Semester- V

Sr. No	Subject Code	Subject	Contact Hours			Credit
			L	T	P	
		Theory				
01	CV 501	Design of Steel Structures	2	2	-	4
02	CV 502	Structural Mechanics-II	2	1	-	3
03	CV 503	Soil Mechanics	3	1	✓	4
04	CV 504	Environmental Engineering	2	-	✓	2
05	CV 505	Transportation Engineering	3	-	✓	3
06	CV E2	**Elective II**	3	-	-	3
		Practical / Drawing and/or Design				
07	CVL 501	Soil Mechanics Laboratory	-	-	2	1
08	CVL 502	Environmental Engineering Laboratory	-	-	2	1
09	CVL 503	Transportation Engineering Laboratory	-	-	2	1
10	CVL 504	Seminar on Topic of Field Visit to works related to Building Services	-	-	1	AU
		Sub-Total	**15**	**4**	**7**	
		Total	**26**			**22**
		Elective II				
	CVE2-501 CVE2-502 CVE2-503	Civil Engineering Materials Computer Aided Drawing Development Engineering	3	-		3

BTCVC 501Design of Steel Structures

Teaching Scheme:(2 Lectures + 2 Tutorial) hours/week

Course Contents

Note: Contents in Module 1 to part of 6 are to be taught with help of IS 800: 1984 and other relevant text or reference books.

Module 1: Introduction and Connections **(8 Lectures)**

Introduction, advantages & disadvantages of steel structures, permissible stresses, factor of safety, methods of design, types of connections, various types of standard rolled sections, types of loads and load combinations

Types: Riveted, Bolted, Welded; Analysis of axially & eccentrically loaded connections (subjected to bending & torsion), Permissible Stresses, Design of connections, failure of joints

Module 2: Axially Loaded Members **(6 Lectures)**

Tension members: Common sections, net effective area, load capacity, connection using weld / bolts, design of tension splice

Compression members: Common sections used, effective length and slenderness ratio, permissible stresses, load carrying capacity, connection using weld / bolt

Module 3: Beams **(6 Lectures)**

Laterally supported & unsupported beams, design of simple beams, built up beams using flange plates, curtailment of flange plates, web buckling & web crippling, secondary and main beam arrangement, beam to beam connections

Module 4: Industrial Roofing **(6 Lectures)**

Gantry girder: Forces acting on a gantry girder, commonly used sections, design of gantry girder as laterally unsupported beam, connection details

Roof trusses: Components of an industrial shed, types of trusses, load calculations and combinations, design of purlins, design of truss members, design of hinge & roller supports

Module 5: Columns and Column Bases **(8 Lectures)**

Simple and built up section, lacing, battening, column subjected to axial force and bending moment, column splices.

Column bases: Analysis and design of: Slab base, gusseted base and moment resisting bases, grillage foundation, design of anchor bolt

Module 6:(4 Lectures)

Introduction to: Plastic Analysis, Hinge Formation, Collapse Mechanism, Recent approaches in Steel Structure design based on Plastic Analysis Method and Limit State Approach, Introduction to Provisions in IS 800-2007

Note: Use of IS 800: 1984 and 2007, IS 875 (All Parts), IS: Handbook No.1 for Steel Section and Steel Table is permitted for theory examination.

Text Books

- Duggal S. K., "Design of Steel Structures", Tata McGraw Hill Pub. Co. Ltd., New Delhi
- Duggal S. K., "Limit State Design of Steel Structures", Tata McGraw Hill Pub. Co. Ltd., New Delhi
- Gambhir, "Fundamentals of Structural Steel Design", Tata McGraw Hill Pub. Co. Ltd., New Delhi
- Negi L. S., "Design of Steel Structures", Tata McGraw Hill Pub. Co. Ltd., New Delhi
- Chandra Ram, "Design of Steel Structures", Vol. I & Vol. II, Standard Book House, New Delhi
- Dayaratnam P., "Design of Steel Structures", Wheeler Publishing, New Delhi
- Subramanian N., "Steel Structures: Design and Practice" Oxford Univ. Press, Delhi
- Vazirani V.N. and Ratwani M.M., "Design and Analysis of Steel Structures", ISBN NO: 978-81-7409-295-3
- Sai Ram K. S., "Design of Steel Structures", Pearson Education, 2nd Edition

Reference Books

- Arya A. S. and Ajamani J.L., "Design of Steel Structures", Nemchand and Brothers, Roorkee
- Vazirani & Ratwani, "Design of Steel Structures", Standard Book House, New Delhi
- Publications of Bureau of Indian Standards, New Delhi, IS 800:1984, 2007, IS 875 (Part I to V)
- Gaylord E.H. and Gaylord C.N., "Design of Steel Structures" McGraw Hill, New York
- Lothers J.E., "Design in Structural Steel" Vol.-I, Prentice Hall New Jersy
- Salmon and Johnson, "Steel Structures: Design and Behaviour", Harper and Row, New York

Introduction

A Civil Engineering Designer has to ensure that the structures and facilities he designs are (i) fit for their purpose (ii) safe and (iii) economical and durable. Thus safety is one of the paramount responsibilities of the designer. However, it is difficult to assess at the design stage how safe a proposed design will actually be – consistent with economy. There is, in fact, a great deal of uncertainty about the many factors, which influence both safety and economy. Firstly, there is a natural variability in the material strengths and secondly it is impossible to predict the loading, which a structure (e.g. a building) may be subjected to on a future occasion. Thus uncertainties affecting the safety of a structure are due to

- uncertainty about loading
- uncertainty about material strength and
- uncertainty about structural dimensions and behaviour.

These uncertainties together make it impossible for a designer to guarantee that a structure will be absolutely safe. All that the designer could ensure is that the risk of failure is extremely small, despite the uncertainties.

Advantage of steel structure :

1. Steel member have high strength/unit weight
2. Steel being a ductile material does not fail suddenly but give visible evidence of independent failure by large deflection
3. Structural steel is tough i.e. they have both strength and ductility
4. Being light steel member can be easily handle and transport properly, steel structure have long life & property of steel mostly do not change
5. Addition and alteration can be made easily to structure
6. The steel can be erected at fast rate
7. Steel has highest scrap value among all building material
8. Steel is the ultimate recyclable material
9. Addition and alteration can be easily made to structure

Disadvantage of steel structure :

1) Steel structure may be more costly than other type of structure
2) The strength of steel reduce subsequently when heated at temperature commonly observed in building fires, hence need fire proof treatment.

3) Steel structure exposed to air and water such as bridges are such suspectical to corrosion and needs regular maintainace.

Permissible Stresses: The permissible stress is defined the ratio of yield stress to factor of safety. So as to keep the stresses within permissible value. Thus

$$\text{Permissible Stress} = \frac{\text{Yield Stress}}{\text{Factor of Safety}}$$

Factor of Safety: The factor of safety is a termed describing the structural capacity of as steel beyond the expected load or actual load. The safety factor are often calculated using detailing analysis because compressive is impractical on many structures , such as bridges, buildings but the structure ability to carry load must be determine to responsible accuracy. Many systems are purposefully deal much stronger than needed for normal uses to alone for emergency situation, unaccepted load, needs or degradation.
Factor of Safety is defined as yield stress to working stress

$$\text{Factor of Safety} = \frac{\text{Yield Stress}}{\text{Working Stress}}$$

Methods of Design:

The aim of design is to design, shape, size and connection details of the members so that the structural beam design will performed satisfactory during its right span. Following methods of design are used in steel structures
1) Working Stress Method (WSM)
2) Ultimate Load Method (ULM)
3) Limit State Method (LSM)

1) Working Stress Method (WSM) :
Working Stress Method is the traditional method of design not only for Reinforced Concrete but also for structural steel and timber design. The conceptual basis of the WSM assumes that the structural material behaves in a linear elastic manner and that appropriate safety can be ensured by suitably limiting the stresses in the material due to the presumed working loads (service loads) on the structure. WSM also assumes that both the steel reinforcement and concrete act together and are perfectly elastic at all stages, and hence the modular ratio can be used to determine the stresses in steel and concrete. The stresses under the working loads are obtained by applying the methods of 'strength of materials' like the simple bending theory. The limitations due to non-linearity and buckling are neglected. The stresses caused by the 'characteristic' or service loads are checked against the permissible (allowable) stress, which is a fraction of the ultimate or yield stress. The permissible stress may be defined in terms of a factor of safety, which takes care of the overload or other unknown factors.

Limitations of Working Stress Method

1.The main assumption of a linear elastic behavior and the implied assumption that the stresses under working loads can be kept within the 'permissible stresses' are found to be unrealistic. Many factors are responsible for this, such as the long-term effects of creep and shrinkage and other secondary effects.

2.The use of the imaginary concept of modular ratio results in larger percentage of compression steel and generally larger member sizes than the members designed using ultimate load or limit states design. However, as a result of the larger member sizes, they result in better performance during service.

2) Ultimate Load Method (ULM)

This is also known as load factor method or ultimate strength method. In this we make use of the nonlinear region of stress strain curves of steel and concrete. The safety is ensured by introducing load factor.

"Load factor is the ratio of ultimate strength to the service loads"

The ULM makes it possible to consider the effects of different loads acting simultaneously thus solving the shortcomings of WSM. As the ultimate strength of the material is considered we will get much slender sections for columns and beams compared to WSM method. But the serviceability criteria is not met because of large deflections and cracks in the sections. The fall-back in the method was that even though the nonlinear stress strain behaviour of was considered sections but the nonlinear analysis of the structural was not carried out for the load effects. Thus the stress distribution at ultimate load was just the magnification of service load by load factor following the linear elastic theory.

3) Limit State Method (LSM)

In limit state design method, the structure shall be designed withstand safety. All loads likely to act on it throughout its life span. It shall not suffer total collapse under accidental load such as from explosion or impact or human error to an extend beyond the local damages. The acceptable limit for safety, serviceability requirement before failure occurs it called limit state.

Steel structure are to be design and constructed to safety. The design requirement with regard to stability, strength, serviceability, brittle, fracture, fatigue, fire and durability such that they need the following points.

a) Remain free adequate re-ability be able to sustain all loads.

b) We have adequate durability under normal maintenance

c) Do not suffer overall damage as collapse

The current revision of the code of practice IS 800-2007 recommended limit state method for design of structure using hot rolled section.

Sl. No.	Working Stress Method	Limit State Method
1.	This method is based on the elastic theory which assumes that concrete and steel are elastic and the stress strain curve is linear for both.	This method is based on the actual stress-strain curves of steel and concrete. For concrete the stress-strain curve is non-linear.
2.	In this method the factor of safety are applied to the yield stresses to get permissible stresses.	In this method, partial safety factors are applied to get design values of stresses.
3.	No factor of safety is used for loads.	Design loads are obtained by multiplying partial safety factors of load to the working loads.
4.	Exact margin of safety is not known.	Exact margin of safety is known.
5.	This method gives thicker, sections, so less economical.	This method is more economical as it gives thinner sections.
6.	This method assumes that the actual loads, permissible stresses and factors of safety are known. So it is called as deterministic method.	This method is based upon the probabilistic approach which depends upon the actual data or experience, hence it is called as non-deterministic method.
7.	Working stress method is also known as the plastic method	Limit State method is also known as the Elastic design
8.	In working stress method, the material follows Hooke's law as stress is not allowed to cross the yield limit.	Limit state method, stress is allowed to cross the yield limit.
9.	This method gives more large sections, therefore less economical.	This method is more economical since it gives thinner sections.
10	This method assumes that the actual loads, permissible pressures and factors of safety have been understood. So it's called a deterministic method.	This way is based upon the probabilistic approach that depends upon the real data or expertise, thus it's referred to as a non-deterministic method.

Types of connection:

The connection provided in steel structure can be classified as following

1) Riveted Connection
2) Bolted Connection
3) Welded connection
4) Pinned connection

1) Riveted Connection: Riveted connection absolute understanding of riveted connection is essential for strength, evaluation and rehabilitation of old structure. The analysis and design of riveted connection as that of bolted connection.

2) Bolted Connection: A bolt metal pin with a head at a one end and the shank is threaded at the outer end in order to receive a nut. Bolts are used for joining together piece of metal by inserting them through the bolts in metal and tightening the nut at the threaded end.

3) Welded connection: Welding is a process of joining to metal pieces at the faces. Render plastic by pressure or heat or both.

4) Pinned connection: When structural members are connected by means of cylindrical shape. Pin the connection is called pinned connection.

Various types of standard rolled section:

[1] Structural steel can be loaded into various shape and size usually having larger modular of section in proportion to their cross-section area are preferred
[2] Steel sections are simply design by cross-sectional shape
[3] The cross section and size are governing by a number of factor such as arrangement of material dimension and capacity of rolling meals and material properties

1. **Rolled steel T-section:-**

[1] I.S.J.T. (I.S. Junior T-section)
[2] I.S.N.T (I.S. Normal T-section)
[3] I.S.H.T (I.S. Heavy T-section)

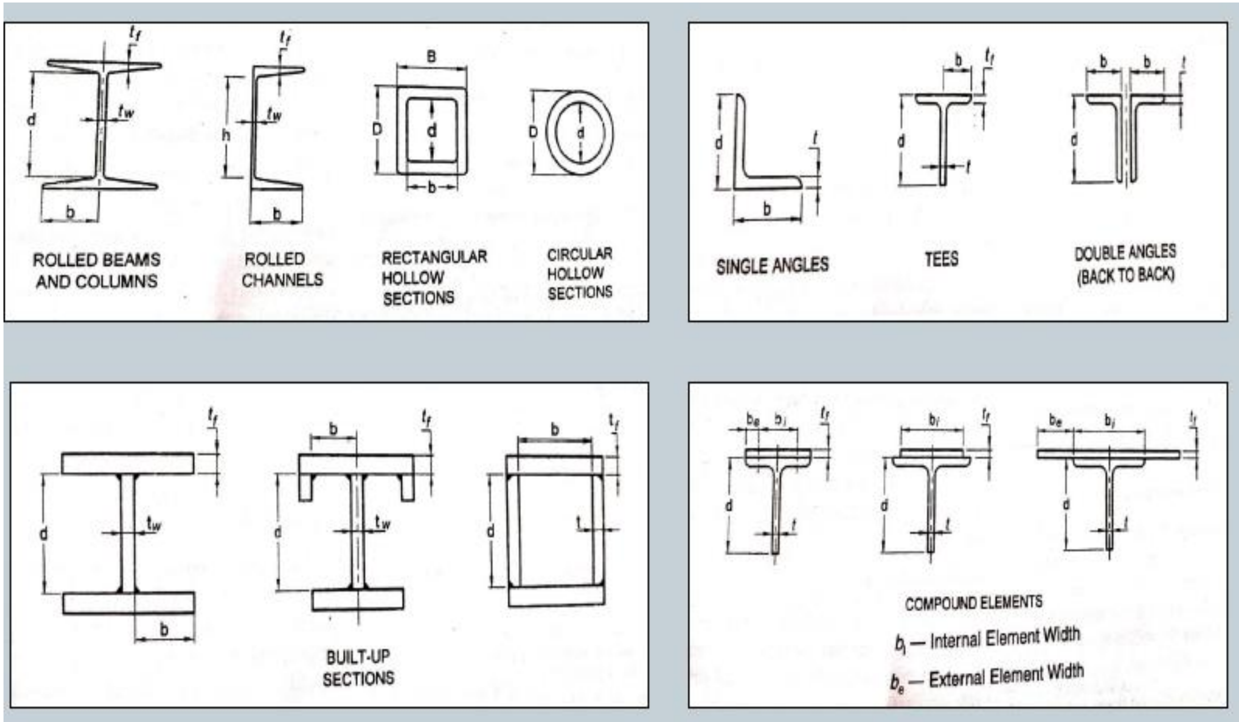
T-section are used to transmit the bracket load to column as tension member.

2. **Rolled steel Angle-section:-**

As per I.S. code angle section are divided into two category

I. Equal angle section
II. Unequal angle section

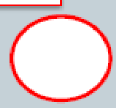
It is denoted by I.S.A.

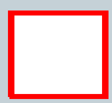


1. Flat hot rolled sections

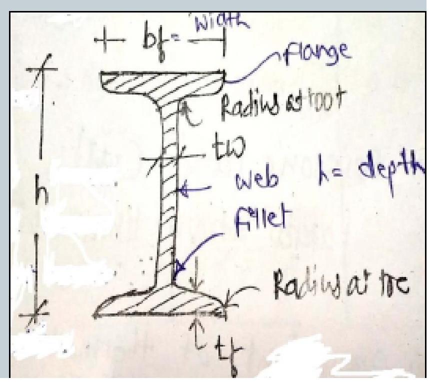
a. Plates (t) 

b. Flat bars (b x t) 

c. Round bar (d) 

d. Square bar (a x a) 

2. Hot rolled beams (B), As per BIS (IS: 808-1989)



ISJB-	Indian Standard Joist/Junior Beam,
ISLB-	Indian Standard Light Beam ,
ISMB-	Indian Standard Medium Weight Beam,
ISHB-	Indian Standard Heavy Weight Beam,
ISWB-	Indian Standard Wide Flange Beam,
ISSC-	Indian Standard Column Section.

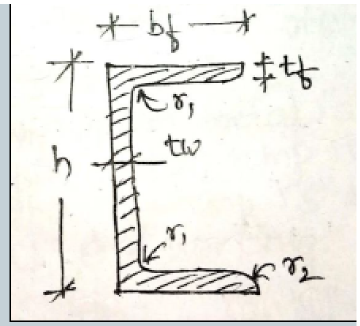
3. Hot rolled channel (C)

ISJC- Indian Standard Joist/Junior Channel,

ISLC- Indian Standard Light Weight Channel,

ISMC- Indian Standard Medium Weight Channel,

ISMCP- Indian Standard Medium Weight Parallel Flange Channel



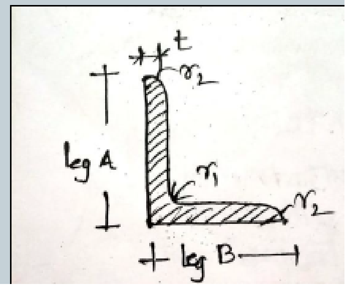
4. Angle sections (A)

ISEA- Indian Standard Equal/Unequal Angle Sections

ISA- Indian Standard Angle

ISRQ- Indian Standard Round Bar

ISSQ- Indian Standard Square Bar



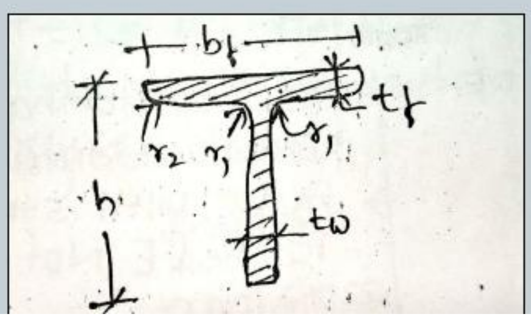
5. Rolled I sections

ISJT- Indian Standard Joist/Junior Tee,

ISLT- Indian Standard Light Tee,

ISHT- Indian Standard Wide Flange Tee,

ISNT- Indian Standard Normal Tee



<u>Types of loads on steels structure:</u>

1) Dead Load:

Dead loads are permanent and stationary load which are transferred to the structure thought there their life span. Dead load is primaralily due to self weight of structural member. Permanent partition wall fixed permanent equipment and weight of different material.

Plain concrete- 25 KN/m^3

R.C.C concrete-25 KN/m^3

Soil-18 KN/m^3

Rolled steel-79 KN/m^3

IS Code used for dead load is IS 875-1987 Part-I

2) Live Load :

Live load are either moveable or moving load without any impact. These are assumed to be produced by the intended use or occupancy of the building including weight of movable portion. Live load is consider according to i.s.875 part-II.

Sr.no	Type of load	Maximum live load
1	Residential building	2 KN/m^2
2	Bank, office	3 KN/m^2
3	Classroom assembly hall	4 KN/m^2
4	Workshop, factory:- Light weight- Medium weight- Heavy weight-	 5 KN/m^2 7.5 KN/m^2 10 KN/m^2

3) Wind Load:

Wind load basically horizontal load causes by movement of air. Wind load is required to be consider in the design specially when the height of building exceeds the two times of dimensions transferred to expose surface. Wind depend upon intensity of wind pressure and shape of structure in case of truss design two type of wind type of wind pressure considered

1) Internal air pressure:-it depend on permeability of structure
2) External air pressure:-it depend on location of structure

   Internal air pressure depends upon permeability of structure and external air pressure depends upon location.

   IS Code used for wind load is 875 part-III.

4) Earthquake Load: If structure is situated in earthquake prone area, earthquake load may be considered due to earthquake shock structure vibration. Earthquake load are horizontal load caused by earthquake and shall be calculated in accordance with IS 1983 and revised IS 2017.

5) Snow Load: This depend upon latitude of placed. Design snow load depends upon shape of roof and this load act vertically and this load can be taken as 2.5 KN/m^2 per mm depends of snow.

6) Imposed Load: Imposed load caused by vibrator or impact or acceleration of person walking, produce live load but soldiers marching or frame supporting lifts produced impact load. Thus impact load is equal to imposed load incremental by some percentage depending on the intensity of impact.

7) Hydrostatic Pressure: Pressure of water is to be taken into account which are below the ground level. Hydrostatics pressure is calculated from established theories.

8) Temperature Effects: Due to change in temperature in structural member, extract or contract and produced the loading effects in member.

Load Combination:

The combination of the loads are necessary to ensure the required safety and economic design. Load combination as per IS 875 Part IV

Dead Load (DL)

Live Load (LL)

Wind Load (WL)

Earthquake Load (EL)

Temporary Load (TL)

Combination

1) 1.5 (DL + LL)

2) 1.2 (DL+LL+EL)

3) 1.2 (DL+LL-EL)

4) 1.5 (DL + EL)

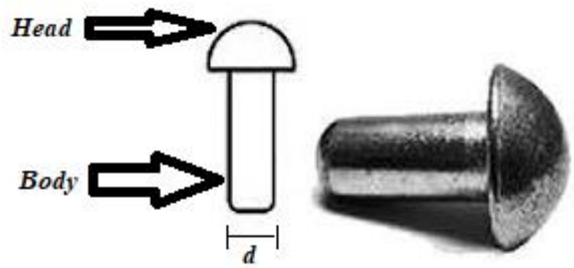
5) 1.5 (DL - EL)

6) 0.9 DL+ 1.5 EL

7) 0.9 DL-1.5 EL etc.

Design of Connection

1) Riveted Connection:- Rivet is a round rod which hold the metal piece permanently.



d= nominal diameter of rivet

D= Gross diameter of rivet

D=d+1.5 mm $\left[d \leq 25\ mm\right]$ **(Page No: 95, C. No: 8.9.3)**

D=d+2 mm $\left[d \succ 25\ mm\right]$

Types of Rivets:

1) Hot Driven Rivet: a) Field 2) Shop

2) Cold Driven Rivet: a) Field 2) Shop

Field and Shop rivet again divided into two categories

Field rivet: Hand Driven and Power Driven

Shop rivet: Hand Driven and Power Driven

Power Driven shop rivet will have better strength than any other rivet

a) Shop Rivet: These rivets are driven in the shop under better control and condition. Therefore they have more strength.

b) Field Rivet: These rivets are driven at site of work, there is less control on process of fabrication which results less strength.

As per IS Code permissible stresses for field rivets are reduced by 10%. Permissible Stresses for shop rivet are given in IS 800-1984, Table No: 8.1

Rivet hole

There are two types of rivet hole

1) Punching 2) Drilling

1) Punching : When rivet hole are made by punching holes are not perfect punch damage material around hole.

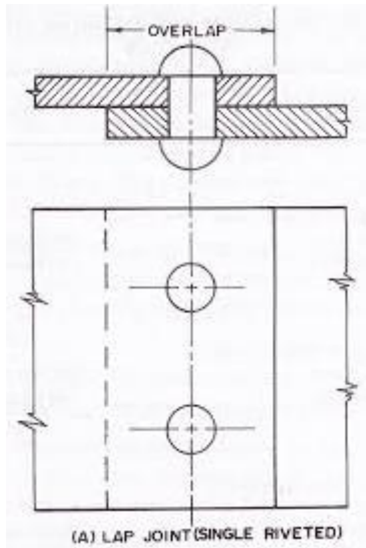
2) Drilling : When rivet hole are made by drilling, holes are perfect and provide good alignment.

Types of riveted joints

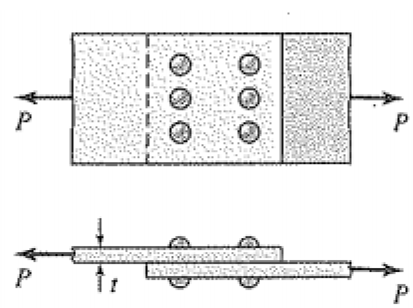
1) Lap Joint   2) Butt Joint

1) Lap Joint: When two or more members are placed one above other by given some overlap and connected to each other by the rivets then joint is called as lap joint. Depending upon number of rows of rivets, lap joint is called

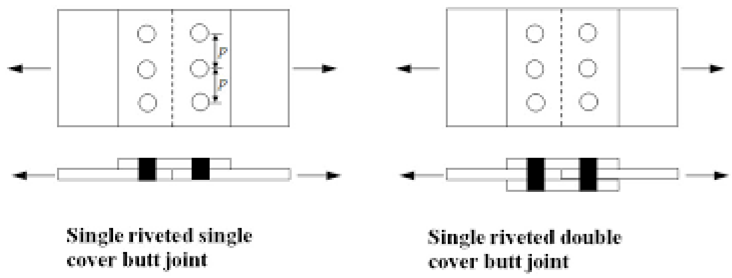
a) Single riveted lap joint

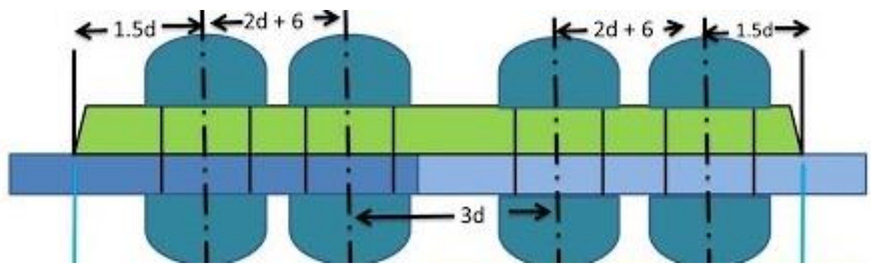


b) Double riveted lap joint



2) Butt Joint: In this joint, members are touched to each other and by providing cover plates they are joined to each other by riveted depending upon cover plates on one or both side, joint is classified as single cover plate or double plate joint.



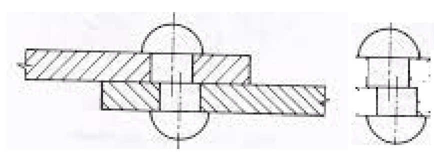


Double riveted single cover plate butt joint

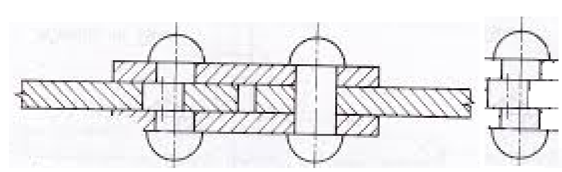
Design of Riveted joints:

Failure of riveted joint

1) Shear failure of rivet



Single shear failure

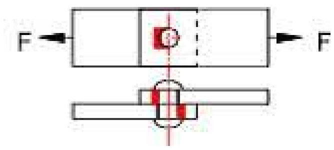


Double shear failure

Shear stress in the rivet many exceed the permissible shear stress in rivets because the plate sleep due to applied force.

2) Bearing failure of rivet

The rivet is crushed around circumference plate may be stronger in bearing.



3) Shear failure of plate

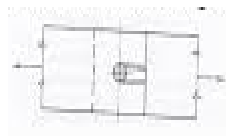
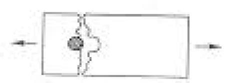


Plate may be failed in shear along two lines as show in figure, this occurs when minimum edge distance is not provided.

4) Tearing failure of plate

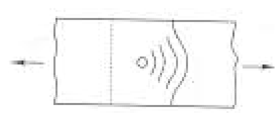


Tearing Failure of Plate

When plate is riveted together are carrying tensile load. Tearing failure of plate is less than strength of rivet . Tearing failure occurs at net sectional area of plate.

Net Area= (Gross Area- Area of Rivet)

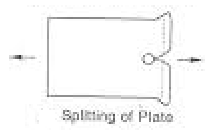
5) Bearing failure of plate



Bearing Failure of Plate

A plate may be crushed when bearing stress in plate exceed permissible stress. Bearing failure of plate may occur when minimum edge distance is not provided.

6) Splitting failure of plate



This may occur when minimum edge distance is not provided.

There are six failure of riveted joint in which shear failure, bearing failure and splitting failure of plate can be provided by providing sufficient edge distance. Only three failure are taken into account in the design and these are

1) Shear failure of rivet

2) Bearing failure of rivet

3) Tearing failure of plate

Strength of riveted joint:

Maximum safe load transferring capacity of the joint one member to another without any failure is called Strength of riveted joint.

Strength of riveted joint is least of the following three values

1) Strength of riveted joint against shearing of rivet (P_s)

2) Strength of riveted joint against bearing failure of rivet (P_b)

3) Strength of riveted joint against tearing of plate (P_t)

1) Strength of riveted joint against shearing of rivet (P_s): Strength of riveted joint against shearing of rivet (P_s) is the load carrying capacity of riveted joint without shear failure of rivet. It is the product of permissible shear stress and the gross cross sectional area of rivet which is in shear and number of rivets

P_S = Number of rivets X Area of rivet in shearing X Permissible shear stress

$P_S = N \text{ x } \frac{\Pi}{4} \text{ x } D^2 \text{ X } \tau_{Vf}$ For Single Shear

$P_S = N \text{ x } 2 \text{ x} \frac{\Pi}{4} \text{ x } D^2 \text{ X } \tau_{Vf}$ For Double Shear

Where N = Number of rivets in a joint

D = Gross diameter of Rivet

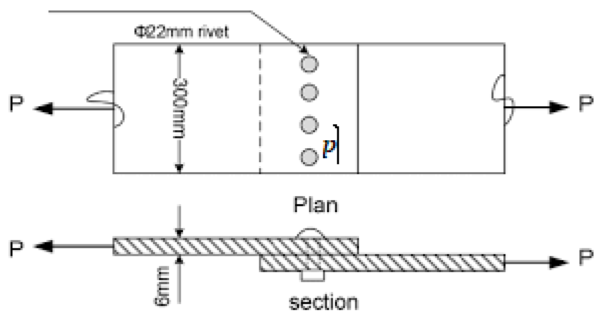
τ_{Vf} = Permissible shear stress in rivet (IS 800-1984 Page No: 95, Table :8.1)

In case of lap joint , total number of rivets are taken into consideration.

Strength of riveted joint against shearing of rivet per pitch (P_s/ pitch):

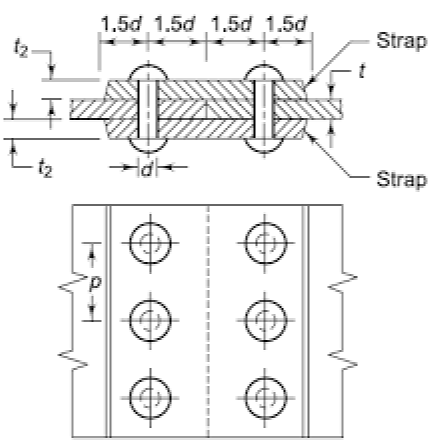
$P_S / pitch = No$ of rivet per pitch $\text{x } \frac{\Pi}{4} \text{ x } D^2 \text{ X } \tau_{Vf}$ For Single Shear

No of rivet per pitch For Single Shear = 1



$P_S / pitch = No$ of rivet per pitch $\text{x } \frac{\Pi}{4} \text{ x } D^2 \text{ X } \tau_{Vf}$ For Double Shear

No of rivet per pitch For Double Shear = 2

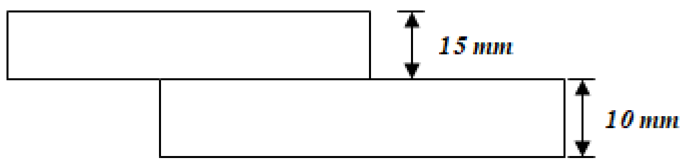


2) Strength of riveted joint against bearing failure of rivet (P_b):

Strength of riveted joint against bearing failure of rivet is the product of number of rivets in a joint and bearing stress in rivet.

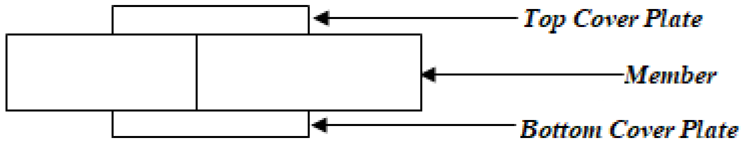
P_b = Number of rivets in joint X Bearing area of rivet X Permissible shear stress in rivet
Bearing area of rivet = area of rectangle
Whose one side is gross diameter of rivet and other side is thickness of plate
$P_b = N$ x D x t X σ_{bf}
Where N = Number of rivets in a joint
D = Gross diameter of Rivet
t=Thickness of plate
σ_{bf} = Permissible bearing stress in rivet (IS 800-1984 Page No: 95, Table :8.1)

In case, in lap joint least thickness out of the thickness of member is to be consider



Lap joint least thickness out of the thickness of member=10 mm

In butt joint , sum of thickness of cover plate compared with the thickness of member and least out of these is taken for calculation



Example : Top cover plate thickness= 6 mm, Bottom cover plate thickness= 6 mm and member thickness= 8mm

Thickness = Least of thickness of cover plate and member= 8 mm or 6+6=12 (Take least)

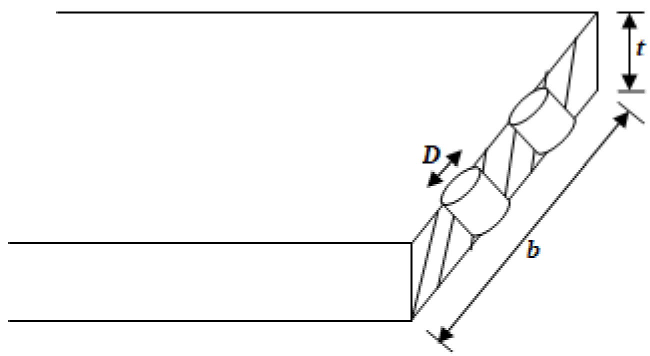
Thickness = 8 mm

Strength of riveted joint against bearing failure of rivet (P_b/ Pitch):

$P_b / pitch =$ Number of rivets per pitch x D x t X σ_{bf}

3) Strength of riveted joint against tearing of plate (P_t):

Strength of riveted joint against tearing of plate is the product of net area of plate and permissible tensile stress.



$P_t =$ Net area X Permissible tensile stress

Net area of rivet $= Gross$ area - Area of rivet hole

Net area of rivet $= (b\ x$ t) -(n x $D\ x$ t)

Net area of rivet $= \left[(b) \text{ -}(n\ x\ D)\right]t$

$P_t = \left[(b) \text{ -}(n\ x\ D)\right]t$ X σ_{at}

Where b = Width of the plate

$n =$ Number of rivet in one row

t=Thickness of plate

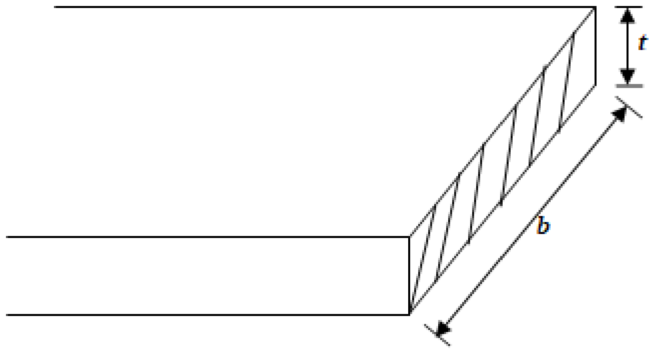
σ_{at} = Permissible axial tensile stress in rivet (IS 800-1984 Page No: 95, Table :8.1)

$P_t / pitch = (p - D) X\ \ t$ X σ_{at}

$p = Pitch$

Strength of solid plate:

Load carrying capacity of solid plate (without rivet holes) is called strength of solid plate. It is product of cross sectional area of plate and permissible tensile stress in plate



P_t = Cross sectional area X Permissible tensile stress

Cross sectional area = (b x t)

$P_t = (b\ x\ \mathrm{t})\ \mathrm{X}\ \sigma_{at}$

Where b = Width of the plate

t=Thickness of plate

σ_{at} = Permissible axial tensile stress in rivet (IS 800-1984 Page No: 95, Table :8.1)

Strength of solid plate per pitch

$P / pitch = p\ X\ t\ \mathrm{X}\ \sigma_{at}$

$p = Pitch$

Efficiency of riveted joint:

Efficiency of riveted joint is denoted by η

It the ratio of riveted joint to strength of solid plate.

$$\eta = \frac{Strength \text{ of riveted joint}}{Strength \text{ of solid plate}}\ X\ 100$$

$$\eta = \frac{Least \text{ of } \mathrm{P_s}, \mathrm{P}_b \text{and } \mathrm{P}_t}{Strength \text{ of solid plate}}\ X\ 100$$

<u>Design of Riveted Joint</u>

<u>Types of problems</u>

1) To find strength, efficiency and rivet value of given of riveted joint

Given Data:

a) Type of joint

b) Nominal diameter of rivet

c) Pitch

d) Permissible stresses from IS 800-1984, Table No:8.1, Page No: 95

<u>Step 1:</u> To find gross diameter of rivet (Page No: 95, C. No: 8.9.3)

d= Nominal diameter of rivet

D= Gross diameter of rivet

D=d+1.5 mm $[d \le 25\ mm]$

D=d+2 mm $[d \succ 25\ mm]$

<u>Step 2:</u> To find shearing strength of rivet

P_S = Number of rivets X Area of rivet in shearing X Permissible shear stress

$P_S = N \text{ x } \frac{\Pi}{4} \text{ x } D^2 \text{ X } \tau_{Vf}$ For Single Shear

$P_S = N \text{ x } 2 \text{ x} \frac{\Pi}{4} \text{ x } D^2 \text{ X } \tau_{Vf}$ For Double Shear

Where N = Number of rivets in a joint

D = Gross diameter of Rivet

τ_{Vf} = Permissible shear stress in rivet (IS 800-1984 Page No: 95, Table :8.1)

Step 3: To find bearing strength of rivet

P_b = Number of rivets in joint X Bearing area of rivet X Permissible bearing stress in rivet
Bearing area of rivet = area of rectangle
Whose one side is gross diameter of rivet and other side is thickness of plate

$P_b = N \text{ x D x t X } \sigma_{bf}$

Where N = Number of rivets in a joint

D = Gross diameter of Rivet

t=Thickness of plate

σ_{bf} = Permissible bearing stress in rivet (IS 800-1984 Page No: 95, Table :8.1)

Step 4: To find tearing strength of plate

$$P_t = (p - D)\ X\ t\ \text{X}\ \sigma_{at}$$
$$p = Pitch$$

Step 5: To find strength of riveted joint

Strength of riveted joint is least of P_s, P_b and P_t

Step 6: To find strength of solid plate

$$P = p\ X\ t\ \text{X}\ \sigma_{at}$$
$$p = Pitch$$

Step 7: Efficiency of joint

$$\eta = \frac{Strength \text{ of riveted joint}}{Strength \text{ of solid plate}}\ X\ 100$$

$$\eta = \frac{Least \text{ of } P_s, P_b \text{and } P_t}{Strength \text{ of solid plate}}\ X\ 100$$

Step 8: Rivet Value

Least of P_s and P_b for single rivet i.e N=1

8.9.4.1 The calculated stress in a mild steel shop rivet or in a bolt of property class 4.6 (*see* IS : 1367-1967) shall not exceed the values given in Table 8.1.

