1.5 times the thickness or 12 mm which ever is ~~better~~ greater.

Design of tension members :-

(P-32, section-6)

Tension member :-

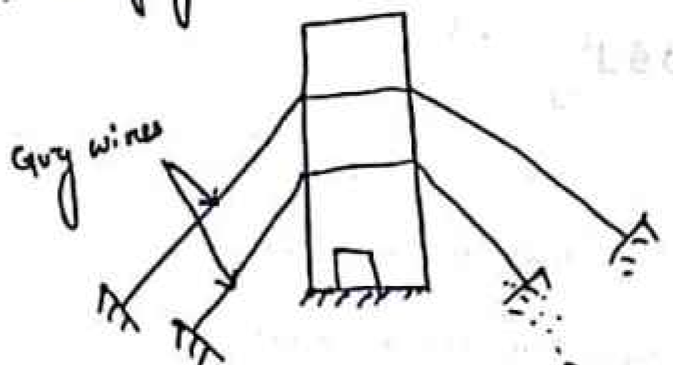
* A structural member subjected to two pulling forces applied at its ends is called a tension member.
* Tension members are also known as tie members.



* **Types of tension members :-**

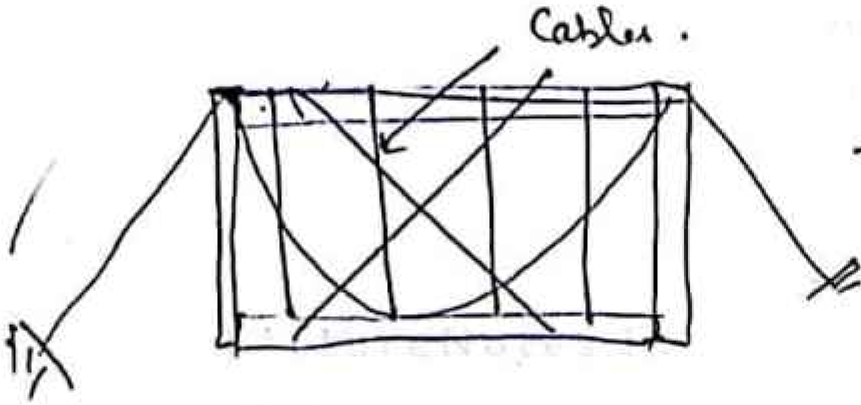
① **Wires and Cable :-**

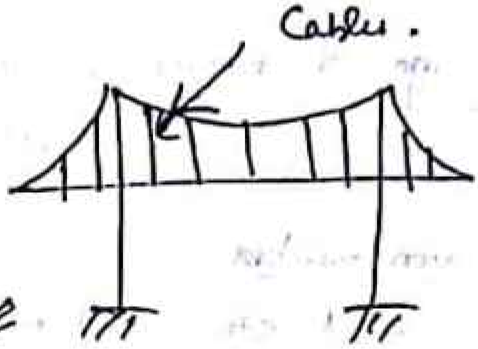
Wire ropes are exclusively used for hoisting purposes and as guy wires in steel stacks and towers.



(Guyed steel stack)

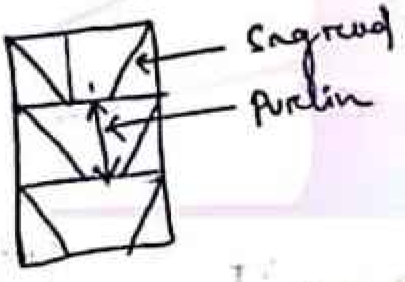
Cables used as floor suspenders in suspension bridges are made from individual strands wound together in rope like fashion.

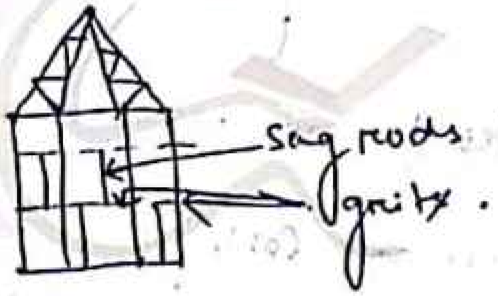




② Bars and Rods :-

These are often used as tension members in bracing system as sag rods to support purlins between trusses. and to support girts in industrial buildings.





③ Plates and flat bars :-

Plates and flat bars are often used as tension members. in transmission towers, foot bridges etc.

* Net sectional area :-

The net sectional area of a tension member

= gross sectional area of the member.

— the sectional area of the maximum no. of holes.

$A_n = (b - nd_0)t$ $\quad$ $A_n = \left[b - nd_0 + \sum_i \frac{p_{si}^2}{4g_i} \right] \times t.$

Types of failures :-

① Gross section yielding :-

Considerable deformation of the member in longitudinal direction may take place before its fractures, making the structure unserviceable.

② Net section Rupture :-

The rupture of the member when the net cross section of the member reaches the ultimate stress.

③ Block Shear failure :-

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

Slenderness Ratio (λ) :-

The slenderness ratio of a tension member is the ratio of its unsupported length L to its least radius of gyration r.

$$\boxed{\lambda = \frac{L}{r} \qquad r = \sqrt{\frac{I}{A}}}$$

Design of tension member :-

(38)

The 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) The block shear (T_{db})

(a) Design strength due to yielding of gross section

$$T_{dg} = \frac{A_g f_y}{\gamma_{m0}} \quad (P-32,\ 6.2)$$

(b) Rupture strength of critical section

(i) for plates

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}} \quad (P-32,\ 6.3.1)$$

(ii) for threaded rods

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}} \quad (P-33,\ 6.3.2)$$

A_n = net ~~...~~ area at the threaded section.

(iii) single angles :-

$$T_{dn} = \frac{0.9 A_{nc} f_u}{\gamma_{m1}} + \frac{\beta A_{go} f_y}{\gamma_{m0}} \quad (P-33,\ 6.3.3)$$

$$\beta = 1.4 - 0.076 (w/t)(f_y/f_u)(b_s/L_c)$$

ordherwise

$$T_{dn} = \frac{9 A_n f_u}{\gamma_{m1}}.$$

T_{dn} will be least be of (iii).

Design strength due to block shear

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

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

T_{db} will be least of the above.

Design of tension member subjected to axial load :-

Step-1

Required net area is determined by using the formula.

$$A_n = \frac{T \times \gamma_{m1}}{0.9 f_u} \text{ for plate.}$$

$T \Rightarrow$ factored tensile load.

Step-2 Required net area as obtained in step-1 will be increased 25% - 40% to compute the gross area A_g.

Step-3

Gross area required 'Ag' also determined by

$$A_g = \frac{T \times \gamma_{m1}}{f_y}$$

Step-4

From steel table looking at the value of Ag, a rolled section is selected.

Step-5

No. of bolts can be determined by using the formula

$$= \frac{\text{Load transmitted}}{\text{Strength of one bolt}}$$

Step-6

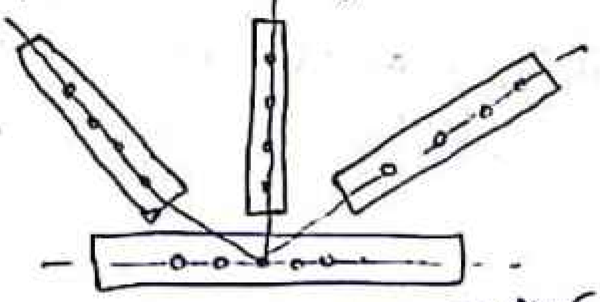
. Design strength T_d of trial section is calculated. This will be minimum of strength T_{dg}, T_{dn} & T_{db}.

The design strength $T_d > T$. So design is OK.

Gusset plate :-

/ A gusset plate is a plate provided to make connection at the place where more than one member is to be joined at the place where more than one member is to be joined i.e. joint of truss, truss girder etc.

* The size and shape of the gusset plates are usually decided from the direction of the members meeting at joint.

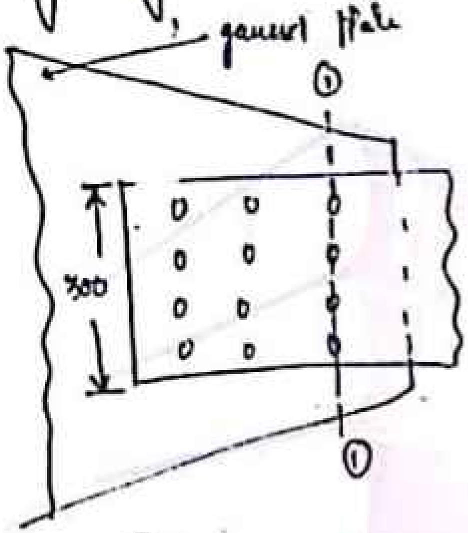


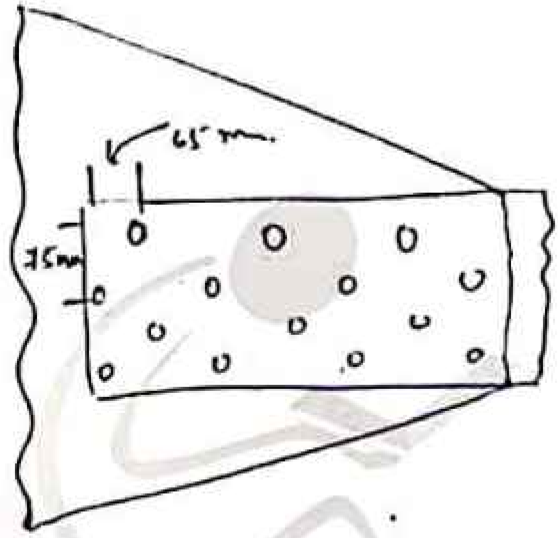
* The lines of action of truss members meeting at a joint are assumed to be coincide as shown in fig.

Q. A 300 ISF 8 mm of grade Fe 410 is used as a tension member in a lattice girder. It is connected to a 12 mm thick gusset plate by 18 mm dia bolt of grade 4.6. Calculate effective net area if.

(a) chain bolting is done as shown in fig..

(b) Zig Zag bolting is done " " ".





Solⁿ:- For Fe. 410 grade of steel,

$f_u = 410\ MPa$. $\qquad d = 18\ mm$.

$f_y = 250\ N/mm^2 \qquad d_0 = 20\ mm$.