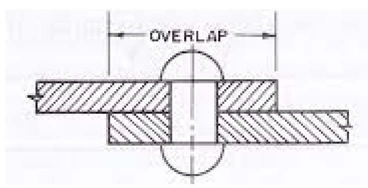
TABLE 8.1 MAXIMUM PERMISSIBLE STRESS IN RIVETS AND BOLTS

Description of Fasteners	Axial Tension, σ_{tf},	Shear, τ_{vf}	Bearing, σ_{pf}
(1)	(2) MPa	(3) MPa	(4) MPa
Power-driven rivets	100	100	300
Hand-driven rivets	80	80	250
Close tolerance and turned bolts	120	100	300
Bolts in clearance holes	120	80	250

1) Find strength, efficiency and rivet value for single riveted lap joint to connect plates of 12 mm and 10 mm thickness by 26 mm diameter of rivet, pitch of rivet is 55 mm. Use hand driven rivet, axial tensile strength of plate $\sigma_{at} = 0.6 f_y,\ f_y = yield$ stress =250 N/ mm^2

Solution: Given Data

Single riveted lap joint



Nominal diameter of rivet= d =26 mm

pitch=P=55 mm (Given)

Thickness =t= 12 mm and 10 mm (Take least value)

Thickness =t= 10 mm

N=1 , Single rivet

$Axial\ tensile\ strength = \ \sigma_{at} = 0.6 f_y,\ f_y = yield$ stress =250 N/ mm^2

$\sigma_{at} = 0.6 f_y$=0.6 x 250= 150 N/ mm^2

Hand driven rivet

IS 800 -1984, Table No: 8.1, Page= 95

$\tau_{vf} = 80$ N/mm^2

$\sigma_{bf} = 250$ N/mm^2

<u>Step 1: </u> To find gross diameter of rivet (Page No: 95, C. No: 8.9.3)

d= Nominal diameter of rivet=26 mm

D= Gross diameter of rivet

D=d+2 mm =26+2=28 mm $\left[d \succ 25\ mm \right]$

Step 2: To find shearing strength of rivet

P_S = Number of rivets X Area of rivet in shearing X Permissible shear stress

$P_S = N$ x $\frac{\Pi}{4}$ x D^2 X τ_{Vf} For Single Shear

$P_S = 1$ x $\frac{\Pi}{4}$ x 28^2 X 80 $= 49.26$ x $10^3 N = 49.26 KN$

Where N = Number of rivets in a joint

D = Gross diameter of Rivet

τ_{Vf} = Permissible shear stress in rivet (IS 800-1984 Page No: 95, Table :8.1)

Step 3: To find bearing strength of rivet

P_b = Number of rivets in joint X Bearing area of rivet X Permissible bearing stress in rivet

Bearing area of rivet = area of rectangle

Whose one side is gross diameter of rivet and other side is thickness of plate

$P_b = N$ x D x t X σ_{bf}

$P_b = 1$ x 28 x 10 X 250 = 70 x $10^3 N = 70KN$

Where N = Number of rivets in a joint

D = Gross diameter of Rivet

t=Thickness of plate (Least Thickness)

σ_{bf} = Permissible bearing stress in rivet (IS 800-1984 Page No: 95, Table :8.1)

Step 4: To find tearing strength of plate

$$P_t = (p - D)\ X\ t\ X\ \sigma_{at} = (55-28) \text{ x } 10 \text{ x } 150 = 40.5 \text{ x } 10^3 N = 40.5KN$$

Step 5: To find strength of riveted joint

Strength of riveted joint is least of P_s, P_b and P_t

Strength of riveted joint = 40.5 KN

Step 6: To find strength of solid plate

$P = p\ X\ t\ \mathrm{X}\ \sigma_{at} = 55\ x\ 10\ x\ 150 = 82.5\ \mathrm{x}\ 10^3 N = 82.5\ \mathrm{KN}$

Step 7: **Efficiency of joint**

$$\eta = \frac{Strength \text{ of riveted joint}}{Strength \text{ of solid plate}}\ X\ 100$$

$$\eta = \frac{Least \text{ of } \mathrm{P_s, P_b \text{ and } P_t}}{Strength \text{ of solid plate}}\ X\ 100 = \frac{40.5}{82.5}\ X\ 100 = 49.04\%$$

Step 8: **Rivet Value**

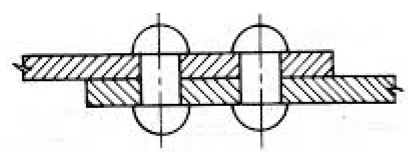
Least of P_s and P_b for single rivet i.e N=1

Rivet Value = 49.26 KN

2) Find strength, efficiency and rivet value for double riveted lap joint to connect plates of 12 mm and 10 mm thickness by 24 mm diameter of rivet, pitch of rivet is 50 mm. Use power driven driven shop rivets, axial tensile strength of plate $\sigma_{at} = 0.6 f_y,\ f_y = yield$ stress =250 N/ mm^2

Solution: Given Data

Double riveted lap joint



Nominal diameter of rivet= d =24 mm

pitch=p=50 mm

Thickness =t= 12 mm and 10 mm (Take least value)

Thickness =t= 10 mm

N=2 , Double rivet

$Axial\ tensile\ strength = \ \sigma_{at} = 0.6 f_y,\ f_y = yield$ stress =250 N/ mm^2

$\sigma_{at} = 0.6 f_y$ =0.6 x 250= 150 N/ mm^2

Power driven shop rivet

IS 800 -1984, Table No: 8.1, Page= 95

$\tau_{vf} = 100$ N/mm^2

$\sigma_{bf} = 300$ N/mm^2

Step 1: **To find gross diameter of rivet (Page No: 95, C. No: 8.9.3)**

d= Nominal diameter of rivet=24 mm

D= Gross diameter of rivet

D=d+1.5 mm = 24+1.5=25.5 mm $\left[d \le 25\ mm\right]$

<u>Step 2:</u> To find shearing strength of rivet

P_S = Number of rivets X Area of rivet in shearing X Permissible shear stress

$$P_S = N \text{ x } \frac{\Pi}{4} \text{ x } D^2 \text{ X } \tau_{Vf}$$

$$P_S = 2 \text{ x} \frac{\Pi}{4} \text{ x } 25.5^2 \text{ X } 100 \quad = 102.14 \text{ x } 10^3 N = 102.14 KN$$

Where N = Number of rivets in a joint

D = Gross diameter of Rivet

τ_{Vf} = Permissible shear stress in rivet (IS 800-1984 Page No: 95, Table :8.1)

<u>Step 3:</u> To find bearing strength of rivet

P_b = Number of rivets in joint X Bearing area of rivet X Permissible bearing stress in rivet

Bearing area of rivet = area of rectangle

Whose one side is gross diameter of rivet and other side is thickness of plate

$$P_b = N \text{ x D x t X } \sigma_{bf}$$

$$P_b = 2 \text{ x } 25.5 \text{ x } 10 \text{ X } 300 = 153 \text{ x } 10^3 N = 153 KN$$

Where N = Number of rivets in a joint

D = Gross diameter of Rivet

t=Thickness of plate (Least Thickness)

σ_{bf} = Permissible bearing stress in rivet (IS 800-1984 Page No: 95, Table :8.1)

<u>Step 4:</u> To find tearing strength of plate

$$P_t = (p - D)\ X\ t\ \text{X}\ \sigma_{at} = (50 - 25.5) \text{ x } 10 \text{ x } 150 = 36.75 \text{ x } 10^3 N = 36.75 KN$$

<u>Step 5:</u> To find strength of riveted joint

Strength of riveted joint is least of P_s, P_b and P_t

Strength of riveted joint = 36.75 KN

Step 6: **To find strength of solid plate**

$$P = p\ X\ t\ \text{X}\ \sigma_{at} = 50\ x\ 10\ x\ 150 = 75\ \text{x}\ 10^3 N = 75\ \text{KN}$$

Step 7: **Efficiency of joint**

$$\eta = \frac{Strength \text{ of riveted joint}}{Strength \text{ of solid plate}}\ X\ 100$$
$$\eta = \frac{Least \text{ of } \mathrm{P_s}, \mathrm{P}_b \text{and } \mathrm{P}_t}{Strength \text{ of solid plate}}\ X\ 100 = \frac{36.75}{75}\ X\ 100 = 49.00\%$$

Step 8: **Rivet Value**

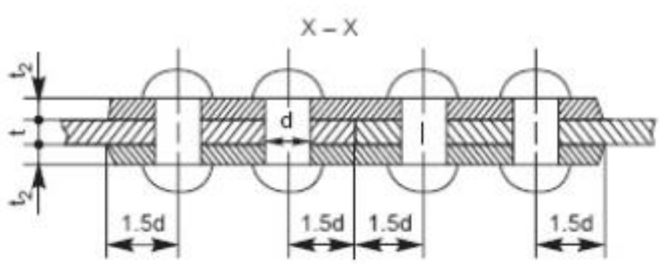
Least of P_s and P_b for single rivet i.e N=1

$$P_s = \frac{102.14}{2} = 51.07KN$$
$$P_b = \frac{153}{2} = 76.5KN$$
$$Rivet\ Value = 51.07KN$$

3) Find strength, efficiency and rivet value for double riveted double cover plate butt joint. The rivet joint two members of 8 mm thickness with cover plate of 6 m thickness , pitch of rivet is 60 mm and diameter of rivet is 20 mm. Use power driven shop rivets, axial tensile strength of plate

$$\sigma_{at} = 0.6 f_y,\ f_y = yield \text{ stress} = 250 \text{ N/ mm}^2$$

Solution: Given Data
Double riveted double cover plate butt joint



Nominal diameter of rivet= d =20 mm

pitch=p=60 mm

Thickness of two cover plate =t= 6+6=12 mm
Thickness of member =t= 8mm
Thickness =12 mm and 8 mm (Take least value)
Thickness =t= 8 mm
N=2 , Double rivet double cover plate

$Axial\ tensile\ strength = \ \sigma_{at} = 0.6 f_y ,\ f_y = yield$ stress =250 N/ mm^2

$\sigma_{at} = 0.6 f_y$ =0.6 x 250= 150 N/ mm^2

Power driven shop rivet
IS 800 -1984, Table No: 8.1, Page= 95

$\tau_{vf} = 100$ N/mm^2

$\sigma_{bf} = 300$ N/mm^2

<u>Step 1:</u> To find gross diameter of rivet (Page No: 95, C. No: 8.9.3)

d= Nominal diameter of rivet=20 mm

D= Gross diameter of rivet

D=d+1.5 mm = 20+1.5=21.5 mm $\left[d \le 25\ mm \right]$

<u>Step 2:</u> To find shearing strength of rivet

P_S = Number of rivets X Area of rivet in shearing X Permissible shear stress

$P_S = 2 \text{ x } N \text{ x } \frac{\Pi}{4} \text{ x } D^2 \text{ X } \tau_{Vf}$ For double Shear

$P_S = 2 \text{ x } 2 \text{ x} \frac{\Pi}{4} \text{ x } 21.5^2 \text{ X } 100 \quad = 145.2 \text{ x } 10^3 N = 145.2 KN$

Where N = Number of rivets in a joint

D = Gross diameter of Rivet

τ_{Vf} = Permissible shear stress in rivet (IS 800-1984 Page No: 95, Table :8.1)

Step 3: To find bearing strength of rivet

P_b = Number of rivets in joint X Bearing area of rivet X Permissible bearing stress in rivet

Bearing area of rivet = area of rectangle

Whose one side is gross diameter of rivet and other side is thickness of plate

$P_b = N$ x D x t X σ_{bf}

$P_b = 2$ x 21.5 x 8 X 300 $= 103.2$ x $10^3 N = 103.2 KN$

Where N = Number of rivets in a joint

$D =$ Gross diameter of Rivet

t=Thickness of plate (Least Thickness)

σ_{bf} = Permissible bearing stress in rivet (IS 800-1984 Page No: 95, Table :8.1)

Step 4: To find tearing strength of plate

$$P_t = (p - D)\ X\ t\ \text{X}\ \sigma_{at} = (60 - 21.5) \text{ x } 8 \text{ x } 150 = 46.2 \text{ x } 10^3 N = 46.2 KN$$

Step 5: To find strength of riveted joint

Strength of riveted joint is least of P_s, P_b and P_t

Strength of riveted joint = 46.2 KN

Step 6: To find strength of solid plate

$$P = p\ X\ t\ \text{X}\ \sigma_{at} = 60\ x\ 8\ x\ 150 = 72 \text{ x } 10^3 N = 72 \text{ KN}$$

Step 7: Efficiency of joint

$$\eta = \frac{Strength \text{ of riveted joint}}{Strength \text{ of solid plate}}\ X\ 100$$

$$\eta = \frac{Least \text{ of } \mathrm{P_s}, \mathrm{P}_b \text{and } \mathrm{P}_t}{Strength \text{ of solid plate}}\ X\ 100 = \frac{46.2}{72}\ X\ 100 = 64.16\%$$

Step 8: Rivet Value

Least of P_s and P_b for single rivet i.e N=1

$$Ps = \frac{145.2}{2} = 72.6KN$$

$$Pb = \frac{103.2}{2} = 51.6KN$$

$$Rivet\ Value = 51.6KN$$

DESIGN OF AXIALLY LOADED RIVETTED JOINT

Assumptions in axially loaded riveted joint

1) Rivet hole is to be assumed completely filled by the rivet
2) In calculations take gross diameter of rivet
3) The bending stress developed in rivet is neglected
4) The friction between contact surface is neglected
5) The actual shear stress distribution is non uniform but shear stress is assumed to be constant throughout the section.

Types of problems

2) To find diameter of rivet, number of rivet, pitch of rivet

Given Data:

a) Total Load

b) Thickness of plate=t

c) Types of joint (Lap or Butt Joint)

d) $\tau_{vf} = ?\ \text{N/mm}^2$

$\sigma_{bf} = ?\ \text{N/mm}^2$

$\sigma_{at} = ?\ \text{N/mm}^2$

Step 1: To find nominal diameter of rivet (By using Unwins Formula)

$$d = 6.04\sqrt{t}$$

t=Least Thickness

Step 2: To find gross diameter of rivet (Page No: 95, C. No: 8.9.3)

d= Nominal diameter of rivet

D= Gross diameter of rivet

D=d+1.5 mm $\left[d \le 25\ mm\right]$

D=d+2 mm $\left[d \succ 25\ mm\right]$

<u>Step 3:</u> To find rivet value

Rivet value is least of P_s and P_b for single rivet i.e N=1

P_S = Number of rivets X Area of rivet in shearing X Permissible shear stress

$P_S = N$ x $\frac{\Pi}{4}$ x D^2 X τ_{Vf} For Single Shear

$P_S = N$ x 2 x $\frac{\Pi}{4}$ x D^2 X τ_{Vf} For Double Shear

Where N = Number of rivets in a joint

D = Gross diameter of Rivet

τ_{Vf} = Permissible shear stress in rivet (IS 800-1984 Page No: 95, Table :8.1)

P_b = Number of rivets in joint X Bearing area of rivet X Permissible bearing stress in rivet

Bearing area of rivet = area of rectangle

Whose one side is gross diameter of rivet and other side is thickness of plate

$P_b = N$ x D x t X σ_{bf}

Where N = Number of rivets in a joint

D = Gross diameter of Rivet

t=Thickness of plate (least thickness)

σ_{bf} = Permissible bearing stress in rivet (IS 800-1984 Page No: 95, Table :8.1)

<u>Step 4:</u> To find number of rivets

$$N = \frac{Total\ \text{Load}}{Rivet\ \text{Value}}$$

Arrange the rivets according to the joint

<u>Step 5:</u> To find pitch of the rivet

$(p - D)$ x t x $\sigma_{at} = Number$ of rivet per pitch x Rivet value

According IS 800-1984, Page No: 96, C No: 8.10.1

Minimum pitch (p_{min}) =2.5 d

Maximum Pitch (p_{max}) = 16 t or 200 mm (which is less)

p< Pmax
p> Pmin
Step 6: To find edge distance of the rivet

According IS 800-1984, Page No: 97, C No: 8.10.2, Use Table No:8.2
Step 7: To find minimum width of plate

1. Design single riveted butt joint to resist a load of 200 KN tensile. The plates are 16 mm thick . Use power driven shop rivets. $\sigma_{at} = 150\ \text{N/mm}^2$

Solution: Given Data
Load= P=200 KN (Tensile)
t=16 mm
For power driven shop rivet
Single riveted butt joint, N=1
thickness of plate = 16 mm
IS 800-1984, Page No: 95, Table No: 8.1

$\tau_{vf} = 100\ \text{N/mm}^2$

$\sigma_{bf} = 300\ \text{N/mm}^2$

$\sigma_{at} = 150\ \text{N/mm}^2$

Step 1: To find nominal diameter of rivet (By using Unwins Formula)

t=Thickness = 16 mm
$d = 6.04\sqrt{t} = 6.04\sqrt{16}$ =24.16 mm $\cong$ 24 mm
Step 2: To find gross diameter of rivet (Page No: 95, C. No: 8.9.3)

d= Nominal diameter of rivet

D= Gross diameter of rivet

D=24+1.5 mm=25.5 mm $[d \le 25\ mm]$

Step 3: To find rivet value

Rivet value is least of P_s and P_b for single rivet i.e N=1

P_S = Number of rivets X Area of rivet in shearing X Permissible shear stress

$P_S = N$ x 2 x $\frac{\Pi}{4}$ x D^2 X τ_{Vf} For Double Shear

$P_S = 1$ x 2 x $\frac{\Pi}{4}$ x D^2 X τ_{Vf} = 1 x 2 x $\frac{\Pi}{4}$ x 25.5^2 X 100 = 102.14 x $10^3 N = 102.14 KN$

$P_b = N$ x D x t X σ_{bf} = 1x 25.5 x 16 x 300= 122.4 x $10^3 N = 122.4 KN$

Where N = Number of rivets in a joint

D = Gross diameter of Rivet

τ_{Vf} = Permissible shear stress in rivet (IS 800-1984 Page No: 95, Table :8.1)

σ_{bf} = Permissible bearing stress in rivet (IS 800-1984 Page No: 95, Table :8.1)

Rivet Value= $102.14 KN$

Step 4: To find number of rivets

$$N = \frac{Total\ \text{Load}}{Rivet\ \text{Value}} = \frac{200}{102.4} = 1.9 \cong 2$$

Providing two rivets on either side of joint in one row

Step 5: To find pitch of the rivet

$(p - D)$ x t x σ_{at} = $Number$ of rivet per pitch x Rivet value

(p-25.5) x 16 x 150 = 1 x 102.14 x 10^3

p = 68.05 mm

According IS 800-1984, Page No: 96, C No: 8.10.1

Minimum pitch (p_{min})=2.5 d=2.5 x 24=60 mm

Maximum Pitch (p_{max})= 16 t or 200 mm (which is less)

= 16 x 16 = 256 or 200 mm

= 200 mm

Providing p= 65 mm $< p_{max}$

$> p_{min}$

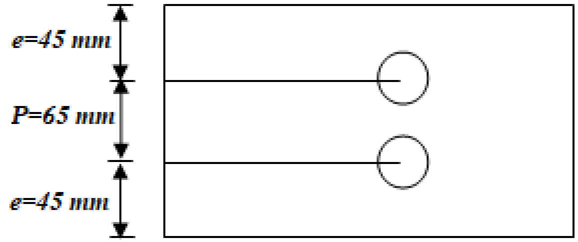
Step 6: To find edge distance of the rivet

According IS 800-1984, Page No: 97, C No: 8.10.2, Use Table No:8.2

Edge Distance =44 mm $\cong$ 45 mm

Note: Edge distance is depend upon gross diameter of rivets i.e (D)

Step 7: To find minimum width of plate



Minimum width of plate = 45+65+45=155 mm

2) Two plates 12 mm thickness are to be connected by double cover butt joint to carry axial pull of 300 KN. Design the riveted joint and width of plate.

Solution: Given Data
Load= P=300 KN (Tensile)
t=12 mm
Assuming power driven shop rivet
Double cover butt joint
IS 800-1984, Page No: 95, Table No: 8.1
Assuming single rivet N=1

$\tau_{vf} = 100$ N/mm^2

$\sigma_{bf} = 300$ N/mm^2

$Assu\min g$ fy= 250 N/mm^2, $\sigma_{at} =$ 0.6x fy = 0.6 x 250 $= 150$ N/mm^2

<u>Step 1:</u> To find nominal diameter of rivet (By using Unwins Formula)

t=Thickness = 12 mm

$d = 6.04\sqrt{t} = 6.04\sqrt{12}$ =20.9 mm $\cong$ 22 mm

<u>Step 2:</u> To find gross diameter of rivet (Page No: 95, C. No: 8.9.3)

d= Nominal diameter of rivet

D= Gross diameter of rivet

D=22+1.5 mm=23.5 mm $[d \leq 25\ mm]$

<u>Step 3: To find rivet value</u>

Rivet value is least of P_s and P_b for single rivet i.e N=1

P_S = Number of rivets X Area of rivet in shearing X Permissible shear stress

$P_S = N$ x 2 x $\frac{\Pi}{4}$ x D^2 X τ_{Vf} For Double Shear

$P_S = 1$ x 2 x $\frac{\Pi}{4}$ x D^2 X τ_{Vf} = 1 x 2 x $\frac{\Pi}{4}$ x 23.5^2 X 100 = 86.74 x $10^3 N = 86.74 KN$

$P_b = N$ x D x t X σ_{bf} = 1x 23.5 x 12 x 300= 84.6 x $10^3 N = 84.6 KN$

Where N = Number of rivets in a joint

D = Gross diameter of Rivet

τ_{Vf} = Permissible shear stress in rivet (IS 800-1984 Page No: 95, Table :8.1)

σ_{bf} = Permissible bearing stress in rivet (IS 800-1984 Page No: 95, Table :8.1)

Rivet Value= 84.6 *KN*

Step 4: To find number of rivets

$$N = \frac{Total\ \text{Load}}{Rivet\ \text{Value}} = \frac{300}{84.6} = 3.54 \cong 4$$

Providing four rivets on either side of joint in one row

Step 5: To find pitch of the rivet

$(p - D)$ x t x σ_{at} = *Number* of rivet per pitch x Rivet value

(p-23.5) x 12 x 150 = 1 x 84.6 x 10^3

p = 70.5 mm

According IS 800-1984, Page No: 96, C No: 8.10.1

Minimum pitch (p_{min})==2.5 d=2.5 x 22=55 mm

Maximum Pitch (p_{max}) = 16 t or 200 mm (which is less)

= 16 x 12 = 192 or 200 mm

= 192 mm

Providing p= 70 mm < Pmax

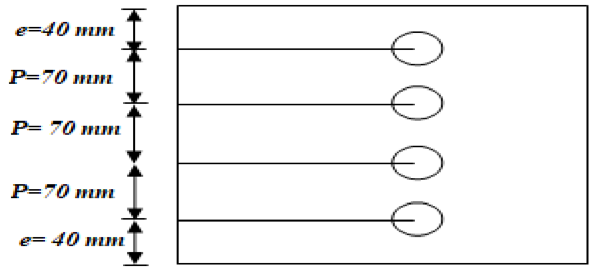
> Pmin

Step 6: To find edge distance of the rivet

According IS 800-1984, Page No: 97, C No: 8.10.2, Use Table No:8.2

Edge Distance =38 mm $\cong$ 40 mm

Step 7: To find minimum width of plate



Minimum width of plate = 40+70+70+70+40=290 mm

3) Double cover butt joint is used to connect the plates of 18 mm thickness to carry axial pull of 300 KN. Design riveted joint .

Solution: Given Data
Load= P=300 KN (Tensile)
t=18 mm
Assuming power driven shop rivet
Double cover butt joint
thickness of plate = 18 mm
IS 800-1984, Page No: 95, Table No: 8.1
Assuming single rivet N=1

$\tau_{vf} = 100$ N/mm^2

$\sigma_{bf} = 300$ N/mm^2

Assu min *g* fy= 250 N/mm^2, $\sigma_{at} =$ 0.6x fy = 0.6 x 250 $= 150$ N/mm^2

<u>Step 1:</u> To find nominal diameter of rivet (By using Unwins Formula)

t=Thickness = 18 mm

$d = 6.04\sqrt{t} = 6.04\sqrt{18} = 25.62 \text{ mm} \cong 26 \text{ mm}$

<u>Step 2:</u> To find gross diameter of rivet (Page No: 95, C. No: 8.9.3)

d= Nominal diameter of rivet

D= Gross diameter of rivet

D=26+2 mm=28 mm $[d \succ 25\ mm]$

<u>Step 3: To find rivet value</u>

Rivet value is least of P_s and P_b for single rivet i.e N=1

P_S = Number of rivets X Area of rivet in shearing X Permissible shear stress

$P_S = N$ x 2 x $\frac{\Pi}{4}$ x D^2 X τ_{Vf} For Double Shear

$P_S = 1$ x 2 x $\frac{\Pi}{4}$ x D^2 X τ_{Vf} = 1 x 2 x $\frac{\Pi}{4}$ x 28^2 X 100 $= 123.15$ x $10^3 N = 123.15 KN$

$P_b = N$ x D x t X σ_{bf} = 1x 28 x 18 x 300= 151.2 x $10^3 N = 151.2$ KN

Where N = Number of rivets in a joint

D = Gross diameter of Rivet

τ_{Vf} = Permissible shear stress in rivet (IS 800-1984 Page No: 95, Table :8.1)

σ_{bf} = Permissible bearing stress in rivet (IS 800-1984 Page No: 95, Table :8.1)

Rivet Value= 123.15 KN

Step 4: To find number of rivets

$$N = \frac{Total \text{ Load}}{Rivet \text{ Value}} = \frac{300}{123.15} = 2.43 \cong 3$$

Providing three rivets on either side of joint in one row

Step 5: To find pitch of the rivet

$(p - D)$ x t x σ_{at} = $Number$ of rivet per pitch x Rivet value

(p-28) x 18 x 150 = 1 x 123.15 x 10^3

$p =$ 73.6 mm

According IS 800-1984, Page No: 96, C No: 8.10.1

Minimum pitch (p_{min})==2.5 d=2.5 x 26=65 mm

Maximum Pitch (p_{max}) = 16 t or 200 mm (which is less)

= 16 x 18 = 288 or 200 mm

= 200 mm

Providing p= 70 mm < Pmax

>Pmin

Step 6: To find edge distance of the rivet

According IS 800-1984, Page No: 97, C No: 8.10.2, Use Table No:8.2

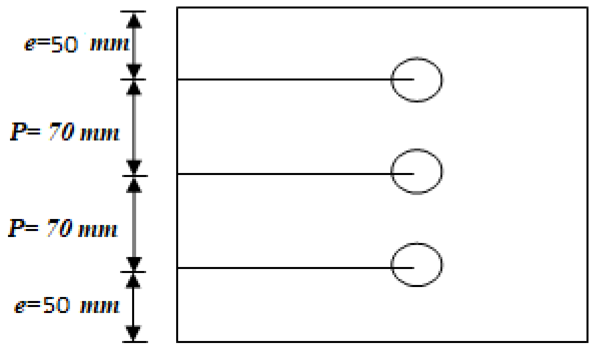
D	Edge Distance
25.5	44
28	?
29	51

By interpolation

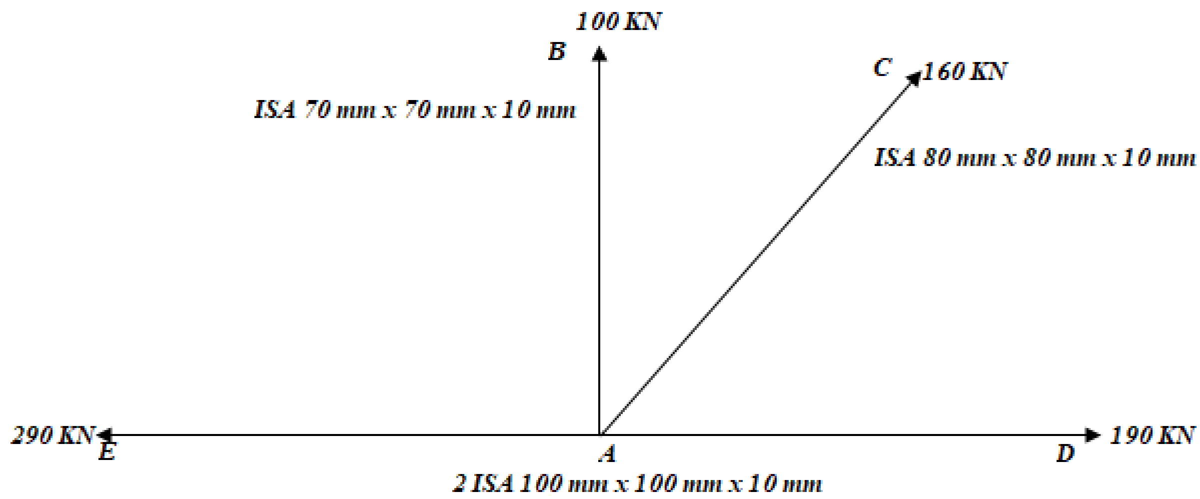
$$Edge \text{ Distance} = 44 + \left[\frac{(51-44)}{(29-25.5)} \text{ X (28-25.5)} \right] = 49 \text{ mm}$$

Edge Distance =49mm $\cong$ 50mm

Step 7: To find minimum width of plate
Minimum width of plate = 50+70+70+70+50=240 mm



4) A lower chord of the truss has vertical member AB, Diagonal member AC meeting at a point A as shown in figure along with axial force. Design joint A using hand driven field rivet. Assume Fy= 250 N/mm^2.



Solution: Given Data
For hand driven field rivets
IS 800-1984, Page No: 95, Table No: 8.1
Assume the thickness of plate = 12 mm

$\tau_{vf} = 80$ N/mm^2, for field rivet reduced 10%

$\tau_{vf} = 72$ N/mm^2,

$\sigma_{bf} = 250$ N/mm^2, for field rivet reduced 10%

$\sigma_{bf} = 225$ N/mm^2

Assu min *g* fy= 250 N/mm^2, σ_{at} = 0.6x fy = 0.6 x 250 = 150 N/mm^2

For member AB

Load =100 KN
Thickness = t= 10 mm (Least thickness 10 mm or 12 mm)

Step 1: To find nominal diameter of rivet (By using Unwins Formula)

t=Thickness = 10 mm

$d = 6.04\sqrt{t} = 6.04\sqrt{10}$ =19.1 mm $\cong$ 20 mm

Step 2: To find gross diameter of rivet (Page No: 95, C. No: 8.9.3)

d= Nominal diameter of rivet

D= Gross diameter of rivet

D=20+1.5 mm=21.5 mm $[d \leq 25\ mm]$

Step 3: To find rivet value

Rivet value is least of P_s and P_b for single rivet i.e N=1

P_S = Number of rivets X Area of rivet in shearing X Permissible shear stress

$P_S = N$ x $\frac{\Pi}{4}$ x D^2 X τ_{Vf} For single Shear

$P_S = 1$ x $\frac{\Pi}{4}$ x D^2 X τ_{Vf} = 1 x $\frac{\Pi}{4}$ x 21.5^2 X72 = 26.14 x 10$^3 N = 26.14 KN$

$P_b = N$ x D x t X σ_{bf} = 1x 21.5 x 10 x 225= 48.37 x 10$^3 N$ = 48.37 KN

Where N = Number of rivets in a joint

D = Gross diameter of Rivet

τ_{Vf} = Permissible shear stress in rivet (IS 800-1984 Page No: 95, Table :8.1)

σ_{bf} = Permissible bearing stress in rivet (IS 800-1984 Page No: 95, Table :8.1)

Rivet Value= 26.14 KN

Step 4: To find number of rivets

$$N = \frac{Total\ \text{Load}}{Rivet\ \text{Value}} = \frac{100}{26.14} = 3.8 \cong 4$$

Pr*oviding* four rivets on either side of joint in one row

Step 5: To find pitch of the rivet

$(p - D)$ x t x σ_{at} = *Number* of rivet per pitch x Rivet value

(p-21.5) x 10 x 150 = 1 x 26.14 x 10^3

$p =$ 38.9 mm

According IS 800-1984, Page No: 96, C No: 8.10.1

Minimum pitch (p_{min})==2.5 d=2.5 x 20=50 mm

Maximum Pitch (p_{max}) = 16 t or 200 mm (which is less)

= 16 x 10 = 160 or 200 mm

= 160 mm

Providing p= 50 mm < Pmax

> Pmin

Step 6: To find edge distance of the rivet

According IS 800-1984, Page No: 97, C No: 8.10.2, Use Table No:8.2

Edge Distance =32mm = 35 mm

For member AC

Load =160 KN

Thickness = t= 10 mm (Least thickness 10 mm or 12 mm)

Step 1: To find nominal diameter of rivet (By using Unwins Formula)

t=Thickness of plate= 10 mm

$d = 6.04\sqrt{t} = 6.04\sqrt{10}$ =19.1 mm ≅ 20 mm

Step 2: To find gross diameter of rivet (Page No: 95, C. No: 8.9.3)

d= Nominal diameter of rivet

D= Gross diameter of rivet

D=20+1.5 mm=21.5 mm $[d \leq 25\ mm]$

Step 3: To find rivet value

Rivet value is least of P_s and P_b for single rivet i.e N=1

P_S = Number of rivets X Area of rivet in shearing X Permissible shear stress

$P_S = N \text{ x} \frac{\Pi}{4} \text{ x D}^2 \text{ X } \tau_{Vf}$ For single Shear

$P_S = 1 \text{ x} \frac{\Pi}{4} \text{ x D}^2 \text{ X } \tau_{Vf} = 1 \text{ x} \frac{\Pi}{4} \text{ x } 21.5^2 \text{ X}72 = 26.14 \text{ x } 10^3 N = 26.14 KN$

$P_b = N$ x D x t X σ_{bf} = 1x 21.5 x 10 x 225= 48.37 x $10^3 N = 48.37$ KN

Where N = Number of rivets in a joint

D = Gross diameter of Rivet

τ_{Vf} = Permissible shear stress in rivet (IS 800-1984 Page No: 95, Table :8.1)

σ_{bf} = Permissible bearing stress in rivet (IS 800-1984 Page No: 95, Table :8.1)

Rivet Value= 26.14 KN

Step 4: To find number of rivets

$$N = \frac{Total \text{ Load}}{Rivet \text{ Value}} = \frac{160}{26.14} = 6.12 \cong 8$$

Pr*oviding* four rivets on either side of joint in two row

Step 5: To find pitch of the rivet

$(p-D)$ x t x σ_{at} = $Number$ of rivet per pitch x Rivet value

(p-21.5) x 10 x 150 = 2 x 26.14 x 10^3 (use 2 because rivet are providinng in two rows)

$p =$ 56.55 mm

According IS 800-1984, Page No: 96, C No: 8.10.1

Minimum pitch (p_{min})= =2.5 d=2.5 x 20=50 mm

Maximum Pitch (p_{max}) = 16 t or 200 mm (which is less)

= 16 x 10 = 160 or 200 mm

= 160 mm

Providing p= 55 mm < Pmax

> Pmin

Step 6: To find edge distance of the rivet

According IS 800-1984, Page No: 97, C No: 8.10.2, Use Table No:8.2

Edge Distance =32mm $\cong$ 35 mm

For member EAD

Member EAD is continuous

Net force =290-190 = 100 KN

Load =100 KN
Double angle section is used
Thickness = t= 12 mm (Least thickness 10+10= 20 mm or 12 mm)

Step 1: To find nominal diameter of rivet (By using Unwins Formula)

t=Thickness of plate= 12 mm

$d = 6.04\sqrt{t} = 6.04\sqrt{12}$ =20.92 mm $\cong$ 22 mm

Step 2: To find gross diameter of rivet (Page No: 95, C. No: 8.9.3)

d= Nominal diameter of rivet

D= Gross diameter of rivet

D=22+1.5 mm=23.5 mm $\left[d \le 25\ mm\right]$

Step 3: To find rivet value

Rivet value is least of P_s and P_b for single rivet i.e N=1

P_S = Number of rivets X Area of rivet in shearing X Permissible shear stress

$P_S = 2x\ N\ \text{x}\ \frac{\Pi}{4}\ \text{x}\ \text{D}^2\ \text{X}\ \tau_{Vf}$ For Double Shear

$P_S = 2x1\ \text{x}\ \frac{\Pi}{4}\ \text{x}\ \text{D}^2\ \text{X}\ \tau_{Vf} = 2\ \text{x}1\ \text{x}\ \frac{\Pi}{4}\ \text{x}\ 23.5^2\ \text{X}72 = 62.45\ \text{x}\ 10^3 N = 62.45KN$

$P_b = N$ x D x t X σ_{bf} = 1x 23.5 x 12 x 225= 63.45 x $10^3 N = 63.45\ KN$

Where N = Number of rivets in a joint

$D =$ Gross diameter of Rivet

τ_{Vf} = Permissible shear stress in rivet (IS 800-1984 Page No: 95, Table :8.1)

σ_{bf} = Permissible bearing stress in rivet (IS 800-1984 Page No: 95, Table :8.1)

Rivet Value= 62.45 KN

Step 4: To find number of rivets

$N = \frac{Total\ \text{Load}}{Rivet\ \text{Value}} = \frac{100}{62.45} = 1.60 \cong 2$

Pr*oviding* two rivets on either side of joint in one row

<u>Step 5:</u> To find pitch of the rivet

$(p-D)$ x t x σ_{at} = $Number$ of rivet per pitch x Rivet value

(p-23.5) x 12 x 150 = 1 x 62.45 x 10^3

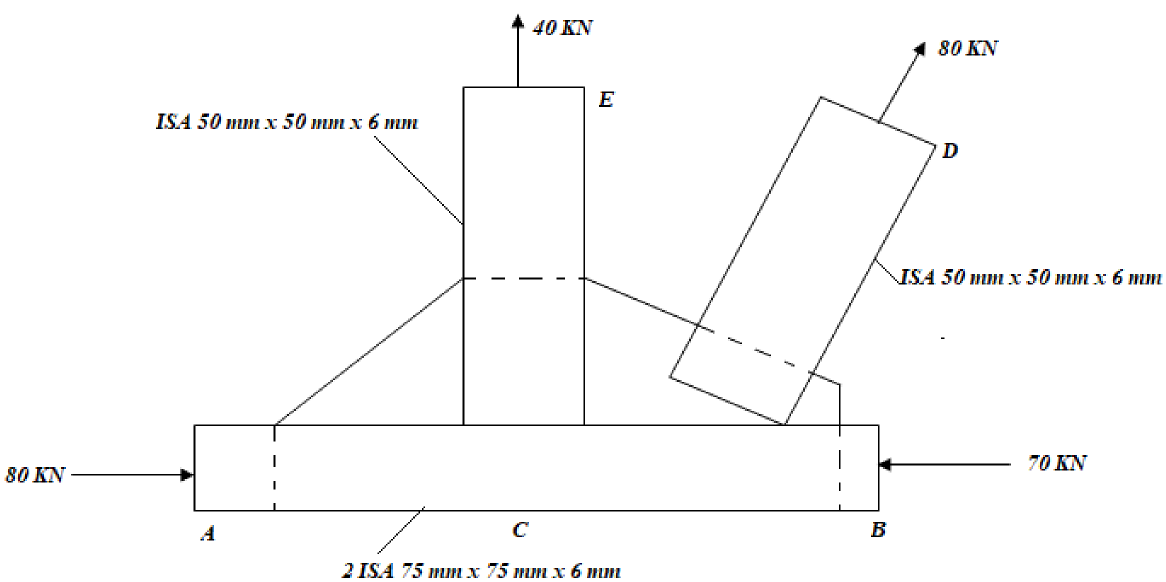
$p =$ 58.16 mm

According IS 800-1984, Page No: 96, C No: 8.10.1
Minimum pitch (p_{min})= =2.5 d=2.5 x 22=55 mm
Maximum Pitch (p_{max}) = 16 t or 200 mm (which is less)
= 16 x 12 = 192 or 200 mm
= 192 mm
Providing P= 55 mm <Pmax
> Pmin

<u>Step 6:</u> To find edge distance of the rivet

According IS 800-1984, Page No: 97, C No: 8.10.2, Use Table No:8.2
Edge Distance =38mm ≅ 40 mm

5) Joint of roof truss is shown in figure. Design riveted connection



Solution: Given Data
Assuming power driven shop rivet
IS 800-1984, Page No: 95, Table No: 8.1
Assume the thickness of plate = 10 mm

$\tau_{vf} = 100$ N/mm^2

$\sigma_{bf} = 300$ N/mm^2

$Assu\min g$ fy= 250 N/mm^2, σ_{at} = 0.6x fy = 0.6 x 250 $= 150$ N/mm^2

<u>For member CE</u>

Load =40 KN
Thickness = t= 6mm (Least thickness 6 mm or 10 mm)

<u>Step 1:</u> To find nominal diameter of rivet (By using Unwins Formula)

t=Thickness = 6 mm

$d = 6.04\sqrt{t} = 6.04\sqrt{6} = 14.79 \text{ mm} \cong 16 \text{ mm}$

<u>Step 2:</u> To find gross diameter of rivet (Page No: 95, C. No: 8.9.3)

d= Nominal diameter of rivet

D= Gross diameter of rivet

D=16+1.5 mm=17.5 mm $[d \leq 25\ mm]$

<u>Step 3: To find rivet value</u>

Rivet value is least of P_s and P_b for single rivet i.e N=1

P_S = Number of rivets X Area of rivet in shearing X Permissible shear stress

$P_S = N$ x $\frac{\Pi}{4}$ x D^2 X τ_{Vf} For single Shear

$P_S = 1$ x $\frac{\Pi}{4}$ x D^2 X τ_{Vf} = 1 x $\frac{\Pi}{4}$ x 17.5^2 X100 $= 24.052$ x $10^3 N = 24.052 KN$

$P_b = N$ x D x t X σ_{bf} = 1x 17.5 x 6 x 300= 31.5 x $10^3 N = 31.5\ KN$

Where N = Number of rivets in a joint

$D =$ Gross diameter of Rivet

τ_{Vf} = Permissible shear stress in rivet (IS 800-1984 Page No: 95, Table :8.1)

σ_{bf} = Permissible bearing stress in rivet (IS 800-1984 Page No: 95, Table :8.1)

Rivet Value= 24.052 KN

<u>Step 4:</u> To find number of rivets

$$N = \frac{Total\ \text{Load}}{Rivet\ \text{Value}} = \frac{40}{24.06} = 1.66 \cong 2$$

Pr*oviding* two rivets on either side of joint in one row

Step 5: To find pitch of the rivet

$(p - D)$ x t x σ_{at} = *Number* of rivet per pitch x Rivet value

(p-17.5) x 6 x 150 = 1 x 24.052 x 10^3

$p =$ 44.22 mm

According IS 800-1984, Page No: 96, C No: 8.10.1

Minimum pitch (p_{min})=2.5 d=2.5 x 16=40 mm

Maximum Pitch (p_{max}) = 16 t or 200 mm (which is less)

= 16 x 6 = 96 or 200 mm

= 96 mm

Providing p= 40 mm < Pmax

> Pmin

Step 6: To find edge distance of the rivet

According IS 800-1984, Page No: 97, C No: 8.10.2, Use Table No:8.2

Edge Distance =29mm $\cong$ 30 mm

For member CD

Load =80 KN

Thickness = t= 6mm (Least thickness 6 mm or 10 mm)

Step 1: To find nominal diameter of rivet (By using Unwins Formula)

t=Thickness = 6 mm

$d = 6.04\sqrt{t} = 6.04\sqrt{6} = 14.79 \text{ mm} \cong 16 \text{ mm}$

Step 2: To find gross diameter of rivet (Page No: 95, C. No: 8.9.3)

d= Nominal diameter of rivet

D= Gross diameter of rivet

D=16+1.5 mm=17.5 mm $[d \le 25\ mm]$

Step 3: To find rivet value

Rivet value is least of P_s and P_b for single rivet i.e N=1

P_S = Number of rivets X Area of rivet in shearing X Permissible shear stress

$P_S = N \text{ x } \frac{\Pi}{4} \text{ x } D^2 \text{ X } \tau_{Vf}$ For single Shear

$P_S = 1 \text{ x } \frac{\Pi}{4} \text{ x } D^2 \text{ X } \tau_{Vf} = 1 \text{ x } \frac{\Pi}{4} \text{ x } 17.5^2 \text{ X}100 = 24.052 \text{ x } 10^3 N = 24.052 KN$

$P_b = N \text{ x D x t X } \sigma_{bf}$ = 1x 17.5 x 6 x 300= 31.5 x $10^3 N = 31.5\ KN$

Where N = Number of rivets in a joint

D = Gross diameter of Rivet

τ_{Vf} = Permissible shear stress in rivet (IS 800-1984 Page No: 95, Table :8.1)

σ_{bf} = Permissible bearing stress in rivet (IS 800-1984 Page No: 95, Table :8.1)

Rivet Value= 24.052 KN

Step 4: To find number of rivets

$$N = \frac{Total \text{ Load}}{Rivet \text{ Value}} = \frac{80}{24.06} = 3.32 \cong 4$$

Providing four rivets on either side of joint in one row

Step 5: To find pitch of the rivet

$(p - D) \text{ x } t \text{ x } \sigma_{at} = Number$ of rivet per pitch x Rivet value

(p-17.5.5) x 6 x 150 = 1 x 24.052 x 10^3

p = 44.22 mm

According IS 800-1984, Page No: 96, C No: 8.10.1

Minimum pitch (p_{min})=2.5 d=2.5 x 16=40 mm

Maximum Pitch (p_{max})= 16 t or 200 mm (which is less)

= 16 x 6 = 96 or 200 mm

= 96 mm

Providing p= 40 mm < Pmax

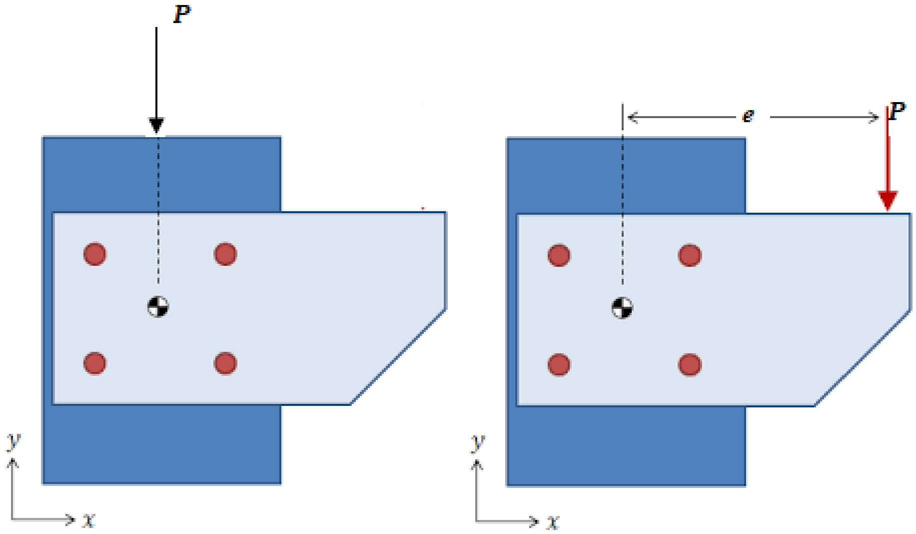
> Pmin

Step 6: To find edge distance of the rivet

According IS 800-1984, Page No: 97, C No: 8.10.2, Use Table No:8.2

Edge Distance =29mm $\cong$ 30 mm

Eccentrically Loaded riveted joint: A riveted joint is said to be eccentrically loaded when the line of action of applied load does not pass through C.G. of rivet. Eccentricity is the perpendicular distance from C.G. of rivet to line of action load



Axially Loaded **Eccentrically Loaded**

Eccentric load P can be replaced by
1) An axial load P passing through C.G. of rivet and parallel to line of action o load
2) A Moment M =P x e
Design Procedure:

<u>Step 1</u>: Direct Force $F_1 = \frac{Total \text{ Load}}{Number \text{ of } Rivet} = \frac{P}{N}$

<u>Step 2</u>: Bending Force

$$F_2 = \frac{M\ r_1}{\sum r^2}$$

$r_1 = D$istance of critical rivet from the C.G. of rivet

$$\sum r^2 = r_1^2 + r_2^2 + r_3^2 + r_4^2$$

$M = $ P x e

Step 3: Resultant Force:

$$F_R = \sqrt{F_1^2 + F_2^2 + 2F_1F_2 \text{ Cos}\,\theta} \qquad (1)$$

$$F_R = \frac{\pi}{4} \text{ x } \mathrm{D}^2 \text{ x } \tau_{\mathrm{vf}} \qquad (2)$$

$D =$ Gross Diameter of rivet

$\tau_{\mathrm{vf}} =$ Permisible shear stress

$\theta =$ Minimum for rivet 1, hence rivet 1 is critical rivet

Equate 1 and 2

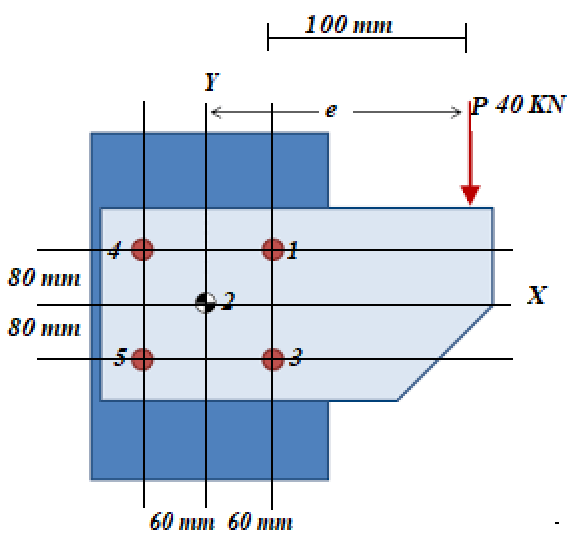
$$\frac{\pi}{4} \text{ x } \mathrm{D}^2 \text{ x } \tau_{\mathrm{vf}} = \sqrt{F_1^2 + F_2^2 + 2F_1F_2 \text{ Cos}\,\theta}$$

$D = ?$

$d = ?$

NOTE: Rivet is critical when Angle θ is minimum

1) **Figure shows a bracket connection. Find diameter if shear stress in rivet is not to exceed 80 N/mm^2.**



Solution:

Eccentricity e= 100+60= 160 mm

Rivet 1 is critical rivet

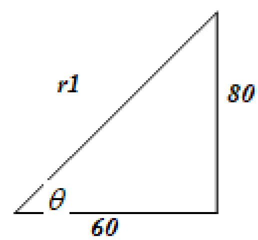
Load =40 KN =40 x 10^3 N

Number of rivet N= 5

Step 1: Direct Force $F_1 = \frac{Total\ \text{Load}}{Number\ \text{of}\ Rivet} = \frac{P}{N} = \frac{40 \text{ x } 10^3}{5} = 8 \text{ x } 10^3 N$

Step 2: Bending Force

$$F_2 = \frac{M\ r_1}{\sum r^2}$$



$M = $ P x e= 40 x 10^3 x 160 = 640 x 10^3 Nmm

$r_1 = D$istance of critical rivet from the C.G. of rivet

$r_1 = \sqrt{60^2 + 80^2} = $ 100 mm

$r_1 = r_3 = r_4 = r_5$ = 100 mm

r_2 =0 mm

$$\sum r^2 = r_1^2 + r_2^2 + r_3^2 + r_4^2 + r_5^2 = 4 \text{ x}(100)^2 + 1 \text{ x } (0)^2 = 40 \text{ x } 10^3 mm^2$$

$$F_2 = \frac{M\ r_1}{\sum r^2} = \frac{640 \text{ x } 10^3 \text{ x } 100}{40 \text{ x } 10^3} = 16 \text{ x } 10^3 N$$

Step 3: Resultant Force:

$$F_R = \sqrt{F_1^2 + F_2^2 + 2F_1F_2 \text{ Cos}\theta} = \sqrt{(8 \text{ x } 10^3)^2 + (16 \text{ x } 10^3)^2 + 2 \text{ x } 8 \text{ x } 10^3 \text{ x } 16 \text{ x } 10^3 \text{ x } (0.6)}$$

$F_R = $ 21.76 x 10^3 N (1)

Cos $\theta = \frac{60}{100} = 0.6$

$F_R = \frac{\pi}{4}$ x D^2 x τ_{vf} (2)

$D = $ Gross Diameter of rivet

$\tau_{vf} = $ Permisible shear stress

$\theta = $ Minimum for rivet 1, hence rivet 1 is critical rivet

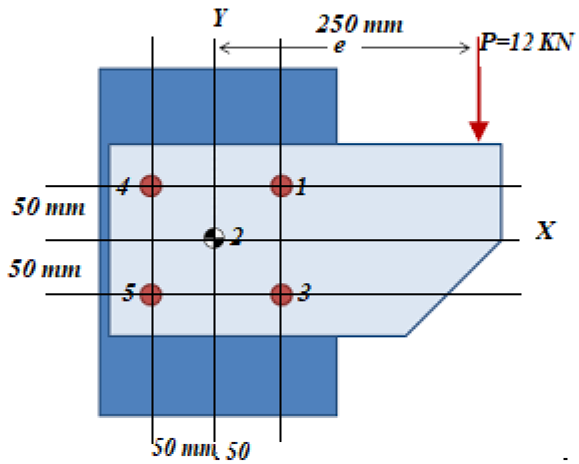
Equate 1 and 2

$$\frac{\pi}{4} \text{ x } D^2 \text{ x } \tau_{vf} = 21.76 \text{ x } 10^3$$

$D = 18.60\ mm$

$Nominal\ diameter = \text{d} = D - 1.5 = 18.60 - 1.5 = 17.10\ mm = 18$ mm

2) A bracket is riveted as shown in figure. Find permissible shear stress if the diameter of rivet is 20 mm.



Solution:

Eccentricity e= 250 mm, Diameter of Rivet =d= 20 mm

Rivet 1 is critical rivet

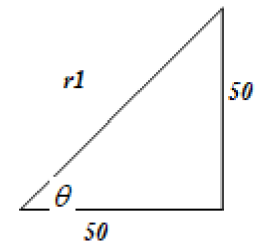
Load =12 KN =12 x 10^3 N

Number of rivet N= 5

Step 1: Direct Force $F_1 = \dfrac{\textit{Total}\ \text{Load}}{\textit{Number of Rivet}} = \dfrac{P}{N} = \dfrac{12 \text{ x } 10^3}{5} = 2.4 \text{ x } 10^3 N$

<u>Step 2</u>: Bending Force

$$F_2 = \frac{M\ r_1}{\sum r^2}$$



$M =$ P x e= 12 x 10^3 x 250 = 3000 x 10^3 Nmm

$r_1 =$ Distance of critical rivet from the C.G. of rivet

$r_1 = \sqrt{50^2 + 50^2} =$ 70.71 mm

$r_1 = r_3 = r_4 = r_5$ = 70.71 mm

r_2 =0 mm

$$\sum r^2 = r_1^2 + r_2^2 + r_3^2 + r_4^2 + r_5^2 = 4 \text{ x}(70.71)^2 + 1 \text{ x } (0)^2 = 20 \text{ x } 10^3 mm^2$$

$$F_2 = \frac{M\ r_1}{\sum r^2} = \frac{3000 \text{ x } 10^3 \text{ x } 70.71}{20 \text{ x } 10^3} = 10.61 \text{ x } 10^3 N$$

<u>Step 3</u>: Resultant Force:

$$F_R = \sqrt{F_1^2 + F_2^2 + 2F_1F_2 \text{ Cos}\,\theta} = \sqrt{(2.4 \text{ x } 10^3)^2 + (10.61 \text{ x } 10^3)^2 + 2 \text{ x } 2.4 \text{ x } 10^3 \text{ x } 10.61 \text{ x } 10^3 \text{ x } 0.707}$$

$F_R =$ 12.41 x 10^3 N (1)

$$\text{Cos } \theta = \frac{50}{70.71} = 0.707$$

$F_R = \frac{\pi}{4}$ x D^2 x τ_{vf} (2)

d=20 mm

$D =$ Gross Diameter of rivet= 20+1.5 =21.5 mm

$\tau_{vf} =$ Permisible shear stress

$\theta =$ Minimum for rivet 1, hence rivet 1 is critical rivet

Equate 1 and 2

$$\frac{\pi}{4} \text{ x } 21.5^2 \text{ x } \tau_{vf} = 12.41 \text{ x } 10^3$$

$$\tau_{vf} = 34.19 \text{ N/mm}^2 < 100 \text{ N/mm}^2 \text{ safe}$$

3) A bracket is riveted to the flange of a column. Find maximum value of P if stress in rivet is not exceed 100 Mpa. Diameter of rivet is 20 mm

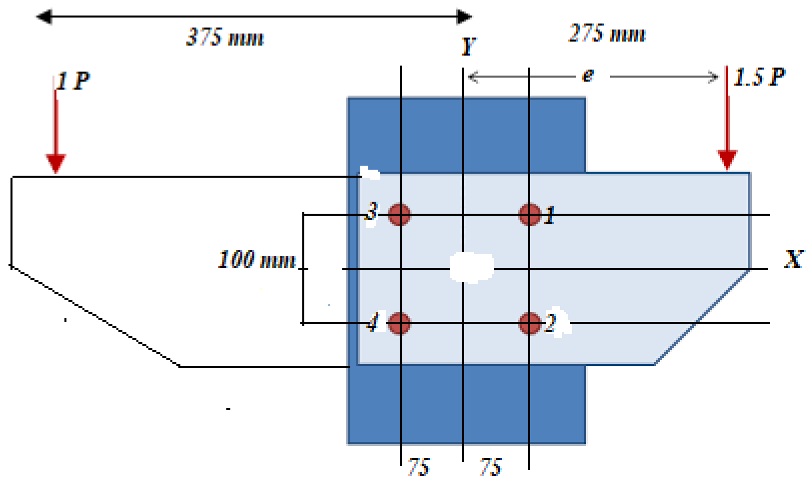
Solution: Total Load= 1 P+1.5 P = 2.5 P, Diameter of Rivet =d= 20 mm

Net B.M= M= 1.5 P x 275 - 1 P x 375 = 37.5 P Nmm (Clockwise)

No of Rivet =N= 4

$\tau_{vf} = 100\ \text{N/mm}^2$

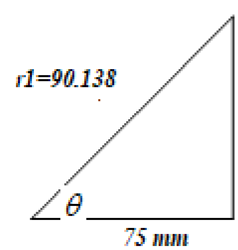
Rivet 1 is critical rivet



<u>Step 1</u>: Direct Force $F_1 = \dfrac{Total\ \text{Load}}{Number\ \text{of}\ Rivet} = \dfrac{2.5P}{4} = 0.625P\ \text{N}$

<u>Step 2</u>: Bending Force

$$F_2 = \frac{M\ r_1}{\sum r^2}$$



$M =$ 37.5 P N.mm

$r_1 =$ Distance of critical rivet from the C.G. of rivet

$r_1 = \sqrt{75^2 + 50^2} =$ 90.138 mm

$r_1 = r_2 = r_3 = r_4 =$ 90.138 mm

$\sum r^2 = r_1^2 + r_2^2 + r_3^2 + r_4^2 = 4 \text{ x}(90.138)^2 = 32.5 \text{ x } 10^3 mm^2$

$$F_2 = \frac{M\ r_1}{\sum r^2} = \frac{37.5 \text{ Px } 90.138}{32.5 \text{ x } 10^3} = 0.104PN$$

<u>Step 3</u>: Resultant Force:

$F_R = \sqrt{F_1^2 + F_2^2 + 2F_1F_2 \text{ Cos}\,\theta} = \sqrt{(0.625P)^2 + (0.104P)^2 + 2 \text{ x}0.625\text{Px}0.104 \text{ P x } 0.832}$

$F_R =$ 0.713 P N (1)

Cos $\theta = \dfrac{75}{90.138} = 0.832$

$F_R = \dfrac{\pi}{4} \text{ x D}^2 \text{ x } \tau_{\text{vf}}$ (2)

d=20 mm

$D =$ Gross Diameter of rivet= 20+1.5 =21.5 mm

$\tau_{\text{vf}} =$ Permisible shear stress

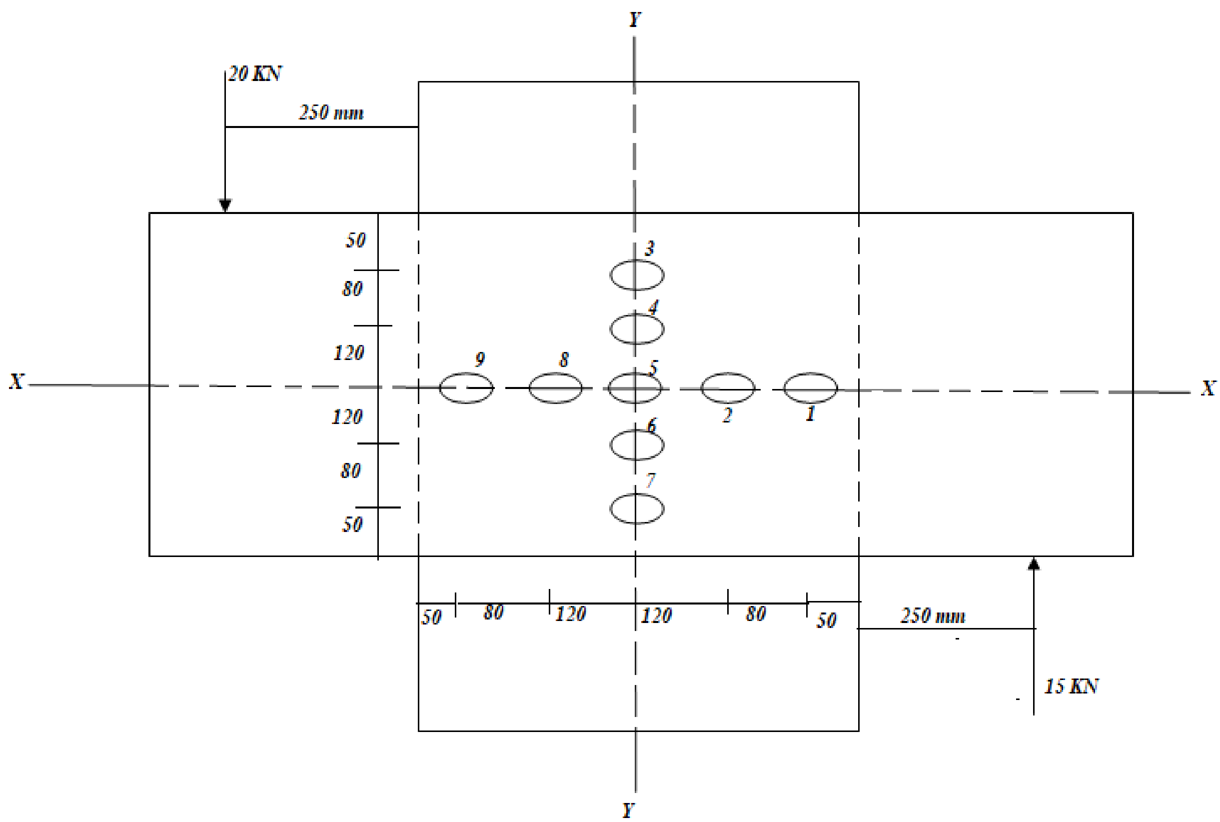
$\theta =$ Minimum for rivet 1, hence rivet 1 is critical rivet

Equate 1 and 2

$\dfrac{\pi}{4} \text{ x } 21.5^2 \text{ x } 100 = 0.713 \text{ P}$

$P = 50.91 \text{ x } 10^3$ N=50.91 KN

4) <u>Two plates are connected together by 18 mm diameter rivets as shown in figure. Find maximum shear induced in the critical rivet</u>



Solution: Eccentricity e= 250+250=500 mm

Net B.M= M= P x e =(20 x 10^3 x500)+ (15 x 10^3 x500)=17.5 x 10^6 Nmm (Anticlockwise)

Total Load = P =(20 x 10^3) -(15 x 10^3) = 5 x 10^3 N

Total Number of rivet N= 9

Rivet number 9 is critical rivet

Step 1: Direct Force $F_1 = \frac{Total\ \text{Load}}{Number\ \text{of}\ Rivet} = \frac{P}{N} = \frac{5 \text{ x } 10^3}{9} = 0.55 \text{ x } 10^3 N$

Step 2: Bending Force

$$F_2 = \frac{M \ r_9}{\sum r^2}$$

$M = 17.5 \text{ x } 10^6 Nmm$

r_9 = Distance of critical rivet from the C.G. of rivet

r_9 = 200 mm

$r_1 = r_9 = r_3 = r_7 = 200$ mm

$r_2 = r_4 = r_6 = r_8 = 120$ mm

$r_5 = 0$

$$\sum r^2 = r_1^2 + r_2^2 + r_3^2 + r_4^2 + r_5^2 + r_6^2 + r_7^2 + r_8^2 + r_9^2$$

$$\sum r^2 = 4 \text{ x}(200)^2 + 4 \text{ x}(120)^2 + 1 \text{ x}(0)^2 = 217.6 \text{ x } 10^3 mm^2$$

$$F_2 = \frac{M \ r_9}{\sum r^2} = \frac{17.5 \text{ x } 10^6 \text{x } 200}{217.6 \text{ x } 10^3} = 16.08 \text{ x } 10^3 N$$

<u>Step 3</u>: Resultant Force:

$$F_R = \sqrt{F_1^2 + F_2^2 + 2F_1F_2 \text{ Cos}\,\theta}$$

$$F_R = = \sqrt{(0.55 \text{ x } 10^3)^2 + (16.08 \text{ x } 10^3)^2 + 2 \text{ x}0.55 \text{ x } 10^3 \text{x}16.08 \text{ x } 10^3 \text{ x Cos } (0^0)}$$

$F_R = 16.63 \text{ x } 10^3$ N (1)

$\theta = 0^0$

$F_R = \frac{\pi}{4} \text{ x D}^2 \text{ x } \tau_{\text{vf}}$ (2)

d=18 mm

D = Gross Diameter of rivet= 18+1.5 =19.5 mm

τ_{vf} = Permisible shear stress

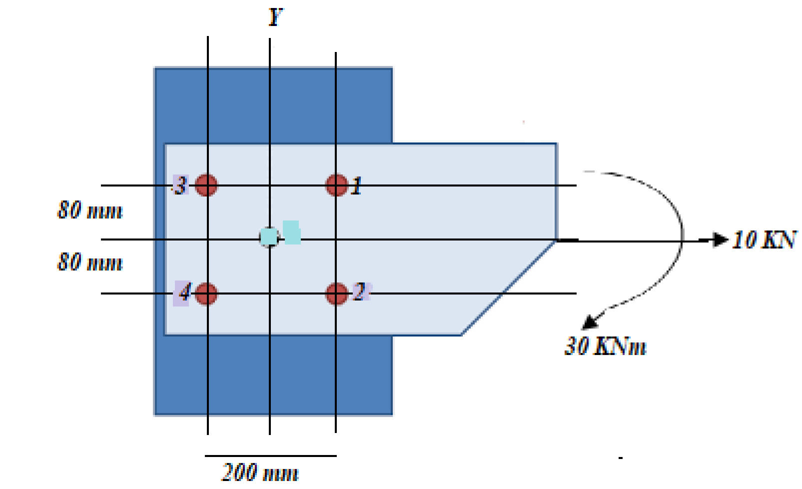
θ = Minimum for rivet 9, hence rivet 9 is critical rivet

Equate 1 and 2

$$\frac{\pi}{4} \text{ x } 19.5^2 \text{ x } \tau_{\text{vf}} = 16.63 \text{ x } 10^3$$

$\tau_{\text{vf}} = 55.67 \text{ N/mm}^2 < 0.4xFy = 0.4X\,250 = 100 \text{ N/mm}^2$ safe

5) A bracket connection is shown in figure. Find diameter of rivet required if permissible shear stress in rivet is 100 Mpa.



Solution: Total Load =10 KN

Moment = M= 30 KNm

τ_{vf} = 100 N/mm^2

Rivet 1 is critical rivet

Total number of rivet = N= 4

Step 1: Direct Force $F_1 = \dfrac{Total\ \text{Load}}{Number\ \text{of}\ Rivet} = \dfrac{P}{N} = \dfrac{10 \text{ x } 10^3}{4} = 2.5 \text{ x } 10^3 N$

Step 2: Bending Force

$$F_2 = \frac{M\ r_1}{\sum r^2}$$

$M = 30 \text{ x } 10^6 Nmm$

r_1 = Distance of critical rivet from the C.G. of rivet

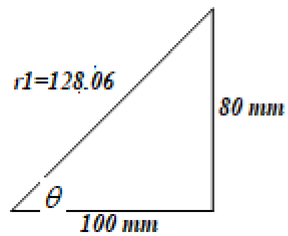
$r_1 = \sqrt{100^2 + 80^2} = 128.06$ mm

$r_1 = r_2 = r_3 = r_4 = 128.06$ mm

$\sum r^2 = r_1^2 + r_2^2 + r_3^2 + r_4^2$

$\sum r^2 = 4 \text{ x} (128.06)^2 = 65.59 \text{ x } 10^3 mm^2$

$$F_2 = \frac{M\ r_1}{\sum r^2} = \frac{30 \text{ x } 10^6 \text{x } 128.06}{65.59 \text{ x } 10^3} = 58.57 \text{ x } 10^3 N$$



Step 3: Resultant Force:

$$F_R = \sqrt{F_1^2 + F_2^2 + 2F_1F_2 \text{ Cos}\,\theta}$$

$$F_R = \sqrt{(2.5 \text{ x } 10^3)^2 + (58.57 \text{ x } 10^3)^2 + 2 \text{ x } 2.5 \text{ x } 10^3 \text{x } 58.57 \text{ x } 10^3 \text{ x } 0.780}$$

$F_R = 60.55 \text{ x } 10^3$ N (1)

$$\text{Cos}\,\theta = \frac{100}{128.06} = 0.780$$

$$F_R = \frac{\pi}{4} \text{ x D}^2 \text{ x } \tau_{\text{vf}} \qquad (2)$$

D = Gross Diameter of rivet

τ_{vf} = Permisible shear stress

θ = Minimum for rivet 1, hence rivet 1 is critical rivet

Equate 1 and 2

$$\frac{\pi}{4} \text{ x D}^2 \text{ x } 100 = 60.55 \text{ x } 10^3$$

$D = 27.66$ mm

d=D-2= 27.66-2=25.66 mm $\cong$ 26 mm

6) A bracket is connected to column as shown in figure. Find the resultant force on critical rivet and suggest diameter of rivet. Assuming power driven shop rivet.

Welded Joint

Welding : It is the process of joining two members by application of heat.

Welding is the least expensive process. It is widely used in fabrication work.

Advantages of Welding

1) The welding joint give more efficiency

2) The welding work is done very quickly

3) The welding joint is rigid joint

4) The welding is more economical

5) The appearance is good as compared to riveted joint

6) The noise is not created at the time of welding

Disadvantages of Welding

1) The inspection of welded joint is very difficult

2) Due to expansion and contraction of joint, the stress is developed in the welded joint

3) More skilled labour is required

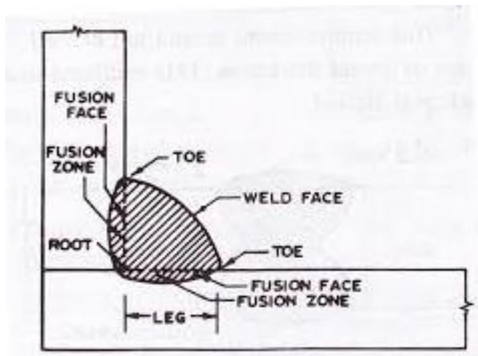
Types of welded joint

1) Fillet Weld 2) Butt Joint weld

1) Fillet Weld: -This type of weld is used when members to be connected overlap each other. Section of fillet weld for design purpose is taken as isosceles right angle triangle and equal side of triangle is called size of weld. Perpendicular distance between the hypogenous of triangle and opposite apex is called throat thickness.

$\text{Sin } 45^0 = t/S$

$t = S \times \text{Sin } 45^0 = 0.707\ S$ (S= Size of weld)



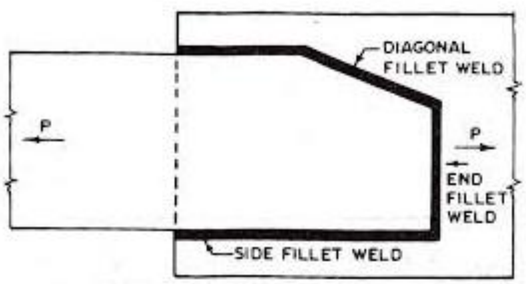
FILLET WELD

Types of Fillet weld

1) Side fillet weld

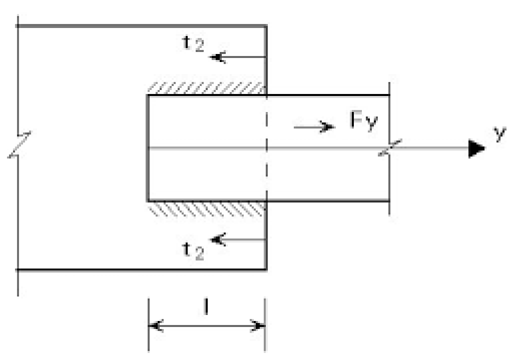
2) End fillet weld

3) Diagonal fillet weld



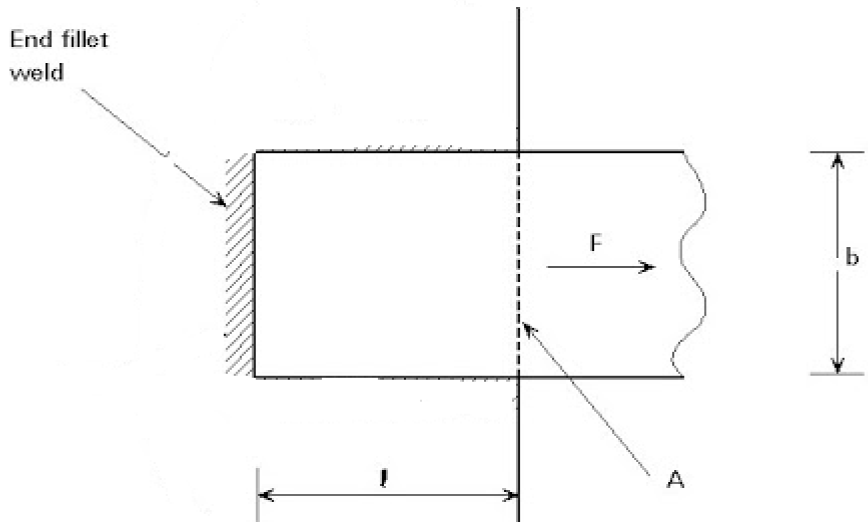
TYPES OF FILLET WELD

1) Side fillet weld: If the weld length is parallel to direction of force, weld is known as side fillet weld.

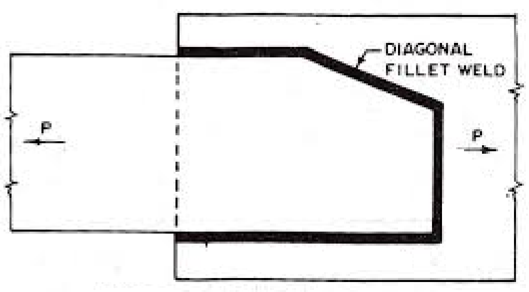


Side fillet welds

2) End fillet weld: If the length of weld is perpendicular to the direction of force the weld is known as end fillet weld.



3) Diagonal fillet weld: If the weld length is inclined to the direction of force , it is called diagonal fillet weld.



Strength of fillet weld:

The load carrying capacity of fillet weld is called strength of the weld.

P_s= Effective length X Throat thickness X Permissible shear stress in weld

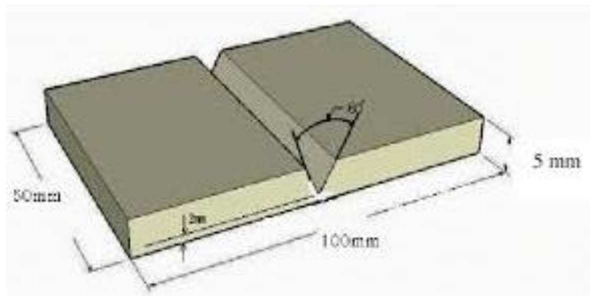
P_s= $L \times 0.707\ S \times \sigma_s$

For design purpose strength of weld per mm length, L =1

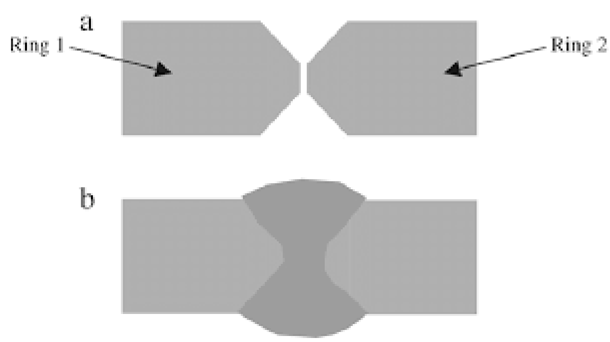
P_s= $1 \times 0.707\ S \times \sigma_s$

Butt Joint Weld: This type of weld is used when members to be connected touch each other, there are many types of butt weld depending on the shape.

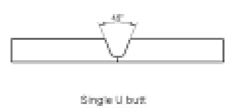
a) Single V- Butt weld



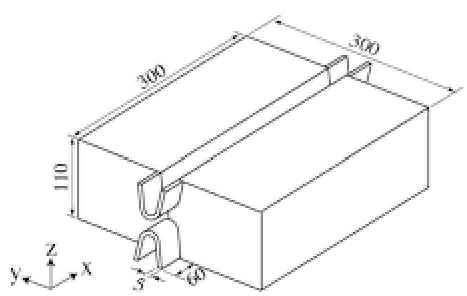
b) Double V- Butt weld



c) Single U- Butt weld



d) Double U- Butt weld



Strength of Butt weld:

P_s= Effective length X Throat thickness X Permissible shear stress in weld

In Butt weld, throat thickness is taken as thickness of thinner plate.

Same Specification:

1) Minimum size of weld= 5 mm

2) Maximum size of weld =Thickness of weld -1.5 mm

3) Spacing between the weld should be less than 16 t

4) Length of weld should be greater than perpendicular distance between weld

Type 1: Axially loaded welded joint

1) Symmetric Section

2) Unsymmetrical Section

1) Symmetric Section

Design Procedure

Given Data

Load= P

Permissible Shear stress= σ_s

Thickness of plate= t

Step 1: <u>Size of weld</u>

Minimum Size = 5 mm

Maximum Size =t-1.5

Step 2: <u>Strength of weld Per mm length</u>

P_s= 1 x 0.707 x S x σ_s

Step 3: <u>Length of the weld</u>

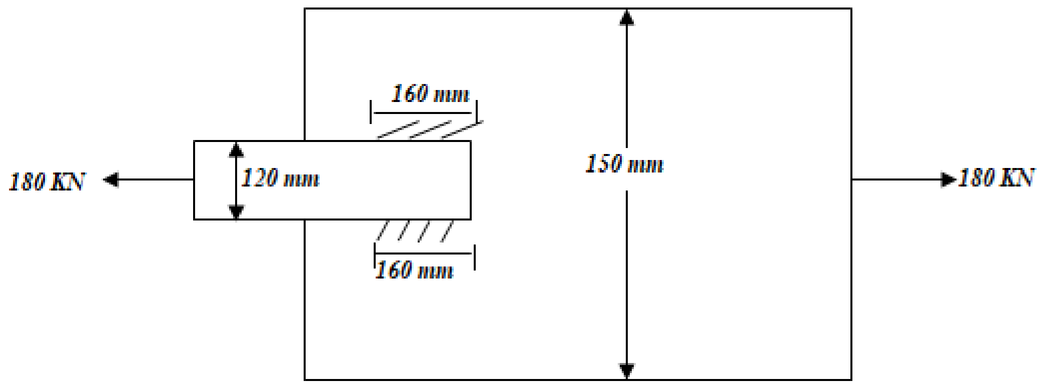
L = P/ P_s

Length of weld on each side= L/2

1) Design a longitudinal fillet weld to connect the plates shown in figure to transmit a pull of 180 KN. The permissible shear stress in weld is 100 N/mm^2. The plates are 10 mm thick.

Solution : Given Data

Load= P= 180 KN



Permissible Shear stress= σ_s=100 N/mm^2

Thickness of plate= t=10 mm

Step 1: <u>Size of weld</u>

Minimum Size = 5 mm

Maximum Size =t-1.5= 10-1.5= 8.5 mm

Assuming size of weld = S= 8 mm

Step 2: <u>Strength of weld per mm length</u>

P_s= 1 x 0.707 x S x σ_s

P_s= 1 x 0.707 x 8 x 100

P_s=565.6 N/mm

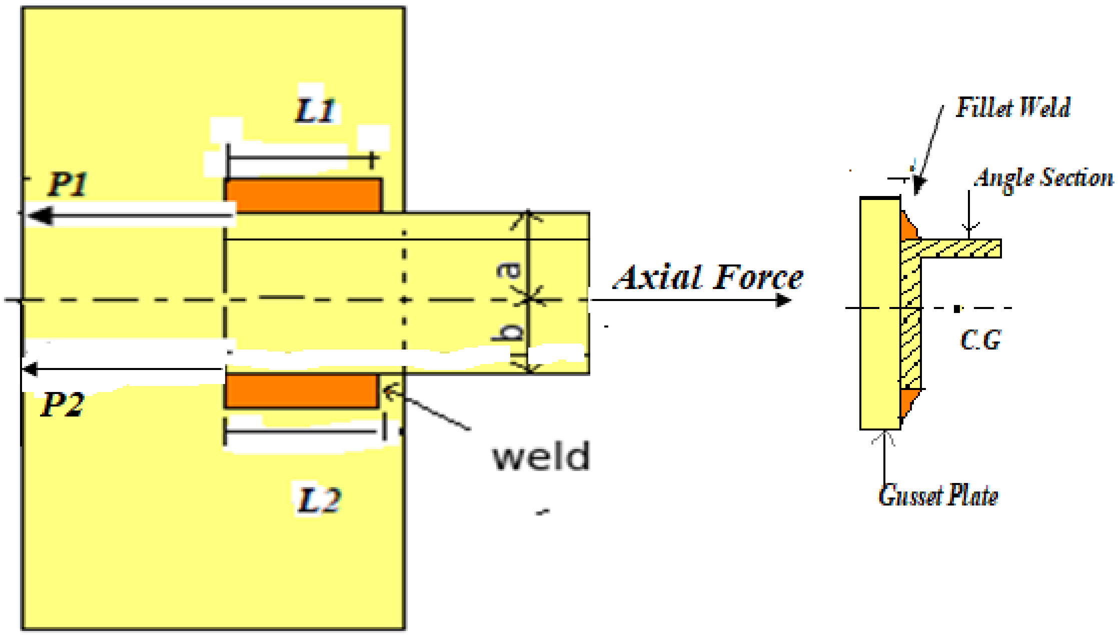
Step 3: <u>Length of the weld</u>

L = P/ P_s= (180 x 10^3)/ 565.6 =318.24 mm

Length of weld on each side= L/2=318.24/2 = 159.12 mm ≈ 160 mm

2) Unsymmetrical Section

Design Procedure



To find L_1 and L_2

When unsymmetrical section is connected by welding the fillet weld is provided such that the C.G of the weld coincide with the neutral axis.

P_1= Pull transmitted by length $\mathbf{L_1}$

P_2= Pull transmitted by length $\mathbf{L_2}$

$P_1 + P_2 = P$ (1)

Taking moment about P_2

$P \times b = P_1\ (a+b)$

P_1=?

$P_2 = P - P_1$ from equation (1)

Strength of weld per mm length $= P_s = 1 \times 0.707 \times S \times \sigma_s$

Where S = Size of Weld

σ_s= Permissible Shear stress

L_1 = P_1 / P_s **,** **L_2 =** P_2 / P_s

1) An ISA 80 x80 x 8 mm carrying axial load of 140 KN is welded to a gusset plate using 6 mm filet weld. Permissible shear stress in weld is 100 Mpa. Distance of C.G. of angle is 22. 7 mm from back. Calculate length of weld required.

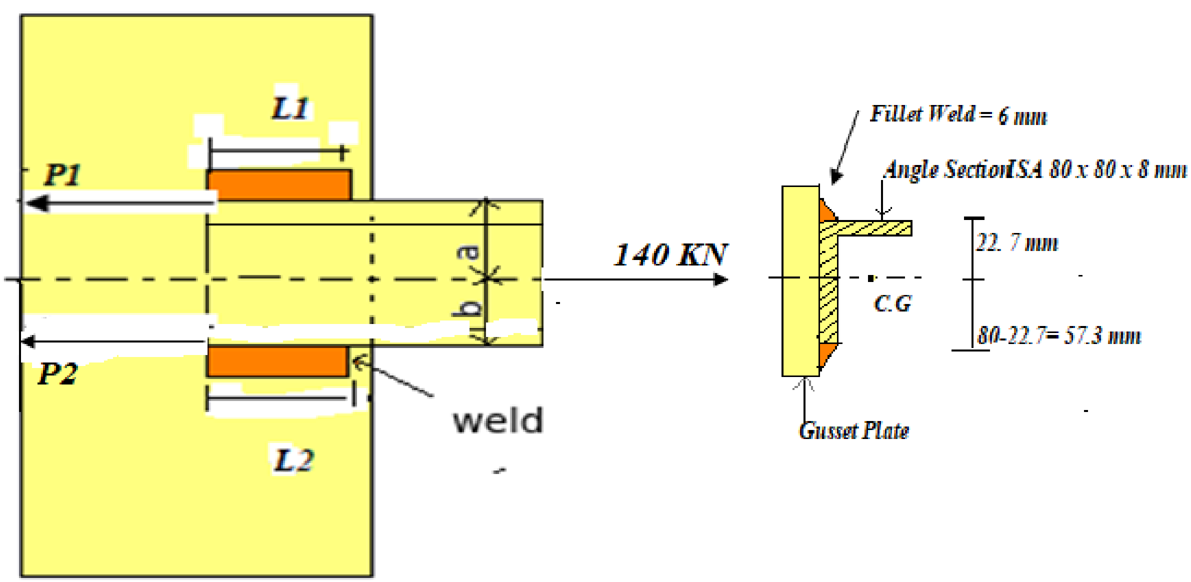
Solution: Given Data ISA 80 x80 x 8

Load = P= 140 KN

Permissible shear stress= σ_s=100 N/mm^2

Size of Weld= S= 6 mm

Distance of C.G. of angle is 22. 7 mm from back.