(a) In chain bolting, the critical section will be 1-1

$$A_n = (B - nd_0)\,t$$

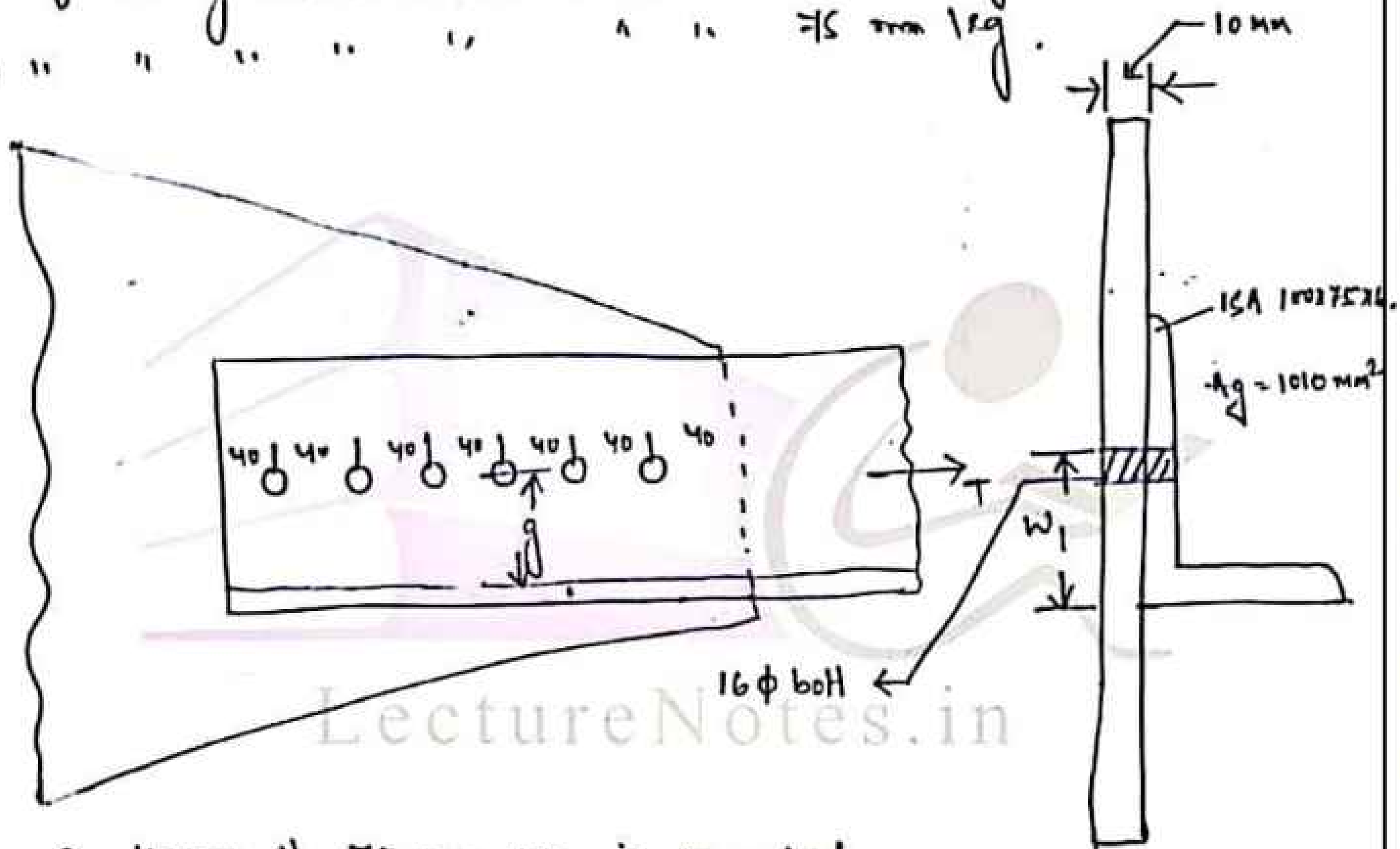
$$= (300 - 4 \times 20) \times 8$$

$$= 1760\ mm^2$$

Q. A single unequal angle 100 x 75 x 6 is connected to a 10 mm thick gusset plate at the ends with 6 16mm dia bolts to transfer tension as shown in fig. Determine the design tensile strength of the angle. Assuming that the yield & ultimate stress of steel used are 250 N/mm2 & 410 N/mm2.

(i) If the gusset is connected to the 100 mm leg.

(ii) " " " " " " 75 mm leg.



g = 40 mm if 75 mm leg is connected.

= 60 mm if 100 mm " " ".

Soln:- For fe 410 grade of steel,

$f_y = 250\ N/mm^2$ $\gamma_{m0} = 1.1$

$f_u = 410\ N/mm^2$ $\gamma_{m1} = 1.25$

$d = 16$ mm

$d_0 = 18$ mm.

(ii) Design strength due to yielding of gross area.
(P-32, 6.2)

$$T_{dg} = \frac{A_g \cdot f_y}{\gamma_{m0}} = \frac{1010 \times 250}{1.1}$$
$$= 229.54 \text{ kN}.$$

(iii) Design strength due to rupture of critical section
(P-33, 6.3.3)

$$T_{dn} = \frac{0.9 A_{nc} f_u}{\gamma_{m1}} + \frac{\beta A_{go} f_y}{\gamma_{m0}}$$ 6.9

A_{nc} = net area of connected leg.
$= \left(100 - 18 - \frac{6}{2}\right) \times 6 = 474 \text{ mm}^2$

A_{go} = gross area of the outstanding leg
$= \left(75 - \frac{6}{2}\right) \times 6 = 432 \text{ mm}^2$

$\beta = 1.4 - 0.076 \left(\frac{w}{t}\right) \times \left(\frac{f_y}{f_u}\right) \times \left(\frac{b_s}{L_c}\right) = 1.02$

w = outstand leg width = 75 mm.

t = 6 mm.

b_s = shear lag width (P-33, fig 6).
$= 75 + 60 - 6 = 129$ mm.
$= w + w_1 - t$

w_1 = 60 mm (as 100 mm leg is connected).

$L_c = 5 \times 40 = 200$ mm.

$$\beta = \frac{f_u \gamma_{m0}}{f_y \gamma_{m1}}$$

$$= \frac{410 \times 1.1}{250 \times 1.25} = 1.44$$

$$0.7 \leq \beta < \frac{f_u \gamma_{m0}}{f_y \gamma_{m1}}$$

$\therefore \beta = 1.02$

$$T_{dn} = \frac{0.9 \times 474 \times 410}{1.25} + \frac{1.02 \times 432 \times 250}{1.1}$$

$$= 240\ kN.$$

Otherwise

$$T_{dn} = \frac{\alpha A_n f_u}{\gamma_{m1}} \ (P-33.) = \frac{0.8 \times 906 \times 410}{1.25}$$

$$= 237.73\ kN.$$

$\alpha = 0.8$ for 6 bolts

A_n = net area of the total c/s.

$= (A_{nc} + A_{go}) = 474 + 432 = 906\ mm^2$

T_{dn} will be least of the following i.e. 240 & 237.73 kN.

$\therefore T_{dn} = 237.73\ kN.$

(ii) Design strength due to block shear (P-33, 6.4.1)

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

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

$A_{vg} = (6 \times 40) \times 6 = 1440 \text{ mm}^2$

$A_{vn} = \left(6 \times 40 - 5 \times 18 - \frac{18}{2}\right) \times 6 = 846 \text{ mm}^2$

$A_{tg} = (100 - 60) \times 6 = 240 \text{ mm}^2$

$A_{tn} = \left(100 - 60 - \frac{18}{2}\right) \times 6 = 186 \text{ mm}^2$.

$$T_{db} = \frac{1440 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 186 \times 410}{1.25}$$

$= 243.85$ KN.

$$T_{db} = \frac{0.9 \times 846 \times 410}{\sqrt{3} \times 1.25} + \frac{240 \times 250}{1.1}$$

$= 198.75$ KN.

T_{db} will least of the above 243.85 & 198.75

$\therefore T_{db} = 198.75$ KN.

Design strength of the member will be least of the following T_{dg}, T_{dn} & T_{db}

$\therefore$ design strength of the member is 198.75 KN. (Ans).

Q. Design a bridge truss diagonal subjected to a factored load of 300 kN. The length of the diagonal is 3 m. The tension member is connected to a gusset plate 16 mm thick with one line of 20 mm dia bolts of grade 8.8.

Soln:- Data given :-

Assuming Fe 410 grade of steel,

$f_u = 410 \ N/mm^2$

$f_y = 250 \ N/mm^2$

$\gamma_{m0} = 1.1$

$\gamma_{m1} = 1.25$

For 8.8 grade of bolt,

$f_{ub} = 830 \ N/mm^2$

$d = 20 \ mm$

$d_0 = 22 \ mm$.

Tensile load $T = 300 \ kN$.

Required net area of the angle section

$$A_n = \frac{T \times \gamma_{m1}}{0.9 \times f_u}$$

$$= \frac{300 \times 10^3 \times 1.25}{0.9 \times 410} = 1016.26 \ mm^2$$

Required gross area

$$A_g = \frac{T \times \gamma_{m0}}{f_y}$$

$$= \frac{300 \times 10^3 \times 1.1}{250} = 1320 \ mm^2$$

(pg-8)

From steel table,

Let's provide ISA $100 \times 75 \times 8$ mm as tension member.

A_g Provided = 1336 mm^2

No. of bolts :-

(i) Shearing strength of bolt in single shear.

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

$$= \frac{830 \times 1 \times 0.78 \times \frac{\pi}{4} \times 20^2}{\sqrt{3} \times 1.25}$$

$$= 93.94\ kN.$$

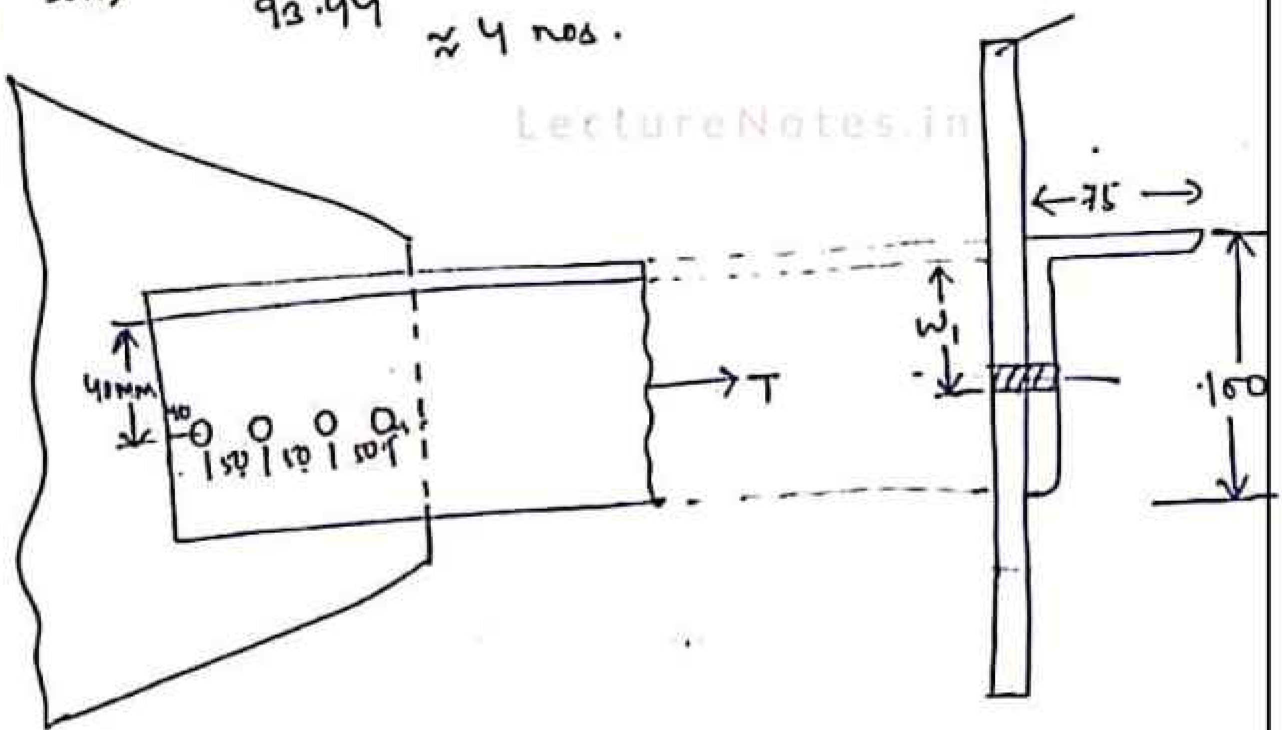
(ii) Bearing strength of bolt.

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

$$= \frac{2.5 \times 1 \times 20 \times 8 \times 410}{1.25} = 131.2\ kN.$$

$\therefore$ strength of bolt $= 93.94\ kN$.

No. of bolts $= \frac{300}{93.94} = 3.19$
≈ 4 nos.



Let's provide 4, 20 mm dia bolt with edge distance 40 mm and minimum pitch 50 mm in one line as shown in fig.