P_1= Pull transmitted by length **L_1**

P_2= Pull transmitted by length **L_2**

P_1+ P_2= P,

P_1+ P_2= 140 x 10^3 N (1)

Taking moment about P_2

140 x 10^3 x 57.3 = P_1 x 80

P_1=100.27 x 10^3 putting in equation (1)

P_2= 140 x 10^3-100.27 x 10^3 = 39.73 x 10^3 N

Strength of weld per mm length =P_s = 1 x 0.707 x S x σ_s

P_s = 1 x 0.707 x 6 x 100 = 424.2 N/mm

Weld length

L_1 = P_1 / P_s= 100.27 x 10^3/424.2 = 236.36 ≈ 240 mm

L_2 = P_2 / P_s= 39.73 x 10^3 /424.2 = 93.69 ≈ 95 mm

2) An ISA 100 x100 x 10 mm carrying axial load of 80 KN is welded to a gusset plate using 8 mm filet weld. Permissible shear stress in weld is 100 Mpa. Calculate length of weld required.

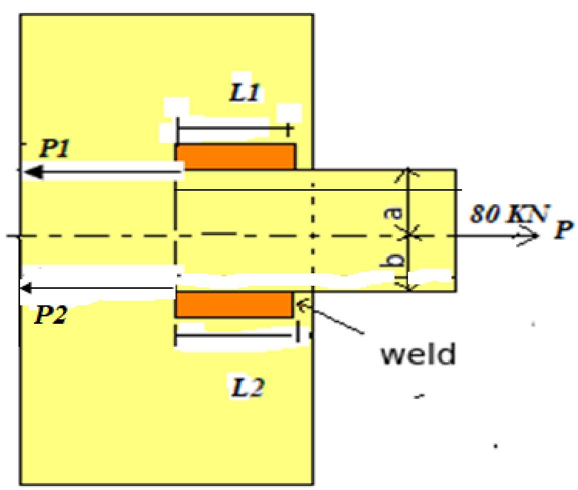
Solution: Given Data ISA 100 x100 x 10

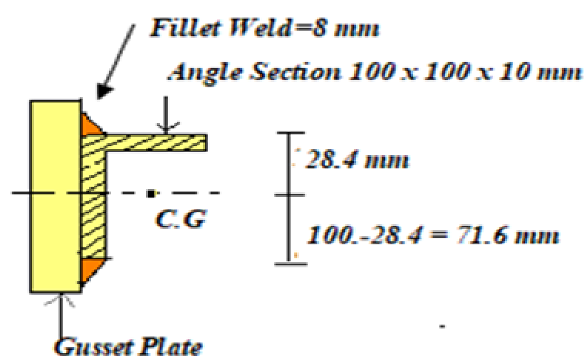
Load = P= 80 KN

Permissible shear stress= σ_s=100 N/mm^2

Size of Weld= S= 8 mm

Distance of C.G. of angle is 28. 4 mm from back. (from steel table)





P_1= Pull transmitted by length **L_1**

P_2= Pull transmitted by length **L_2**

P_1+ P_2= P,

$P_1 + P_2 = 80 \times 10^3$ N (1)

Taking moment about P_2

$80 \times 10^3 \times 71.6 = P_1 \times 100$

$P_1 = 57.28 \times 10^3$ putting in equation (1)

$P_2 = 80 \times 10^3 - 57.28 \times 10^3 = 22.72 \times 10^3$ N

Strength of weld per mm length $= P_s = 1 \times 0.707 \times S \times \sigma_s$

$P_s = 1 \times 0.707 \times 8 \times 100 = 565.6$ N/mm

Weld length

$\mathbf{L_1} = P_1 / P_s = 57.28 \times 10^3 / 565.6 = 101.27 \approx 105$ mm

$\mathbf{L_2} = P_2 / P_s = 22.72 \times 10^3 / 565.6 = 40.17 \approx 45$ mm

3) An ISA 125 x 95 x 10 mm is welded to a gusseted plate of thickness 12 mm, longer leg is connected to gusset plate, the angle section is subjected to a force of 120 KN. Permissible shear stress in weld is 100 Mpa. Design the welded joint.

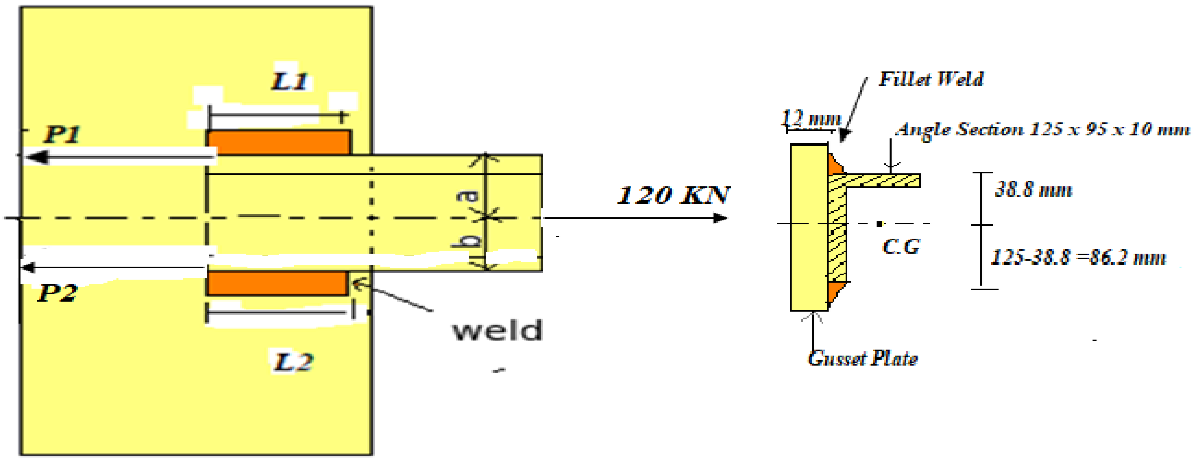
Solution: Given Data ISA 125 x 95 x 10

Load = P= 120 KN

Permissible shear stress= $\sigma_s = 100$ N/mm^2

Thickness of gusset plate= 12 mm

Distance of C.G. of angle is 38.8 mm from back. (from steel table)



P_1= Pull transmitted by length $\mathbf{L_1}$

P_2= Pull transmitted by length $\mathbf{L_2}$

$P_1 + P_2 = P$,

$P_1 + P_2 = 120 \times 10^3$ N (1)

Taking moment about P_2

$120 \times 10^3 \times 86.2 = P_1 \times 125$

$P_1 = 82.752 \times 10^3$ putting in equation (1)

$P_2 = 120 \times 10^3 - 82.752 \times 10^3 = 37.248 \times 10^3$ N

Strength of weld per mm length $= P_s = 1 \times 0.707 \times S \times \sigma_s$

Minimum size of weld = 5 mm

Maximum Size =10-1.5=8.5 mm (10 mm angle thickness and 12 mm gusset plate thickness, take least)

Assuming Size of weld =S= 8 mm

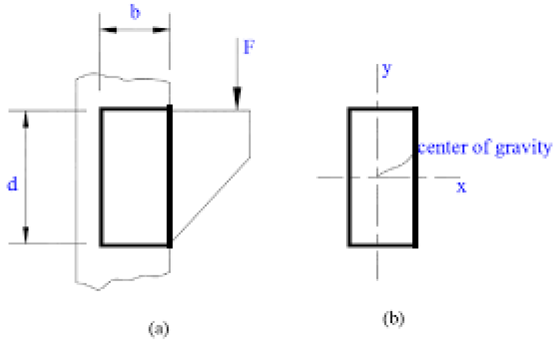
$P_s = 1 \times 0.707 \times 8 \times 100 = 565.6$ N/mm

Weld length

$\mathbf{L_1} = P_1 / P_s = 82.752 \times 10^3 / 565.6 = 146.30 \approx 150$ mm

L_2 = P_2 / P_s= 37.248 x 10^3 /565.6 = 65.85 ≈ 70 mm

Eccentrically Welded Connections



Design Procedure

Step 1: Direct force per mm length

$$F_1 = \frac{Total\ \text{Load}}{Total\ \text{Length of weld}} = \frac{P}{L}$$

Step 2: Bending force per mm length

$$F_2 = \frac{M\ r}{I_P}$$

$M = P$ x e

$I_P =$ Polar moment of Inertia

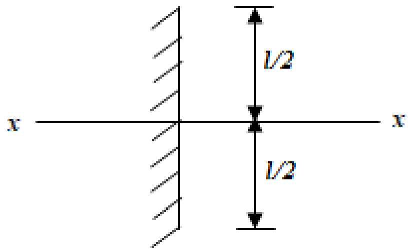
$I_P = I_X + I_Y$

$I_X = M.I$ of weld about X- axis

$I_y = M.I$ of weld about Y- axis

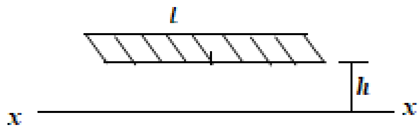
If weld length is perpendicular to the axis

Moment of Inertia = $\dfrac{(Length)^3}{12}$



If weld length is parallel to the axis

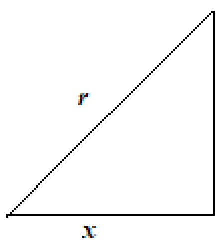
Moment of Inertia = $(l \text{ x } h)^2$



<u>Step 3</u>: Resultant Force:

$$F_R = \sqrt{F_1^2 + F_2^2 + 2F_1F_2 \,\mathrm{Cos}\,\theta}$$

Cos $\theta = \dfrac{x}{r}$

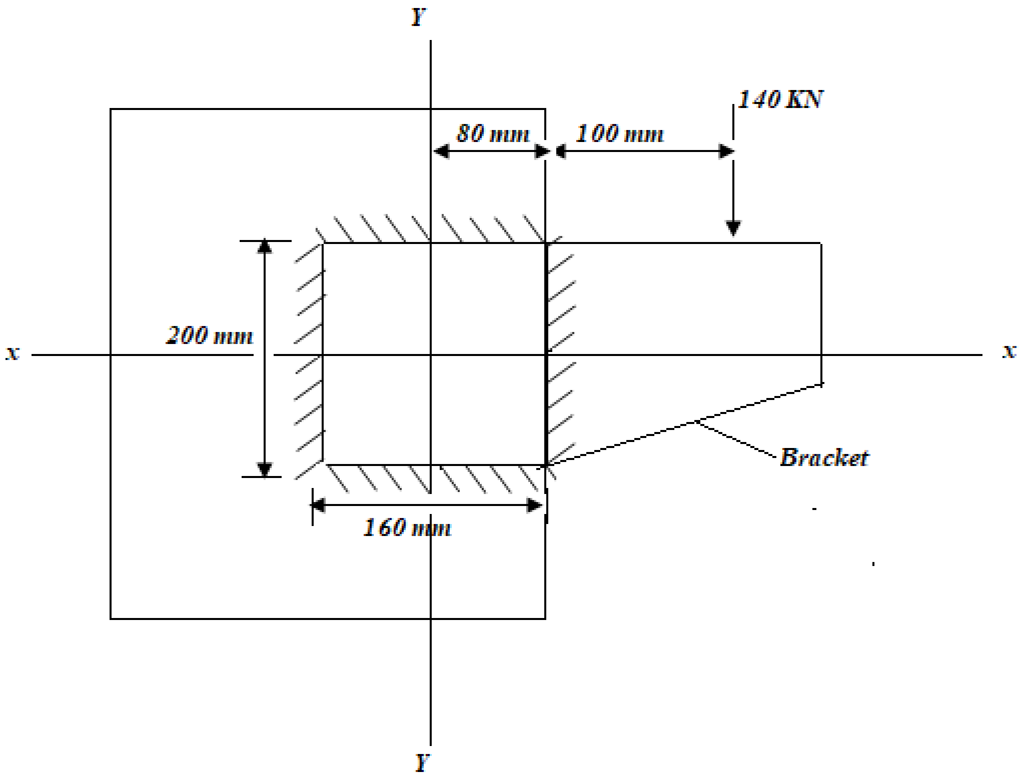


<u>Step 4</u>: Resultant Force: Fr= 0.707 x S x σ_S

S = Size of weld

σ_S = Permissible shear stress in weld

1) A bracket plate is welded to a column carries a load of 140 KN as shown in figure. If permissible shear stress is 100 Mpa. Find size of weld



Solution : Given data

Load =P= 140 KN= 140 X 10^3 N

Eccentricity = e= 80+100= 180 mm

Permissible shear stress= σ_s = 100 N/mm^2

Total Length of weld =L= 200+200+160+160=720 mm

<u>Step 1:</u> Direct force per mm length

$$F_1 = \frac{\textit{Total}\ \text{Load}}{\textit{Total}\ \text{Length of weld}} = \frac{P}{L} = \frac{140 \text{ x } 10^3}{720} = 194.44 \text{ N/mm}$$

<u>Step 2:</u> Bending force per mm length

$$F_2 = \frac{M\ r}{I_P}$$

$M = P$ x e = 140 x $10^3 x$ 180= 25.20 x 10^6 Nmm

X=160/2=80 mm

Y= 200/2= 100 mm

r=$\sqrt{X^2 + Y^2} = \sqrt{80^2 + 100^2} = 128.06\ mm$

$I_P =$ Polar moment of Inertia

$I_P = I_X + I_Y$

$$I_X = \left[\frac{200^3}{12}\right] x\ 2 + (160 \text{ x } 100^2) x 2 = 4.53 \text{ x } 10^6 \text{ mm}^3$$

$$I_Y = \left[\frac{160^3}{12}\right] x\ 2 + (200 \text{ x } 80^2) x 2 = 3.24 \text{ x } 10^6 \text{ mm}^3$$

$$I_P = 4.53 \text{ x } 10^6 + 3.24 \text{ x } 10^6 = 7.73 \text{ x } 10^6 \text{ mm}^3$$

$$F_2 = \frac{M\ r}{I_P} = \frac{25.20 \text{ x} 10^6 x\ 128.06}{7.77 x\ 10^6} = 415.03 \text{ N/mm}$$

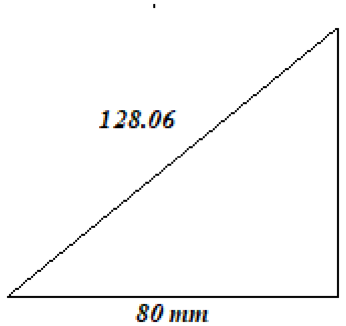
<u>Step 3</u>: Resultant Force:

$$F_R = \sqrt{F_1^2 + F_2^2 + 2F_1F_2 \text{ Cos}\,\theta}$$

$$\text{Cos}\,\theta = \frac{80}{128.06} = 0.624$$

$$F_R = \sqrt{(194.44)^2 \text{ x}(415.03)^2 + 2 \text{ x} 194.44 x 415.03 x 0.624}$$

$F_R =$ 557.56 N/mm^2



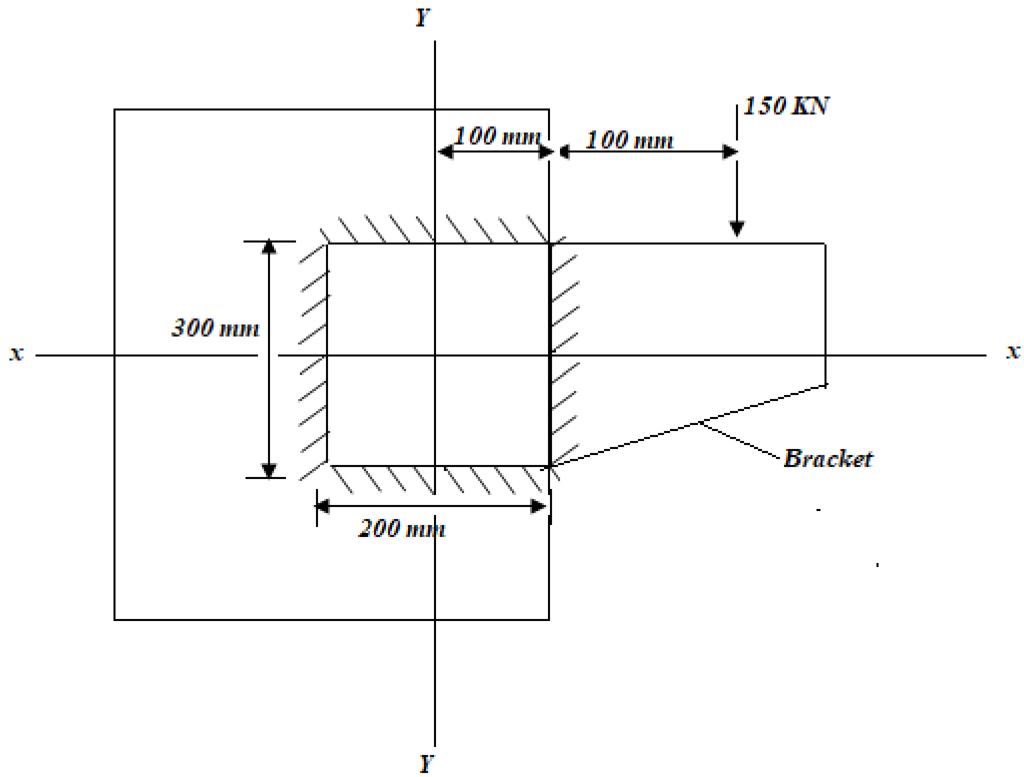
<u>Step 4</u>: Resultant Force:

Fr= 0.707 x S x σ_s

557.56 = 0.707 x S x 100

S= 7.88 mm ≈ 8 mm

2) A bracket plate is connected to flange of column as shown in figure. Find maximum size of weld. Permissible shear stress is 100 Mpa.



Solution : Given data

Load =P= 150 KN= 150 X 10^3 N

Eccentricity = e= 100+100= 200 mm

Permissible shear stress= σ_s = 100 N/mm^2

Total Length of weld =L= 300+300+200+200=1000 mm

<u>Step 1:</u> Direct force per mm length

$$F_1 = \frac{Total \text{ Load}}{Total \text{ Length of weld}} = \frac{P}{L} = \frac{150 \text{ x } 10^3}{1000} = 150 \text{ N/mm}$$

<u>Step 2:</u> Bending force per mm length

$$F_2 = \frac{M\ r}{I_P}$$

$M = P$ x e = 150 x $10^3 x$ 200= 30 x 10^6 Nmm

X=200/2=100 mm

Y= 300/2= 150 mm

r=$\sqrt{X^2+Y^2}$ = $\sqrt{100^2+150^2} = 180.27\ mm$

$I_P =$ Polar moment of Inertia

$I_P = I_X + I_Y$

$$I_X = \left[\frac{300^3}{12}\right] x\ 2 + (200 \text{ x } 150^2) x2 = 13.5 \text{ x } 10^6 \text{ mm}^3$$

$$I_Y = \left[\frac{200^3}{12}\right] x\ 2 + (300 \text{ x } 100^2) x2 = 7.33 \text{ x } 10^6 \text{ mm}^3$$

$$I_P = 13.5 \text{ x } 10^6 + 7.33 \text{ x } 10^6 = 20.83 \text{ x } 10^6 \text{ mm}^3$$

$$F_2 = \frac{M\ r}{I_P} = \frac{30 \text{ x} 10^6 x\ 180.27}{20.83 x\ 10^6} = 259.63 \text{ N/mm}$$

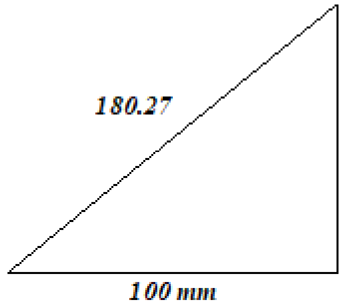
<u>Step 3</u>: Resultant Force:

$$F_R = \sqrt{F_1^{\ 2} + F_2^{\ 2} + 2F_1F_2 \text{ Cos}\,\theta}$$

$$\text{Cos}\,\theta = \frac{100}{180.27} = 0.554$$

$$F_R = \sqrt{(150)^2 \text{ x}(259.63)^2 + 2 \text{ x} 150 x 259.63 x 0.554}$$

$$F_R = 364.7 \text{ N/mm}^2$$



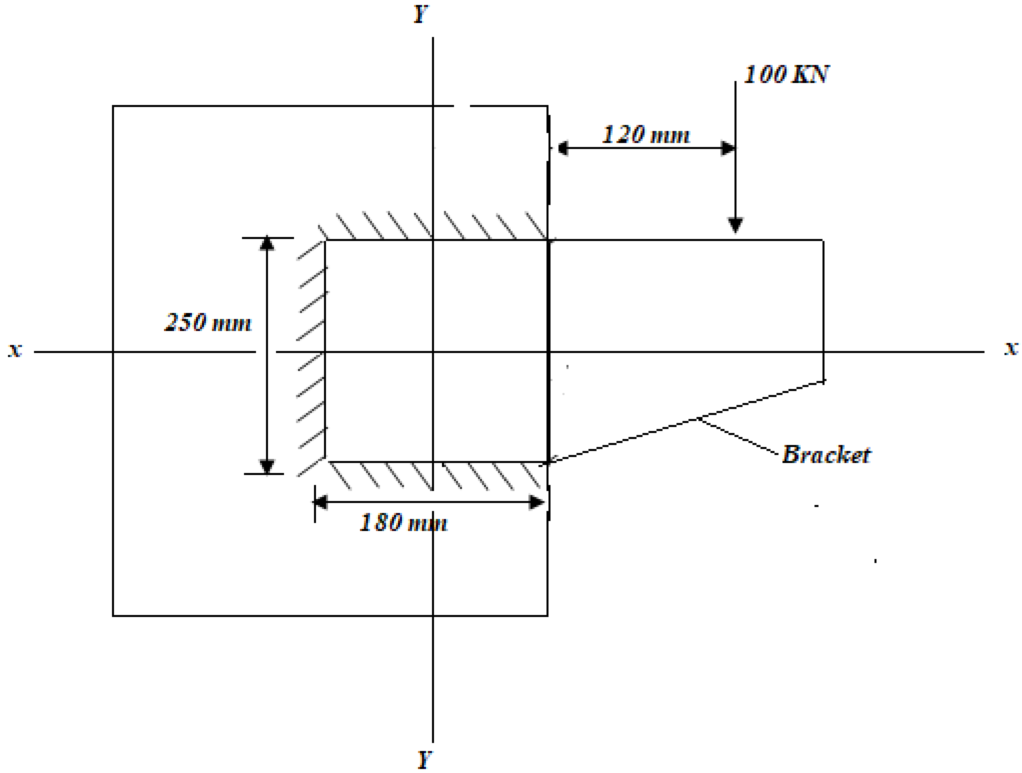
<u>Step 4</u>: Resultant Force:

Fr= 0.707 x S x σ_s

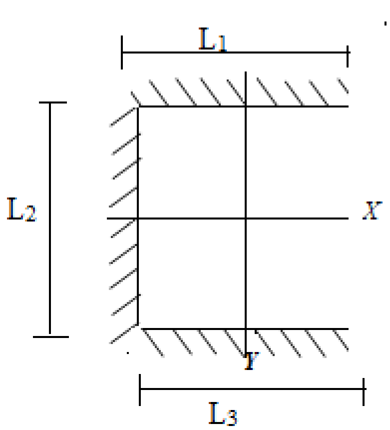
364.7 = 0.707 x S x 100

S= 5.15mm ≈ 6 mm

3) A bracket plate is welded to the column as shown in figure. Design suitable fillet weld. Permissible shear stress is 100 Mpa.



Solution : Given data



$$\bar{x} = \frac{L_1 x_1 + L_2 x_2 + L_3 x_3}{L_1 + L_2 + L_3}$$

$L_1 = 180$ mm, $x_1 = \frac{180}{2} = 90mm$

$L_2 = 250$ mm, $x_2 = 0mm$

$L_3 = 180$ mm, $x_3 = \frac{180}{2} = 90mm$

$\bar{x} = \frac{(180x90) + (250x0) + (180x90)}{180 + 250 + 180} = 53.11mm$ from left hand side

$\bar{x} = 180 - 53.11 =$ 126.89 mm from right hand side

Load =P= 100 KN= 100 X 10^3 N

Eccentricity = e= 120+126.89= 246.89 mm

Permissible shear stress= σ_s = 100 N/mm^2

Total Length of weld =L= 180+250+180=610 mm

__Step 1:__ Direct force per mm length

$$F_1 = \frac{Total \text{ Load}}{Total \text{ Length of weld}} = \frac{P}{L} = \frac{100 \text{ x } 10^3}{610} = 163.93 \text{ N/mm}$$

__Step 2:__ Bending force per mm length

$$F_2 = \frac{M\ r}{I_P}$$

$M = P$ x e = 100 x $10^3 x$ 246.89= 24.689 x 10^6 Nmm

X=126.89 mm

Y= 250/2= 125 mm

r=$\sqrt{X^2 + Y^2}$ = $\sqrt{126.89^2 + 125^2} = 178.12\ mm$

$I_P =$ Polar moment of Inertia

$I_P = I_X + I_Y$

$$I_X = \left[\frac{250^3}{12}\right] + (180 \text{ x } 125^2) x2 = 6.93 \text{ x } 10^6 \text{ mm}^3$$

$$I_Y = \left[\frac{180^3}{12}\right] x\ 2 + \left[180 \text{ x}(90\text{-}53.11)^2\right] x2 + (250 \text{ x } 53.11^2) = 2.16 \text{ x } 10^6 \text{ mm}^3$$

$I_P = 6.93 \text{ x } 10^6 + 2.16 \text{ x } 10^6 = 9.097 \text{ x } 10^6 \text{ mm}^3$

$$F_2 = \frac{M\ r}{I_P} = \frac{24.69\ \text{x}10^6 x\ 178.12}{9.097 x\ 10^6} = 483.48\ \text{N/mm}$$

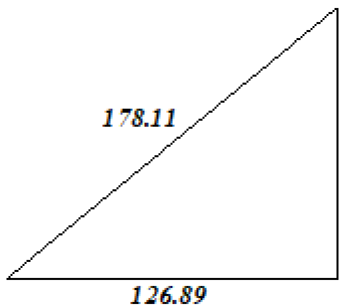
<u>Step 3</u>: Resultant Force:

$$F_R = \sqrt{F_1^{\ 2} + F_2^{\ 2} + 2F_1F_2\ \text{Cos}\,\theta}$$

$$\text{Cos}\,\theta = \frac{126.89}{178.11} = 0.712$$

$$F_R = \sqrt{(163.93)^2\ \text{x}(483.48)^2 + 2\ \text{x}163.93 x 483.48 x 0.712}$$

$$F_R = \ 611.137\ \text{N/mm}^2$$



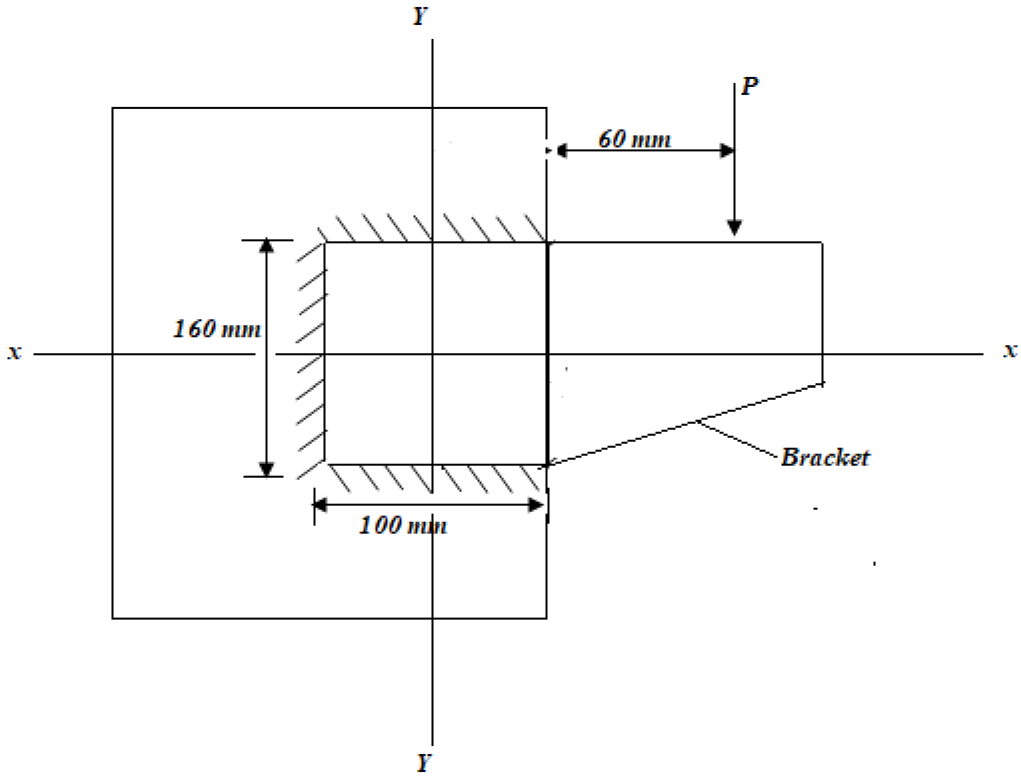
<u>Step 4</u>: Resultant Force:

Fr= 0.707 x S x σ_S

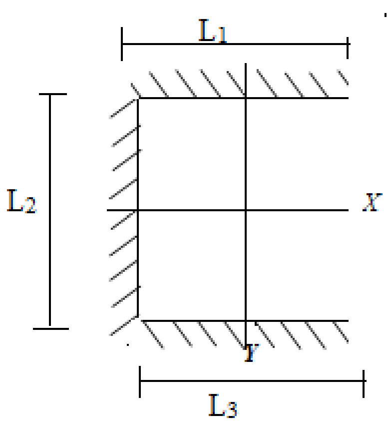
611.137 = 0.707 x S x 100

S= 8.64 mm ≈ 10 mm

4) A bracket plate is connected as shown in figure using 8 mm fillet weld. Calculate maximum value of load P if the permissible shear stress is 110 Mpa.



Solution : Given data



$$\bar{x} = \frac{L_1 x_1 + L_2 x_2 + L_3 x_3}{L_1 + L_2 + L_3}$$

$L_1 = 100$ mm, $x_1 = \dfrac{100}{2} = 50mm$

$L_2 = 160$ mm, $x_2 = 0mm$

$L_3 = 100$ mm, $x_3 = \dfrac{100}{2} = 50mm$

$\bar{x} = \dfrac{(100x50) + (160x0) + (100x50)}{100 + 160 + 100} = 27.78mm$ from left hand side

$\bar{x} = 100 - 27.78 =$ 72.22 mm from right hand side

Load =P

Eccentricity = e= 60+72.22= 132.22 mm

Permissible shear stress= σ_s = 110 N/mm^2

Total Length of weld =L= 100+160+100=360 mm

<u>Step 1:</u> Direct force per mm length

$$F_1 = \frac{Total\ \text{Load}}{Total\ \text{Length of weld}} = \frac{P}{L} = \frac{P}{360} = 2.78\ x\ 10^{-3}\ \text{P}$$

<u>Step 2:</u> Bending force per mm length

$$F_2 = \frac{M\ r}{I_P}$$

$M = P$ x e = Px 132.22= 132.22 P

X=72.22 mm

Y= 160/2= 80 mm

r=$\sqrt{X^2 + Y^2}$ = $\sqrt{72.22^2 + 80^2} = 107.77\ mm$

$I_P =$ Polar moment of Inertia

$I_P = I_X + I_Y$

$$I_X = \left[\frac{160^3}{12}\right] + (100 \text{ x } 80^2)x2 = 1.62 \text{ x } 10^6 \text{ mm}^3$$

$$I_Y = \left[\frac{100^3}{12}\right] x\ 2 + [100 \text{ x}(50\text{-}27.78)^2]x2 + (160 \text{ x } 27.78^2) = 0.38 \text{ x } 10^6 \text{ mm}^3$$

$$I_P = 1.62 \text{ x } 10^6 + 0.38 \text{ x } 10^6 = 2 \text{ x } 10^6 \text{ mm}^3$$

$$F_2 = \frac{M\ r}{I_P} = \frac{\text{P } x \text{ 132.22 x 107.77}}{2\ x\ 10^6} = 7.12x10^{-3}\ \text{P N/mm}$$

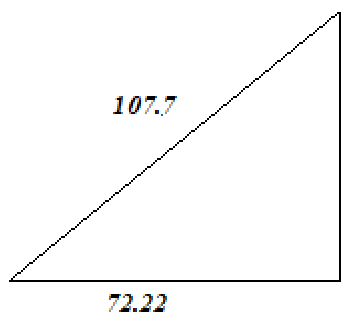
<u>Step 3</u>: Resultant Force:

$$F_R = \sqrt{F_1^{\ 2} + F_2^{\ 2} + 2F_1F_2 \text{ Cos}\,\theta}$$

$$\text{Cos}\,\theta = \frac{72.22}{107.77} = 0.67$$

$$F_R = \sqrt{(2.78x\ 10^{-3}P)^2\ \text{x}(7.12x\ 10^{-3}P)^2 + 2\ \text{x}2.78x\ 10^{-3}Px7.12x\ 10^{-3}Px0.67}$$

$$F_R =\ 9.20x\ 10^{-3}P\ \text{N/mm}^2$$



<u>Step 4</u>: Resultant Force:

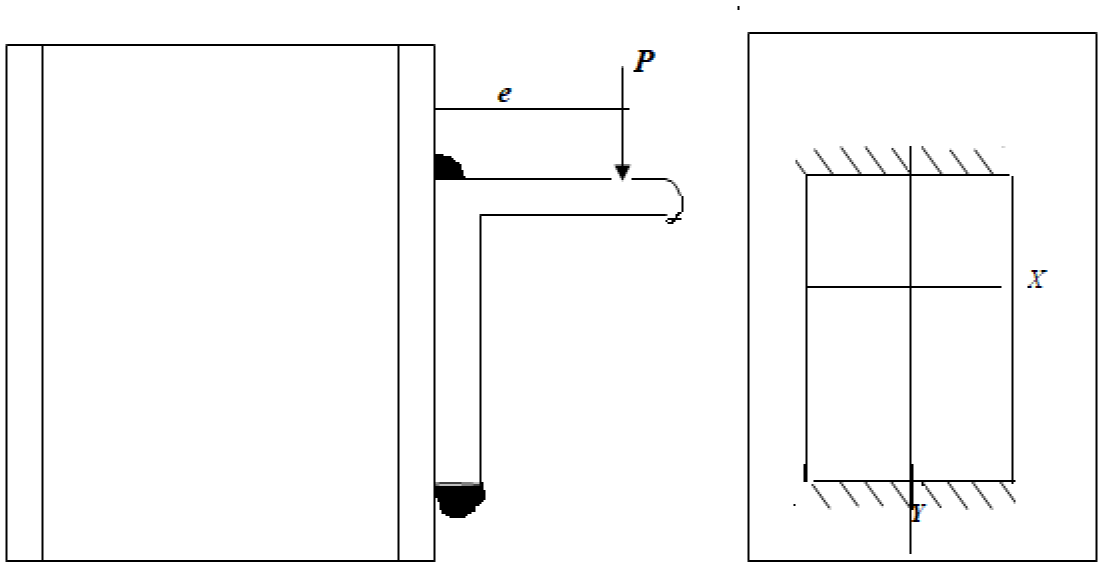
Fr= 0.707 x S x σ_S

9.20 x 10^{-3} P = 0.707 x 8 x 110

P= 67.52 X 10^3 N

P= 67.52 KN

Type 2: Welded connection subjected to moment in the plane perpendicular to plane of connection



Design Procedure:

<u>Step 1:</u> Direct force per mm length

$$F_1 = \frac{Total\ \text{Load}}{Total\ \text{Length of weld}} = \frac{P}{L}$$

<u>Step 2:</u> Bending force per mm length

$$F_2 = \frac{M\ y}{I_{xx}}$$

$$M = P \text{ x e}$$

$$I_{xx} = \left[l \text{ x (d/2)}^2 \right] x2$$

<u>Step 3</u>: Resultant Force:

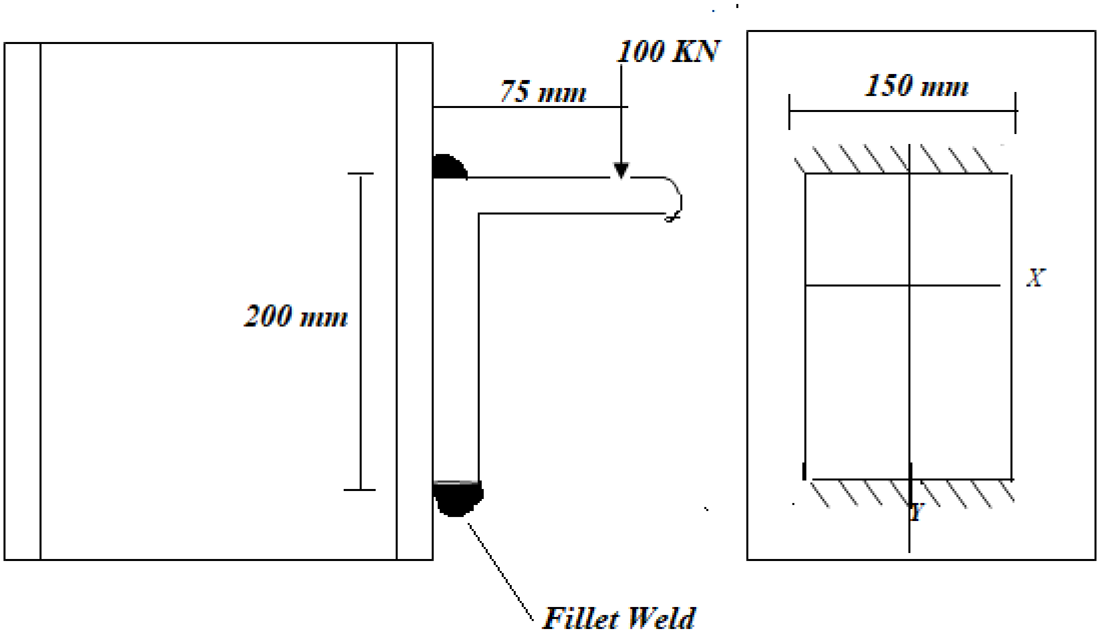
$$F_R = \sqrt{F_1^{\ 2} + F_2^{\ 2}}$$

<u>Step 4</u>: Resultant Force: Fr= 0.707 x S x σ_S

S = Size of weld

σ_S = Permissible shear stress in weld

1) A load of 100 KN is applied to a bracket as shown in figure. Find the size of weld, length of each weld is 150 mm and permissible shear stress is 100 N/mm^2.



Solution : Given data

Load =P= 100 KN= 100 X 10^3 N

Eccentricity = e= 75 mm

Permissible shear stress= σ_s = 100 N/mm^2

Total Length of weld =L= 150+150=300 mm

<u>Step 1:</u> Direct force per mm length

$$F_1 = \frac{Total\ \text{Load}}{Total\ \text{Length of weld}} = \frac{P}{L} = \frac{100 \text{ x } 10^3}{300} = 0.3 \text{ x } 10^3 N$$

<u>Step 2:</u> Bending force per mm length

$$F_2 = \frac{M\ y}{I_{xx}}$$

$$M = P \text{ x e} = 100 \text{ x } 10^3 \text{ x } 75 = 7.5 \text{ x } 10^6 \text{ N mm}$$

$$I_{xx} = \left[l \text{ x } (d/2)^2 \right] x2$$

$$I_{xx} = \left[150 \text{ x } (200/2)^2 \right] x2 = 3 \text{ x } 10^6 mm^4$$

$$F_2 = \frac{M\ y}{I_{xx}} = \frac{7.5 \text{ x } 10^6 \text{ x } 100}{3 \text{ x } 10^6} = 250 N/mm$$

<u>Step 3</u>: Resultant Force:

$$F_R = \sqrt{F_1^{\ 2} + F_2^{\ 2}}$$

$$F_R = \sqrt{(0.3\ x10^3)^2 + (250)^2} = 416.\ 66 \text{ N/mm}$$

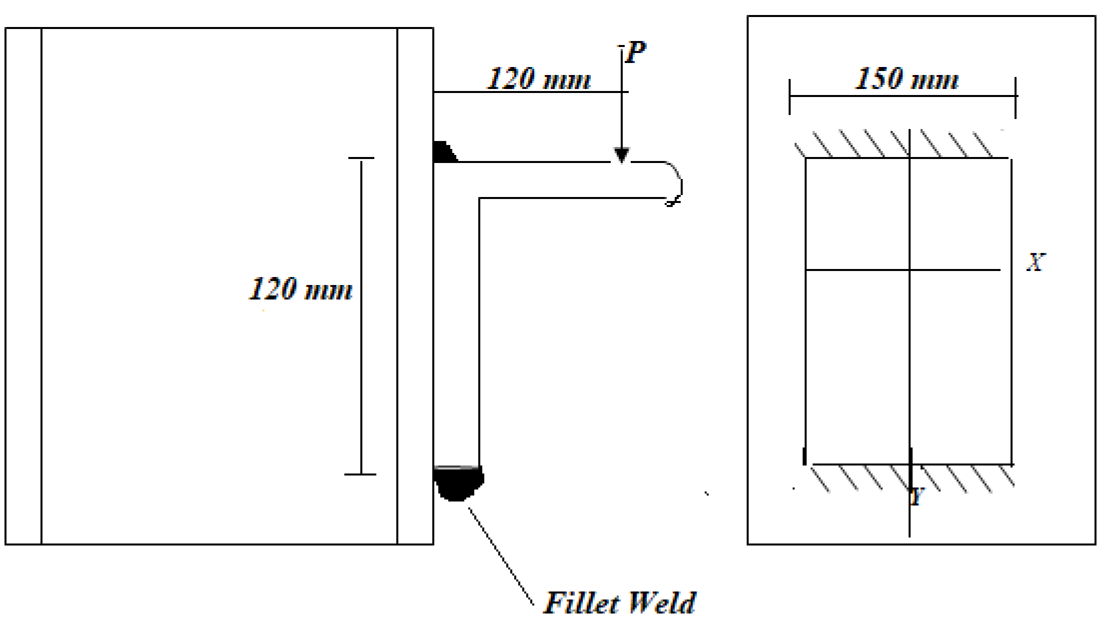
<u>Step 4</u>: Resultant Force:

Fr= 0.707 x S x σ_s

416.66 =0.707 x S X100

S =5.89 mm ≈ 6 mm

2) A bracket carrying load P as shown in figure is connected to the column by means of two fillet welds ach 150 mm long and 8 mm size. Find maximum load the bracket can carry. Permissible shear stress is 100 Mpa.



Solution : Given data

Load =P

Eccentricity = e= 120 mm

Permissible shear stress= σ_s = 100 N/mm^2

Total Length of weld =L= 150+150=300 mm

Size of weld =S =8 mm

<u>Step 1:</u> Direct force per mm length

$$F_1 = \frac{Total\text{ Load}}{Total\text{ Length of weld}} = \frac{P}{L} = \frac{P}{300} = 3.33 \text{ x } 10^{-3} P\ N$$

<u>Step 2:</u> Bending force per mm length

$$F_2 = \frac{M\ y}{I_{xx}}$$

$$M = P \text{ x e =P x 120 = } 120P \text{ N mm}$$

$$I_{xx} = \left[l \text{ x (d/2)}^2 \right] x2$$

$$I_{xx} = \left[150 \text{ x (120/2)}^2 \right] x2 = 1.08 \text{ x } 10^6 mm^4$$

$$F_2 = \frac{M\ y}{I_{xx}} = \frac{120P \text{ x 60}}{1.08 \text{ x } 10^6} = 6.67x10^{-3} P$$

<u>Step 3</u>: Resultant Force:

$$F_R = \sqrt{F_1^{\ 2} + F_2^{\ 2}}$$

$$F_R = \sqrt{(3.333x10^{-3} P)^2 + (6.67x10^{-3} P)^2} \text{ = 7.45 x } 10^{-3} P \text{ N/mm}$$

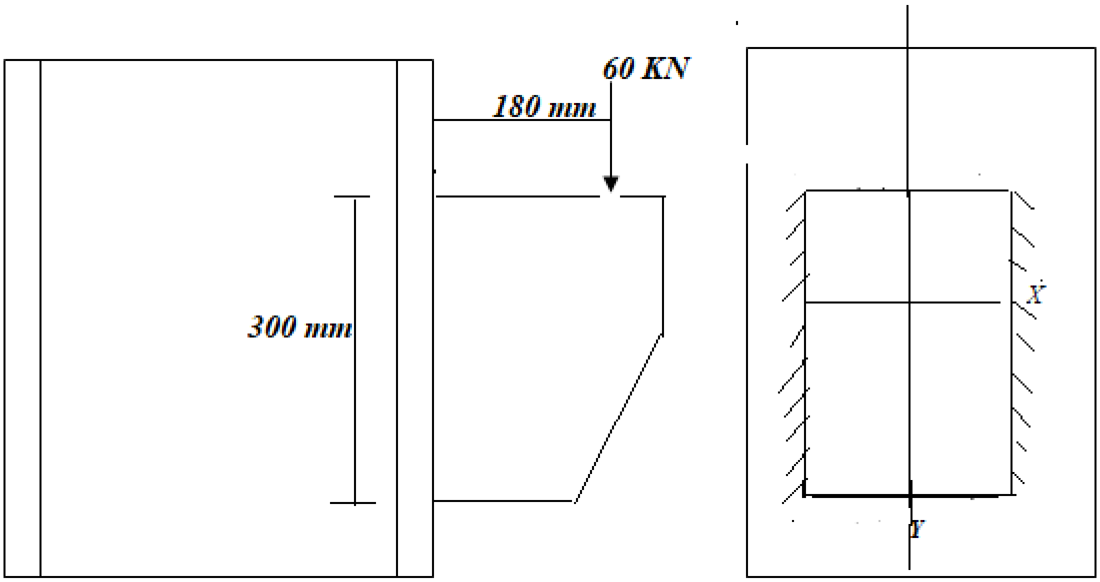
<u>Step 4</u>: Resultant Force:

Fr= 0.707 x S x σ_s

7.45 x 10^{-3} P =0.707 x 8 X100

P =75.919 x 10^3 N =75.919 KN

3) Find maximum size of the fillet weld required to connect the bracket to the column as shown in figure. Permissible shear stress in weld should not exceed 102.5 N/mm^2.



Solution : Given data

Load =P= 60 KN= 60 X 10^3 N

Eccentricity = e= 180 mm

Permissible shear stress= σ_s = 102.5 N/mm^2

Total Length of weld =L= 300+300=600 mm

Step 1: **Direct force per mm length**

$$F_1 = \frac{Total \text{ Load}}{Total \text{ Length of weld}} = \frac{P}{L} = \frac{60\ x 10^3}{600} = 100 N/mm$$

Step 2: **Bending force per mm length**

$$F_2 = \frac{M\ y}{I_{xx}}$$

$$M = P \text{ x e } = 60 \text{ x } 10^3 \text{ x } 180 = 10.8\ x\ 10^6 \text{ N mm}$$

$$I_{xx} = \left[\frac{300^3}{12}\right] x2 = 4.5 \text{ x } 10^6 \text{ mm}^3$$

$$y = \frac{300}{2} = 150 mm$$

$$F_2 = \frac{M\ y}{I_{xx}} = \frac{10.8\ x\ 10^6 \text{ x } 150}{4.5 \text{ x } 10^6} = 360 N/mm$$

<u>Step 3</u>: Resultant Force:

$$F_R = \sqrt{F_1^2 + F_2^2}$$

$$F_R = \sqrt{(100)^2 + (360)^2} = 373.63 \text{ N/mm}$$

<u>Step 4</u>: Resultant Force:

Fr= 0.707 x S x σ_s

373.63 =0.707 x S X100

S=5.155 mm ≈ 6 mm

TENSION MEMBER

A member carries axial tensile force is called tensile member. Tension member carry axial load causing elongation. A tension member can be sustain the load up to the ultimate load without fracture but the elongation of the member at their load would be much larger resulting in the structure supported by the member would become non-functional hence the general design of tension member the yield load is generally taken as failure load. In some cases, may get rupture (suddenly failure of structure at the ultimate stress of material at critical section.)

The various section used as a tension member are wire cables, circular bars, square bars and flat bars are used to carry light loads.

Steel section, such as angle, ‘I’, channel and tee section are used to carry moderate loads.

Double angle section is preferred then single angle as a single angle section develops bending stress due to the eccentricity between the end connections and the center of gravity of the angle section; double angle sections develop relatively less eccentricity.

Definition of tension member: It is defined as structural member subjected to tensile force acting along the direction of longitudinal axis.

Ex. The tie member of roof truss, cable in suspension bridge

Types of tension member

1) Wires, cables and rods

2) Angle sections: Angle section are more rigid intension, angle sections used in roof trusses.

3) Channel Sections: They are having high strength rigidity in direction of web.

4) I-Sections: I-Section is having more rigidity.

5) Built up section: Two or more members are connected together. The section is called built up section. In case of built up section moment of inertia increased.

Ties of trusses [Fig 1(*a*)], suspenders of cable stayed and suspension bridges [Fig.1 (*b*)], suspenders of buildings systems hung from a central core [Fig.1(*c*)] (such buildings are used in earthquake prone zones as a way of minimising inertia forces on the structure), and sag rods of roof purlins [Fig 1(*d*)] are other examples of tension members. Tension members are also encountered as bracings used for the lateral load resistance. In X type bracings [Fig.1 (*e*)] the member which is under tension, due to lateral load acting in one direction, undergoes compressive force, when the direction of the lateral load is changed and vice versa. Hence, such members may have to be designed to resist tensile and compressive forces.

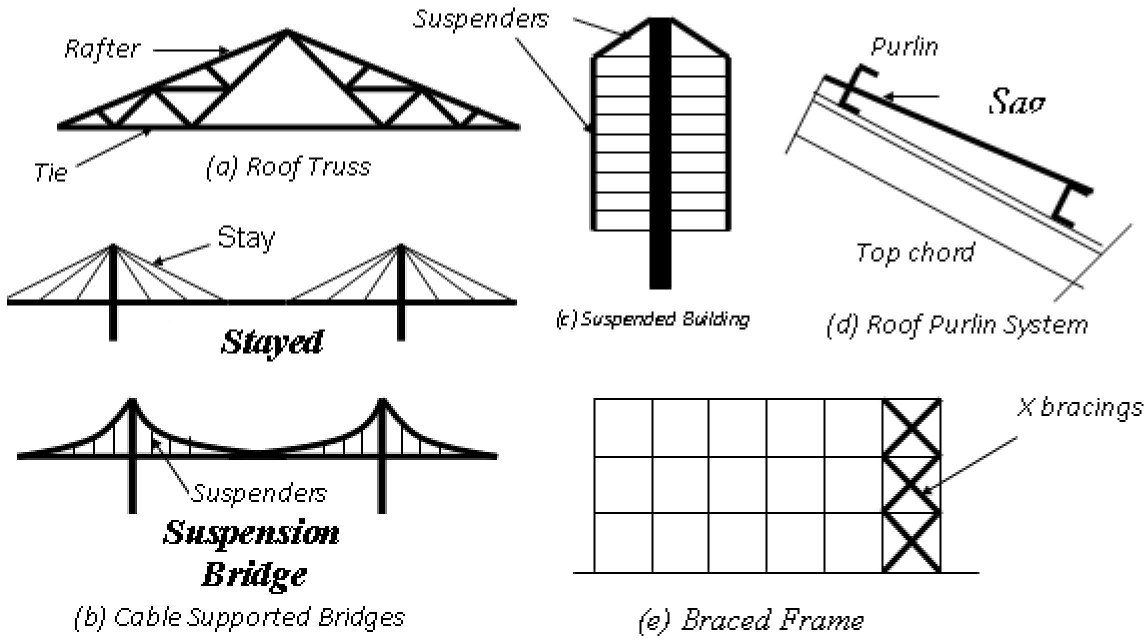


Fig. 1 Tension Members in Structures

Different Section of Tension Member

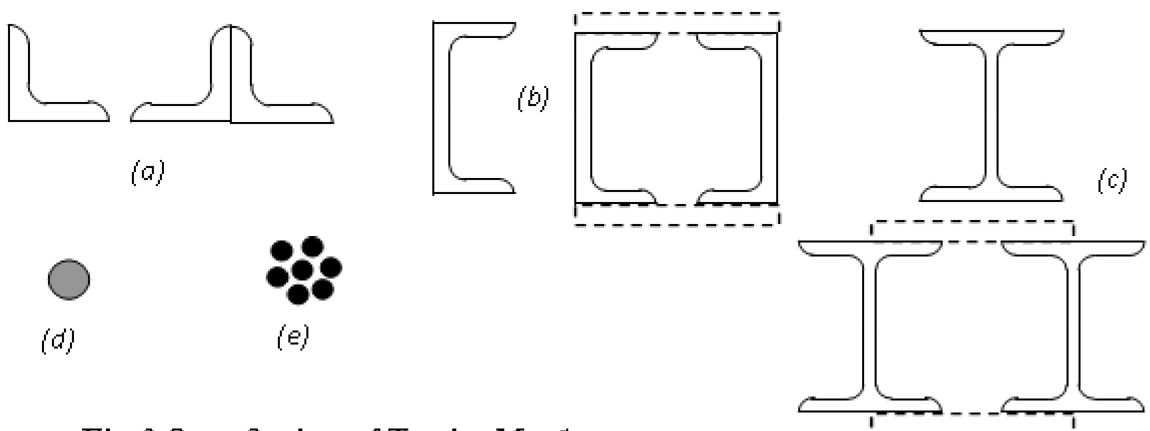


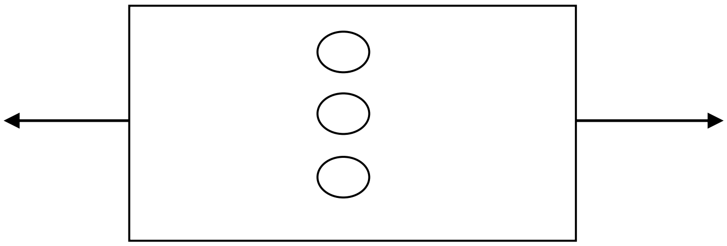
Fig. 2 Cross Sections of Tension Members

The tension members can have a variety of cross sections. The single angle and double angle sections [Fig 2(*a*)] are used in light roof trusses as in industrial buildings. The tension members in bridge trusses are made of channels or I sections, acting individually or built-up [Figs. 2(*c*) and 2(*d*)]. The circular rods [Fig.2 (*d*)] are used in bracings designed to resist loads in tension only. They buckle at very low compression and are not considered effective. Steel wire ropes [Fig.2 (*e*)] are used as suspenders in the cable suspended bridges and as main stays in the cable-stayed bridges.

Load carrying capacity of tension member:

$$P_t = \text{ Net effective area X Axial tensile stress}$$
$$P_t = A_{net} \text{ x } \sigma_{\text{at}}$$
$$\sigma_{\text{at}} = \mathit{Axial\ tensile}\ \text{permissible stress}$$
$$\sigma_{\text{at}} = 0.6xF_y$$
$$F_y = Yield\ Stress$$

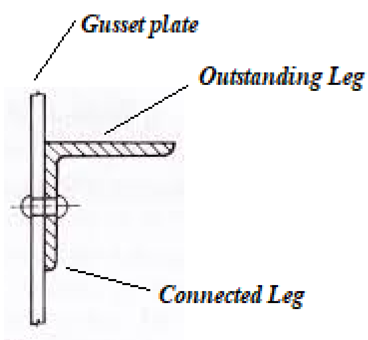
2) Net effective sectional area



$$A_{net} = \text{ Gross area - Area of rivets}$$
$$A_{net} = [(bt) - (ndt)]$$
$$A_{net} = [(b) - (nd)]t$$

Effective area of angle section

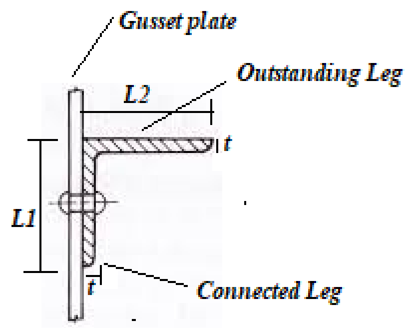
1) Single angle section connected by one leg



$$A_{net} = \text{ A}_1\text{+A}_2K$$
$$K = \frac{3A_1}{3A_1 + A_2}$$ IS Code P.No: 37, C. No:4.2.1.1

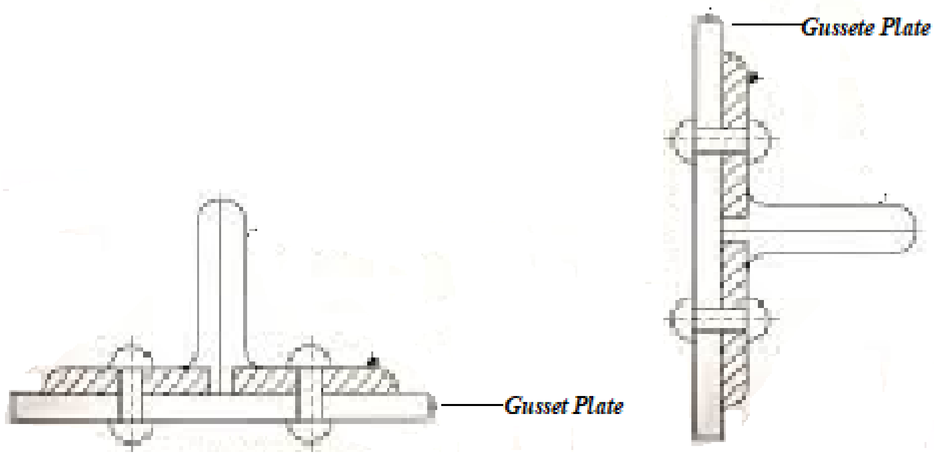
$$A_1 = \text{Net area of connected leg}$$
$$A_2 = \text{Net area of outstanding leg}$$

$$A_1 = \text{Net area of connected leg}$$
$$A_1 = (L_1\text{xt}) - (\text{Dxt}) - \left(t\frac{t}{2}\right)$$
$$A_1 = \left[L_1\text{-D-}\frac{t}{2}\right]t$$
$$A_2 = \text{Net area of outstanding leg}$$
$$A_2 = (L_2\text{xt}) - \left(t\frac{t}{2}\right)$$



$$A_2 = \left[L_2 - \frac{t}{2}\right] t$$

2) Double angle connected on same side of gusset plate



$$A_{net} = \mathrm{A}_1 + \mathrm{A}_2 K$$

$$K = \frac{5A_1}{5A_1 + A_2}$$ IS Code P.No: 37, C. No:4.2.1.2

A_1 = Net area of connected leg

A_2 = Net area of outstanding leg

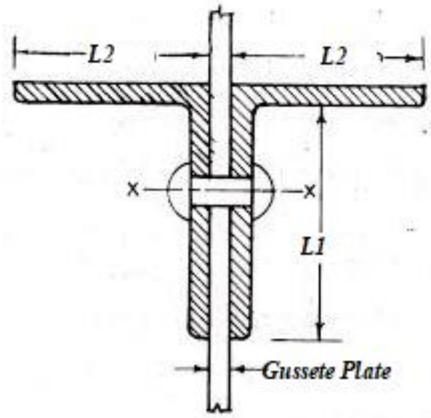
A_1 = Net area of connected leg

$$A_1 = \left[L_1\text{-D-}\frac{t}{2}\right] t$$

A_2 = Net area of outstanding leg

$$A_2 = \left[L_2 - \frac{t}{2}\right] t$$

3) Double angle connected on both side of gusset plate



$A_{net} = A_1 + A_2 K$

$K = 1$ IS Code P.No: 37, C. No:4.2.1.3

A_1 = Net area of connected leg

A_2 = Net area of outstanding leg

A_1 = Net area of connected leg

$$A_1 = \left[L_1\text{-D-}\frac{t}{2}\right]t$$

A_2 = Net area of outstanding leg

$$A_2 = \left[L_2 - \frac{t}{2}\right]t$$

Strength of Tension Member

$P_t = A_{net} \times \sigma_{at}$

$\sigma_{at} = 0.6$ Fy IS Code P No:37, C No:4.1.1

Fy = Minimum yield stress of steel in Mpa

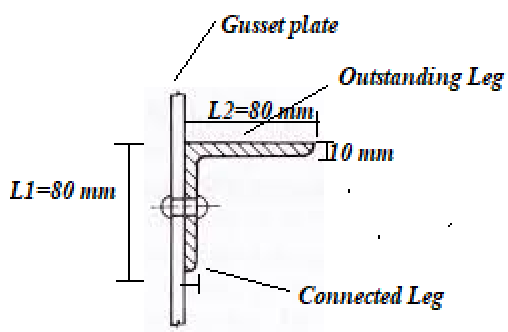
1) In a roof truss a diagonal member consist of ISA 80 x 80 x 10 mm is connected to gusset plate by one leg only by 20 mm diameter rivet in row. Find tensile strength of member.

Solution: Nominal diameter of rivet=d= 20 mm

Gross diameter of rivet=D= 20+1.5=21.5 mm

Assuming fy=250 N/mm^2

σ_{at}=0.6 fy =0.6 x250 = 150 N/mm^2



$A_{net} = A_1 + A_2 K$

$$K = \frac{3A_1}{3A_1 + A_2}$$ IS Code P.No: 37, C. No:4.2.1.1

A_1 = Net area of connected leg

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A_2 = Net area of outstanding leg

$\underline{Step}$ 1: To find net area

A_1 = Net area of connected leg

$$A_1 = \left[L_1 \text{- D -} \frac{t}{2}\right] t = \left[80 \text{ - } 21.5 \text{ - } \frac{10}{2}\right] 10 = 535 \text{ mm}^2$$

A_2 = Net area of outstanding leg

$$A_2 = \left[L_2 - \frac{t}{2}\right] t = \left[80 - \frac{10}{2}\right] 10 = 750 \text{ mm}^2$$

$$K = \frac{3A_1}{3A_1 + A_2} = \frac{3 \text{ x } 535}{3 \text{ x } 535 + 750} = 0.6815$$

$$A_{net} = \text{A}_1 + \text{A}_2 K$$

$$A_{net} = 535 + (750 \text{ x } 0.6815) = 1046.125 \text{ mm}^2$$

Step 2: Load carrying capacity

$$P_t = A_{net} \text{ x } \sigma_{\text{at}}$$

$$P_t = 1046.125 \text{ x } 150 = 156.918 \text{ x } 10^3 N$$

$$P_t = 156.918 \; KN$$

2) In ISA 100 x 65 x 10 mm is to be connected to the gusset plate by 18 mm diameter of rivets. Find tensile strength

1) If longer leg connected to gusset plate

2) If shorter leg connected to gusset plate

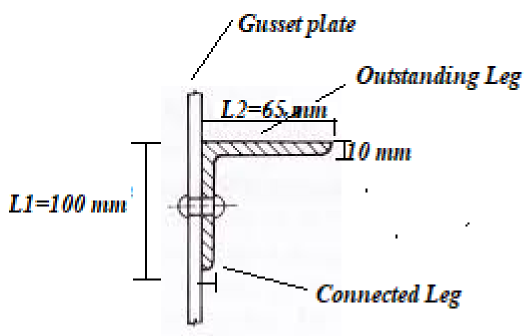
Solution: Nominal diameter of riveted=d= 18 mm

Gross diameter of rivet=D= 18+1.5=19.5 mm

Assuming fy=250 N/mm^2

σ_{at}=0.6 fy =0.6 x250 = 150 N/mm^2

Case I: If longer leg connected to gusset plate



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$A_{net} = A_1 + A_2 K$

$K = \dfrac{3A_1}{3A_1 + A_2}$ IS Code P.No: 37, C. No:4.2.1.1

A_1 = Net area of connected leg

A_2 = Net area of outstanding leg

$\underline{Step\ 1}$: To find net area

A_1 = Net area of connected leg

$$A_1 = \left[L_1 - D - \frac{t}{2}\right] t = \left[100 - 19.5 - \frac{10}{2}\right] 10 = 755 \text{ mm}^2$$

A_2 = Net area of outstanding leg

$$A_2 = \left[L_2 - \frac{t}{2}\right] t = \left[65 - \frac{10}{2}\right] 10 = 600 \text{ mm}^2$$

$$K = \frac{3A_1}{3A_1 + A_2} = \frac{3 \times 755}{3 \times 755 + 600} = 0.7905$$

$$A_{net} = A_1 + A_2 K$$

$$A_{net} = 755 + (600 \times 0.7905) = 1229.3 \text{ mm}^2$$

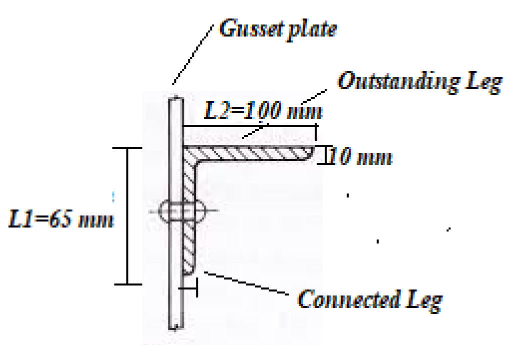
Step 2: Load carrying capacity

$P_t = A_{net} \times \sigma_{at}$

$P_t = 1229.3 \times 150 = 184.395 \times 10^3 N$

$P_t = 184.395\ KN$

Case II: If shorter leg connected to gusset plate



$A_{net} = A_1 + A_2 K$

$K = \dfrac{3A_1}{3A_1 + A_2}$ IS Code P.No: 37, C. No:4.2.1.1

A_1 = Net area of connected leg

A_2 = Net area of outstanding leg

$\underline{Step}$ 1: To find net area

A_1 = Net area of connected leg

$$A_1 = \left[L_1 - D - \frac{t}{2}\right] t = \left[65 - 19.5 - \frac{10}{2}\right] 10 = 405 \text{ mm}^2$$

A_2 = Net area of outstanding leg

$$A_2 = \left[L_2 - \frac{t}{2}\right] t = \left[100 - \frac{10}{2}\right] 10 = 950 \text{ mm}^2$$

$$K = \frac{3A_1}{3A_1 + A_2} = \frac{3 \text{ x } 405}{3 \text{ x } 405 + 950} = 0.5612$$

$$A_{net} = A_1 + A_2 K$$

$$A_{net} = 405 + (950 \text{ x } 0.5612) = 938.140 \text{ mm}^2$$

$\underline{\text{Step 2}}$: Load carrying capacity

$$P_t = A_{net} \text{ x } \sigma_{at}$$

$$P_t = 938.140 \text{ x } 150 = 140.721 \text{ x } 10^3 N$$

$$P_t = 140.721\ KN$$

Note: Longer leg connected to gusset plate having more tensile strength.

3) A tension member of a steel roof truss has to carry axial pull of 100 KN. Compare tensile strength when single a single angle section is used in following cases

1) ISA 70 x 70 x 8 mm with 20 mm diameter of rivet

2) ISA 70 x 70 x 6 mm with fillet weld

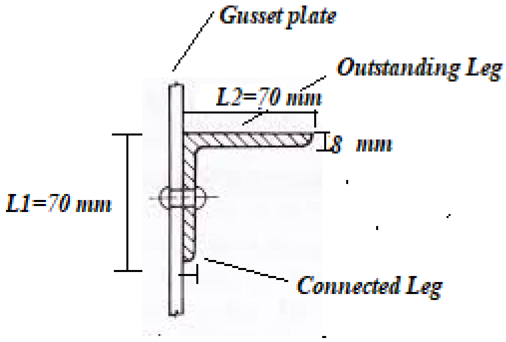
Solution : **Case I:** ISA 70 x 70 x 8 mm with 20 mm diameter of rivet

Nominal diameter of rivet=d= 20 mm

Gross diameter of rivet=D= 20+1.5=21.5 mm

Assuming fy=250 N/mm^2

σ_{at}=0.6 fy =0.6 x250 = 150 N/mm^2



$$A_{net} = A_1 + A_2 K$$

$$K = \frac{3A_1}{3A_1 + A_2}$$ IS Code P.No: 37, C. No:4.2.1.1

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A_1 = Net area of connected leg
A_2 = Net area of outstanding leg

$Step$ 1: To find net area

A_1 = Net area of connected leg

$$A_1 = \left[L_1 \text{- D -} \frac{t}{2}\right] t = \left[70 \text{ - } 21.5 \text{ -} \frac{8}{2}\right] 8 = 356 \text{ mm}^2$$

A_2 = Net area of outstanding leg

$$A_2 = \left[L_2 - \frac{t}{2}\right] t = \left[70 - \frac{8}{2}\right] 8 = 528 \text{ mm}^2$$

$$K = \frac{3A_1}{3A_1 + A_2} = \frac{3 \text{ x } 356}{3 \text{ x } 356 + 528} = 0.6691$$

$$A_{net} = \mathrm{A_1 + A_2}K$$

$$A_{net} = 356 + (528 \text{ x } 0.6691) = 709.323 \text{ mm}^2$$

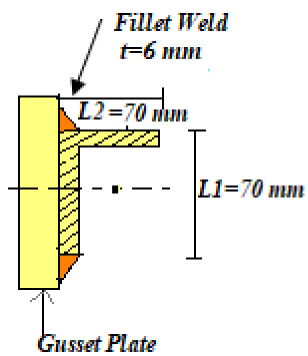
Step 2: Load carrying capacity

$$P_t = A_{net} \text{ x } \sigma_{\text{at}}$$

$$P_t = 709.323 \text{ x } 150 = 106.398 \text{ x } 10^3 N$$

$$P_t = 106.398\ KN > 100\ KN \text{ safe}$$

Case II: ISA 70 x 70 x 6 mm with fillet weld



$$A_{net} = \mathrm{A_1 + A_2}K$$

$$K = \frac{3A_1}{3A_1 + A_2}$$ IS Code P.No: 37, C. No:4.2.1.1

A_1 = Net area of connected leg
A_2 = Net area of outstanding leg

$Step$ 1: To find net area

A_1 = Net area of connected leg

$$A_1 = \left[L_1 \text{-} \frac{t}{2}\right] t = \left[70 \text{ -} \frac{6}{2}\right] 6 = 402 \text{ mm}^2$$

A_2 = Net area of outstanding leg

$$A_2 = \left[L_2 - \frac{t}{2}\right]t = \left[70 - \frac{6}{2}\right]6 = 402 \text{ mm}^2$$
$$K = \frac{3A_1}{3A_1 + A_2} = \frac{3 \text{ x } 402}{3 \text{ x } 402 + 402} = 0.75$$
$$A_{net} = A_1 + A_2K$$
$$A_{net} = 402 + (402 \text{ x } 0.75) = 703.5 \text{ mm}^2$$

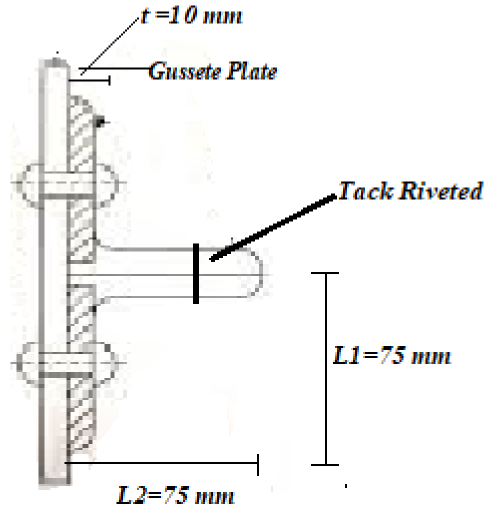
<u>Step 2</u>: Load carrying capacity

$$P_t = A_{net} \text{ x } \sigma_{at}$$
$$P_t = 703.5 \text{ x } 150 = 105.525 \text{ x } 10^3 N$$
$$P_t = 105.525\ KN > 100\ KN \text{ safe}$$

Note: Both the angles are having load carrying greater than 100 KN. Therefore both sections are suitable but second section i.e weld section is more economical.

4) A section consists of two angle ISA 75 x 75 x 10 mm connected on same side of gusset plate back to back using 20 mm diameter of rivet. Find the tensile strength.
If a) The angles are tack riveted
b) The angles are not tack riveted
Solution : 2ISA 75 x 75 x 10 mm
Nominal diameter of rivet=d= 20 mm
Gross diameter of rivet=D= 20+1.5=21.5 mm
Assuming fy=250 N/mm^2
σ_{at}=0.6 fy =0.6 x250 = 150 N/mm^2

<u>Case I:</u> The angles are tack riveted



$$A_{net} = A_1 + A_2K$$
$$K = \frac{5A_1}{5A_1 + A_2}$$ IS Code P.No: 37, C. No:4.2.1.2

A_1 = Net area of connected leg
A_2 = Net area of outstanding leg

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$\underline{Step}$ 1: To find net area

A_1 = Net area of connected leg

$$A_1 = 2\left[L_1 \text{- D -} \frac{t}{2}\right]t = 2\left[75 \text{ - } 21.5 \text{ - } \frac{10}{2}\right]10 = 970 \text{ mm}^2$$

A_2 = Net area of outstanding leg

$$A_2 = 2\left[L_2 - \frac{t}{2}\right]t = 2\left[75 - \frac{10}{2}\right]10 = 1400 \text{ mm}^2$$

$$K = \frac{5A_1}{5A_1 + A_2} = \frac{5 \text{ x } 970}{5 \text{ x } 970 + 1400} = 0.776$$

$$A_{net} = A_1 + A_2K$$

$$A_{net} = 970 + (1400 \text{ x } 0.776) = 2056.4 \text{ mm}^2$$

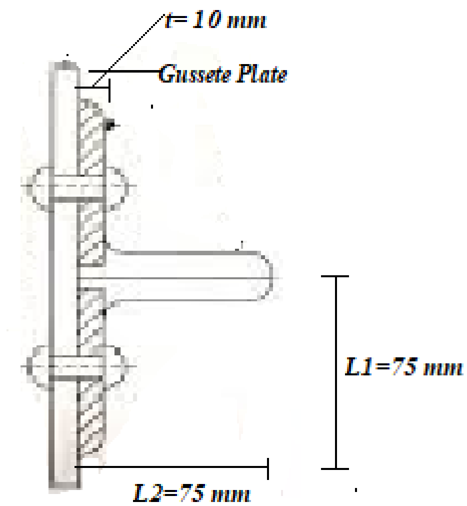
Step 2: Load carrying capacity

$$P_t = A_{net} \text{ x } \sigma_{at}$$

$$P_t = 2056.4 \text{ x } 150 = 308.46 \text{ x } 10^3 N$$

$$P_t = 308.46 \; KN$$

Case II: The angle are not tack riveted (As the section is not tack riveted then according IS Code 800-1984 , Page No :38, C. No: 4.2.1.4 then angle shall be designed as a single angle connected through one legs only in accordance with 4.2.1.1.
The angle are not tack riveted , designed as a single angle



$$A_{net} = A_1 + A_2K$$

$$K = \frac{3A_1}{3A_1 + A_2}$$ IS Code P.No: 37, C. No:4.2.1.1

A_1 = Net area of connected leg

A_2 = Net area of outstanding leg

$\underline{Step}$ 1: To find net area

A_1 = Net area of connected leg

$$A_1 = 2\left[L_1 \text{- D -} \frac{t}{2}\right]t = 2\left[75 \text{ - } 21.5 \text{ - } \frac{10}{2}\right]10 = 970 \text{ mm}^2$$

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A_2 = Net area of outstanding leg

$$A_2 = 2\left[L_2 - \frac{t}{2}\right] t = 2\left[75 - \frac{10}{2}\right] 10 = 1400 \text{ mm}^2$$

$$K = \frac{3A_1}{3A_1 + A_2} = \frac{3 \text{ x } 970}{3 \text{ x } 970 + 1400} = 0.6751$$

$$A_{net} = A_1 + A_2 K$$

$$A_{net} = 970 + (1400 \text{ x } 0.6751) = 1915.24 \text{ mm}^2$$

Step 2: Load carrying capacity

$$P_t = A_{net} \text{ x } \sigma_{at}$$

$$P_t = 1915.24 \text{ x } 150 = 287.286 \text{ x } 10^3 N$$

$$P_t = 287.286\ KN$$

Note: The angle section with tack riveted have more strength.

5) A section consist of two ISA 100 x 65 x 8 mm connected on same side of gusset plate by longer leg using 12 mm diameter of rivet. Find the tensile strength of section

If a) The angles are tack riveted

b) The angles are not tack riveted

Solution: 2ISA 100 x 65 x 8 mm

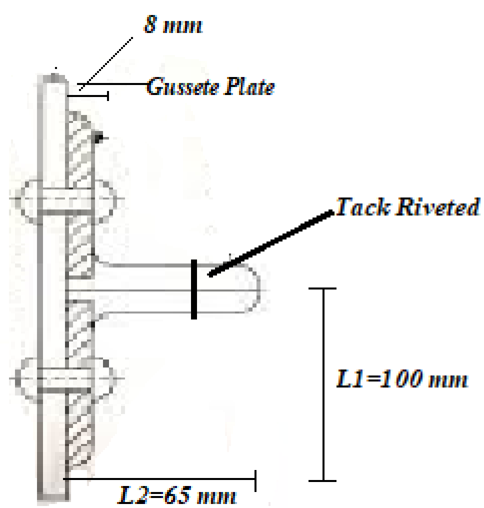
Nominal diameter of rivet=d= 12 mm

Gross diameter of rivet=D= 12+1.5=13.5 mm

Assuming fy=250 N/mm^2

σ_{at}=0.6 fy =0.6 x250 = 150 N/mm^2

<u>Case I:</u>The angle are tack riveted



$$A_{net} = A_1 + A_2K$$

$$K = \frac{5A_1}{5A_1 + A_2}$$ IS Code P.No: 37, C. No:4.2.1.2

A_1 = Net area of connected leg

A_2 = Net area of outstanding leg

$\underline{Step}$ 1: To find net area

A_1 = Net area of connected leg

$$A_1 = 2\left[L_1 - D - \frac{t}{2}\right]t = 2\left[100 - 13.5 - \frac{8}{2}\right]8 = 1320 \text{ mm}^2$$

A_2 = Net area of outstanding leg

$$A_2 = 2\left[L_2 - \frac{t}{2}\right]t = 2\left[65 - \frac{8}{2}\right]8 = 976 \text{ mm}^2$$

$$K = \frac{5A_1}{5A_1 + A_2} = \frac{5 \times 1320}{5 \times 1320 + 976} = 0.8711$$

$$A_{net} = A_1 + A_2K$$

$$A_{net} = 1320 + (976 \times 0.8711) = 2170.26 \text{ mm}^2$$

Step 2: Load carrying capacity

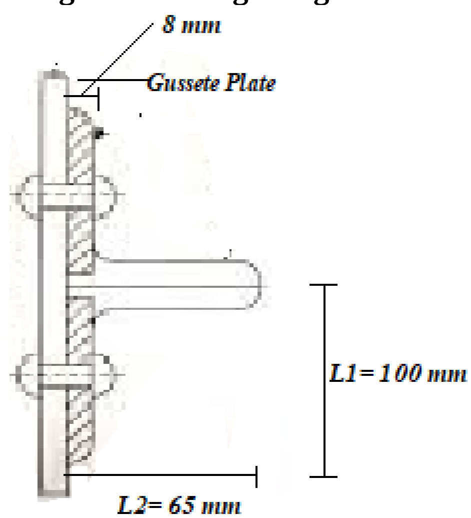
$$P_t = A_{net} \times \sigma_{at}$$

$$P_t = 2170.26 \times 150 = 325.539 \times 10^3 N$$

$$P_t = 325.539\ KN$$

Case II: The angle are not tack riveted (As the section is not tack riveted then according IS Code 800-1984 , Page No :38, C. No: 4.2.1.4 then angle shall be designed as a single angle connected through one legs only in accordance with 4.2.1.1.

The angles are not tack riveted , designed as a single angle



$$A_{net} = A_1 + A_2K$$

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$$K = \frac{3A_1}{3A_1 + A_2}$$ IS Code P.No: 37, C. No:4.2.1.1

A_1 = Net area of connected leg

A_2 = Net area of outstanding leg

$\underline{Step}$ 1: To find net area

A_1 = Net area of connected leg

$$A_1 = 2\left[L_1 - D - \frac{t}{2}\right] t = 2\left[100 - 13.5 - \frac{8}{2}\right] 8 = 1320 \text{ mm}^2$$

A_2 = Net area of outstanding leg

$$A_2 = 2\left[L_2 - \frac{t}{2}\right] t = 2\left[65 - \frac{8}{2}\right] 8 = 976 \text{ mm}^2$$

$$K = \frac{3A_1}{3A_1 + A_2} = \frac{3 \text{ x } 660}{3 \text{ x } 660 + 488} = 0.8022$$

$$A_{net} = A_1 + A_2 K$$

$$A_{net} = 1320 + (976 \text{ x } 0.8022) = 2103.014 \text{ mm}^2$$

AStep 2: Load carrying capacity

$$P_t = A_{net} \text{ x } \sigma_{at}$$

$$P_t = 2103.014 \text{ x } 150 = 315.4521 \text{ x } 10^3 N$$

$$P_t = 315.4521\ KN$$

Note: The angle section with tack riveted have more strength.

6) A double angle ISA 125 x 95 x 10 mm are used in roof truss and connected to the gusset plate by 20 mm diameter of rivet. Find tensile strength in two cases.