~~Design~~ Check for design tensile strength (48)

① Design strength due to yielding (P-32, 6.2)

$$T_{dg} = \frac{A_g \times f_y}{\gamma_{m0}} = \frac{1336 \times 250}{1.1} = 303 \text{ KN} > T.$$

So it is ok.

② Design strength due to rupture. (P-33, 6.3.3)

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

A_{nc} = net area of connected leg

$= \left(100 - 22 - \frac{8}{2}\right) \times 8 = 592 \text{ mm}^2.$

A_{go} = gross area of outstanding leg.

$= \left(75 - \frac{8}{2}\right) \times 8 = 568 \text{ mm}^2.$

$\beta = 1.4 - 0.076 \times \left(\frac{w}{t}\right) \times \left(\frac{f_y}{f_u}\right) \times \left(\frac{b_s}{L_c}\right) = 1.09$

w = outstand leg width = 75 mm.

t = 8 mm.

b_s = shear ~~length~~ lag width.

$= w + w_1 - t = 75 + 40 - 8 = 107$ mm.

w_1 = 40 mm.

L_c = distance betn end bolts $= 3 \times 50$

$= 150$ mm.

$$\beta = \frac{f_u \times \gamma_{m0}}{f_y \times \gamma_{m1}} = \frac{410 \times 1.1}{250 \times 1.25} = 1.44$$

$$0.7 < \beta < \frac{f_u \gamma_{m0}}{f_y \gamma_{m1}}.$$

$$\therefore \beta = 1.09$$

$$T_{dn} = \frac{0.9 \times 592 \times 410}{1.25} + \frac{1.09 \times 568 \times 250}{1.1}$$

$$= 315 \text{ KN} > 300 \text{ KN}.$$

So it is OK.

(iii) Design strength due to block shear. (P. 33, 6.4.1)

$$T_{db_1} = \frac{A_{vg} \times f_y}{\sqrt{3} \times \gamma_{m0}} + \frac{0.9 \times A_{tn} f_u}{\gamma_{m1}} = 315.16 \text{ KN}.$$

$$T_{db_2} = \frac{0.9 A_{vn} f_u}{\sqrt{3} \gamma_{m1}} + \frac{A_{tg} f_y}{\gamma_{m0}} = 263.16 \text{ KN}.$$

$$A_{vg} = (3 \times 50 + 40) \times 8 = 1520 \text{ mm}^2$$

$$A_{vn} = \left(3 \times 50 + 40 - 3 \times 22 - \frac{22}{2}\right) \times 8 = 904 \text{ mm}^2$$

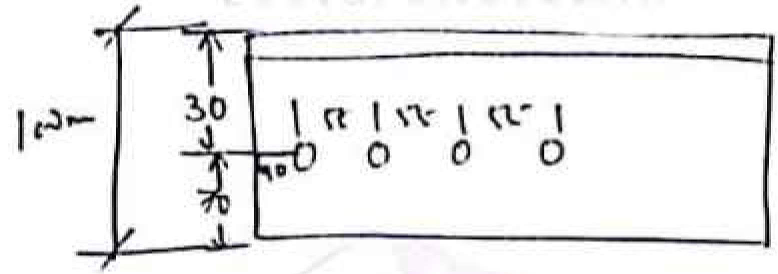
$$A_{tg} = (100 - 40) \times 8 = 480 \text{ mm}^2$$

$$A_{tn} = \cancel{(100-40)}\left(60 - \frac{22}{2}\right) \times 8 = 392 \text{ mm}^2.$$

$$\therefore T_{db} = 263.13 \text{ KN} < 300 \text{ KN}.$$

Since T_{db} is less than T, the member will fail in block shear.

The section can be made safe by increasing the distance of the bolt line from the toe and that by increasing the pitch as shown in fig.



$$A_{vg} = (3\times55+40)\times8 = \cancel{1024}\ 1640\ mm^2$$

$$A_{vn} = \left(3\times55+40-3\times22-\frac{22}{2}\right)\times8 = 1024\ mm^2$$

$$A_{tg} = (100-30)\times8 = 560\ mm^2$$

$$A_{tn} = \left(70-\frac{22}{2}\right)\times8 = 472\ mm^2$$

$$T_{db_1} = 354.52\ kN$$

$$T_{db_2} = 301.79\ kN$$

$$\therefore\ T_{db} = 301.79 > T$$

$\therefore$ So it is OK. (Ans).

Compression Member
(P- 34, section-7)

A compression member is a structural member which is straight and subjected to two equal and opposite compressive forces applied at its ends.

Ex: column, post, struct etc.

* **Effective length :-** (P-45, 7.2)

Effective length of a compression member is the product of effective length factor 'K' and the actual length 'L'.

Mathematically,

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

* The value of K depends upon the rotational and relative translational condition at the end of the member.

for K values → (P-45, T-11)

* **Slenderness ratio :- (λ)** ~~(P-36, T-3(a))~~

Slenderness ratio of a column is defined as the ratio of the effective length to corresponding radius of gyration of the secation.

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

L → actual length of compression member.

l → effective length.

r → radius of gyration.

~~Types of Cross-section:-~~

~~Tabulation~~

* The tendency of a member to buckle is usually measured by the slenderness ratio.

Maximum values of slenderness ratio

(P-20, T-3)

* Design of compression member

Step-1:-

Design stress in compression is to be assumed.

Step-2

Effective sectional area required is

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

Step-3

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

Step-4

Knowing the end conditions and deciding the type of connection determine effective length.

Step-5

Find the slenderness ratio and hence design stress f_{cd} and load carrying capacity P_d.

Step-6

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

Design of axially loaded compression Member :—

Assumptions

(i) The ideal column is assumed to be absolutely straight having no crookedness.

(ii) The modulus of elasticity is assumed to be const. in a built up column.

Step-1

A slenderness ratio is assumed w.r.t. to height of column.

Step-2

For the assumed value of the slenderness ratio in step-1, the design compressive stress for that value is determined from appropriate curve and buckling class.

Step-3

The cross sectional area required to carry the factored load at the assumed compressive stress is computed.

$$A_g = \frac{P}{\text{assumed Comp. stress.}}$$

Step-4

A section that provides the estimated required area is selected from steel book.

Step-5

The effective length of the column is calculated on the basis of end condition.

Step-6

For the estimated value of λ, the design comp. stress f_{cd} is calculated from Table of page 40, 41 - -

Step-7

For a single angle section loaded concentrically the design strength is determined by using formula

$$P_d = A_e \times f_{cd} \quad (P-34,\ 7.1.2)$$

and the design compressive stress using the formula and curve

$$f_{cd} = \frac{f_y/\gamma_{m0}}{\phi + \left[\phi^2 - \lambda^2\right]^{0.5}} \quad (P-34,\ 7.1.2.1)$$

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

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

$\alpha \rightarrow$ Table no. 7 (P-35).

However, a single angle when loaded through one of its leg is subjected to flexural torsional buckling. The equivalent slenderness ratio in such case

$$\lambda_e = \sqrt{K_1 + K_2 \lambda_{vv}^2 + K_3 \lambda_\phi^2} \quad (P-48,\ 7.5.1.2)$$

$(K_1, K_2, K_3 \rightarrow T-12)$ P-48

$$\lambda_{vv} = \frac{\frac{l}{r_{vv}}}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}} \quad \text{and} \quad \lambda_\phi = \frac{(b_1 + b_2)/2t}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}} \quad \left(P-48,\ 7.5.1.2\right)$$

Step-8

The design strength of the member is computed by formula $P_d = A_e \times f_{cd}$.

Q Calculate the design compressive load for a strut 350 @ 710.2 N/m, 3.5 m high. The column is restrained in direction and position at both the ends. ~~It is to be assumed~~ Use steel of grade Fe 410.

Sol^n:- For Fe 410 grade of steel

$f_u = 410\ N/mm^2$

$f_y = 250\ N/mm^2$

$\gamma_{m0} = 1.25$

from steel table ISHB 350 @ 710.2 N/m. (P-14)

$h = 350$ mm. $t_f = 11.6$ MM $t_w = 10.1$

$b_f = 250$ mm. $r_x = 146.5$ mm

$A = 9221\ mm^2$ $r_y = 52.2$ mm.

$\frac{h}{b_f} = \frac{350}{250} = 1.4 > 1.2$ (P-44, T-10).

t_f = 11.6 mm < 40 mm.

Buckling about axis z-z and buckling curve class is a.

Buckling " " y-y " " " " " b.

Design compressive stress from table 9 (a) and buckling about z-z axis

$$\lambda_z = \frac{KL}{r_z} = \frac{0.65 \times 3.5 \times 10^3}{146.5}$$

$$= 15.52.$$

From table 9 (a) (P-40)

λ		f_{cd} (for $f_y = 250$)
10	$\longrightarrow$	227
20	$\longrightarrow$	226

$$\cancel{\lambda(15.52)}\; f_{cd(15.52)} = 227 - \frac{227-226}{20-10} \times (15.52 - 10)$$

$$= 226.448 \ N/mm^2.$$

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

$$= 9221 \times 226.448$$

$$= 2088 \ kN$$

Design compressive stress from table 9 (b) and buckling about y-y axis

$$\lambda_y = \frac{KL}{r_y} = \frac{0.65 \times 3.5 \times 10^3}{52.2} = 43.5 \ mm.$$

from table 9(b). (P-42)

λ		f_{cd}
40	$\longrightarrow$	206
50	$\longrightarrow$	194

$$f_{cd}(43.5) = 206 - \frac{206 - 194}{50 - 40} \times (43.5 - 40)$$

$$= 201.8 \ N/mm^2$$

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

$= 9221 \times 201.8$

$= 1860.79 \ kN$ (Ans).

least of above

$\therefore$ The design compressive strength of the column is 1860.79 kN (Ans)

Otherwise

(i) About Z-Z axis and class (a).

$P_d = A \times (f_{cd})_z$. (P-34, 7.1.2.1)

$$f_{cd} = \frac{f_y / \gamma_{m0}}{\phi_z + (\phi_z^2 - \lambda_z^2)^{0.5}}$$

$$\lambda_z = \sqrt{f_y \left(\frac{KL}{r_z}\right)^2 / \pi^2 E}$$

$$= \sqrt{\frac{250 \times \left(\frac{0.65 \times 3.5 \times 10^3}{146.5}\right)^2}{\pi^2 \times 2 \times 10^5}} = 0.175$$

Note :- when $\lambda < 0.2$, then the magnitude of λ to be considered will be 0.2.

$\therefore \lambda_z = 0.2$ (58)

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

$$\alpha = 0.21 \quad (P-35, T-7).$$

$$\phi_z : 0.5\left[1+0.21(0.2-0.2)+0.2^2\right]$$

$$= 0.52.$$

$$f_{cd} = \frac{250/1.1}{0.52+(0.52^2-0.2^2)^{0.5}} = 227.2\ N/mm^2.$$

$$(P_d)_z = A_e \times (f_{cd})_z.$$

$$= 9221 \times 227.2$$

$$= 2095\ kN.$$

(ii) About Y-Y axis and class 9(b)

$$(P_d)_y = A_e \times (f_{cd})_y \quad = 9221 \times 202.11 = 1863.65\ kN.$$

$$f_{cd} = \frac{f_y/\gamma_{m0}}{\phi_y + (\phi_y^2 - \lambda_y^2)^{0.5}} = 202.11\ N/mm^2$$

$$\lambda_y = \sqrt{f_y\left(\frac{KL}{r_y}\right)^2/\pi^2 E} = 0.490.$$

$$\phi_y = 0.5\left(1+\alpha(\lambda_y - 0.2)+\lambda_y^2\right)$$

$$= 0.5\left(1+0.34(0.490-0.2)+0.490^2\right)$$

$$= 0.669.$$

$$\alpha = 0.34 \quad (P-35, T-7)$$

$\therefore P_d = 1863\ kN$ Ans.

Q. Design a single angle discontinuous strut to carry a factored axial compressive load of 65 kN. The length of strut is 3 m betn intersection. It is connected to 12 mm thick gusset plate by 20 mm dia 4.6 grade bolts. Use steel of grade Fe 410.

Soln:- For Fe 410,

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

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

For bolt of grade 4.6,

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

$d = 20\ mm$

$d_0 = 22\ mm$

Let's assume slenderness ratio $\lambda = 120$ and class 'c'.

From table 9(c)

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

Area required, $A = \frac{P}{f_{cd}}$

$= \frac{65 \times 10^3}{83.7} = 777\ mm^2.$

From steel table, let's provide ISA 70 x 70 x 6 mm.

Provided area = 806 mm² (P-4)

$r_{vv} = 13.6\ mm$

Considering both end fixed (P-45)

Effective length $l = K \times L$

$= 1 \times 3 = 3000\ mm.$

No. of bolts

(i) Shearing strength of bolt

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

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

$$= 45.25 \text{ KN}.$$

(ii) Bearing strength of bolt

$$V_{dpb} = \frac{2.5 \times K_b \times d \times t \times f_u}{\gamma_{m2}}$$

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

∴ Strength of bolt = 45.25 KN.

No. of bolts required for end connection

$$= \frac{65}{45.25} = 1.436 \approx 2 \text{ Nos}.$$

Provide 2, 20 mm dia bolts for making the end connections of the strut.

Considering the end fixity,

$K_1 = 0.2$, $K_2 = 0.35$, $K_3 = 20$. (P-48, T-12)

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

$$\lambda_{vv} = \frac{l/r_{vv}}{\epsilon\sqrt{\frac{\pi^2 E}{250}}} = \frac{3000/13.5}{1 \times \sqrt{\frac{\pi^2 \times 2 \times 10^5}{250}}} \qquad \epsilon = \sqrt{\frac{f_y}{250}} = 1$$

$$= 2.482.$$

$$\lambda_\phi = \frac{b_1 + b_2}{\varepsilon\sqrt{\frac{\pi^2 E}{250}} \times 2t} = \frac{70+70}{1\times\sqrt{\frac{\pi^2 \times 2\times10^5}{250}}\times(2\times6)} = 0.131$$

$$\lambda_e = \sqrt{0.2 + 0.35 \times 2.482^2 + 20 \times 0.131^2}$$

$$= 1.642$$

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

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

$$= 2.20$$

$$f_{cd} = \frac{f_y/\gamma_{m0}}{\phi + (\phi^2 - \lambda_e^2)^{0.5}} = \frac{250/1.1}{2.20 + (2.20^2 - 1.642^2)^{0.5}}$$

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

design comp. strength

$$P_d = A_e \times f_{cd} = 806 \times 62.02$$
$$= 49.98\ kN < 65\ kN.$$

So design is not OK.

Next, Let's provide ISA $70 \times 70 \times 8$ mm.

Provided area $= 1058\ mm^2$ (P-4)

$r_{vv} = 13.5$ mm.

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

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

$$= \frac{(70+70)/2\times 8}{\sqrt{\frac{\pi^2\times 2\times 10^5}{250}}} = 0.098$$

$$\lambda_e = \sqrt{0.2 + 0.35\times 2.5^2 + 20\times 0.098^2}$$

$$= 1.606$$

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

$$= 0.5\times\left[1+0.49(1.606-0.2)+1.606^2\right].$$

$$= 2.13$$

$$f_{cd} = \frac{f_y/\gamma_{m0}}{\phi+(\phi^2-\lambda_e^2)^{0.5}} = \frac{250/1.1}{2.13+(2.13^2-1.606^2)^{0.5}}$$

$$= 64.4 N/mm^2.$$

design compressive stress.

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

$$= 1058\times 64.4$$

$$= 68.13 kN > 65 kN.$$

So design is OK.

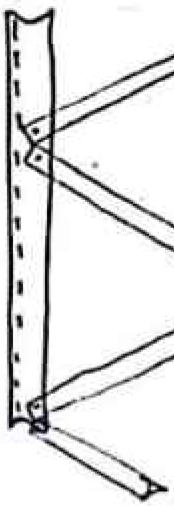
Laced & Battened Columns

To achieve maximum value for minimum radius of gyration, without increasing the area of the section, a no. of elements are placed away from the principal axis using suitable lateral system. The commonly used lateral systems are (a) lacing or latticing

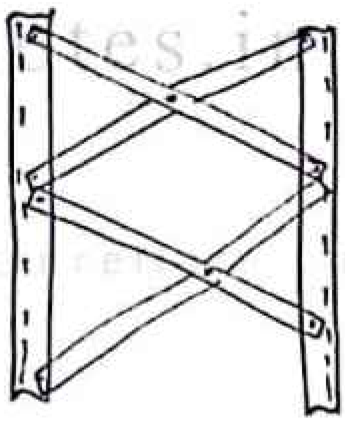
(b) battening.

Lacing:-

Rolled steel plate and angles are used for lacing. The object of providing lateral system is to keep the main members of the column away from principal ones.



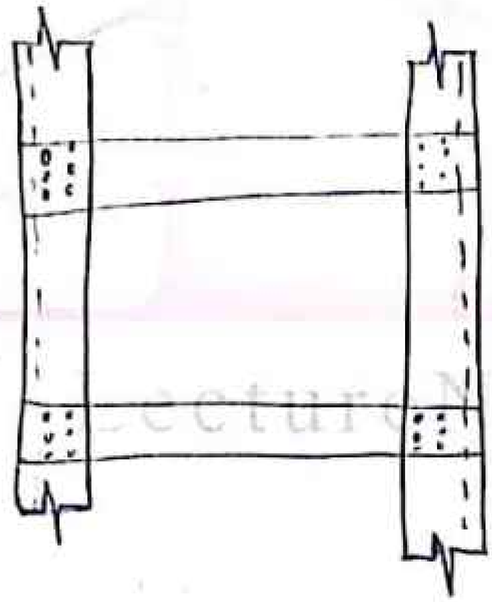
(fig: single laced system)



(fig: Double laced system)

Battens :-

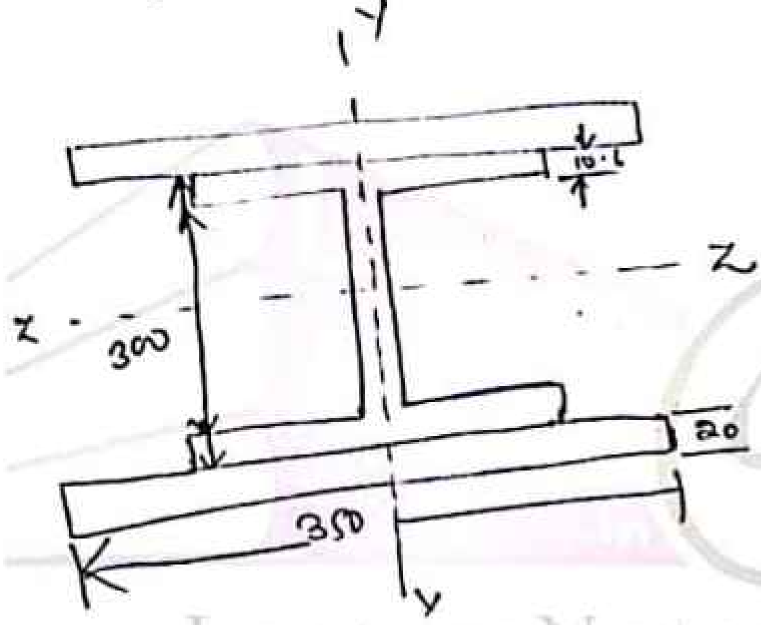
Instead of lacing one can use battens to keep members of columns at required distances.



(fig: Battened column)

Q. Calculate the compressive resistance of a compound column consisting of ISHB 300 with one cover plate of 350 x 20 mm on each flange and having a length of 5 m. Assume that the bottom of column is fixed and top is rotation fixed translational free.

Sol^n:-



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

$L = 5000 \; mm$

From steel table, for ISHB 300. (P-14)

$A = 7485 \; mm^2$

$I_{zz} = 12545.2 \times 10^4 \; mm^4$

$I_{yy} = 2193.6 \times 10^4 \; mm^4$

Total area of built up section.

$A = 7485 + 2 \times (350 \times 20)$

$= 21485 \; mm^2$.

$$I_{zz} = 12545.2 \times 10^4 + 2\left[\frac{350 \times 20^3}{12} + 350 \times 20 \times \left(\frac{350}{2} + \frac{20}{2}\right)^2\right]$$

$$60506.8 \times 10^4 \text{ mm}^4$$

$$= \cancel{49431.866 \times 10^4} \text{ mm}^4$$

$$I_{yy} = 2193.6 \times 10^4 + 2 \times \frac{20 \times 350^3}{12}$$

$$= 16485.26 \times 10^4 \text{ mm}^4.$$

Column will buckle about the axis of least moment of resistance or least radius of gyration.

$$\therefore r_y = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{16485.26 \times 10^4}{21485}}$$

$$= 87.59 \text{ mm}.$$

Calculation for design stress.

$$\lambda_y = \frac{KL}{r_y} = \frac{1.2 \times 5000}{87.59}$$

$$= 68.50.$$

From table 12

Column will buckle about class 'c'.

from 9(c)

λ	f_{cd}
60	168
70	152.

$$f_{cd}(68.50) = 168 - \frac{168 - 152}{70 - 60} \times (68.5 - 60)$$

$$= 154.4 \text{ N/mm}^2.$$

$$P_d = 21485 \times 154.4 = 3317.284 \text{ kN (Ans)}.$$

Design of column base

(P-46, 7.4)

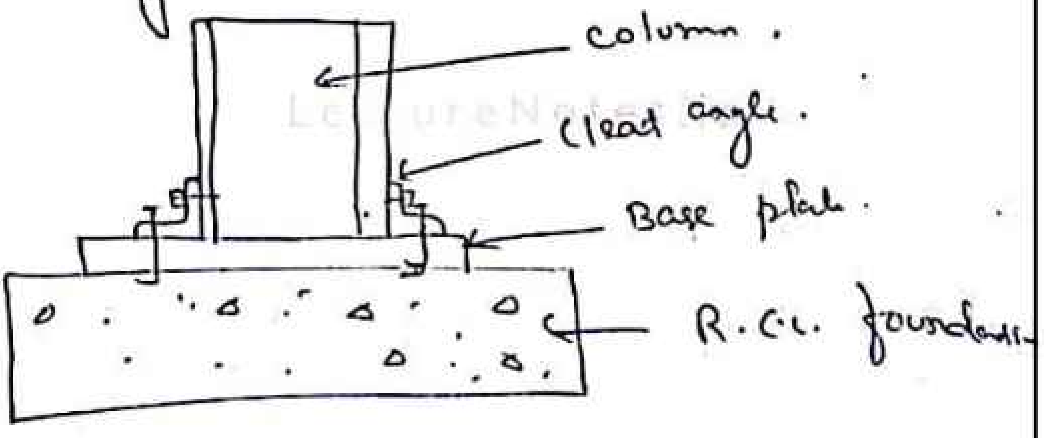
Column bases transmit the column load to the concrete or masonry foundation blocks. The column base spreads the load on wider area so that the intensity of bearing pressure on the foundation block is within the bearing strength. There are two types of column bases commonly used.

(i) Slab base

(ii) Gusseted base.