Case I: Longer leg connected on same side of gusset plate

Case II: Longer leg connected on both side of gusset plate

Solution: : 2ISA 125 x 95 x 10 mm

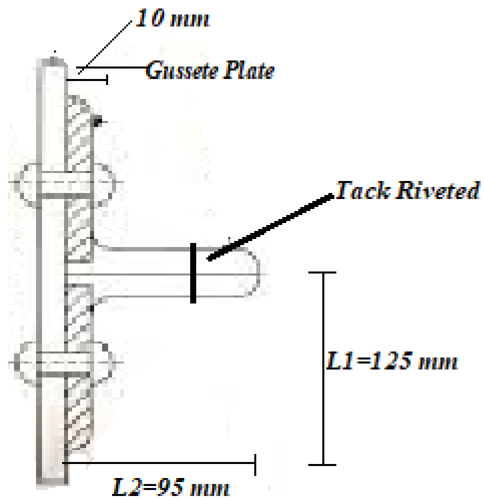
Nominal diameter of rivet=d= 20 mm

Gross diameter of rivet=D= 20+1.5=21.5 mm

Assuming fy=250 N/mm^2

σ_{at}=0.6 fy =0.6 x250 = 150 N/mm^2

Case I: Longer leg connected on same side of gusset plate



$A_{net} = A_1 + A_2 K$

$K = \dfrac{5A_1}{5A_1 + A_2}$ IS Code P.No: 37, C. No:4.2.1.2

A_1 = Net area of connected leg

A_2 = Net area of outstanding leg

Step 1: To find net area

A_1 = Net area of connected leg

$$A_1 = 2\left[L_1 - D - \frac{t}{2}\right]t = 2\left[125 - 21.5 - \frac{10}{2}\right]10 = 1970 \text{ mm}^2$$

A_2 = Net area of outstanding leg

$$A_2 = 2\left[L_2 - \frac{t}{2}\right]t = 2\left[95 - \frac{10}{2}\right]10 = 1800 \text{ mm}^2$$

$$K = \frac{5A_1}{5A_1 + A_2} = \frac{5 \text{ x } 1970}{5 \text{ x } 1970 + 1800} = 0.8454$$

$$A_{net} = A_1 + A_2 K$$

$$A_{net} = 1970 + (1800 \text{ x } 0.8454) = 3491.72 \text{ mm}^2$$

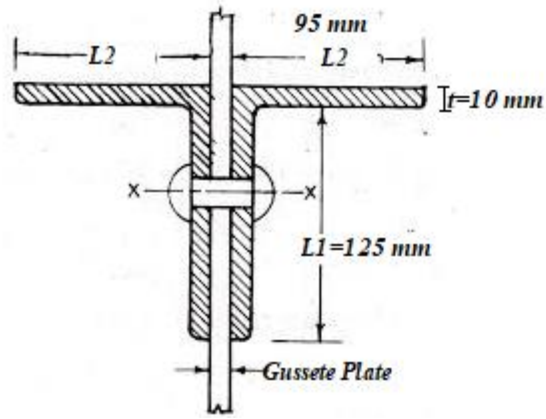
Step 2: Load carrying capacity

$P_t = A_{net} \text{ x } \sigma_{at}$

$P_t = 3491.72 \text{ x } 150 = 523.758 \text{ x } 10^3 N$

$P_t = 523.758 \; KN$

Case II: Longer leg connected on both side of gusset plate



$A_{net} = A_1 + A_2 K$

$K = 1$ IS Code P.No: 37, C. No:4.2.1.4

A_1 = Net area of connected leg

A_2 = Net area of outstanding leg

Step 1: To find net area

A_1 = Net area of connected leg

$$A_1 = 2\left[L_1 - D - \frac{t}{2}\right]t = 2\left[125 - 21.5 - \frac{10}{2}\right]10 = 1970 \text{ mm}^2$$

A_2 = Net area of outstanding leg

$$A_2 = 2\left[L_2 - \frac{t}{2}\right]t = 2\left[95 - \frac{10}{2}\right]10 = 1800 \text{ mm}^2$$

$K = 1$

$A_{net} = A_1 + A_2 K$

$A_{net} = 1970 + (1800 \text{ x } 1) = 3770 \text{ mm}^2$

Step 2: Load carrying capacity

$P_t = A_{net} \text{ x } \sigma_{at}$

$P_t = 3770 \text{ x } 150 = 565.5 \text{ x } 10^3 N$

$P_t = 565.5\ KN$

Note: When angles are connected on both side of gusset plate , load carrying capacity is more.

7) A tension member consisting of 4 ISA 100 x 100 x 10 mm connected to gusset plate by 18 mm diameter of rivets as shown in figure. Find load carrying capacity of the section.

If 1) Angle are tack riveted

2) Angle are not tack riveted

Solution : 4 ISA 100 x 100 x 10

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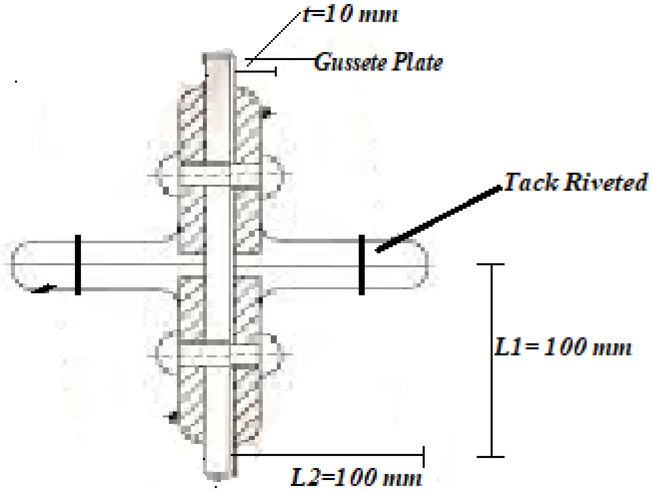
Nominal diameter of rivet=d= 18 mm
Gross diameter of rivet=D= 18+1.5=19.5 mm
Assuming fy=250 N/mm^2
σ_{at}=0.6 fy =0.6 x250 = 150 N/mm^2



1) Angle are tack riveted

$$A_{net} = A_1 + A_2 K$$

$$K = \frac{5A_1}{5A_1 + A_2}$$ IS Code P.No: 37, C. No:4.2.1.2

A_1 = Net area of connected leg

A_2 = Net area of outstanding leg

<u>*Step* 1</u>: To find net area

A_1 = Net area of connected leg

$$A_1 = 4\left[\mathrm{L_1} - \mathrm{D} - \frac{t}{2}\right]t = 4\left[100 - 19.5 - \frac{10}{2}\right]10 = 3020 \text{ mm}^2$$

A_2 = Net area of outstanding leg

$$A_2 = 4\left[\mathrm{L_2} - \frac{t}{2}\right]t = \left[100 - \frac{10}{2}\right]10 = 3800 \text{ mm}^2$$

$$K = \frac{5A_1}{5A_1 + A_2} = \frac{5 \text{ x } 3020}{5 \text{ x } 3020 + 3600} = 0.7989$$

$$A_{net} = A_1 + A_2 K$$

$$A_{net} = 3020 + (3600 \text{ x } 0.7989) = 6055.82 \text{ mm}^2$$

Step 2: Load carrying capacity

$P_t = A_{net} \text{ x } \sigma_{at}$

$P_t = 6055.82 \text{ x } 150 = 908.373 \text{ x } 10^3 N$

$P_t = 908.373\ KN$

2) Angle are not tack riveted

$A_{net} = \mathrm{A_1 + A_2 K}$

$K = \dfrac{3A_1}{3A_1 + A_2}$ IS Code P.No: 37, C. No:4.2.1.1

A_1 = Net area of connected leg

A_2 = Net area of outstanding leg

Step 1: To find net area

A_1 = Net area of connected leg

$$A_1 = 4\left[\mathrm{L_1 - D -} \frac{t}{2}\right]t = 4\left[\ 100\text{- } 19.5 \text{ -}\frac{10}{2}\right]10 = 3020 \text{ mm}^2$$

A_2 = Net area of outstanding leg

$$A_2 = 4\left[\mathrm{L_2} - \frac{t}{2}\right]t = 4\left[100 - \frac{10}{2}\right]10 = 3800 \text{ mm}^2$$

$$K = \frac{3A_1}{3A_1 + A_2} = \frac{3 \text{ x } 3020}{3 \text{ x } 3020 + 3800} = 0.7045$$

$A_{net} = \mathrm{A_1 + A_2 K}$

$A_{net} = 3020 + (3800 \text{ x } 0.7045) = 5697.1 \text{ mm}^2$

Step 2: Load carrying capacity

$P_t = A_{net} \text{ x } \sigma_{at}$

$P_t = 5697.1 \text{ x } 150 = 854.565 \text{ x } 10^3 N$

$P_t = 854.565\ KN$

Design of Tension Member

Design Procedure
Given Data
Tensile Load =P
σ_{at}=0.6 fy =0.6 x250 = 150 N/mm^2
Step 1: Find net effective sectional area

$$A_{net} = \frac{P}{\sigma_{at}}$$

Step 2: Find gross area assuming 20 % to 40 % more

Step 3: Try the section from steel table

Step 4: Find load carrying capacity of trial section

$P_t = A_{net}$ x σ_{at}
$If\ P_t >$ P ------------------ Section is safe
$P_t <$ P ------------------ Section is unsafe, then try another section
Step 5: Design of connection (Riveted or welded)

1. Design tension member in a roof truss to carry a load of 80 KN. The diameter of connecting rivet is 16 mm. Design the connection also, Take Fy= 250 N/mm^2
   Solution: Given Data
   Load= P= 80 KN= 80 X 10^3 N

   σ_{at}=0.6 fy =0.6 x250 = 150 N/mm^2
   d= 16 mm
   D= 16+1.5 = 17.5 mm
   $A_{net} = A_1 + K\ A_2$
   Step 1: Find net effective sectional area

   $$A_{net} = \frac{P}{\sigma_{at}}$$

   $$A_{net} = \frac{80X\ 10^3}{150} = 533.33 \text{ mm}^2$$

   Step 2: Find gross area assuming 30 %
   Assuming gross area 30% more

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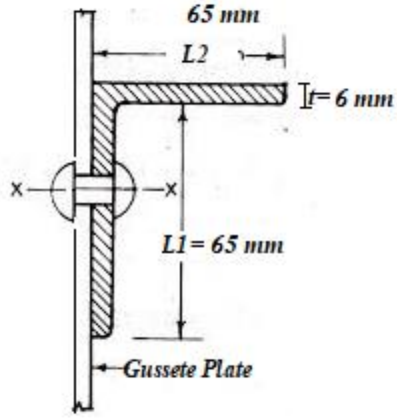
A_{gross}= 1.3 X 533.33 = 693.33 mm^2

<u>Step 3</u>: Try the section from steel table
From Steel Section , try section
ISA 65 X 65 X 6 mm



$A_{net} = A_1 + K\,A_2$

A_1 = Net area of connected leg

$$A_1 = \left[\mathrm{L}_1 \text{- D - } \frac{t}{2}\right]t = \left[\ 65\text{- }17.5\text{ -}\frac{6}{2}\right]6 = 267\ \mathrm{mm}^2$$

A_2 = Net area of outstanding leg

$$A_2 = \left[\mathrm{L}_2 - \frac{t}{2}\right]t = \left[65 - \frac{6}{2}\right]6 = 372\ \mathrm{mm}^2$$

$$K = \frac{3A_1}{3A_1 + A_2} = \frac{3 \text{ x } 267}{3 \text{ x } 267 + 372} = 0.682$$

$$A_{net} = \mathrm{A}_1 + \mathrm{A}_2\mathrm{K}$$

$$A_{net} = 267 + (372 \text{ x } 0.682) = 520.704\ \mathrm{mm}^2$$

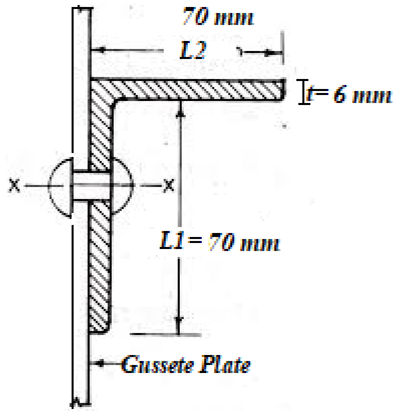
<u>Step 4</u>: Find load carrying capacity of trial section

$$P_t = A_{net} \text{ x } \sigma_{\mathrm{at}} = 520.704 \text{ x } 150 = 78.10 \text{ x } 10^3 \text{ N} = 78.10 \text{ KN } \prec 80 \text{ KN}$$

Unsafe, Try another section

ISA 70 X 70 X 6 mm

$A_{net} = A_1 + K\,A_2$



A_1 = Net area of connected leg

$$A_1 = \left[L_1 - D - \frac{t}{2} \right] t = \left[70 - 17.5 - \frac{6}{2} \right] 6 = 297 \text{ mm}^2$$

A_2 = Net area of outstanding leg

$$A_2 = \left[L_2 - \frac{t}{2} \right] t = \left[70 - \frac{6}{2} \right] 6 = 402 \text{ mm}^2$$

$$K = \frac{3A_1}{3A_1 + A_2} = \frac{3 \text{ x } 297}{3 \text{ x } 297 + 402} = 0.689$$

$$A_{net} = A_1 + A_2 K$$

$$A_{net} = 297 + (402 \text{ x } 0.689) = 573.98 \text{ mm}^2$$

$$P_t = A_{net} \text{ x } \sigma_{at} = 573.98 \text{ x } 150 = 86.098 \text{ x } 10^3 \text{ N} = 86.098 \text{ KN} \succ 80 \text{ KN}$$

Safe

Step 5: Design of connection (Riveted)

Nominal Diameter=d= = 16 mm

Gross Diameter of rivet =D= 16 +1.5 = 17.5 mm

Assuming power driven shop rivets (PDS) (P No:95 Tno:8.1)

τ_{VF}= 100 N/mm^2

σ_{bf}= 300 N/mm^2

$$P_S = \frac{\Pi}{4} \text{ x } D^2 \text{ X } \tau_{Vf}$$

$$P_S = \frac{\Pi}{4} \text{ x } 17.5^2 \text{ X } 100 \quad = 24.052 \text{ x } 10^3 N = 24.052 KN$$

$$P_b = D \text{ x } t \text{ X } \sigma_{bf}$$

$$P_b = 17.5 \text{ x } 6 \text{ X } 300 = 31.5 \text{ x } 10^3 N = 31.5 KN$$

Rivet value= least of Ps and Pb

Rivet value= 24.052 KN

$$N = \frac{Total \text{ Load}}{Rivet \text{ Value}} = \frac{P}{Rv} = \frac{80}{24.052} = 3.32 \cong 4$$

Minimum Pitch = 2.5 X d = 2.5 X16= 40 mm (P no:96, C No:8.10)

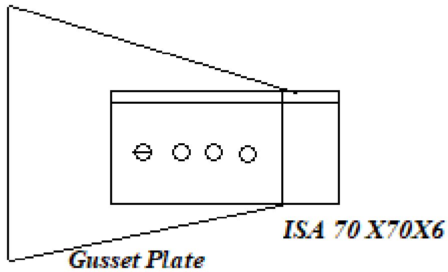
Maximum Pitch = 16 t or 200 mm (Which is less)

= 16 x 6= 96 or 200

= 96 mm

Assuming pitch = P= 50 mm

Edge Distance= 30 mm (P no 97, T No:8.2)



2. Design tension member in a roof truss to carry a load of 100 KN. The diameter of connecting rivet is 20mm. Design the connection also, Take Fy= 250 N/mm^2

Solution: Given Data

Load= P= 100 KN= 100 X 10^3 N

σ_{at}=0.6 fy =0.6 x250 = 150 N/mm^2

d= 20 mm

D= 20+1.5 = 21.5 mm

$A_{net} = A_1 + K\ A_2$

<u>Step 1</u>: Find net effective sectional area

$$A_{net} = \frac{P}{\sigma_{at}}$$

$$A_{net} = \frac{100 X\ 10^3}{150} = 666.67\ \text{mm}^2$$

<u>Step 2</u>: Find gross area assuming 30 %

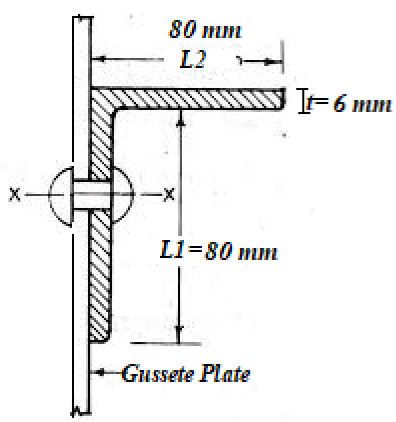
Assuming gross area 30% more

A_{gross}= 1.3 X 666.66 = 866.67 mm^2

<u>Step 3</u>: Try the section from steel table

From Steel Section , try section

ISA 80 X 80 X 6 mm



$A_{net} = A_1 + K\ A_2$

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A_1 = Net area of connected leg

$$A_1 = \left[\text{L}_1 \text{- D - } \frac{t}{2} \right] t = \left[\text{ 80- 21.5 -} \frac{6}{2} \right] 6 = 333 \text{ mm}^2$$

A_2 = Net area of outstanding leg

$$A_2 = \left[\text{L}_2 - \frac{t}{2} \right] t = \left[80 - \frac{6}{2} \right] 6 = 462 \text{ mm}^2$$

$$K = \frac{3A_1}{3A_1 + A_2} = \frac{3 \text{ x } 333}{3 \text{ x } 333 + 462} = 0.683$$

$$A_{net} = \text{A}_1 + \text{A}_2\text{K}$$

$$A_{net} = 333 + (462 \text{ x } 0.683) = 648.55 \text{ mm}^2$$

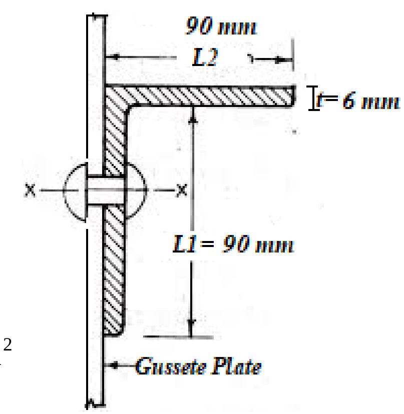
<u>Step 4</u>: Find load carrying capacity of trial section

$$P_t = A_{net} \text{ x } \sigma_{\text{at}} = 648.55 \text{ x } 150 = 97.28 \text{ x } 10^3 \text{ N} = 97.28 \text{ KN } \prec 100 \text{ KN}$$

Unsafe, Try another section

ISA 90 X 90 X 6 mm

$A_{net} = A_1 + K\ A_2$



A_1 = Net area of connected leg

$$A_1 = \left[\text{L}_1 \text{- D - } \frac{t}{2} \right] t = \left[\text{ 90- 21.5 -} \frac{6}{2} \right] 6 = 393 \text{ mm}^2$$

A_2 = Net area of outstanding leg

$$A_2 = \left[\text{L}_2 - \frac{t}{2} \right] t = \left[90 - \frac{6}{2} \right] 6 = 520 \text{ mm}^2$$

$$K = \frac{3A_1}{3A_1 + A_2} = \frac{3 \text{ x } 393}{3 \text{ x } 393 + 520} = 0.693$$

$$A_{net} = \text{A}_1 + \text{A}_2\text{K}$$

$$A_{net} = 393 + (520 \text{ x } 0.689) = 753.36 \text{ mm}^2$$

$$P_t = A_{net} \text{ x } \sigma_{\text{at}} = 753.36 \text{ x } 150 = 113.00 \text{ x } 10^3 \text{ N} = 113.00 \text{ KN } \succ 100 \text{ KN}$$

Safe

<u>Step 5</u>: Design of connection (Riveted)

Nominal Diameter=d= = 20 mm
Gross Diameter of rivet =D= 20 +1.5 = 21.5 mm
Assuming power driven shop rivets (PDS)
τ_{VF}= 100 N/mm^2
σ_{bf}= 300 N/mm^2

$$P_S = \frac{\Pi}{4} \text{ x } D^2 \text{ X } \tau_{Vf}$$

$$P_S = \frac{\Pi}{4} \text{ x } 21.5^2 \text{ X } 100 = 36.31 \text{ x } 10^3 N = 36.31 KN$$

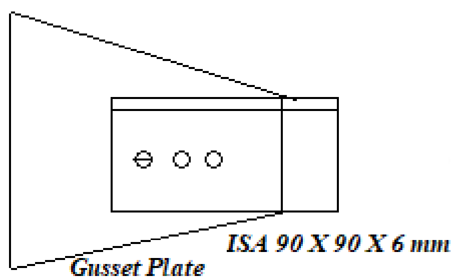
$$P_b = \text{D x t X } \sigma_{bf}$$

$$P_b = 21.5 \text{ x } 6 \text{ X } 300 = 38.7 \text{ x } 10^3 N = 38.7 KN$$

Rivet value= least of Ps and Pb
Rivet value= 36.31 KN

$$N = \frac{\textit{Total} \text{ Load}}{\textit{Rivet} \text{ Value}} = \frac{P}{Rv} = \frac{100 \text{ x } 10^3}{36.31} = 2.75 \cong 3$$

Minimum Pitch = 2.5 X d = 2.5 X 20= 50 mm
Maximum Pitch = 16 t or 200 mm (Which is less)
= 16 x 6= 96 or 200
= 96 mm
Assuming pitch = P= 60 mm
Edge Distance= 35 mm



3. Design tension member in a roof truss to carry a load of 100 KN. Shorter leg of angle section is Connected to gusset plate diameter of rivet is 20 mm. Take Fy= 250 N/mm^2
Solution: Given Data
Load= P= 100 KN= 100 X 10^3 N
σ_{at}=0.6 fy =0.6 x250 = 150 N/mm^2
d= 20 mm
D= 20+1.5 = 21.5 mm
$A_{net} = A_1 + K\,A_2$
<u>Step 1</u>: Find net effective sectional area

$$A_{net} = \frac{P}{\sigma_{at}}$$

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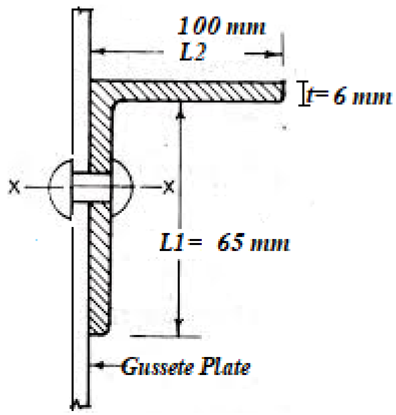
$$A_{net} = \frac{100X\ 10^3}{150} = 666.67\ \text{mm}^2$$

<u>Step 2</u>: Find gross area assuming 30 %
Assuming gross area 30% more
A_{gross}= 1.3 X 666.66 = 866.67 mm^2

<u>Step 3</u>: Try the section from steel table
From Steel Section , try section
ISA 100 X 65 X 6 mm



A_{net} = A_1+K A_2

A_1 = Net area of connected leg

$$A_1 = \left[\text{L}_1\text{- D - } \frac{t}{2} \right] t = \left[\text{ 65- 21.5 -} \frac{6}{2} \right] 6 = 243\ \text{mm}^2$$

A_2 = Net area of outstanding leg

$$A_2 = \left[\text{L}_2 - \frac{t}{2} \right] t = \left[100 - \frac{6}{2} \right] 6 = 582\ \text{mm}^2$$

$$K = \frac{3A_1}{3A_1 + A_2} = \frac{3 \text{ x } 243}{3 \text{ x } 243 + 582} = 0.556$$

$$A_{net} = \text{ A}_1\text{+A}_2\text{K}$$

$$A_{net} = 243\text{+(582 x 0.556) = 566.69 mm}^2$$

<u>Step 4</u>: Find load carrying capacity of trial section

$$P_t = A_{net} \text{ x } \sigma_{at} = 648.55 \text{ x } 150 = 97.28 \text{ x } 10^3 \text{ N =97.28 KN } \prec 100 \text{ KN}$$

Unsafe, Try another section

ISA 100 X 65 X 8 mm
A_{net} = A_1+K A_2

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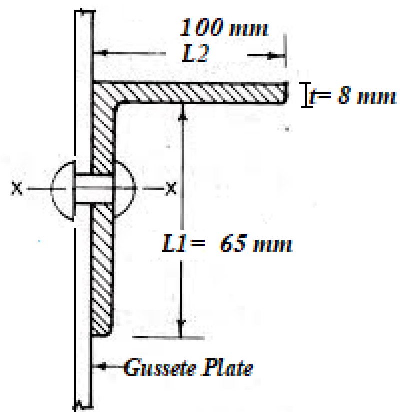

A_1 = Net area of connected leg

$$A_1 = \left[L_1 - D - \frac{t}{2} \right] t = \left[65 - 21.5 - \frac{8}{2} \right] 8 = 316 \text{ mm}^2$$

A_2 = Net area of outstanding leg

$$A_2 = \left[L_2 - \frac{t}{2} \right] t = \left[100 - \frac{8}{2} \right] 8 = 768 \text{ mm}^2$$

$$K = \frac{3A_1}{3A_1 + A_2} = \frac{3 \text{ x } 316}{3 \text{ x } 316 + 768} = 0.55$$

$$A_{net} = A_1 + A_2 K$$

$$A_{net} = 316 + (768 \text{ x } 0.550) = 738.4 \text{ mm}^2$$

Step 4: Find load carrying capacity of trial section

$$P_t = A_{net} \text{ x } \sigma_{at} = 738.4 \text{ x } 150 = 110.76 \text{ x } 10^3 \text{ N} = 110.76 \text{ KN} \succ 100 \text{ KN}$$

Safe

4. Design tension member consisting pair of angles back to back connected shorter leg to the same side of gusset plate. The member is to carry pull of 250 KN, Take Fy= 250 N/mm^2

Solution: Given Data

Load= P= 250 KN= 250 X 10^3 N

σ_{at}=0.6 fy =0.6 x250 = 150 N/mm^2

Step 1: Find net effective sectional area

$$A_{net} = \frac{P}{\sigma_{at}}$$

$$A_{net} = \frac{250X \ 10^3}{150} = 1666.67 \text{ mm}^2$$

Step 2: Find gross area assuming 30 %

Assuming gross area 30% more

A_{gross}= 1.3 X 1666.67 = 2166.67 mm^2 (Gross area for double angle section)

Area of single angle section = 2166.67/2 = 1083.33 mm^2

Step 3: Try the section from steel table

From Steel Section, try section

ISA 100X 65 X 8 mm

Nominal diameter

$$d = 6.04 \sqrt{t} = 6.04 \sqrt{8} = 17.08 \text{ mm} \cong 18 \text{ mm}$$

Gross Diameter= D= 18+1.5 =19.5 mm

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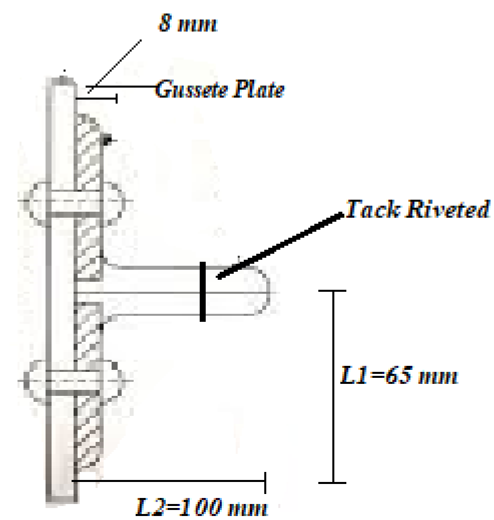


$A_{net} = A_1 + K\ A_2$

A_1 = Net area of connected leg

$$A_1 = 2\left[\mathrm{L}_1 \text{ - D - } \frac{t}{2}\right]t = 2\left[\ 65\text{- }19.5\text{ -}\frac{8}{2}\right]8 = 664\ \mathrm{mm}^2$$

A_2 = Net area of outstanding leg

$$A_2 = 2\left[\mathrm{L}_2 - \frac{t}{2}\right]t = 2\left[100 - \frac{8}{2}\right]8 = 1536\ \mathrm{mm}^2$$

$$K = \frac{5A_1}{5A_1 + A_2} = \frac{5 \text{ x } 664}{5 \text{ x } 664 + 1536} = 0.684$$

$A_{net} = \mathrm{A}_1 + \mathrm{A}_2\mathrm{K}$

$A_{net} = 664 + (1536 \text{ x } 0.684) = 1714.62\ \mathrm{mm}^2$

<u>Step 4</u>: Find load carrying capacity of trial section

$P_t = A_{net} \text{ x } \sigma_{\mathrm{at}} = 1714.62 \text{ x } 150 = 257.19 \text{ x } 10^3 \text{ N} = 257.19 \text{ KN} \succ 250 \text{ KN}$

Safe

5. Design tension member consisting of two angles section connected shorter leg to the both side of gusset plate. The member is to carry pull of 270 KN, Take Fy= 250 N/mm^2

   Solution: Given Data

   Load= P= 270 KN= 270 X 10^3 N

   σ_{at}=0.6 fy =0.6 x250 = 150 N/mm^2

   <u>Step 1</u>: Find net effective sectional area

$$A_{net} = \frac{P}{\sigma_{at}}$$

$$A_{net} = \frac{270 X \; 10^3}{150} = 1800 \text{ mm}^2$$

Step 2: Find gross area assuming 30 %

Assuming gross area 30% more

A_{gross}= 1.3 X 1800 = 2340 mm^2 (Gross area for double angle section)

Area of single angle section = 2340/2 = 1170 mm^2

Step 3: Try the section from steel table

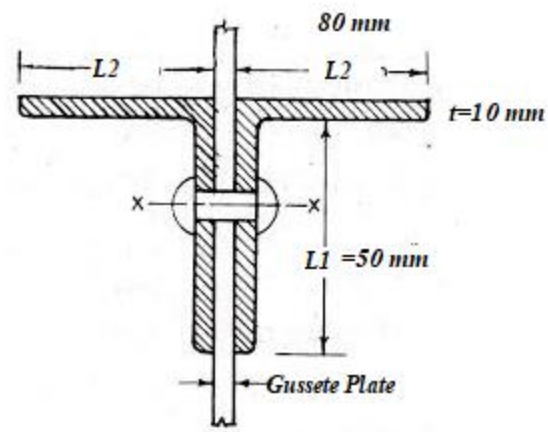
From Steel Section, try section

ISA 80X 50 X 10 mm

Nominal diameter

$$d = 6.04\sqrt{t} = 6.04\sqrt{10} = 19.08 \text{ mm} \cong 20 \text{ mm}$$

Gross Diameter= D= 20+1.5 =21.5 mm



$A_{net} = A_1 + K\,A_2$

A_1 = Net area of connected leg

$$A_1 = 2\left[\text{L}_1\text{- D - }\frac{t}{2}\right]t = 2\left[\text{ 50- 21.5 -}\frac{10}{2}\right]10 = 470 \text{ mm}^2$$

A_2 = Net area of outstanding leg

$$A_2 = 2\left[\text{L}_2 - \frac{t}{2}\right]t = 2\left[80 - \frac{10}{2}\right]10 = 1500 \text{ mm}^2$$

$K = 1$

$A_{net} = \text{ A}_1\text{+A}_2\text{K}$

$A_{net} = 470\text{+(1500 x 1) = 1970 mm}^2$

Step 4: Find load carrying capacity of trial section

$P_t = A_{net} \text{ x } \sigma_{\text{at}} = 1970 \text{ x 150 = 295.5 x } 10^3 \text{ N =295.5 KN } \succ 270 \text{ KN}$

Safe

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6. A single angle section ISA 125 X 75 X 8 mm connected by its longer leg to gusset plate 10 mm thick carries an axial load to its full capacity. Design welded connection with 8 mm fillet weld, load should be axial to weld, permissible shear stress is 100 Mpa.
   Solution: Given Data

Tension member subjected to Direct tension and bearing

1. **Design a tension member to carry axial load of 1500 KN, length of member is 5 m, consider the effect of deflection due to its own weight.** Take Fy= 250 N/mm^2

Solution: Given Data

Load= P= 1500 KN=1500 X 10^3 N

σ_{at}=0.6 fy =0.6 x250 = 150 N/mm^2

Step 1: Find net effective sectional area

$$A_{net} = \frac{P}{\sigma_{at}}$$

$$A_{net} = \frac{1500X\ 10^3}{150} = 10000\ \text{mm}^2$$

Step 2: Find gross area assuming 30 %

Assuming gross area 30% more

A_{gross}= 1.3 X 10000 = 13000 mm^2

Step 3: Try the section from steel table

From Steel Section, try section

2ISA 200X 200 X 18 mm

A=6881 mm^2 (Single angle section)

A=2X6881=13762 mm^2 (Double angle section)

Self-weight= 108kg/m

rxx=61.3 mm

Ixx= 2588.7 x 10^4 mm^4 (For Single section)

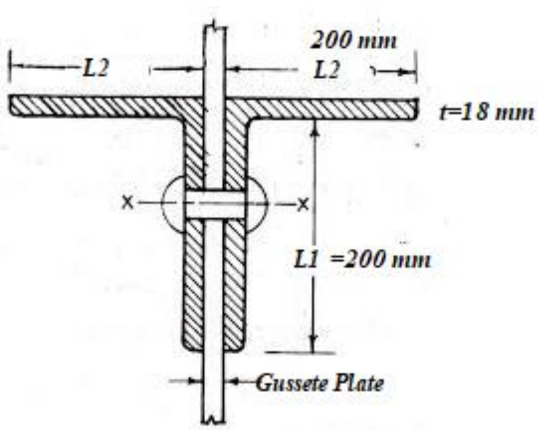
Ixx= 5177.4 x 10^4 mm^4 (For double section)

Assuming angles are connected to gusset plate on both side

Nominal diameter

$$d = 6.04\sqrt{t} = 6.04\sqrt{18} = 25.7\text{ mm} \cong 26\text{ mm}$$

Gross Diameter= D= 26+2 =28 mm



$A_{net} = A_1 + K\,A_2$

A_1 = Net area of connected leg

$$A_1 = 2\left[\mathrm{L_1}\text{- D - }\frac{t}{2}\right]t = 2\left[\ 200\text{- }28\text{ -}\frac{18}{2}\right]18 = 5868\ \mathrm{mm}^2$$

A_2 = Net area of outstanding leg

$$A_2 = 2\left[\mathrm{L_2} - \frac{t}{2}\right]t = 2\left[200 - \frac{18}{2}\right]18 = 6876\ \mathrm{mm}^2$$

$K = 1$

$A_{net} = \mathrm{A_1 + A_2 K}$

$A_{net} = 5868 + (6876 \text{ x } 1) = 12745\ \mathrm{mm}^2$

<u>Step 4</u>: Find load carrying capacity of trial section

$P_t = A_{net} \text{ x } \sigma_{\mathrm{at}} = 12745 \text{ x } 150 = 1911.75 \text{ x } 10^3$ N $= 1911.75$ KN $\succ 1500$ KN

Safe

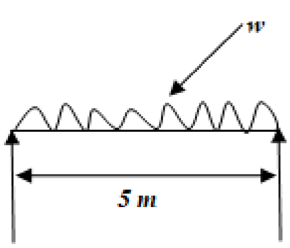
<u>Step 5</u>: Self weight

Self-weight= 54 kg/m (Single Angle section)

Self-weight= 108 kg/m

Self-weight=108 x 9.81 = 1059.48 N/m

B.M Due to self weight



$$M = \frac{Wl^2}{8} = \frac{1059.48\ x\ 5^2}{8} = 3310.875\,Nm = 3310.875 \text{ x } 10^3\,Nmm$$

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$$Anet_{req} = \frac{P}{\sigma_{at}} + \frac{M\ \mathrm{C_{xx}}}{\sigma_{at}\ \mathrm{r^2}_{xx}} = \frac{1500 \text{ x } 10^3}{150} + \frac{3310.87 \text{ x } 10^3 \text{x } 56.1}{150 \text{ x } 61.3^2} = 10329.5\ mm^2$$

$$Anet_{\mathrm{Provided}} = 12475 \text{ mm}^2$$

$$Anet_{\mathrm{Provided}} \succ Anet_{req}, \quad \text{ok}$$

<u>Step 6</u>: Consider effect of deflection

$$M^{'} = \frac{M}{1 + \frac{Pl^2}{10EI}} = \frac{3310.875 \text{ x } 10^3}{1 + \frac{1500 \text{ x } 10^3 \text{ x } 5000^2}{10 \text{ x } 2 \text{ x } 10^5 \text{ x } 5177.4 \text{ x } 10^4}} = 2.427 \text{ x } 10^6 Nmm$$

$$Anet^{'}_{req} = \frac{P}{\sigma_{at}} + \frac{M^{'}\ \mathrm{C_{xx}}}{\sigma_{at}\ \mathrm{r^2}_{xx}} = \frac{1500 \text{ x } 10^3}{150} + \frac{2.427 \text{ x } 10^6 \text{x } 56.1}{150 \text{ x } 61.3^2} = 10241.5\ mm^2$$

$$Anet_{\mathrm{Provided}} = 12475 \text{ mm}^2$$

$$Anet_{\mathrm{Provided}} \succ Anet^{'}_{req}, \quad \text{ok}$$

Check

$$\frac{\sigma^{'}_{at}}{\sigma_{at}} + \frac{\sigma^{'}_{bt}}{\sigma_{bt}} \le 1$$

$$\sigma^{'}_{at} = \frac{P}{A_{Net}(provided)} = \frac{1500 \text{ x } 10^3}{12745} = 117.69\ N/mm^2$$

$$\sigma_{at} = 0.6 \text{ x } \mathrm{F_y} = 0.6x250 = 150\ N/mm^2$$

$$\sigma^{'}_{bt} = \frac{M}{Ixx}Cxx = \frac{3310.87x10^3}{5177.4x10^4}x56.1 = 3.58\ N/mm^2$$

$$\sigma_{bt} = 0.66 \text{ x } \mathrm{F_y} = 0.66x250 = 165\ N/mm^2$$

$$\frac{117.69}{150} + \frac{3.58}{165} \le 1$$

$$0.806 \le 1$$

$$\mathrm{Pr}\,oviding\ 200 \text{ x } 200 \text{ x } 18 \text{ mm}$$

2. **Design 8 m long tension member of heavy truss to carry axial load of 2500 KN, Considering effect of deflection due to its own weight.** Take Fy= 250 N/mm^2

Design of tension splice

When a butt joint is covered by plates on both sides, then it called splicing. The cover plates used to join tension members are known as tension splice. Tension splicing is done in the following two cases

1) When the size of tension member changes at different lengths.
2) When the length of the section available is less than the required

In the tension of tension splice, splice plates and rivets are designed for the pull required to be transmitted by the tension member.

If the tension members to be joined are unequal thickness, then, packing is required to fill the gap. As per IS-800-1984, If the thickness of packing is grater than 6 mm, the number of rivets required by normal calculation shall be increased by 2.5% for each 2 mm thickness of the packing. For double shear condition packed on both sides, the number of additional rivets required shall be determined from the thickness of the thicker packing. The additional rivets should preferably be placed in an extension of the tacking.

The design of the tension splice is similar to that of other tension members.