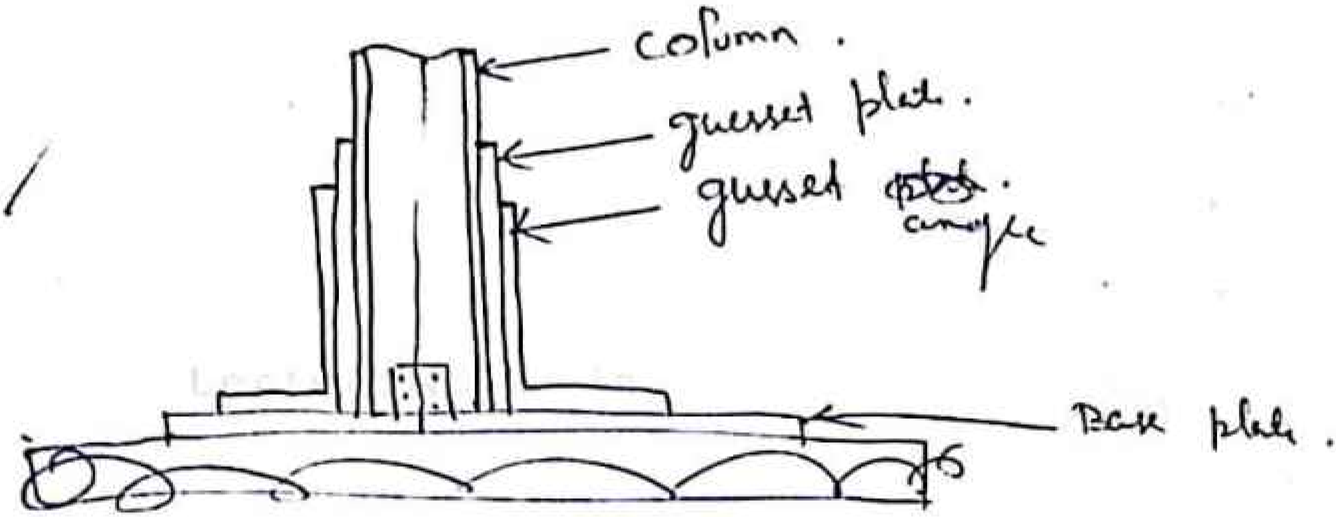
① **Slab base**

These are used in columns carrying small loads. In this type, the column is directly connected to the base plate through cleat angles. The load is transferred to the base plate through bearing.



② **Gusseted base :-**

For columns carrying heavy loads gusseted bases are used. In gusseted base, the column is connected to base plate through gussets. The load is transferred to the base partly through gussets.



Design of slab base

The design of slab base consists in finding the size and thickness of slab base.

Size of base slab

Step-1 Find the bearing strength of concrete which is given by $0.45 f_{ck}$.

Step-2 Therefore, area of base plate required

$$= \frac{P_u}{0.45 f_{ck}}$$, where P_u is factored load.

Step-3 Select the size of base plate. For economy as far as possible keep the projections a and b equal.

Thickness of base plate :-

Thickness of base plate

① Find intensity of pressure

$$w = \frac{P_u}{\text{Area of base plate}}$$

② Minimum thickness required is given by (P-47, 7.4.3.1)

$$t_s = \left[\frac{2.5w(a^2 - 0.3b^2)\gamma_{mo}}{f_y}\right]^{0.5} > t_f .$$

ts : thickness of base plate

t_f - thickness of flange.

Design of gusset base:—

① Area of base plate = $\frac{\text{Factored load}}{0.45 f_{ck}}$

② Assume various members of gusset base.

(a) Thickness of gusset plate is assumed as 16 mm.

(b) Size of the gusset angle is assumed such that its vertical leg is assumed in which one bolt can be provided.

(c) Thickness of angle is kept approximately equal to the thickness of gusset plate.

③ Width of gusset base is kept such that it will just project outside the gusset angle and hence.

$$\text{length} = \frac{\text{Area of plate}}{\text{width}}$$

④ When the end ~~load~~ of the column is machined for complete bearing on the base plate, 50% of load is assumed to be transferred by the bearing & 50% by the fastenings.

~~When the ends of the column shaft and gusset plates~~ are not faced for complete bearing, the fastenings connecting them to the base plate shall be designed to transmit all the forces to which the base is subjected.

⑤ The thickness of the base plate is computed by flexural strength at the critical sections.

Foundation bolts :—

Foundation bolts also known as anchor bolts are generally provided to check the uplift of the base plate. These bolts are either anchored into the foundation by a hook or by a washer plate or by some other appropriate load distributing member embedded in the concrete. A min^{m} two anchor bolts are to be provided even if the column is subjected to only axial load.

Q. Design a ~~the base~~ slab base, for a column ISHB 350 @ 710.2 N/m subjected to an factor axial compressive load of 1500 KN for the following conditions.

(a) Load is transferred to the base plate by direct bearing of column flange.

(b) Load is transferred to the base plate by welded connections; the column end and the base plate are not machined for bearing.

(c) whether anchor bolts are required?

The base rests on concrete pedestal of grade M20.

Sol^{n}:- For Fe 410 grade of steel,

$f_u = 410$ MPa.

$f_y = 250$ N/mm².

For M20 grade of concrete,

$f_{ck} = 20$ N/mm².

- Bearing strength of concrete

$= 0.45 \times f_{ck}$

$= 0.45 \times 20$

$= 9\ N/mm^2.$

$\gamma_{m0} = 1.1$

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

For ISHB 350 @ 710.2 N/m. (P-14)

$t_f = 11.6\ mm.$

$t_w = 10.1\ mm.$

$h = 350\ mm.$

$b = 250\ mm.$

Required area of slab base

$$A = \frac{1500 \times 10^3}{9} = 166666.67\ mm^2.$$

Let's provide a square base plate.

Side of the base plate $L = B = \sqrt{166666.67}$

$= 408.24\ mm$

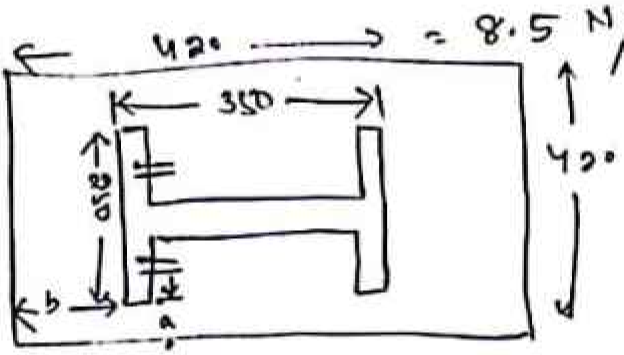
$\approx 420\ mm.$

Let's provide base plate of $420 \times 420\ mm$.

The bearing pressure of concrete

$$w = \frac{P}{A} = \frac{1500 \times 10^3}{420 \times 420}.$$

$= 8.5\ N/mm^2 < 9\ N/mm^2.$



The greater projection

$$a = \frac{420 - 250}{2} = 85 \text{ mm}.$$

The small projection

$$b = \frac{420 - 350}{2} = 35 \text{ mm}.$$

Thickness of slab base.

$$t_s = \sqrt{\frac{2.5 w \left(a^2 - 0.3 b^2\right) \gamma_{m0}}{f_y}} \quad (\text{P-46, } 7.4.3.1)$$

$$= \sqrt{\frac{2.5 \times 8.5 \left(85^2 - 0.3 \times 35^2\right) \times 1.1}{250}}$$

$$= 25.32 \simeq 28 \text{ mm} > t_f = 11.6 \text{ mm}.$$

∴ Provide a base plate 420 × 420 × 28 mm in size.

(a) The load is transferred to the base plate by direct bearing. So there is no bending moment ∴ connection of column with base need not be designed.

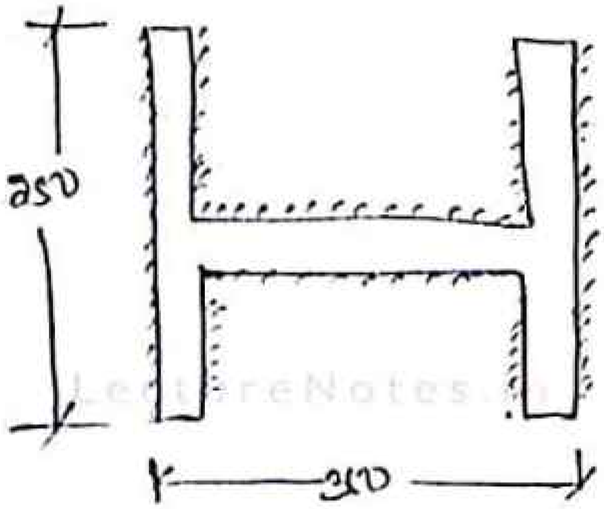
However to keep the column in position, two cleat angles of nominal size 60 × 60 × 8 mm may be provided connecting the column flanges with the base plate.

(b) Column end and base plate have not been machined for perfect bearing. Therefore, the load from the column will be transferred to the base plate through welded connection.

Length available for welding around column profile.

$$L_a = 2 \times 250 + 2 \times (250 - 10.1) + 2 \times (350 - 2 \times 11.6)$$

$$= 1633.4 \text{ mm}.$$



Let's provide 8 mm fillet weld. Since welding will not be possible at toes and fillets of the ISHB section, End returns (2s) will have to be subtracted at the end of each fillet weld length to get the effective length, that can be provided.

No. of total end returns = 12

$$\text{Effective length } (l_w) = 1633.4 - 12 \times (2 \times s)$$
$$= 1633.4 - 12 \times 16 \quad (\because s = 8\ mm).$$
$$= 1441.4\ mm.$$

Effective throat thickness $= 0.7 \times s$.
$$= 0.7 \times 8 = 5.6\ mm.$$

Strength of the fillet weld

$$P_{dw} = \frac{l_w \times t_t \times f_u}{\sqrt{3} \times \gamma_{mw}}$$

$$= \frac{1441.4 \times 5.6 \times 410}{\sqrt{3} \times 1.25} = 1528\ kN > 1500\ kN.$$

∴ design is OK.

Since the base is subjected to only axial compressive load and there is no BM, the base is not subjected to tension in any of its part. Therefore, provide nominal 20 mm dia bolts, 2 in no. to keep the base in position.

Q. A column ISHB 350 @ 661.2 N/m carries an axial comp. factored load of 1700 kN. Design a suitable bolted gusset base. The base rests on M15 grade concrete pedestal. Use 24 mm dia bolts of grade 4.6 for making the connections.

Solⁿ:- For Fe 410 grade of steel,

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

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

$f_{ck} = 15\ N/mm^2$

Bearing strength of concrete $= 0.45 \times f_{ck}$

$= 0.45 \times 15$

$= 6.75\ N/mm^2$.

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

For ISHB 350 @ 661.2 N/m.

$t_f = 11.6$ mm

$t_w = 8.3$ mm.

$h = 350$ mm.

$b = 250$ mm.

For 4.6 grade of bolt

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

$d = 24$ mm.

$d_0 = 26$ mm.

Assuming pitch $(P) = 2.5 \times d$

$= 60$ mm.

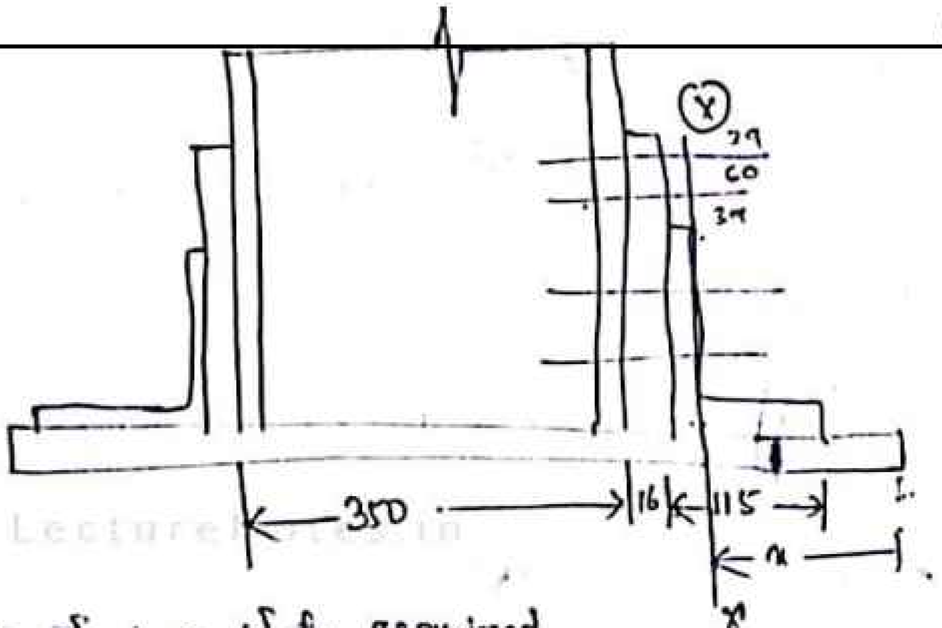
edge distance $(e) = 1.5\, d_0 = 39$ mm.

factored load $P = 1700$ kN.

Required area of base plate $A = \dfrac{1700 \times 10^3}{6.75} = 251.85 \times 10^3\ mm^2$

Let's provide 16 mm thick gusset plates on the two flanges of column section and two gusset angles.

ISA 150 × 115 × 15 mm.



Min^m width of base plate required

$= 350 + 2\times16 + 2\times115 = 612$ mm.

≈ 620 mm.

Projection of base plate beyond flange angle toe

$= \frac{620 - 612}{2} = 4$ mm.

Length of base plate $= \frac{A}{B}$

$= \frac{251.85 \times 10^3}{620} = 406.2$ mm

$\simeq 410$ mm.

Let's provide a base plate 620 × 410 mm in size.

Bearing pressure of concrete $w = \frac{P}{A}$.

$= \frac{1700 \times 10^3}{620 \times 410} = 6.68\ N/mm^2$

$< 6.75\ N/mm^2$

Thickness of base plate :-

Let 't_s' be the thickness of base plate.

The critical section of the base for bolted gusset base will be at section X-X as shown in fig.

The length of base plate at critical section

$(x) = 115 + 4 - 15 = 104$ mm.

Let t The combined thickness of base plate and gusset angle at critical Section.

Max^n moment $M_x = \frac{w \times x^2}{2}$

$= \frac{6.68 \times 104^2}{2} = 36125.44$ Nmm. —(1)

Assuming simply supported.

$M_d = \frac{1.2 \times Z_e f_y}{\gamma_{m0}}$ (P-53, 8.2.1.2)

$= 1.2 \times \frac{250}{1.1} \times \left(\frac{1 \times t^2}{6}\right)$

$= 45.45\, t^2$ —(2)

Equating (1) & (2)

$45.45\, t^2 = 36125.44$

$\Rightarrow t = 28.19$ mm.

thickness of base plate $t_s = t - 15$

$= 28.19 - 15$

$= 13.19$

≈ 16 mm $> t_f = 11.6$ mm.

Let's provide a base plate of 620x410x16 mm.

<u>Bolted connection</u> :-

Connection bet^n gusset plate & flange, each bolt is in single shear and bearing.

$V_{dsb} = \frac{f_{ub}}{\sqrt{3}}(n_n A_{nb} + n_s A_{sb}) = 65.21$ kN.

$V_{dpb} = \frac{2.5 k_b \times d \times t \times f_u}{\gamma_{mb}} = \frac{2.5 \times 0.541 \times 24 \times 11.6 \times 410}{1.25} = 123$ kN.

K_b least of the following

i) $\frac{e}{3d_0} = 0.541$ iii) $\frac{f_{ub}}{f_u} = 0.975$

ii) $\frac{p}{3d_0} - 0.25 = 0.58$ iv) $= 1$ $\therefore K_b = 0.541$

∴ Strength of bolt = 65.21 kN.

Assuming column end and gusset material to have complete bearing, 50% of the load will be assumed to pass directly and 50% of the load will pass through the connection.

No. of bolts required to connect the column flanges with gusset plates

$$= \frac{0.5 \times 1700}{65.21} = 13.03 \approx 16\ \text{nos}.$$

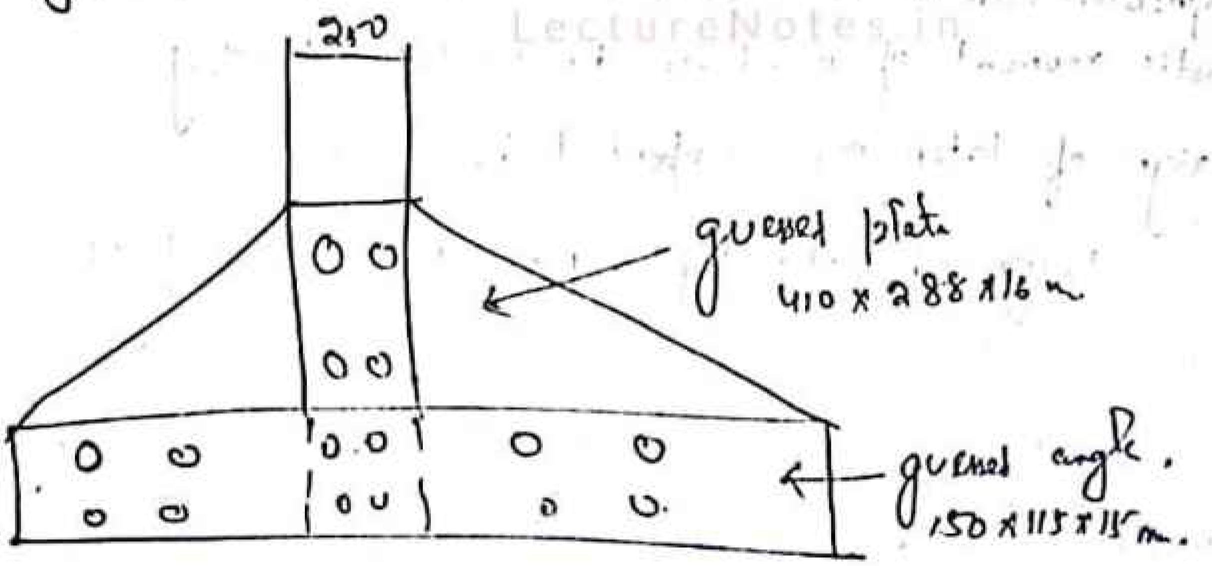
Let's provide 24 mm dia bolt on each flange in two rows.

The no. of bolts required to connect the gusset angle with gusset plate will be same.

Height of gusset plate = 150 + 2x39 + 1x60 = 288 mm.

length of gusset plate = length of base plate = 410 mm.

Provide gusset plate 410x288x16 mm in size.



Beam :-

A structural member subjected to transverse loads is called a beam. When provided in buildings to support roofs, they are called joists.
- A large beam supporting a number of joists is called a girder.

Classification Types of C/S :-

(1) **Class 1 (Plastic) C/S :-**

These sections can develop plastic hinges and have the rotation capacity required for failure of the structure by formation of plastic mechanisim.

(2) **Class 2 (compact) C/S :-**

Such sections can develop plastic moment of resistance, but have inadequate plastic hinge rotation capacity for formation of plastic mechanism, due to local buckling.

(3) **Class 3 (Semi compact) C/S :-**

These are the sections in which the extreme fibre in compression can reach yield stress, but cannot develop the plastic moment of resistance, due to local buckling.

Design of laterally supported beam :-

Design of laterally supported beam consist of selecting a section on the basis of modulus of section and checking it for shear capacity, hig/low shear, web buckling, web creeping and deflection etc.

Procedure for :

(1) The service load expected on the beam are as certain, the service load are multiplied with the load factor γ_{mf} (1.5) Determine the factored load.

(2) The max^m bending moment 'M', Max^m shear force 'V' calculated for the beam. These forces are referred to as design forces.

(3) A trial plastic section modulus for the beam is worked out.

$$Z_p = \frac{M_f \times \gamma_{m0}}{\beta_b \times f_y} \quad (P\text{-}53,\ 8.2.1.2)$$

(4) Looking at the value of plastic section modulus a suitable section having plastic section modulus more than required is selected. ISLB, ISMB, ISWB section are preferred.

~~(5) ISLBs~~

(5) Classification of section is checked. from (T-2, P-18).

(6) checked for web buckling. (P-53, 8.2.1.1)

(7) The trial section is check for shear capacity.

$$V_d = \frac{A_v \times f_{yw}}{\sqrt{3} \times \gamma_{m0}} \quad (P\text{-}59,\ 8.4)$$

or ~~V_p~~

(8) The trial section is check for designed bending strength.

$$M_d = \frac{\beta_b \times Z_p \times f_y}{\gamma_{m0}} \quad (P\text{-}53,\ 8.2.1.2)$$

(9). The trial section is check for ~~buckling~~ bearing.

$$F_w = \frac{(b_1 + n_2)\, t_w \cdot f_{yw}}{\gamma_{m0}} \quad (P\text{-}67,\ 8.7.4).$$

(9) check for high / low shear.

$V < 0.6 V_d$ (low shear) P-53, 8.2.1.2).

$V > 0.6 V_d$ (high shear)

(10) Check for deflection.

permissible defl^n (P-31, T-6)

Design for latterally unsupported beam :-

Procedure :-

(1) The service load expected on the beam are as certain, the service load are multiplied with the load factor 'γ_{mf}' (1.5). determine the factored load.

(2) The max^m bending moment 'M', Max^m shear force 'V' calculated for the beam. These forces are referred to as design forces.

(3) A trial plastic section modulus for the beam is worked out.

(4) Looking at the value of plastic section modulus a suitable section having plastic section modulus more than required is selected.

(5) classification of section is checked. (P-18, T-2)

(6) Check for design bending strength.

$$M_{cr} = \sqrt{\left\{\left(\frac{\pi^2 E I_y}{(L_{LT})^2}\right)\left(G I_t + \frac{\pi^2 E I_w}{(L_{LT})^2}\right)\right\}} \quad \text{(P.54 8.2.2.1)}$$

$$I_t = \sum_{i=1} \frac{b_i t_i^3}{3} \quad \text{(P-129)}$$

$$I_w = (1-\beta_f)\,\beta_f\, I_y\, h_y^2 \quad \text{(P-129)}.$$

For safe design

$$M_d = \beta_b \times Z_p \times f_{bd} > M_{max} \quad (P - 54, 8.2.2)$$

$$f_{bd} = \frac{\chi_{LT} f_y}{\gamma_{m0}}$$

$$\chi_{LT} = \frac{1}{\left\{\phi_{LT} + \left(\phi_{LT}^2 - \lambda_{LT}^2\right)^{0.5}\right\}} \leq 1$$

$$\phi_{LT} = 0.5\left[1 + \alpha_{LT}\left(\lambda_{LT} - 0.2\right) + \lambda_{LT}^2\right].$$

$$\lambda_{LT} = \sqrt{\frac{\beta_b \times Z_p \times f_y}{M_{cr}}}$$

Q. A simply supported steel joist of 4m effective span is laterally supported throughout. It carries a total uniformly distributed load of 40kN (including self wt.). Design an appropriate section using steel of grade Fe 410.