Design Procedure

Given Data:

<u>Step 1:</u> To find shearing strength of rivet

P_S = Number of rivets X Area of rivet in shearing X Permissible shear stress

$$P_S = 2 \text{ x } \frac{\Pi}{4} \text{ x } D^2 \text{ X } \tau_{Vf} \qquad \text{For Double Shear}$$

Where N = Number of rivets in a joint

D = Gross diameter of Rivet

τ_{Vf} = Permissible shear stress in rivet (IS 800-1984 Page No: 95, Table :8.1)

<u>Step 2:</u> To find bearing strength of rivet

P_b = Number of rivets in joint X Bearing area of rivet X Permissible bearing stress in rivet

Bearing area of rivet = area of rectangle

Whose one side is gross diameter of rivet and other side is thickness of plate

$P_b = \text{ D x t X } \sigma_{bf}$

D = Gross diameter of Rivet

t=Thickness of plate

σ_{bf} = Permissible bearing stress in rivet (IS 800-1984 Page No: 95, Table :8.1)

<u>Step 3:</u> To find rivet value

Rivet value is least of P_s and P_b

<u>Step 4:</u> To find number of rivets

$$N = \frac{Total\ \text{Load}}{Rivet\ \text{Value}} = \frac{P}{Rv}$$

Step 5: To find strength of splice plate

$Strength$ of splice plate = 2 x (b-nD) x t X σ_{at} (for one flat of size)

$Strength$ of splice plate = 1 x (b-nD) x t X σ_{at} (for two flat of size)

1. **Design a splice for a tie member in a bridge. Tie bar is composed of one flat of size 260 mm X 15 mm and carries a maximum load of 500 KN. The diameter of rivet is 20 mm and use power driven shop rivets.**
   Solution: Given Data
   P= 500KN =500 x 10^3 N
   Flat Size 260 mm X 15 mm
   Thickness t=15 mm
   Nominal Diameter= d=20 mm
   Gross Diameter =D= 20+1.5=21.5 mm
   power driven shop rivets
   $\tau_{vf} = 100\ \text{N/mm}^2$
   $\sigma_{bf} = 300\ \text{N/mm}^2$

Step 1: To find shearing strength of rivet

P_S = Number of rivets X Area of rivet in shearing X Permissible shear stress

$$P_S = 2 \text{ x} \frac{\Pi}{4} \text{ x D}^2 \text{ X } \tau_{Vf} \qquad \text{For Double Shear}$$

$$P_S = 2 \text{ x} \frac{\Pi}{4} \text{ x } 21.5^2 \text{ X } 100$$

$$P_S = 72610N = 72.610KN$$

Where N = Number of rivets in a joint

D = Gross diameter of Rivet

τ_{Vf} = Permissible shear stress in rivet (IS 800-1984 Page No: 95, Table :8.1)

<u>Step 2:</u> To find bearing strength of rivet

P_b = Number of rivets in joint X Bearing area of rivet X Permissible bearing stress in rivet

Bearing area of rivet = area of rectangle

Whose one side is gross diameter of rivet and other side is thickness of plate

P_b = D x t X σ_{bf}

P_b = 21.5 x 15 X 300

$P_b = 96750N = 96.750KN$

D = Gross diameter of Rivet

t=Thickness of plate

σ_{bf} = Permissible bearing stress in rivet (IS 800-1984 Page No: 95, Table :8.1)

<u>Step 3:</u> To find rivet value

Rivet value is least of P_s and P_b

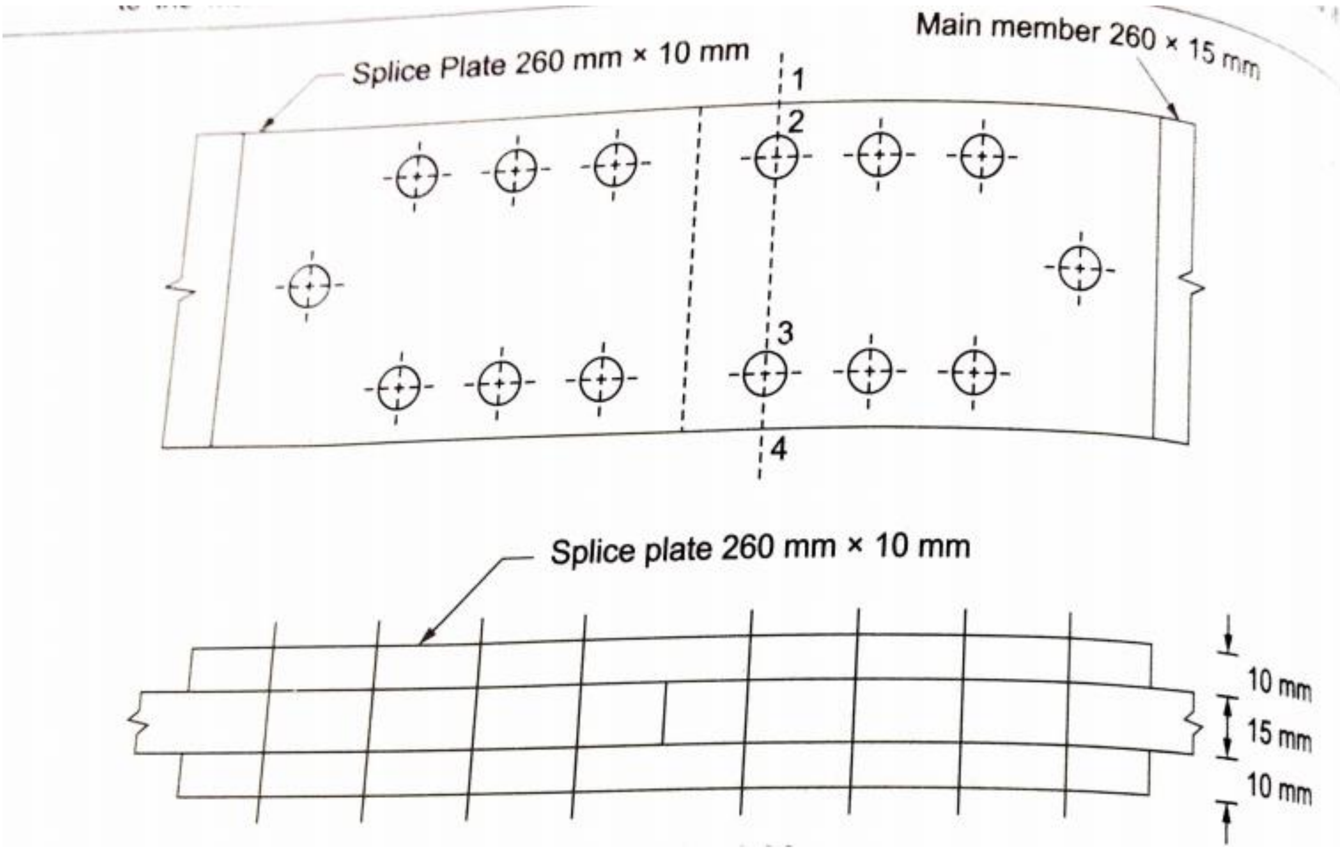
Rivet value=72.610 KN

<u>Step 4:</u> To find number of rivets

$$N = \frac{Total \text{ Load}}{Rivet \text{ Value}} = \frac{P}{Rv} = \frac{500}{72.61} = 6.89 \cong 7$$

<u>Step 5:</u> To find strength of splice plate

Let Size of splice plate be 260 mm X 10 mm



N=2 (LOOK THE DIAGRAM)

Strength of splice plate = 2 x (b-nD) x t X σ_{at} (for one flat of size)

Strength of splice plate = 2 x (260-2 x 21.5) x 10 X 150

Strength of splice plate $= 651000N = 651KN \succ 500KN$ (OK)

2. **Design a splice to connect two flats of sizes 250 mm X 20 mm and 250 mm X 14 mm. The maximum load in the member is 300 KN. The diameter of rivet is 20 mm and use power driven shop rivets.**
   Solution: Given Data
   P= 300KN =300 x 10^3 N
   Flat Size 250 mm X 20 mm
   250 mm X 14 mm
   Thickness t=14 mm (take least thickness)
   Nominal Diameter= d=20 mm
   Gross Diameter =D= 20+1.5=21.5 mm
   power driven shop rivets
   $\tau_{vf} = 100$ N/mm^2
   $\sigma_{bf} = 300$ N/mm^2

<u>Step 1:</u> To find shearing strength of rivet

P_S = Number of rivets X Area of rivet in shearing X Permissible shear stress

$P_S = 2 \text{ x} \frac{\Pi}{4} \text{ x } D^2 \text{ X } \tau_{Vf}$ For Double Shear

$P_S = 2 \text{ x} \frac{\Pi}{4} \text{ x } 21.5^2 \text{ X } 100$

$P_S = 72610N = 72.610KN$

Where N = Number of rivets in a joint

D = Gross diameter of Rivet

τ_{Vf} = Permissible shear stress in rivet (IS 800-1984 Page No: 95, Table :8.1)

<u>Step 2:</u> To find bearing strength of rivet

P_b = Number of rivets in joint X Bearing area of rivet X Permissible bearing stress in rivet

Bearing area of rivet = area of rectangle

Whose one side is gross diameter of rivet and other side is thickness of plate

$P_b = \text{ D x t X } \sigma_{bf}$

$P_b = 21.5 \text{ x } 14 \text{ X } 300$

$P_b = 90300N = 90.3KN$

D = Gross diameter of Rivet

t=Thickness of plate

σ_{bf} = Permissible bearing stress in rivet (IS 800-1984 Page No: 95, Table :8.1)

<u>Step 3:</u> To find rivet value

Rivet value is least of P_s and P_b

Rivet value=72.610 KN

<u>Step 4:</u> To find number of rivets

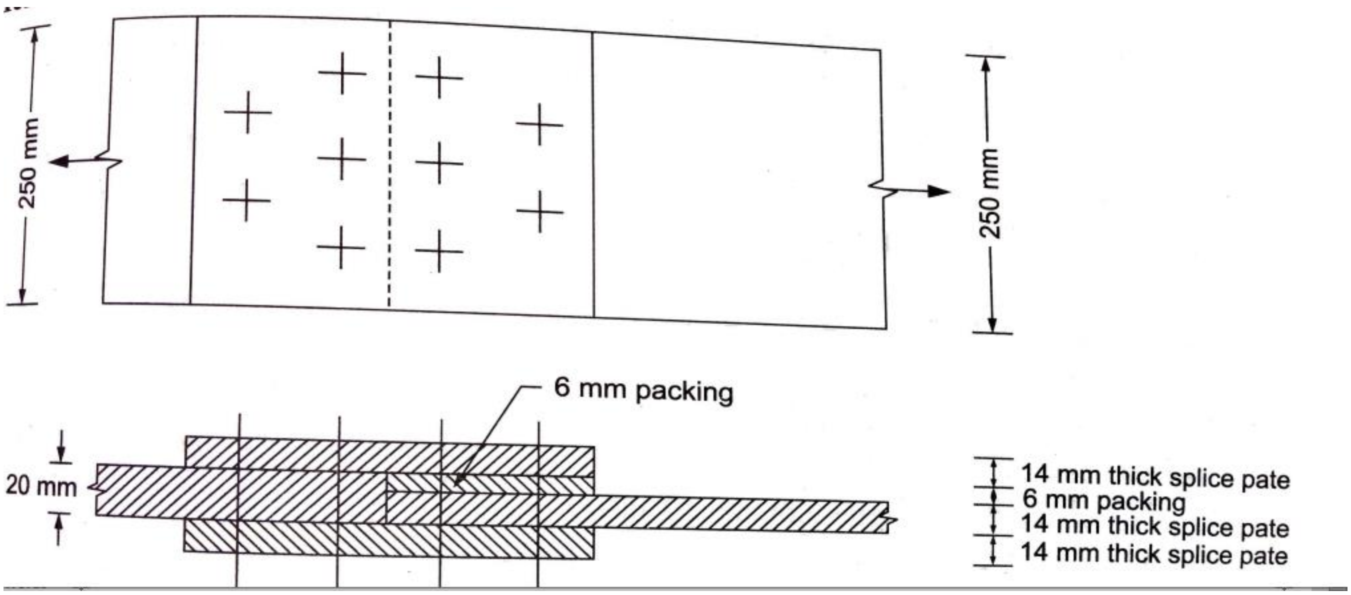
$$N = \frac{Total \text{ Load}}{Rivet \text{ Value}} = \frac{P}{Rv} = \frac{300}{72.61} = 4.13 \cong 5$$

Thickness of packing = 20-14= 6 mm

As the thickness of packing is 6 mm, no extra rivet is required

<u>Step 5:</u> To find strength of splice plate

Assuming Size of splice plate be 250 mm X 14 mm



N=3 (LOOK THE DIAGRAM)

Strength of splice plate = (b-nD) x t X σ_{at} (for two flat of size)

Strength of splice plate = (250-3 x 21.5) x 14 X 150

Strength of splice plate $= 389550N = 389.55KN \succ 300KN$ (OK)

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Compression Member

A member subjected to compressive force is called compression member. In the structure vertical member carrying compressive force is called as column or stanchion.

In roof truss member carrying compressive force are called strut. Compression member in crane is called boom.

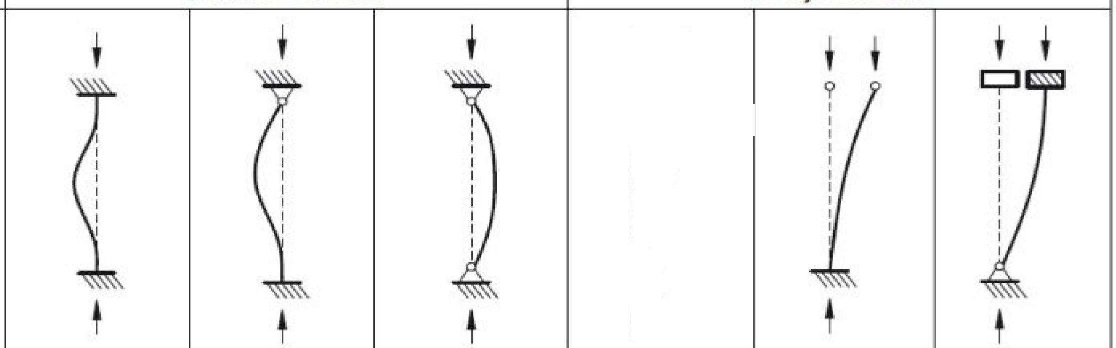
Compression members consist of following types

1. To find strength of compression member
2. Design of axially loaded compression member
3. Design of lacing
4. Design of battening
5. Design of eccentrically loaded compression member

Buckling load: A load at which column collapse is called buckling load. The direction of buckling of column depends upon EI of the column. Column buckles in direction about which MI is minimum.

Effective length of compression member:

It depends upon end condition, as per IS Code P. No:41

	Braced member			Sway member		
Buckled shape						
Effective length factor (k_e)	l=0.65 L	l=0.8 L	l=L		l=2 L	l= 2 L
Symbols for end restraint conditions	= Rotation fixed, translation fixed			= Rotation fixed, translation free		
	= Rotation free, translation fixed			= Rotation free, translation free		

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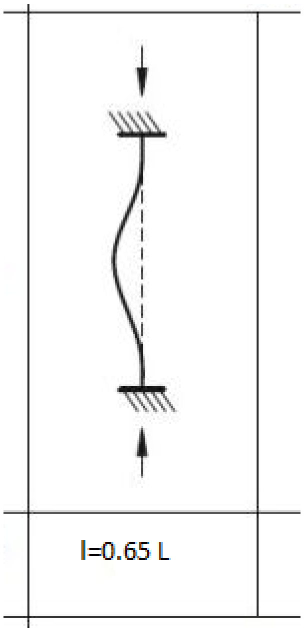
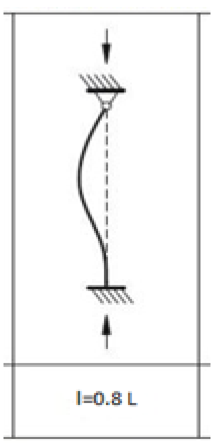
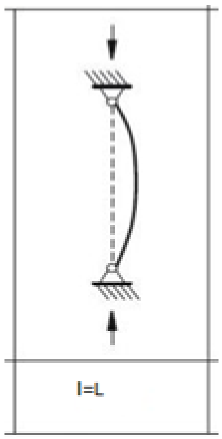
1. Both end fixed
   l= 0.65 L



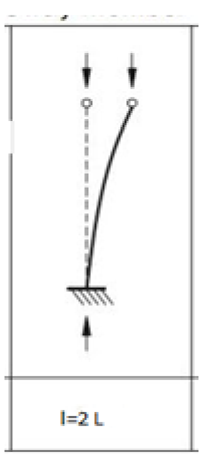
2. One end fixed other hinged
   l= 0.8 L



3. Both ends hinged
   $l = L$



4. One end fixed other free



$l = 2L$

Slenderness ratio: It is the ratio of effective length to minimum radius of gyration.

$$\lambda = \frac{l_{eff}}{r_{\min}}$$

l_{eff}=Effective length

r_{min}= Minimum radius of gyration

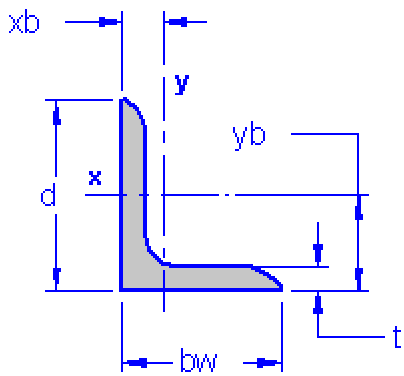
Permissible axial compressive stress:

It depend upon slenderness ratio, λ and Fy

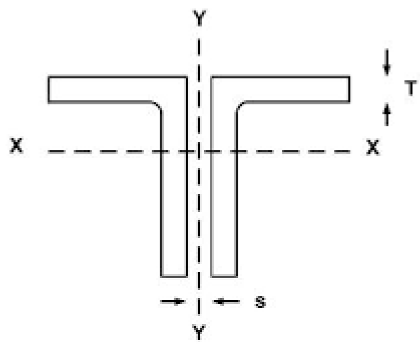
Types of compression member

Sections for compression members

1) Single angle section

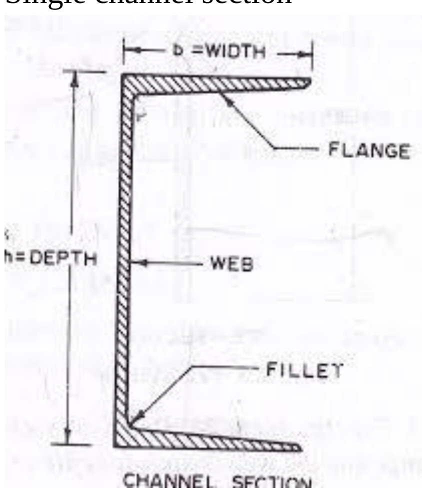


2) Double angle section

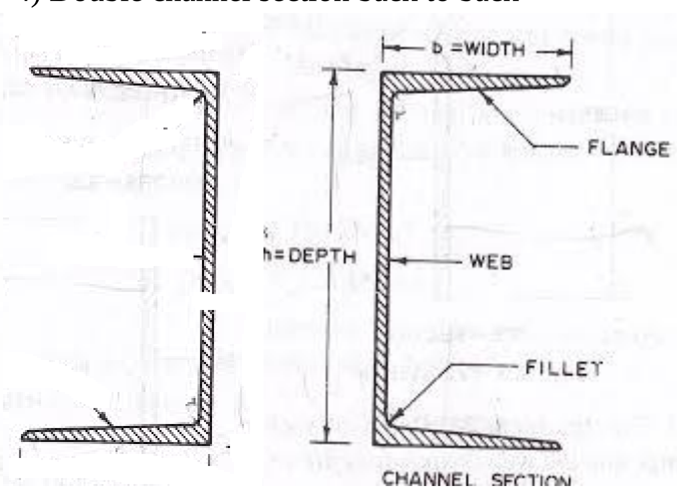


Double Angles

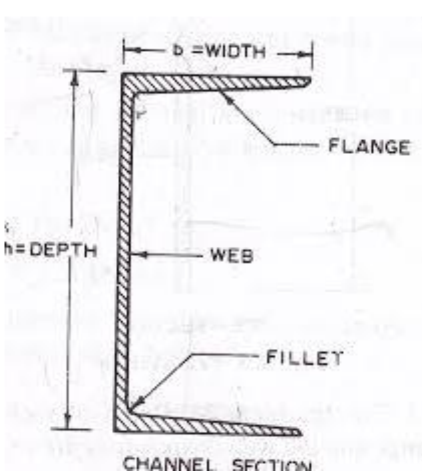
3) Single channel section



4) Double channel section back to back



4) Double channel section face to face



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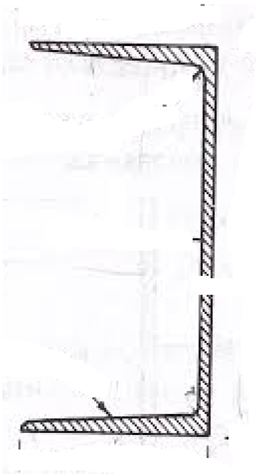
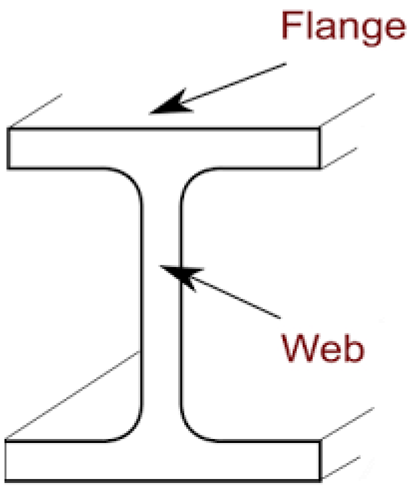
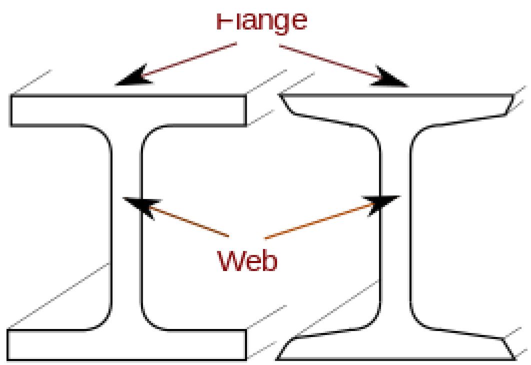


5) I section

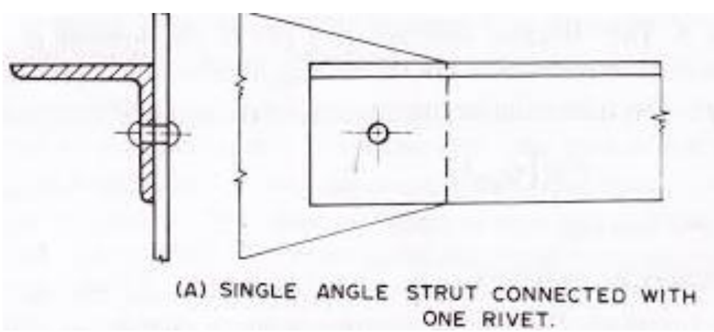
6) Double I section





7) IS 800-1984, Page No: 46

a) Single angle discontinuous strut (Connected by one rivet only)



l_{eff}=L , σac=0.8 X σac (Calculated)

To find σac From table 5.1, P No:39

b) Single angle discontinuous strut (Connected by two or more rivet only)

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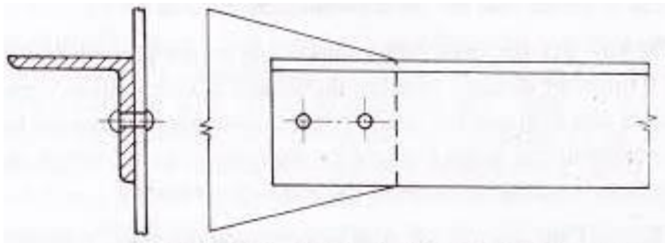
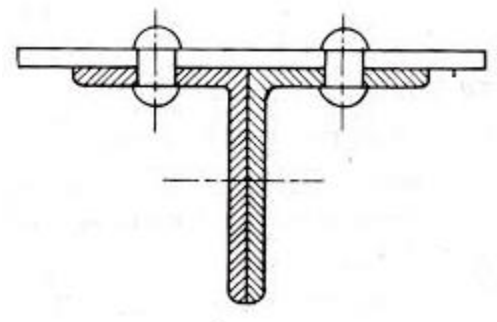


(B) SINGLE ANGLE STRUT CONNECTED TWO OR MORE RIVETS.

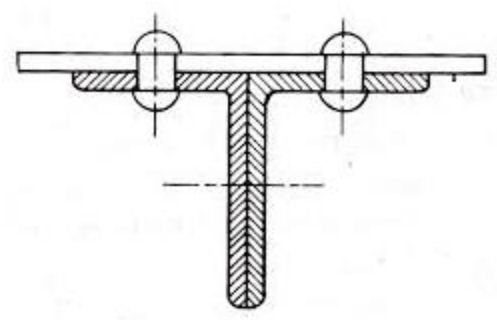
l_{eff}=0.85L
σac= σac (Calculated)

c) Double angle discontinues strut (same side of gusset plate, only one rivet/weld)



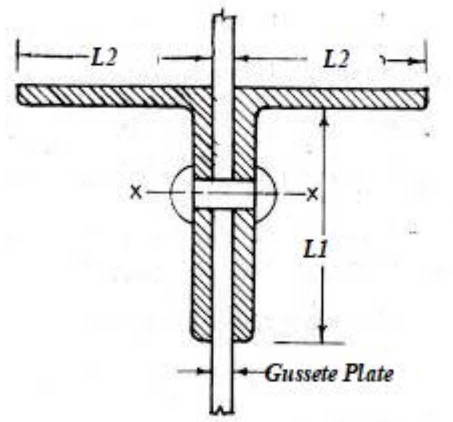
l=L
σac=0.8 X σac (Calculated)

d) Double angle discontinues strut (same side of gusset plate, connected by two or more rivet/weld)



l=0.85L
σac=σac (Calculated)

e) Double angle discontinues strut back to back (not less than two rivets)



l=0.7L to 0.85 L

σac=σac (Calculated)

Continuous member : Compression member consisting of a single or double angle which are continuous over a number of joints known as continuous member. The top chord member of truss girder or principle rafter of roof truss is known as continuous member.

For continuous strut

l=0.7L to L

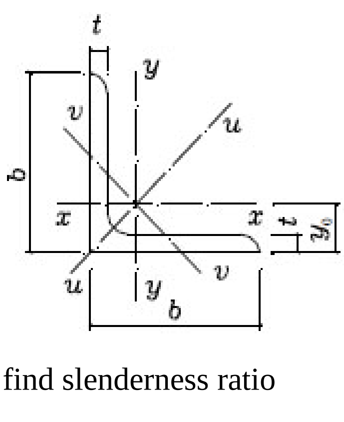
σac=σac (Calculated)

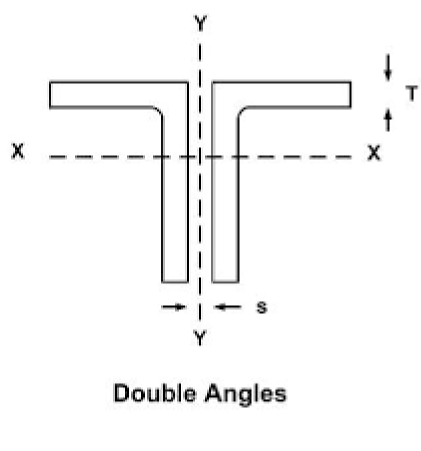
Type I : To find strength of member

Design Procedure:

Given data: End condition , effective length and Fy

1) From steel table , find the sectional properties of the given section
   A, rxx, ryy, ruu, rvv





Double Angles

2) To find slenderness ratio

$$\lambda = \frac{l_{eff}}{r_{\min}}$$

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3) From IS 800-1984, Page No: 39 Table No:5.1
   Find σac
   λ σac
4) Load carrying capacity or strength of compression member

$$P_c = A_{gross} \text{ x } \sigma_{ac}$$

1. Find the capacity of single angle strut ISA 80 X 80 X 8 mm, length of strut between centre to centre of intersection is 2.1 m and Single rivet is used. Fy= 250 N/mm^2

   Solution: Given data
   ISA 80 X 80 X 8 mm

   L=2.1 m=2100 mm

   For single angle and single rivet (one rivet)
   l_{eff}=L=2100 mm
   σac=0.8 X σac (Calculated)
   Fy= 250 N/mm^2

1. From steel table , find the sectional properties of the given section

   From steel table
   A_{gross} = 1221 mm^2
   rxx= 24.4 mm
   ryy= 24.4 mm
   ruu= 30.8 mm
   rvv= 15.5 mm
   rmin=15.5 mm

2. To find slenderness ratio

$$\lambda = \frac{l_{eff}}{r_{min}} = \frac{2100}{15.5} = 135.48$$

3. From IS 800-1984, Page No: 39 Table No:5.1, Fy= 250 N/mm^2

λ	σac
130	57
135.48	?
140	51

By interpolation

$$\sigma ac \ \left(Calculated\right) = 57 + \left[\frac{(51-57)}{(140-130)} \text{ X } (135.48\text{-}130)\right] = 53.7 \text{ N/mm}^2$$

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σac=0.8 X σac (Calculated) = 0.8 x 53.7 = 42.96 N/mm^2

4. Load carrying capacity or strength of compression member

$$P_c = A_{gross} \text{ x } \sigma_{ac}$$

P_c = 1221 x 42.96 = 52.45 x 10^3 N

P_c = 52.45 KN

2. Find the capacity of single angle strut ISA 70 X 45 X 8 mm, length of strut between centre to centre of intersection is 2 m and connected by more than two rivets. Use Fy= 250 N/mm^2

Solution: Given data

ISA 70 X 45 X 8 mm

L=2 m=2000 mm

For single angle and connected by more than two rivets

l_{eff} =0.85 L=0.85 x 2000=1700 mm

σac= σac (Calculated)

Fy= 250 N/mm^2

1. From steel table , find the sectional properties of the given section

From steel table

A_{gross} = 858 mm^2

rxx= 21.2 mm

ryy= 12.4 mm

ruu= 23.2mm

rvv= 9.5 mm

rmin=9.5 mm

2.To find slenderness ratio

$$\lambda = \frac{l_{eff}}{r_{\min}} = \frac{1700}{9.5} = 178.947$$

3.From IS 800-1984, Page No: 39 Table No:5.1, Fy= 250 N/mm^2

λ	σac
170	37
178.947	?
180	33

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By interpolation

$$\sigma ac\ \left(Calculated\right) = 37 + \left[\frac{(33-37)}{(180-170)}\ \text{X}\ (178.95\text{-}170)\right] = 33.42\ \text{N/mm}^2$$

σac= σac (Calculated) = 33.42 N/mm^2

4.Load carrying capacity or strength of compression member

$$P_c = A_{gross}\ \text{x}\ \sigma_{ac}$$

P_c =858 x 33.42 = 28.67x 10^3 N

P_c = 28.67 KN

3. Find the capacity of strut 2.5 m long with 2 ISA 90 X 60 X 8 mm connected on each side of gusset plate of 10 mm thick. Use Fy= 250 N/mm^2

   Solution: Given data

   2ISA 90 X 60 X 8 mm

   L=2.5 m=2500 mm

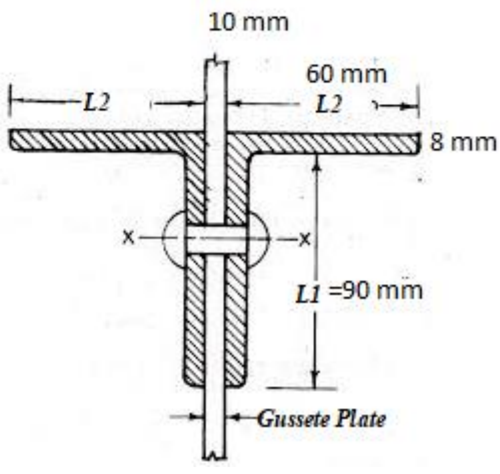
   For angle connected on each side of gusset plate

   l_{eff} =0.85 L=0.85 x 2500=2125 mm

   σac= σac (Calculated)

   Fy= 250 N/mm^2

1.From steel table , find the sectional properties of the given section



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From steel table
2ISA 90 X 60 X 8 mm
A_{gross} = 2274 mm^2
rxx= 28.4 mm
ryy= 26 mm (Back to back distance 10 mm thick gusset plate)
rmin=26 mm

2.To find slenderness ratio

$$\lambda = \frac{l_{eff}}{r_{\min}} = \frac{2125}{26} = 81.73$$

3.From IS 800-1984, Page No: 39 Table No:5.1, Fy= 250 N/mm^2

λ	σac
80	101
81.73	?
90	90

By interpolation

$$\sigma ac\ \left(Calculated\right) = 101 + \left[\frac{(90-101)}{(90-80)}\ \text{X (81.73-80)}\right] = 99.09\ \text{N/mm}^2$$

σac= σac (Calculated) = 99.09 N/mm^2

4.Load carrying capacity or strength of compression member

$$P_c = A_{gross} \text{ x } \sigma_{ac}$$

P_c =2274 x 99.09 = 225.33x 10^3 N
P_c = 225.33 KN

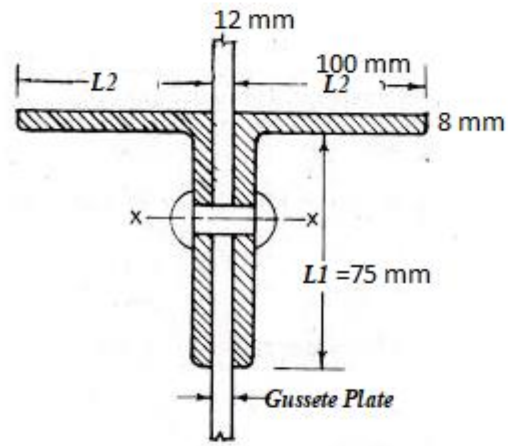
4. Find the capacity of strut 2 ISA 100 X75 X 8 mm connected to the each side of gusset plate, shorter leg is connected to gusset plate 12 mm thick, length of strut is 2 m. Use Fy= 250 N/mm^2
   Solution: Given data
   2ISA 100 X 75 X 8 mm

   L=2 m=2000 mm

   For angle connected on each side of gusset plate
   l_{eff} =0.85 L=0.85 x 2000=1700 mm

σac= σac (Calculated)
Fy= 250 N/mm^2

1.From steel table , find the sectional properties of the given section



From steel table
Shorter leg connected to gusset plate
2ISA 100 X 75 X 8 mm
A $_{gross}$ = 2672 mm^2
rxx= 21.8 mm
ryy= (47.8+49.3)/2= 48.55 mm (Back to back distance 12 mm thick gusset plate)
rmin=21.8 mm

2.To find slenderness ratio

$$\lambda = \frac{l_{eff}}{r_{\min}} = \frac{1700}{21.8} = 77.98$$

3.From IS 800-1984, Page No: 39 Table No:5.1, Fy= 250 N/mm^2

λ	σac
70	112
77.98	?
80	101

By interpolation

$$\sigma ac\ \left(Calculated\right) = 112 + \left[\frac{(101-112)}{(80-70)}\ \text{X (77.98-70)}\right] = 103.22\ \text{N/mm}^2$$

σac= σac (Calculated) = 103.22 N/mm^2

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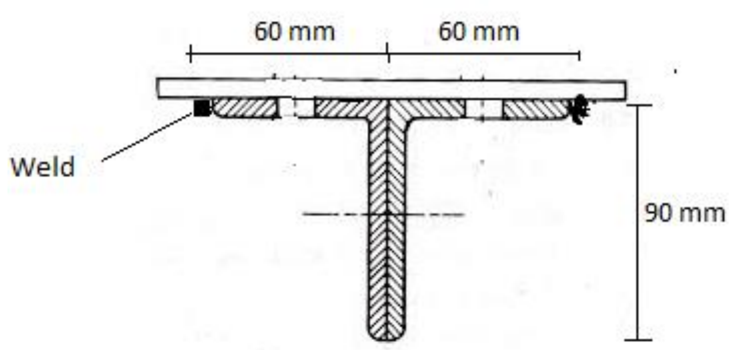
4.Load carrying capacity or strength of compression member

$$P_c = A_{gross} \times \sigma_{ac}$$
$$P_c = 2672 \times 103.22 = 275.80 \times 10^3 \text{ N}$$
$$P_c = 275.80 \text{ KN}$$

5. A member in a structure has cross section consisting of 2 ISA 90 X60 X 8 mm connected on same side of gusset plate with shorter leg welded to gusset plate by 6 mm fillet weld it is 2.5 m long with tack welding along length.
   Find load carrying capacity in the member for the following cases
   a) Member carrying tension under (Dead load+ Live Load)
   b) Member carrying compression under (Dead load+ Wind Load)
      Fy= 250 N/mm^2

      Solution: Given data
      2ISA 90 X 60 X 8 mm
      l=2.5 m=2500 mm
      Fy= 250 N/mm^2



   a) Member carrying tension under (Dead load+ Live Load)

$\underline{Step}$ 1: To find net area

A_1 = Net area of connected leg

$$A_1 = 2\left[L_1 - \frac{t}{2}\right]t = 2\left[60 - \frac{8}{2}\right]8 = 896 \text{ mm}^2$$

A_2 = Net area of outstanding leg

$$A_2 = 2\left[L_2 - \frac{t}{2}\right]t = 2\left[90 - \frac{8}{2}\right]8 = 1376 \text{ mm}^2$$

$$K = \frac{5A_1}{5A_1 + A_2} = \frac{5 \times 896}{5 \times 896 + 1376} = 0.765$$

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$A_{net} = A_1 + A_2 K$

$A_{net} = 896 + (1376 \text{ x } 0.765) = 1948.64 \text{ mm}^2$

Step 2: Load carrying capacity

$P_t = A_{net} \text{ x } \sigma_{at}$

$P_t = 1948.64 \text{ x } 150 = 292.296 \text{ x } 10^3 N$

$P_t = 292.296\ KN$

b) Member carrying compression under (Dead load+ Wind Load)

2ISA 90 X 60 X 8 mm

L=2.5 m=2500 mm

Fy= 250 N/mm^2

l_{eff} =0.85 L (For weld)

l_{eff} =0.85 x 2500=2125 mm

σac= σac (Calculated)

1.From steel table , find the sectional properties of the given section

From steel table

Shorter leg connected to gusset plate

2ISA 90 X 60 X 8 mm

A_{gross} = 2274 mm^2

rxx= 16.9 mm

ryy= 41 mm (Back to back distance 0 mm thick gusset plate)

rmin=16.9 mm

2.To find slenderness ratio

$$\lambda = \frac{l_{eff}}{r_{\min}} = \frac{2125}{16.9} = 125.73$$

3.From IS 800-1984, Page No: 39 Table No:5.1, Fy= 250 N/mm^2

λ	σac
120	64
125.73	?
130	57

By interpolation

$$\sigma ac\ \left(Calculated\right) = 64 + \left[\frac{(64-57)}{(130-120)} \text{ X } (125.73\text{-}120)\right] = 59.98 \text{ N/mm}^2$$

From IS Code 800, P No: 31

σac=1.33 x σac (Calculated)

= 1.33 x 59.98 = 79.77 N/mm^2

4.Load carrying capacity or strength of compression member

$$P_c = A_{gross} \text{ x } \sigma_{ac}$$

P_c =2274 x 79.77 = 181.40x 10^3 N

P_c = 181.40 KN

Type II : Design of the compression member

Design Procedure:

Given data: Compressive Load= P

Length of member = L

Yield Stress =Fy

1. Assuming slenderness ratio
   For strut or angle section = λ =110 to 130
   For rolled steel section= λ =70 to 90 (I section or channel section)
   For large load = λ =40

2. Find σ_{ac} from IS 800:1984 Page No: 39 Table No:5.1
3. Find sectional area required

   $$A = \frac{P}{\sigma_{ac}}$$
4. Try section from steel table of the required area

5. To find slenderness ratio

$$\lambda = \frac{l_{eff}}{r_{\min}}$$

6. From IS 800-1984, Page No: 39 Table No:5.1
   Find σac
   λ σac
7. Load carrying capacity or strength of compression member

$$P_c = A \text{ x } \sigma_{ac} > P \text{ , SAFE}$$

$$P_c = A \text{ x } \sigma_{ac} < P \text{ , UNSAFE, Try other section}$$

8. Design of Connection

1. Design a suitable angle section for a strut to carry an axial load of 150 KN over a length of 2.5 m also design the end connection by using 12 mm diameter of rivet. Fy = 250 N/mm^2

Solution : Given Data

Compressive load =P= 150 KN = 150 X 10^3 N

Length of member = L= 2.5 m =2500 mm

Nominal Diameter of rivet =d= 12 mm

Gross Diameter of rivet = D= 12+1.5 = 13.5 mm

1. Assuming λ = 110 for angle section
 Fy = 250 N/mm^2
2. σac = 72 N/mm^2 (From IS CODE) Page No: 39 Table No:5.1
3. Find sectional area required

$$A = \frac{P}{\sigma_{ac}} = \frac{150 X 10^3}{72} = 2083.33\ mm^2$$

4. Try section from steel table of the required area
 From steel table Try ISA 100 X 100 X 12 mm
 A = 2259 mm^2
 rxx= 30.3 mm
 ryy= 30.3 mm

 ruu= 38.2 mm
 rvv=19.4 mm
 rmin=19.4 mm

5.To find slenderness ratio

Assuming two or more rivets

L_{eff} = 0.85 L=0.85 X 2500 = 2125 mm

$$\lambda = \frac{l_{eff}}{r_{\min}} = \frac{2125}{19.4} = 109.53 \prec 180$$

6.From IS 800-1984, Page No: 39 Table No:5.1, Fy= 250 N/mm^2

λ σac

100	80
109.53	?
110	72

By interpolation

$$\sigma ac \ \left(Calculated\right) = 80 + \left[\frac{(72-80)}{(110-100)} \text{ X } (109.53\text{-}100)\right] = 72.376 \text{ N/mm}^2$$

σac= σac (Calculated)

= 72.376 N/mm^2

7.Load carrying capacity or strength of compression member

$P_c =$ A x σ_{ac}

P_c =2259 x 72.376 = 163.48x 10^3 N

P_c = 163.48 KN > 150 KN , SAFE

8.Design of connection (Riveted)

Nominal Diameter=d= = 12 mm

Gross Diameter of rivet =D= 12 +1.5 = 13.5 mm

Assuming power driven shop rivets (PDS) (P No:95 Tno:8.1)

τ_{VF}= 100 N/mm²

σ_{bf}= 300 N/mm²

$$P_S = \frac{\Pi}{4} \text{ x } D^2 \text{ X } \tau_{Vf}$$

$$P_S = \frac{\Pi}{4} \text{ x } 13.5^2 \text{ X } 100 \quad = 14.31 \text{ x } 10^3 N = 14.31 KN$$

$$P_b = \text{D x t X } \sigma_{bf}$$

$$P_b = 13.5 \text{ x } 12 \text{ X } 300 = 48.6 \text{ x } 10^3 N = 48.6 KN$$

Rivet value= least of Ps and Pb

Rivet value= 14.31 KN

$$N = \frac{Total \text{ Load}}{Rivet \text{ Value}} = \frac{P}{Rv} = \frac{150}{14.31} = 10.48 \cong 12$$

Minimum Pitch = 2.5 X d = 2.5 X12= 30 mm (P no:96, C No:8.10)

Maximum Pitch = 12 t or 200 mm (Which is less) (12t for compression member) (P no:96, C No:8.10)

= 12 x 12= 144 or 200

= 144 mm

Assuming pitch = P= 40 mm

Edge Distance= 20 mm (P no 97, T No:8.2)

2. Design a suitable angle section for a strut to carry an axial load of 130 KN over a length of 3 m also design the end connection. Fy = 250 N/mm^2

Solution : Given Data

Compressive load =P= 130 KN = 130 X 10^3 N

Length of member = L= 3m =3000 mm

1. Assuming λ = 110 for angle section

Fy = 250 N/mm^2

2. σac = 72 N/mm^2 (From IS CODE) Page No: 39 Table No:5.1

3.Find sectional area required

$$A = \frac{P}{\sigma_{ac}} = \frac{130X10^3}{72} = 1805.55\ mm^2$$

4.Try section from steel table of the required area

From steel table Try ISA 100 X 100 X 10 mm

A = 1903 mm^2

rxx= 30.5 mm

ryy= 30.5 mm

ruu= 35.5 mm

rvv=19.4 mm

rmin=19.4 mm

5.To find slenderness ratio

Assuming two or more rivets

L_{eff} = 0.85 L=0.85 X 3000 = 2550 mm

$$\lambda = \frac{l_{eff}}{r_{\min}} = \frac{2550}{19.4} = 131.44 \prec 180$$

6.From IS 800-1984, Page No: 39 Table No:5.1, Fy= 250 N/mm^2

λ	σac
130	57
131.44	?
140	51

By interpolation

$$\sigma ac\ \left(Calculated\right) = 57 + \left[\frac{(51-57)}{(140-130)}\ \text{X (131.44-130)}\right] = 56.13\ \text{N/mm}^2$$

σac= σac (Calculated)

= 56.13 N/mm^2

7.Load carrying capacity or strength of compression member

$P_c =$ A x σ_{ac}

P_c =1903x 56.16 = 106.81 X 10^3 N

P_c = 106.81 KN < 130 KN , UNSAFE

Try ISA 110 X 110 X 10 mm

A = 2106 mm^2

rxx= 33.6 mm

ryy= 33.6 mm

ruu= 42.5 mm

rvv=21.4 mm

rmin=21.4 mm

Assuming two or more rivets

L_{eff} = 0.85 L=0.85 X 3000 = 2550 mm

$$\lambda = \frac{l_{eff}}{r_{\min}} = \frac{2550}{21.4} = 119.15 \prec 180$$

From IS 800-1984, Page No: 39 Table No:5.1, Fy= 250 N/mm^2

λ	σac
110	72
119.15	?
120	64

By interpolation

$$\sigma ac\ \left(Calculated\right) = 72 + \left[\frac{(64-72)}{(120-110)}\ \text{X (119.15-110)}\right] = 64.58\ \text{N/mm}^2$$

σac= σac (Calculated)

= 64.58 N/mm^2

Load carrying capacity or strength of compression member

$P_c = $ A x σ_{ac}

P_c =2106x 64.58 = 136X 10^3 N

P_c = 136 KN > 130 KN , SAFE

Design of connection (Riveted)

Thickness =t= 10 mm

Nominal Diameter=d

$d = 6.04\sqrt{t} = 6.04\sqrt{10}$ =19.10 mm ≅ 20 mm

Gross Diameter of rivet =D= 20 +1.5 = 21.5 mm

Assuming power driven shop rivets (PDS) (P No:95 Tno:8.1)

τ_{VF}= 100 N/mm^2

σ_{bf}= 300 N/mm^2

$$P_S = \frac{\Pi}{4} \text{ x } D^2 \text{ X } \tau_{Vf}$$

$$P_S = \frac{\Pi}{4} \text{ x } 21.5^2 \text{ X } 100 \quad = 36.30 \text{ x } 10^3 N = 36.30 KN$$

$$P_b = \text{D x t X } \sigma_{bf}$$

$$P_b = 21.5 \text{ x } 10 \text{ X } 300 = 64.5 \text{ x } 10^3 N = 64.5 KN$$

Rivet value= least of Ps and Pb

Rivet value= 36.30 KN

$$N = \frac{Total\ \text{Load}}{Rivet\ \text{Value}} = \frac{P}{Rv} = \frac{130}{36.30} = 3.58 \cong 4$$

Minimum Pitch = 2.5 X d = 2.5 X20= 50 mm (P no:96, C No:8.10)

Maximum Pitch = 12 t or 200 mm (Which is less) (12t for compression member) (P no:96, C No:8.10)

= 12 x 10= 120 or 200

= 120 mm

Assuming pitch = P= 60 mm

Edge Distance= 35 mm (P no 97, T No:8.2)

3. A compression member of a roof truss is to carry an axial load of 110 KN. Design the section if you are providing double angle section. Length of member is 2 m. Design connection also. Take Fy= 250 N/mm^2.