Solⁿ:- For Fe 410 grade of steel,

$f_u = 410\ N/mm^2$ $\quad \gamma_{m0} = 1.1$

$f_y = 250\ N/mm^2$ $\quad \gamma_{mf} = 1.5$

To Service load = $W = wl = 40\ kN$.

Factored load $= 1.5 \times 40$

$(W_u) = 60\ kN = w_u \times l$

Max^m bending moment $= M = \frac{wl^2}{8}$

$$= \frac{w_u l \times l}{8} = \frac{60 \times 4}{8} = 30\ kNm.$$

Max^m shear force = $\frac{W_u l}{2}$

$= \frac{60}{2} = 30$ kN.

Plastic section modulus required

Z_p (required) $= \frac{M \times \gamma_{m0}}{\beta_b \times f_y}$ (P-53, 8.2.1.2).

$= \frac{30 \times 1.1}{1 \times 250} = 132 \times 10^3$ mm^3

Let's select ISLB 200 @ 194.2 N/m (T-46, P-138 - - .1)

$t_f = 7.3$ mm. $Z_{pz} = 184.34 \times 10^3$ mm^3

$t_w = 5.4$ mm. $Z_{ez} = 169.70 \times 10^3$ mm^3.

$h = 200$ mm. $I_{xx} = 1696.66 \times 10^4$ mm^4 (Steel table).

$b_f = 100$ mm.

$R_1 = 9.5$ mm. (From steel table (P-12, 13))

Depth of web $= d = h - 2(t_f + R_1)$

$= 200 - 2(7.3 + 9.5)$

$= 166.4$ mm.

<u>Classification of section</u> (P-18)

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

Outstand of flange $= \frac{b_f}{2} = b = \frac{100}{2} = 50$ mm.

$$\frac{b}{t_f} = \frac{50}{7.3} = 6.85 < 9.4$$

$$\frac{d}{t_w} = \frac{166.4}{5.4} = 30.81 < 84$$

$\therefore$ The section is plastic.

Check for web buckling :- (P-53, 8.2.1.1)

Since $\frac{d}{t_w} = 30.51 < 67$

Since $\frac{d}{t_w} < 67\varepsilon$, shear buckling check of web will not be required.

Check for shear capacity :- (P-59, 8.4)

Factored force = 30 kN.

$V_d = \frac{A_v f_y}{\sqrt{3} \times \gamma_{m0}} = \frac{200 \times 5.4 \times 250}{\sqrt{3} \times 1.1} = 141.713 \text{ kN}$.

$A_v = h \times t_w$ for hot rolled section (8.4.1.1)

$= 200 \times 5.4$

$V_d > V$. So design is OK.

Check for designed bending strength

$M_d = \frac{\beta_b \times Z_p \times f_y}{\gamma_{m0}}$ (P-53, 8.2.1.2)

$= \frac{1 \times 184.34 \times 10^3 \times 250}{1.1} = 41.895 \text{ kNm}$

$M_d \leq 1.2 Z_e \frac{f_y}{\gamma_{m0}} = \frac{1.2 \times 169.7 \times 10^3 \times 250}{1.1} = 46 \text{ kNm} > 30 \text{ kNm}$

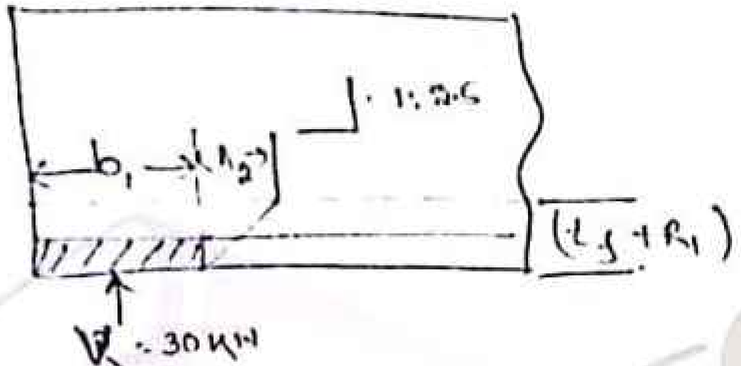
$M_d > M$.

So design is OK.

Check for web bearing :- P-61, 8.7.4)

$$F_w = \frac{(b_1 + n_2)\, t_w f_{yw}}{\gamma_{m0}} = 143.59\,kN > 30$$

~~Assuming stiff bearing~~



Assuming stiff bearing length $b_1 = 75$ mm.

n_1 = length obtained by dispersion.

$= 2.5(t_f + R_1) = 2.5(7.3 + 9.5)$

$= 42$ mm.

$F_w > F$

So design is OK.

Check for high/low shear (P-53, 8.2.1.2)

$0.6 V_d = 0.6 \times 141.73$

$= 85.02$ kN.

Since $V < 0.6 V_d$.

It is a low shear.

Check for deflection. (P-31, T-6).

Permissible deflection $= \frac{\text{Span}}{300} = \frac{4\times10^3}{300}$

$= 13.33$ mm.

Max^m deflection $= \frac{5}{384} \times \frac{Wl^4}{EI}$

$= \frac{5}{384} \times \frac{(Wl)\times l^3}{EI}$

$= \frac{5}{384} \times \frac{40\times10^3 \times (4\times10^3)^3}{2\times10^5 \times 1696.66\times10^4}$

$= 9.82 < 13.33$ mm.

So design is OK.

Q. Design a laterally unsupported beam for the following data.

Effective span = 4m

Max bending moment = 550 kNm

Max^m shear force = 200 kN

$G = 76.923\times10^3$ N/mm2 (Shear modulus)

steel of grade: Fe 410.

Sol^n:- For Fe 410 grade of steel,

$f_y = 250$ N/mm2

$f_u = 410$ N/mm2.

$\gamma_{m0} = 1.1$

$\gamma_{mf} = 1.5$

Max^m BM = 550 kN.

Plastic section modulus required.

$$Z_{px} \text{ (required)} = \frac{1.3 \times M_d \times \gamma_{m0}}{f_y}$$

$$= \frac{1.3 \times 550 \times 10^3 \times 1.1}{250}$$

$$= 3146 \times 10^3 \text{ mm}^3$$

Let's select ISMB 600 @ 1200.71 N/m. (P-138, T-46).

$h = 600$ mm
$b_f = 210$ mm
$t_f = 20.8$ mm
$t_w = 12$ mm.

$Z_{px} = 3510.63 \times 10^3 \text{ mm}^3$.
$Z_{ex} = 3060.4 \times 10^3 \text{ mm}^3$.

From steel table (P-14)

$R_1 = 20$ mm.
$I_z = 91813 \times 10^4 \text{ mm}^4$
$I_y = 2651 \times 10^4 \text{ mm}^4$

$d = h - 2(t_f + R_1)$
$= 600 - (20.8 + 20)$
$= 518.4$ mm

__Section classification__

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

outstand flange $b = \frac{b_f}{2} = \frac{210}{2} = 105$ mm.

$$\frac{b}{t_f} = 5.04 < 9.4$$

$$\frac{d}{t_w} = 43.2 < 84.$$

$\therefore$ section is plastic.

__Check for design bending strength__

$$M_{cr} = \sqrt{\frac{\pi^2 E I_y}{(L_{LT})^2}\left(G I_t + \frac{\pi^2 E I_w}{(L_{LT})^2}\right)}$$

- $E = 2\times10^5\ N/mm^2$

$G = 76.923\times10^3\ N/mm^2$ (shear modulus).

$L_{LT} = 4000\ mm$.

$I_t = \Sigma \dfrac{b_i t_i^3}{3}$ (P-129).

$= 2\cdot\dfrac{210\times20.8^3}{3} + \dfrac{(600-2\times20.8)\times12^3}{3}$

$= 1.58\times10^6\ mm^4$

$I_w = (1-\beta_f)(\beta_f I_y h_y^2) = (1-0.5)\,0.5\times2651\times10^4\times579.2^2$

$= 2.22\times10^{12}\ mm^6$.

$\beta_f = \dfrac{I_{fc}}{I_{fc}+I_{ft}} = 0.5$

Assuming $I_{fc} = I_{ft}$

h_y = distance betn shear senter of two flanges of the c/s.

$= 600 - \dfrac{20.8}{2} - \dfrac{20.8}{2} = 579.2\ mm$.

$$M_{cr} = \sqrt{\frac{\pi^2\times2\times10^5\times2651\times10^4}{4000^2}\times\left(76.923\times10^3\times1.59\times10^6 + \frac{\pi^2\times2\times10^5\times2.22\times10^{12}}{4000^2}\right)}$$

$= 1138.3\ kNm$.

Design bending Moment:

$M_d = \beta_b \times Z_p \times f_{bd}$. (P-54, 8.2.2) $= 597\ kNm$. $> 550\ kNm$.

$f_{bd} = \dfrac{\chi_{LT} f_y}{\gamma_{m0}} = 170.06\ N/mm^2$.

$$\chi_{LT} = \frac{1}{\phi_{LT} + \{(\phi_{LT})^2 - (\lambda_{LT})^2\}^{0.5}} = 0.7483$$

$$\phi_{LT} = 0.5\left[1 + \alpha\left(\lambda_{LT} - 0.2\right) + \lambda_{LT}^2\right] = 0.9566$$

$(\alpha = 0.21)$

$$\lambda_{LT} = \sqrt{\frac{\beta_b \times Z_p \times f_y}{M_{cr}}} = \frac{1 \times 3570.63 \times 10^3 \times 250}{1183.3 \times 10^6}$$

$= 0.878$

$M_d > M$.

So design is OK.

Check for shear capacity :—

Design shear force $V = 200$ KN.

Design shear strength of the section

$$V_d = \frac{f_y A_v}{\sqrt{3}\,\gamma_{m0}}$$

$$= \frac{h \times t_w \times f_y}{\sqrt{3} \times \gamma_{m0}} = \frac{600 \times 12 \times 250}{\sqrt{3} \times 1.1} = 944.75 \text{ KN}$$

$V_d > V$. So design is OK.

Check for web buckling

$\frac{d}{t_w} < 67\,\epsilon$. check for web buckling is not required.

If $\frac{d}{t_w} > 67\varepsilon$.

we have to check capacity of section.

capacity of section = $A_b \times f_{cd} > V$.

$A_b = (b + n) \times t_w = (100 + 300) \times 12 = 4800 \text{ mm}^2$.

b = bearing length = 100 mm (assuming).

$n = \frac{h}{2} = \frac{600}{2} = 300$ mm.

$\lambda = \frac{l_e}{r} = \frac{362.88}{3.46} = 104.88$.

l_e : effective length of web = 0.7 d

= 0.7×518.4

= 362.88 mm.

$r = \sqrt{\frac{I_{eff}}{A_{eff}}} = \sqrt{\frac{14400}{1200}} = 3.46$

I_{eff} of web $= \frac{100 \times 12^3}{12} = 14400 \text{ mm}^4$

A_{eff} of web $= 100 \times 12 = 1200 \text{ mm}^2$

Assuming class 'c', for $\lambda = 104.88$, from 9 (c)

λ	f_{cd}
100	107
110	94.6

$f_{cd(104.88)} = 107 - \frac{107 - 94.6}{110 - 100}(104.88 - 100)$

$= 100.94 \text{ N/mm}^2$.

capacity of section $= 4800 \times 100.94$

$= 484.56 \text{ KN} > 200 \text{ KN}$.

So design is OK.

Check for web bearing

$$F_w = \frac{(b + n_1) t_w f_{yw}}{\gamma_{m0}}.$$

$b = 100$ mm.

$n_1 = 2.5(t_f + R_1) = 102$ mm.

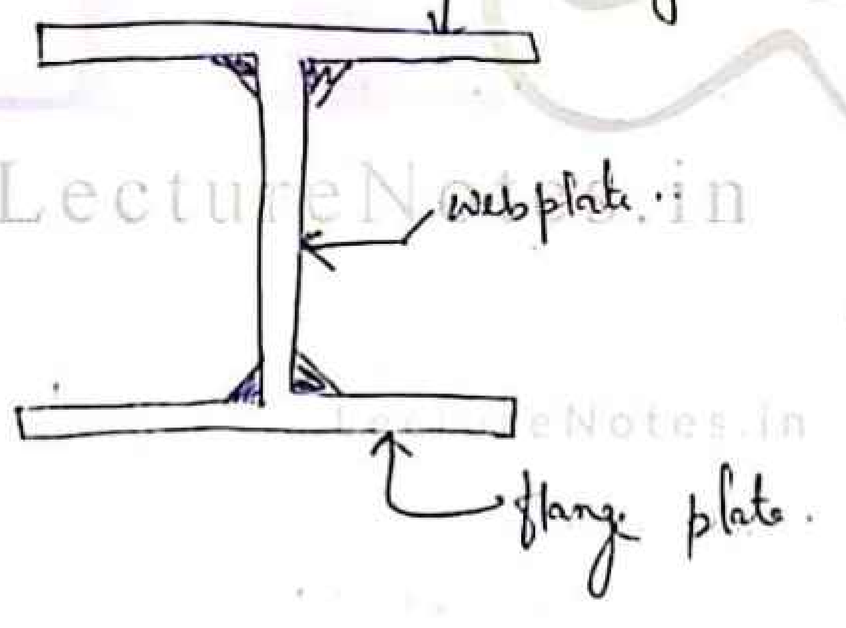
$F_w = 550$ KN $> V$.

So design is OK.

Plate Girder

When span and load increases, the available rolled section may not sufficient, even after strengthening with cover plates, ~~Such situations are common in the following :-~~

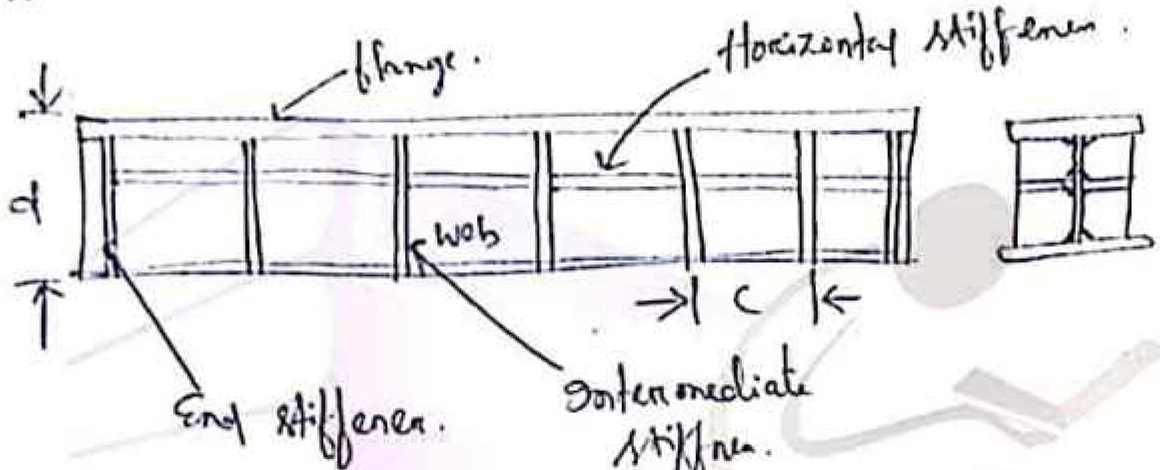
~~① Larger~~ In such situation one of the remedies is to go for a built up I-section with two flanges plates connected to a web plate of required depth. The depth of such I-beams may vary from 1.5 m to 5 m. This type of I-beams are known as plate girder.



* Elements of plate girders.

Following are the elements of a typical girder.

(1) web
(2) Flanges.
(3) Stiffeners.



(1) web

webs of required depth and thickness are provided to.

(a) keep flange plates at required distances.

(b) resist the shear in the beam.

(2) Flange:-

Flanges of required width and thickness are provided to resist bending moment acting on the beam by developing compressive force in one flange and tensile force in another flange.

(3) Stiffeners:-

Stiffeners are provided to safeguard the web against local buckling failure. The stiffeners provided may be classified as

(a) Transverse (vertical) stiffeners,

(b) Longitudinal (horizontal) " .

(a) Transverse stiffeners are of two types.

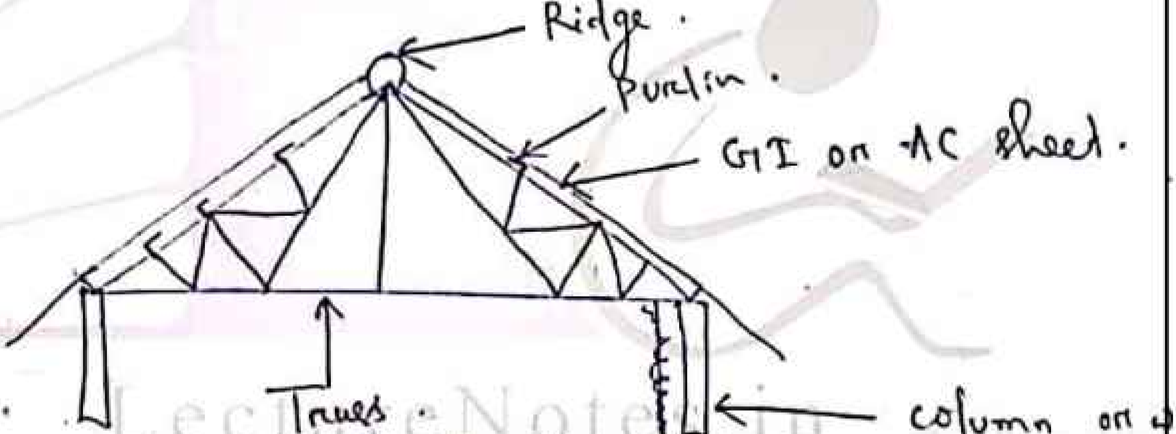
(i) Bearing stiffener.

(ii) Intermediate " .

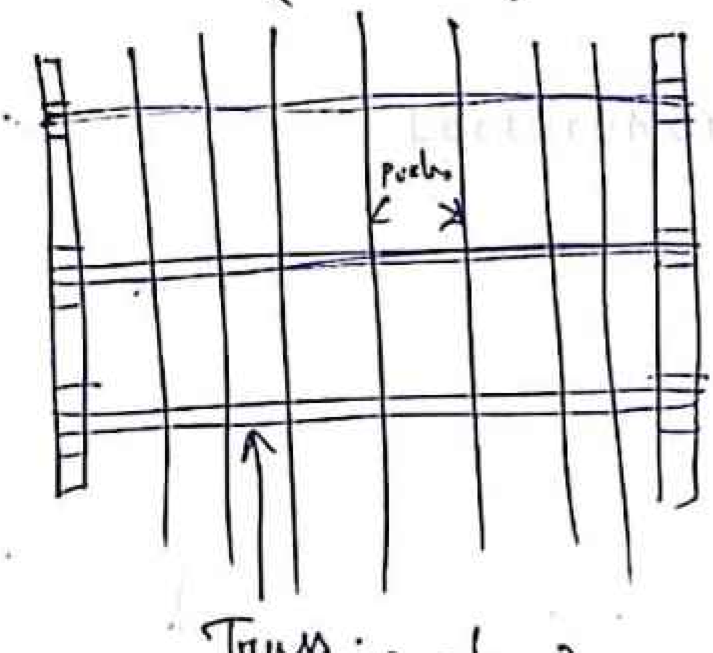
End bearing stiffeners are provided to transfer the load from beam to the support. At the end certain portion of web of beam acts as a compression member and hence there is possibility of crushing of web. Hence web needs stiffeners to transfer the load to the support. If concentrated loads are acting on the plate girder, intermediate bearing stiffeners are required.

Roof trusses

Large column free areas are required for auditorium, assembly halls, workshops etc. To get such column free area one of the commonly used roofing system is to provide a set of steel roof trusses, interconnected with purlins which in turn support GI (Galvanised Iron) or AC (Asbestos Cement) sheets. The roof trusses are supported on walls or a series of columns.

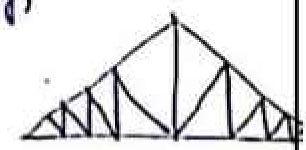


Truss.
(Elevation)



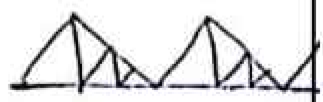
Truss. (plan)

(g)



Belgian truss

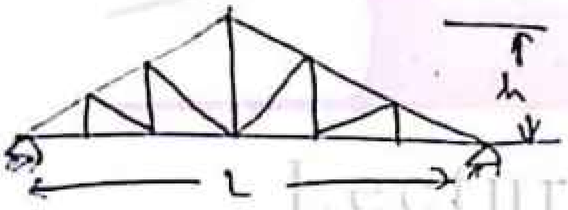
(h)



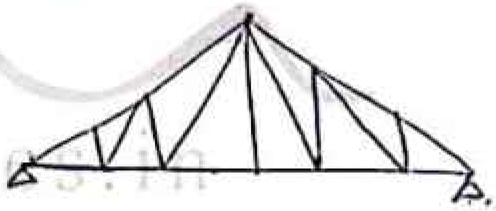
saw tooth or north light roof truss.

Types of roof trusses :-

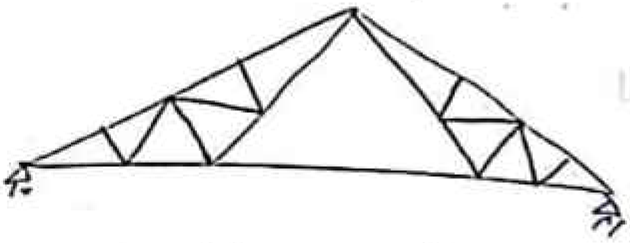
* On the basis of structural behaviour, roof trusses can be classified as simple roof trusses supported over masonry / concrete walls / columns and steel columns.
* A roof truss may also be classed a plane truss or space truss.
* In a plane truss the external loads and the component members lie in the same plane. Whereas in a space truss the component members are oriented in 3 dimension in space and loads may also act in any direction.



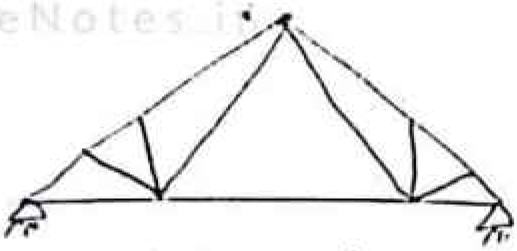
(a) Pratt truss.



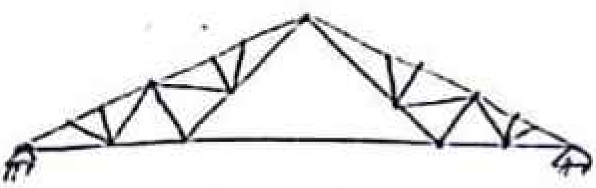
(b) Howe truss.



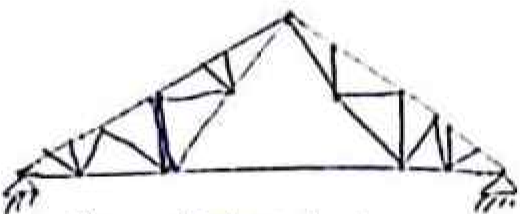
(c) Flink or French truss.



(d) Fan truss.



(e) Fink fan truss.



(b) compound fan truss.