Solution : Given Data

Compressive load =P= 110 KN = 110 X 10^3 N

Length of member = L= 2m =2000 mm

1.Assuming λ = 110 for angle section

Fy = 250 N/mm^2

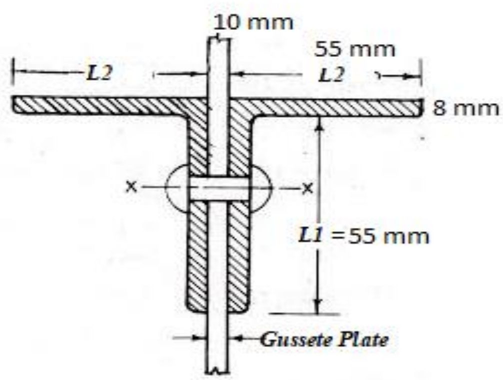
2.σac = 72 N/mm^2 (From IS CODE) Page No: 39 Table No:5.1

3.Find sectional area required

$$A = \frac{P}{\sigma_{ac}} = \frac{110 X 10^3}{72} = 1527.77\ mm^2$$

4.Try section from steel table of the required area

Assuming double angle section connected on both side of gusset plate



From steel table Try 2ISA 55 X 55 X 8 mm
Assuming thickness of gusset plate 10 mm

A = 1636 mm^2
rxx= 16.4 mm
ryy= 27 mm (Back to back distance for gusset plate thickness 10 mm)
rmin=16.4 mm

5.To find slenderness ratio

Assuming two or more rivets

L_{eff} = 0.85 L=0.85 X 2000 = 1700 mm

$$\lambda = \frac{l_{eff}}{r_{min}} = \frac{1700}{16.4} = 103.65 \prec 180$$

6.From IS 800-1984, Page No: 39 Table No:5.1, Fy= 250 N/mm^2

λ σac

100	80
103.65	?
110	72

By interpolation

$$\sigma ac\ \left(Calculated\right)=80+\left[\frac{(72-80)}{(110-100)}\ \text{X}\ (103.65\text{-}100)\right]=77.08\ \text{N/mm}^2$$

σac= σac (Calculated)

= 77.08 N/mm^2

7.Load carrying capacity or strength of compression member

$P_c =$ A x σ_{ac}

P_c =1636x 77.08 = 126.10 X 10^3 N

P_c = 126.10 KN > 110 KN , SAFE

8. Design of connection (Riveted)

Thickness =t= 8 mm

Nominal Diameter=d

$$d=6.04\ \sqrt{t}=6.04\ \sqrt{8}\ =17.08\ \text{mm}\cong 18\ \text{mm}$$

Gross Diameter of rivet =D= 18 +1.5 = 19.5 mm

Assuming power driven shop rivets (PDS) (P No:95 Tno:8.1)

τ_{VF}= 100 N/mm^2

σ_{bf}= 300 N/mm^2

$$P_S = 2\frac{\Pi}{4}\ \text{x D}^2\ \text{X}\ \tau_{Vf}$$

$$P_S = 2\frac{\Pi}{4}\ \text{x } 19.5^2\ \text{X } 100 \qquad = 59.72 \text{ x } 10^3 N = 36.30KN$$

$$P_b = \text{D x t X}\ \sigma_{bf}$$

$$P_b = 19.5 \text{ x } 8 \text{ X } 300 = 46.8 \text{ x } 10^3 N = 46.8KN$$

Rivet value= least of Ps and Pb

Rivet value= 58.5 KN

$$N=\frac{Total\ \text{Load}}{Rivet\ \text{Value}}=\frac{P}{Rv}=\frac{110}{46.8}=2.35\cong 3$$

Minimum Pitch = 2.5 X d = 2.5 X18= 45 mm (P no:96, C No:8.10)

Maximum Pitch = 12 t or 200 mm (Which is less) (12t for compression member) (P no:96, C No:8.10)

= 12 x 8= 96 or 200

= 96 mm

Assuming pitch = P= 50 mm

Edge Distance= 35 mm (P no 97, T No:8.2)

4.A compression member of a roof truss is to carry an axial load of 120 KN. Design the section if you are providing double angle section. Length of member is 4 m. Design connection also. Take Fy= 250 N/mm^2.

Solution : Given Data

Compressive load =P= 120 KN = 120 X 10^3 N

Length of member = L= 4m =4000 mm

1.Assuming λ = 110 for angle section

Fy = 250 N/mm^2

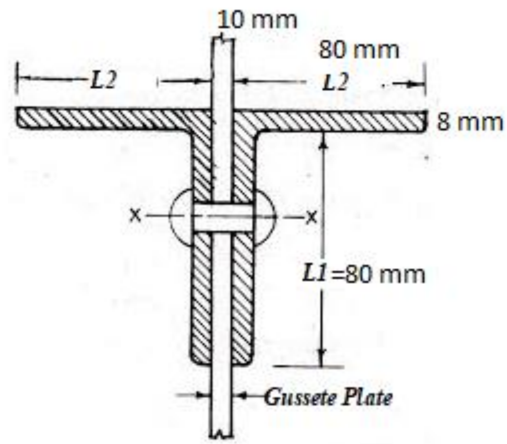
2.σac = 72 N/mm^2 (From IS CODE) Page No: 39 Table No:5.1

3.Find sectional area required

$$A = \frac{P}{\sigma_{ac}} = \frac{120X10^3}{72} = 1666.66\ mm^2$$

4.Try section from steel table of the required area

Assuming double angle section connected on both side of gusset plate



From steel table Try 2ISA 80X 80 X 8 mm

Assuming thickness of gusset plate 10 mm

A = 2442 mm^2

rxx= 24.4 mm

ryy= 36.9 mm (Back to back distance for gusset plate thickness 10 mm)

rmin=24.4 mm

5.To find slenderness ratio

Assuming two or more rivets

L_{eff} = 0.85 L=0.85 X 4000 = 3400 mm

$$\lambda = \frac{l_{eff}}{r_{\min}} = \frac{3400}{24.4} = 139.34 \prec 180$$

6.From IS 800-1984, Page No: 39 Table No:5.1, Fy= 250 N/mm^2

λ	σac
130	57
139.34	?
140	51

By interpolation

$$\sigma ac\ \left(Calculated\right) = 57 + \left[\frac{(51-57)}{(150-140)}\ \text{X (139.34-140)}\right] = 51.39\ \text{N/mm}^2$$

σac= σac (Calculated)

= 51.39N/mm^2

7.Load carrying capacity or strength of compression member

$P_c =$ A x σ_{ac}

P_c =2442x 51.39 = 125.5 X 10^3 N

P_c = 125.5 KN > 120 KN , SAFE

5.Two members AB and AC of a roof truss meeting at a point carry force as shown in table. Design the member using equal angle section also design end connection using 16 mm diameter of rivet. Use Fy= 250 N/mm^2

Member	**FORCE IN MEMBER (KN)**			**Length (m)**	**Note**
	DEAD LOAD	**LIVE LOAD**	**WIND LOAD**		
AB	**30**	**35**	**-80**	**2.1**	**+ Tension**
AC	**-40**	**-48**	**110**	**2.35**	**- Compression**

Solution: Design of member AB

DL+LL =30+35=65 KN (T)

DL+WL=30-80=-50 KN (C)

Design member AB as a tension member

P= 65 KN = 65 x 10^3 N

Nominal diameter of rivet=d= 16 mm

Gross diameter of rivet=D= 16+1.5=17.5 mm

fy=250 N/mm^2

σ_{at}=0.6 fy =0.6 x250 = 150 N/mm^2

<u>Step 1</u>: Find net effective sectional area

$$A_{net} = \frac{P}{\sigma_{at}}$$

$$A_{net} = \frac{65X\ 10^3}{150} = 433.33\ \text{mm}^2$$

<u>Step 2</u>: Find gross area assuming 30 %

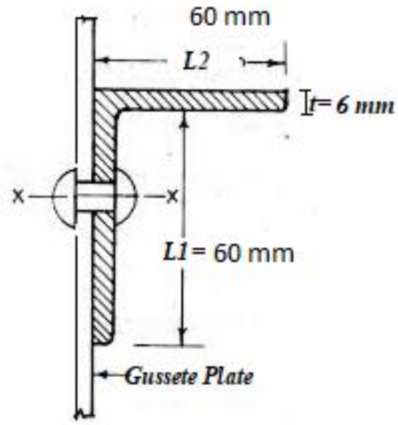
Assuming gross area 30% more

A_{gross}= 1.3 X 433.33 = 563.33 mm^2

<u>Step 3</u>: Try the section from steel table

From Steel Section , try section

ISA 60 X 60 X 6 mm



A_{net} = A_1+K A_2

A_1 = Net area of connected leg

$$A_1 = \left[\text{L}_1\text{- D - }\frac{t}{2}\right]t = \left[\ 60\text{- }17.5\text{ -}\frac{6}{2}\right]6 = 237\ \text{mm}^2$$

A_2 = Net area of outstanding leg

$$A_2 = \left[\text{L}_2 - \frac{t}{2}\right]t = \left[60 - \frac{6}{2}\right]6 = 342\ \text{mm}^2$$

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$$K = \frac{3A_1}{3A_1 + A_2} = \frac{3 \text{ x } 237}{3 \text{ x } 237 + 342} = 0.67$$

$A_{net} = A_1 + A_2 K$

$A_{net} = 237 + (342 \text{ x } 0.67) = 467.9 \text{ mm}^2$

<u>Step 4</u>: Find load carrying capacity of trial section

$P_t = A_{net} \text{ x } \sigma_{at} = 467.9 \text{ x } 150 = 70.18 \text{ x } 10^3 \text{ N} = 70.18 \text{ KN} > 65 \text{ KN}$

Safe

Check as a compression member

1. Assuming single angle angle strut connected by 2 or more rivets

L_{eff} = 0.85 L=0.85 X 2100 = 1785 mm

Try same section from steel table of the required area

From steel table Try ISA 60 X 60 X 6 mm

A = 684 mm^2

rxx= 18.2 mm

ryy= 18.2mm

ruu= 22.9 mm

rvv=11.5 mm

rmin=11.5 mm

2..To find slenderness ratio

Assuming two or more rivets

$$\lambda = \frac{l_{eff}}{r_{\min}} = \frac{1785}{11.5} = 155.1 \prec 180$$

3.From IS 800-1984, Page No: 39 Table No:5.1, Fy= 250 N/mm^2

λ	σac
150	45
155.1	?
160	41

By interpolation

$$\sigma ac \ \left(Calculated\right) = 45 + \left[\frac{(41 - 45)}{(150 - 140)} \text{ X } (155.1\text{-}150) \right] = 42.96 \text{ N/mm}^2$$

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σac= σac (Calculated)

= 42.96 N/mm^2

For wind load =1.33 X σac (Calculated)

= 1.33 X 42.96 =573136 N/mm^2

4.Load carrying capacity or strength of compression member

$P_c = A \times \sigma_{ac}$

$P_c = 684 \times 57.136 = 39.08 \times 10^3$ N

P_c = 39.08 KN < 50 KN , UNSAFE

Try section from steel table of the required area

1. From steel table Try ISA 75 X 75 X 6 mm

A = 866 mm^2

rxx= 23 mm

ryy= 23 mm

ruu= 29.1 mm

rvv=14.6 mm

rmin=14.6 mm

2.To find slenderness ratio

Assuming two or more rivets

$$\lambda = \frac{l_{eff}}{r_{\min}} = \frac{1785}{14.6} = 122.26 \prec 180$$

3.From IS 800-1984, Page No: 39 Table No:5.1, Fy= 250 N/mm^2

λ	σac
120	64
122.26	?
130	57

By interpolation

$$\sigma ac\ \left(Calculated\right) = 64 + \left[\frac{(57-64)}{(130-120)}\ \text{X (122.26-120)}\right] = 62.42\ \text{N/mm}^2$$

σac= σac (Calculated)

= 62.42 N/mm^2

For wind load =1.33 X σac (Calculated)

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$= 1.33 \text{ X } 62.42 = 83.01 \text{ N/mm}^2$

4.Load carrying capacity or strength of compression member

$P_c = \text{A x } \sigma_{ac}$

$P_c = 866 \text{ x } 83.01 = 71.89 \text{x } 10^3 \text{ N}$

$P_c = 71.89$ KN > 50 KN , SAFE (OK)

5.Design of connection (Riveted)

Nominal diameter of rivet=d= 16 mm

Gross diameter of rivet=D= 16+1.5=17.5 mm

Assuming power driven shop rivets (PDS) (P No:95 Tno:8.1)

τ_{VF}= 100 N/mm^2

σ_{bf}= 300 N/mm^2

$$P_S = 1 \text{x} \frac{\Pi}{4} \text{ x } D^2 \text{ X } \tau_{Vf}$$

$$P_S = 1x \frac{\Pi}{4} \text{ x } 17.5^2 \text{ X } 100 \quad = 24.05 \text{ x } 10^3 N = 24.05 KN$$

$$P_b = \text{D x t X } \sigma_{bf}$$

$$P_b = 17.5 \text{ x } 6 \text{ X } 300 = 31.5 \text{ x } 10^3 N = 31.5 KN$$

Rivet value= least of Ps and Pb

Rivet value= 24.05 KN

$$N = \frac{Total \text{ Load}}{Rivet \text{ Value}} = \frac{P}{Rv} = \frac{65}{24.05} = 2.7 \cong 3$$

Minimum Pitch = 2.5 X d = 2.5 X16= 40 mm (P no:96, C No:8.10)

Maximum Pitch = 16 t or 200 mm (Which is less) (16t for tension member) (P no:96, C No:8.10)

= 16 x 6= 96 or 200

= 96 mm

Assuming pitch = P= 50 mm

Edge Distance= 30 mm (P no 97, T No:8.2)

Design of member AC

DL+LL =-40-48=-88 KN (C)

DL+WL=-40+110=70 KN (T)

Design member AB as a Compression member

P= 88 KN = 88 x 10^3 N

Nominal diameter of rivet=d= 16 mm

Gross diameter of rivet=D= 16+1.5=17.5 mm

Assuming λ = 110 for angle section

Fy = 250 N/mm^2

σac = 72 N/mm^2 (From IS CODE) Page No: 39 Table No:5.1

1.Assuming single angle angle strut connected by 2 or more rivets

L_{eff} = 0.85 L=0.85 X 2350 = 1997.5 mm

Try section from steel table of the required area
From steel table Try ISA 90 X 90 X 8 mm

A = 1379 mm^2
rxx= 27.5 mm
ryy= 27.5 mm
ruu= 34.4 mm
rvv=17.5 mm
rmin=17.5 mm

2.To find slenderness ratio

Assuming two or more rivets

$$\lambda = \frac{l_{eff}}{r_{\min}} = \frac{1997.5}{17.5} = 114.14 \prec 180$$

3..From IS 800-1984, Page No: 39 Table No:5.1, Fy= 250 N/mm^2

λ	σac
110	72
114.14	?
120	64

By interpolation

$$\sigma ac \ \left(Calculated\right) = 72 + \left[\frac{(64-72)}{(120-110)} \text{ X (114.14-110)} \right] = 68.68 \text{ N/mm}^2$$

σac= σac (Calculated)
= 68.68 N/mm^2

4.Load carrying capacity or strength of compression member

$P_c = $ A x σ_{ac}
P_c =1379 x 68.68 = 94.7 x 10^3 N
P_c = 94.7 KN > 88 KN , SAFE (OK)

Check as a tension member

Nominal diameter of rivet=d= 16 mm
Gross diameter of rivet=D= 16+1.5=17.5 mm

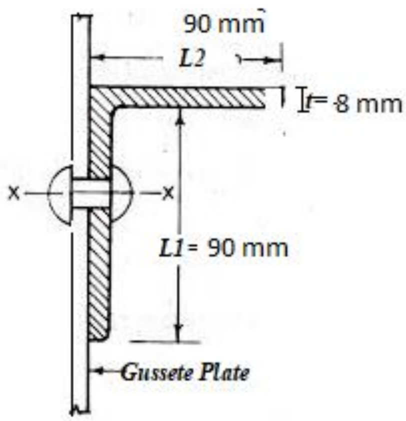
$fy = 250\ N/mm^2$

$\sigma_{at} = 0.6\ fy = 0.6 \times 250 = 150\ N/mm^2$

Try the section from steel table

From Steel Section , try section

ISA 90 X 90 X 8 mm



$A_{net} = A_1 + K\ A_2$

A_1 = Net area of connected leg

$$A_1 = \left[L_1 - D - \frac{t}{2} \right] t = \left[90 - 17.5 - \frac{8}{2} \right] 8 = 548\ mm^2$$

A_2 = Net area of outstanding leg

$$A_2 = \left[L_2 - \frac{t}{2} \right] t = \left[90 - \frac{8}{2} \right] 8 = 688\ mm^2$$

$$K = \frac{3A_1}{3A_1 + A_2} = \frac{3 \times 548}{3 \times 548 + 688} = 0.704$$

$$A_{net} = A_1 + A_2 K$$

$$A_{net} = 548 + (688 \times 0.704) = 1032.04\ mm^2$$

Find load carrying capacity of trial section

$$P_t = A_{net} \times \sigma_{at} = 1032.04 \times 150 = 154.8 \times 10^3\ N = 154.8\ KN > 70\ KN$$

Safe

Design of connection (Riveted)

Nominal Diameter=d= = 16mm

Gross Diameter of rivet =D= 16 +1.5 = 17.5 mm

Assuming power driven shop rivets (PDS) (P No:95 Tno:8.1)

$\tau_{VF} = 100\ N/mm^2$

$\sigma_{bf} = 300\ N/mm^2$

$$P_S = \frac{\Pi}{4} \text{ x } D^2 \text{ X } \tau_{Vf}$$

$$P_S = \frac{\Pi}{4} \text{ x } 17.5^2 \text{ X } 100 \quad = 24.052 \text{ x } 10^3 N = 24.052 KN$$

$$P_b = D \text{ x t X } \sigma_{bf}$$

$$P_b = 17.5 \text{ x } 8 \text{ X } 300 = 42 \text{ x } 10^3 N = 42 KN$$

Rivet value= least of Ps and Pb

Rivet value= 24.052 KN

$$N = \frac{Total \text{ Load}}{Rivet \text{ Value}} = \frac{P}{Rv} = \frac{88}{24.05} = 3.64 \cong 4$$

Minimum Pitch = 2.5 X d = 2.5 X16= 40 mm (P no:96, C No:8.10)

Maximum Pitch = 12 t or 200 mm (Which is less) (12t for compression member) (P no:96, C No:8.10)

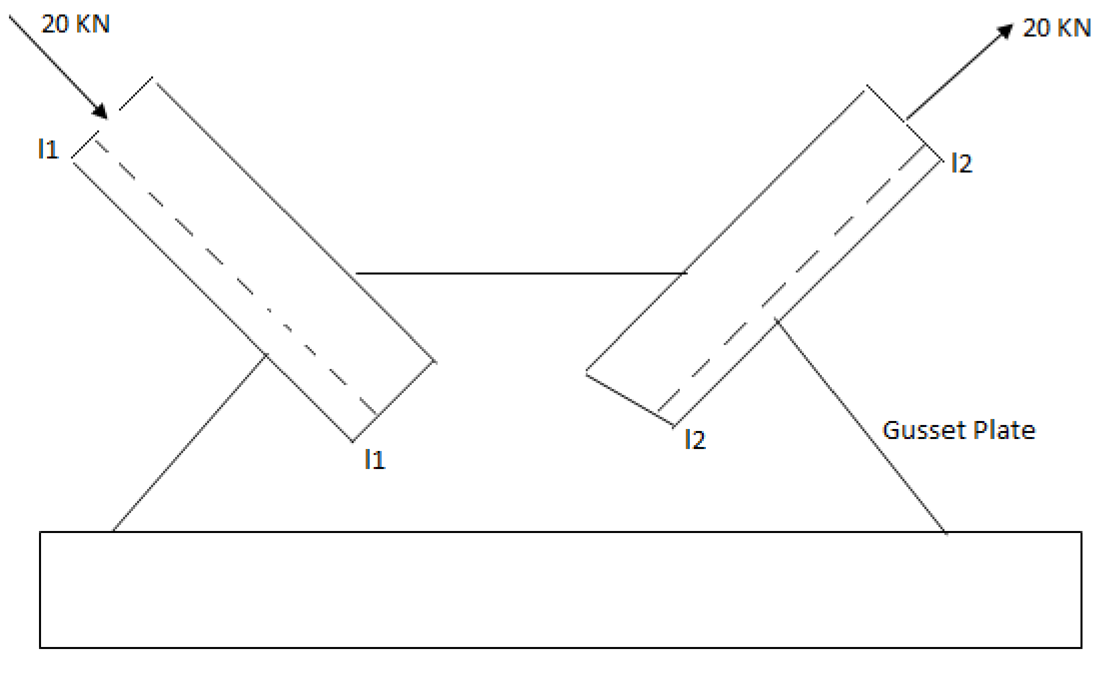
= 12 x 8= 96 or 200

= 96 mm

Assuming pitch = P= 50 mm

Edge Distance= 30 mm (P no 97, T No:8.2)

4. Figure shows truss joint , the force in different member as shown in figure . The length of each member is 2 m, Design
   a) The member l_1l_1
   b) The member l_2l_2
   Assuming 16 mm diameter of rivet



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Solution: Given Data

Design of member l_1l_1 (Design as a compression member)

P= 20 KN = 20 x 10^3
Nominal diameter of rivet=d= 16 mm
Gross diameter of rivet=D= 16+1.5=17.5 mm
Assuming λ = 110 for angle section
Fy = 250 N/mm^2
σac = 72 N/mm^2 (From IS CODE) Page No: 39 Table No:5.1

Find sectional area required

$$A = \frac{P}{\sigma_{ac}} = \frac{20X10^3}{72} = 277.7\ mm^2$$

1. Assuming single angle angle strut connected by 2 or more rivets

L_{eff} = 0.85 L=0.85 X 2000 = 1700 mm

Try section from steel table of the required area
From steel table Try ISA 55 X 55 X 6 mm (Minimum angle section)

A = 626 mm^2
rxx= 16.6 mm
ryy= 16.6 mm
ruu= 21 mm
rvv=10.6 mm
rmin=10.6 mm

2.To find slenderness ratio

Assuming two or more rivets

$$\lambda = \frac{l_{eff}}{r_{\min}} = \frac{1700}{10.6} = 160.37 \prec 180$$

3.From IS 800-1984, Page No: 39 Table No:5.1, Fy= 250 N/mm^2

λ	σac
160	41
160.37	?
170	37

By interpolation

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$$\sigma ac\ \left(Calculated\right)=41+\left[\frac{(37-41)}{(170-160)}\ \text{X (160.3-10)}\right]=40.85\text{N/mm}^2$$

σac= σac (Calculated)

=40.85 N/mm^2

4.Load carrying capacity or strength of compression member

$P_c =$ A x σ_{ac}

P_c =626 x 40.85= 25.57x 10^3 N

P_c = 25.57 KN > 20 KN , SAFE (OK)

5. **Design of connection (Riveted)**

Nominal diameter of rivet=d= 16 mm

Gross diameter of rivet=D= 16+1.5=17.5 mm

Assuming power driven shop rivets (PDS) (P No:95 Tno:8.1)

τ_{VF}= 100 N/mm^2

σ_{bf}= 300 N/mm^2

$$P_S = 1\text{x}\frac{\Pi}{4}\ \text{x D}^2\ \text{X}\ \tau_{Vf}$$

$$P_S = 1x\frac{\Pi}{4}\ \text{x } 17.5^2\ \text{X } 100 \qquad = 24.05 \text{ x } 10^3 N = 24.05KN$$

$$P_b = \text{D x t X } \sigma_{bf}$$

$$P_b = 17.5 \text{ x } 6 \text{ X } 300 = 31.5 \text{ x } 10^3 N = 31.5KN$$

Rivet value= least of Ps and Pb

Rivet value= 24.05 KN

$$N = \frac{Total\ \text{Load}}{Rivet\ \text{Value}} = \frac{P}{Rv} = \frac{20}{24.05} = 0.831 \cong 2$$

Minimum Pitch = 2.5 X d = 2.5 X16= 40 mm (P no:96, C No:8.10)

Maximum Pitch = 12 t or 200 mm (Which is less) (12t for compression member) (P no:96, C No:8.10)

= 12 x 6= 72 or 200

= 72 mm

Assuming pitch = P= 50 mm

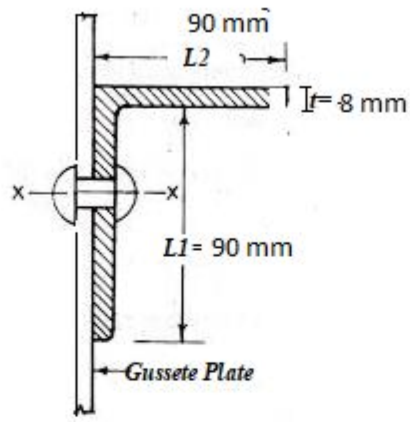
Edge Distance= 30 mm (P no 97, T No:8.2)

c) Design the member l2l2 (Tension Member)

Try the section from steel table

From Steel Section , try section

ISA 55 X 55 X 8 mm



$A_{net} = A_1 + K\,A_2$

A_1 = Net area of connected leg

$$A_1 = \left[L_1 - D - \frac{t}{2} \right] t = \left[55 - 17.5 - \frac{6}{2} \right] 6 = 207 \text{ mm}^2$$

A_2 = Net area of outstanding leg

$$A_2 = \left[L_2 - \frac{t}{2} \right] t = \left[55 - \frac{6}{2} \right] 6 = 312 \text{ mm}^2$$

$$K = \frac{3A_1}{3A_1 + A_2} = \frac{3 \text{ x } 207}{3 \text{ x } 207 + 312} = 0.6655$$

$$A_{net} = A_1 + A_2 K$$

$$A_{net} = 207 + (312 \text{ x } 0.6655) = 414.66 \text{ mm}^2$$

Find load carrying capacity of trial section

$$P_t = A_{net} \text{ x } \sigma_{at} = 414.66 \text{ x } 150 = 62.19 \text{ x } 10^3 \text{ N} = 62.19 \text{ KN} > 20 \text{ KN}$$

Safe

Design of connection (Riveted)

Nominal diameter of rivet=d= 16 mm

Gross diameter of rivet=D= 16+1.5=17.5 mm

Assuming power driven shop rivets (PDS) (P No:95 Tno:8.1)

τ_{VF}= 100 N/mm²

σ_{bf}= 300 N/mm²

$$P_S = 1 \text{x} \frac{\Pi}{4} \text{ x } D^2 \text{ X } \tau_{Vf}$$

$$P_S = 1x \frac{\Pi}{4} \text{ x } 17.5^2 \text{ X } 100 = 24.05 \text{ x } 10^3 N = 24.05 KN$$

$P_b = \text{D x t X } \sigma_{bf}$

$P_b = 17.5 \text{ x } 6 \text{ X } 300 = 31.5 \text{ x } 10^3 N = 31.5 KN$

Rivet value= least of Ps and Pb

Rivet value= 24.05 KN

$N = \frac{Total \text{ Load}}{Rivet \text{ Value}} = \frac{P}{Rv} = \frac{20}{24.05} = 0.831 \cong 2$

Minimum Pitch = 2.5 X d = 2.5 X16= 40 mm (P no:96, C No:8.10)

Maximum Pitch = 16 t or 200 mm (Which is less) (16t for tension member)

(P no:96, C No:8.10)

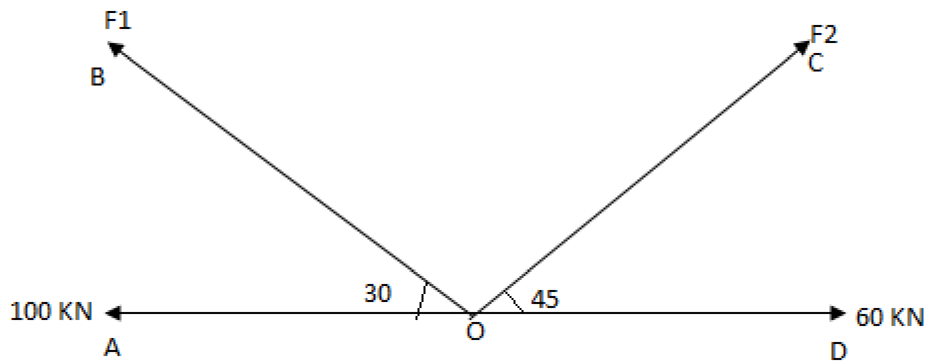
= 16 x 6= 96 or 200

= 96 mm

Assuming pitch = P= 50 mm

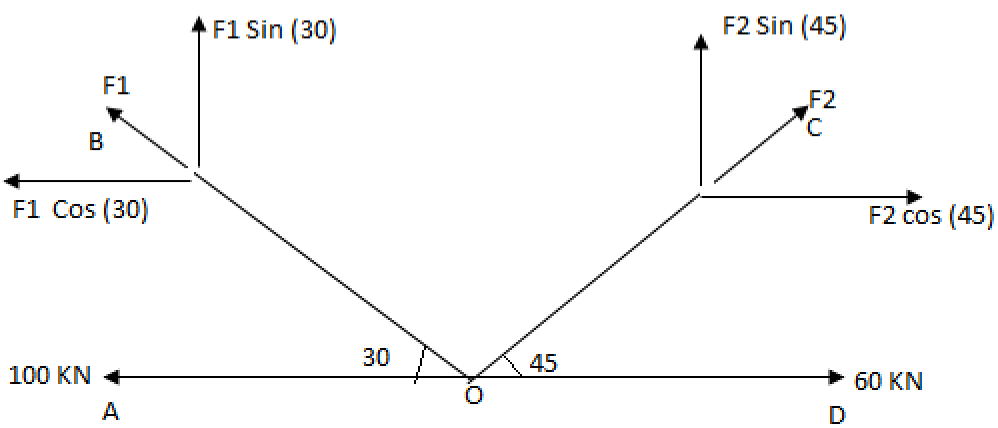
Edge Distance= 30 mm (P no 97, T No:8.2)

6. Four members AO, BO, CO and DO are carrying the axial forces as shown in figure. Design a suitable angle section for all members assuming length of each member is 2 m and using 16 mm diameter of rivet. Use Fy= 250 N/mm^2



F1= Force in member OB

F2= Force in member OC



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$\sum Fx = 0$

-100 +60-F1 Cos (30^0)+ F2 Cos (45^0) =0 (1)

$\sum Fy = 0$

F1 Sin (30^0) + F2 Cos (45^0) = 0
F1 Sin (30^0) = - F2 Cos (45^0)
F1 = - F2 Cos (45^0) /((Sin (30^0))
F1= -1.414 F2 (2)
Substitute F1 in equation (1)
-100 +60-(-1.414 F2) X Cos (30^0)+ F2 Cos (45^0) =0
F2= 20.71 KN (Tensile)
F1= -29.28 KN (Compression)

Force in member OB=29.28 KN (C)
Force in member OC=20.70 KN (T)
Force in member OA=100 KN (T)
Force in member OD=60 KN (T)

Design member OB as compression member

Force in member OB=29.28 KN (C)

Compressive load =P= 29.28 KN = 29.28 X 10^3 N

Length of member = L= 2m =2000 mm

1.Assuming λ = 110 for angle section

Fy = 250 N/mm^2

2.σac = 72 N/mm^2 (From IS CODE) Page No: 39 Table No:5.1

3.Find sectional area required

$$A = \frac{P}{\sigma_{ac}} = \frac{29.28X10^3}{72} = 406.66\ mm^2$$

4.Try section from steel table of the required area

From steel table Try ISA 60 X 60 X 6 mm

A = 684 mm^2
rxx= 18.2 mm
ryy= 18.2 mm

ruu= 22.9 mm
rvv=11.5 mm

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rmin=11.5 mm

5.To find slenderness ratio

Assuming two or more rivets

L_{eff} = 0.85 L=0.85 X 2000 = 1700 mm

$$\lambda = \frac{l_{eff}}{r_{\min}} = \frac{1700}{11.5} = 147.82 \prec 180$$

6.From IS 800-1984, Page No: 39 Table No:5.1, Fy= 250 N/mm^2

λ	σac
140	51
147.82	?
150	45

By interpolation

$$\sigma ac\ \left(Calculated\right) = 51 + \left[\frac{(45-51)}{(150-140)}\ \text{X (147.82-140)}\right] = 46.30\ \text{N/mm}^2$$

σac= σac (Calculated)

= 46.30 N/mm^2

7.Load carrying capacity or strength of compression member

$P_c = $ A x σ_{ac}

P_c =684x 46.30 = 31.67 X 10^3 N

P_c = 31.67 KN >29.28 KN , SAFE

8.Design of connection (Riveted)

Nominal Diameter=d=16 mm

Gross Diameter of rivet =D= 16 +1.5 = 17.5 mm

Assuming power driven shop rivets (PDS) (P No:95 Tno:8.1)

τ_{VF}= 100 N/mm^2

σ_{bf}= 300 N/mm^2

$$P_S = \frac{\Pi}{4}\ \text{x D}^2\ \text{X}\ \tau_{Vf}$$

$$P_S = \frac{\Pi}{4}\ \text{x } 17.5^2\ \text{X } 100 \quad = 24.05 \text{ x } 10^3 N = 24.05 KN$$

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$P_b = \text{D x t X } \sigma_{bf}$

$P_b = 17.5 \text{ x } 6 \text{ X } 300 = 31.5 \text{ x } 10^3 N = 31.5 KN$

Rivet value= least of Ps and Pb

Rivet value= 24.05 KN

$$N = \frac{Total\ \text{Load}}{Rivet\ \text{Value}} = \frac{P}{Rv} = \frac{29.28}{24.05} = 1.21 \cong 2$$

Minimum Pitch = 2.5 X d = 2.5 X16= 40 mm (P no:96, C No:8.10)

Maximum Pitch = 12 t or 200 mm (Which is less) (12t for compression member) (P no:96, C No:8.10)

= 12 x 6= 72 or 200

= 72 mm

Assuming pitch = P= 50 mm

Edge Distance= 30 mm (P no 97, T No:8.2)

Design member OC as tension member
Force in member OC=20.71 KN (T)

Nominal diameter of rivet=d= 16 mm

Gross diameter of rivet=D= 16+1.5=17.5 mm

Assuming fy=250 N/mm^2

σ_{at}=0.6 fy =0.6 x250 = 150 N/mm^2

Step 1: Find net effective sectional area

$$A_{net} = \frac{P}{\sigma_{at}}$$

$$A_{net} = \frac{20.73X\ 10^3}{150} = 138 \text{ mm}^2$$

Step 2: Find gross area assuming 30 %

Assuming gross area 30% more

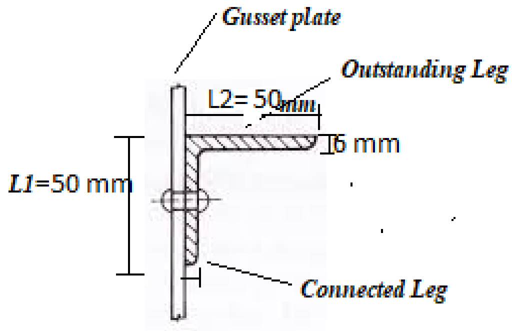
A_{gross}= 1.3 X 138 = 179.4 mm^2

Step 3: Try the section from steel table

From Steel Section , try section

For truss minimum angle section is ISA 50 X 50 X 6 mm

Try ISA 50 X 50 X 6 mm



$A_{net} = A_1 + A_2 K$

$K = \dfrac{3A_1}{3A_1 + A_2}$ IS Code P.No: 37, C. No:4.2.1.1

A_1 = Net area of connected leg

A_2 = Net area of outstanding leg

Step 1: To find net area

A_1 = Net area of connected leg

$A_1 = \left[L_1 - D - \dfrac{t}{2}\right] t = \left[50 - 17.5 - \dfrac{6}{2}\right] 6 = 177 \text{ mm}^2$

A_2 = Net area of outstanding leg

$A_2 = \left[L_2 - \dfrac{t}{2}\right] t = \left[50 - \dfrac{6}{2}\right] 6 = 282 \text{ mm}^2$

$K = \dfrac{3A_1}{3A_1 + A_2} = \dfrac{3 \times 177}{3 \times 177 + 282} = 0.653$

$A_{net} = A_1 + A_2 K$

$A_{net} = 177 + (282 \times 0.653) = 361.8 \text{ mm}^2$

Step 2: Load carrying capacity

$P_t = A_{net} \times \sigma_{at}$

$P_t = 361.8 \times 150 = 54.17 \times 10^3 N$

$P_t = 54.17\ KN > 20.71\ KN$

Design of connection (Riveted)

Nominal Diameter=d=16 mm

Gross Diameter of rivet =D= 16 +1.5 = 17.5 mm

Assuming power driven shop rivets (PDS) (P No:95 Tno:8.1)

τ_{VF}= 100 N/mm^2

σ_{bf}= 300 N/mm^2

$$P_S = \frac{\Pi}{4} \times D^2 \times \tau_{Vf}$$

$$P_S = \frac{\Pi}{4} \times 17.5^2 \times 100 = 24.05 \times 10^3 N = 24.05 KN$$

$P_b = \text{D x t X } \sigma_{bf}$

$P_b = 17.5 \text{ x } 6 \text{ X } 300 = 31.5 \text{ x } 10^3 N = 31.5 KN$

Rivet value= least of Ps and Pb

Rivet value= 24.05 KN

$$N = \frac{Total \text{ Load}}{Rivet \text{ Value}} = \frac{P}{Rv} = \frac{20.70}{24.05} = 0.86 \cong 2$$

Minimum Pitch = 2.5 X d = 2.5 X16= 40 mm (P no:96, C No:8.10)

Maximum Pitch = 16 t or 200 mm (Which is less) (16t for tension member)

(P no:96, C No:8.10)

= 16 x 6= 96 or 200

= 96 mm

Assuming pitch = P= 50 mm

Edge Distance= 30 mm (P no 97, T No:8.2)

Design member OD as tension member

Force in member OD=20.71 KN (T)

Nominal diameter of rivet=d= 16 mm

Gross diameter of rivet=D= 16+1.5=17.5 mm

Assuming fy=250 N/mm^2

σ_{at}=0.6 fy =0.6 x250 = 150 N/mm^2

Step 1: Find net effective sectional area

$$A_{net} = \frac{P}{\sigma_{at}}$$

$$A_{net} = \frac{60X \; 10^3}{150} = 400 \text{ mm}^2$$

Step 2: Find gross area assuming 30 %

Assuming gross area 30% more

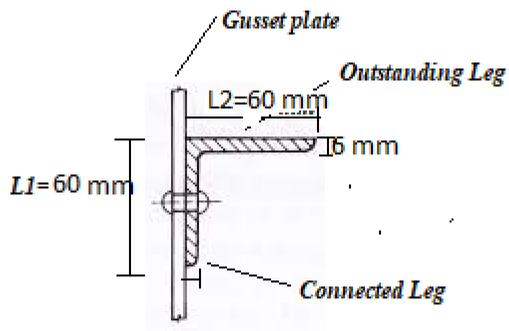
A_{gross}= 1.3 X 400 = 520 mm^2

Step 3: Try the section from steel table

From Steel Section , try section

For truss minimum angle section is ISA 50 X 50 X 6 mm

Try ISA 60 X 60 X 6 mm



$A_{net} = A_1 + A_2 K$

$K = \dfrac{3A_1}{3A_1 + A_2}$ IS Code P.No: 37, C. No:4.2.1.1

A_1 = Net area of connected leg

A_2 = Net area of outstanding leg

Step 1: To find net area

A_1 = Net area of connected leg

$$A_1 = \left[L_1 - D - \frac{t}{2}\right] t = \left[60 - 17.5 - \frac{6}{2}\right] 6 = 237\text{mm}^2$$

A_2 = Net area of outstanding leg

$$A_2 = \left[L_2 - \frac{t}{2}\right] t = \left[60 - \frac{6}{2}\right] 6 = 342 \text{ mm}^2$$

$$K = \frac{3A_1}{3A_1 + A_2} = \frac{3 \text{ x } 237}{3 \text{ x}237 + 342} = 0.67$$

$$A_{net} = A_1 + A_2 K$$

$$A_{net} = 237 + (342 \text{ x } 0.67) = 467.9 \text{ mm}^2$$

Step 2: Load carrying capacity

$$P_t = A_{net} \text{ x } \sigma_{at}$$

$$P_t = 467.9 \text{ x } 150 = 70.18 \text{ x } 10^3 N$$

$$P_t = 70.18\ KN > 60\ KN$$

4.Design of connection (Riveted)

Nominal Diameter=d=16 mm

Gross Diameter of rivet =D= 16 +1.5 = 17.5 mm

Assuming power driven shop rivets (PDS) (P No:95 Tno:8.1)

τ_{VF}= 100 N/mm^2

σ_{bf}= 300 N/mm^2

$$P_S = \frac{\Pi}{4} \text{ x } D^2 \text{ X } \tau_{Vf}$$

$$P_S = \frac{\Pi}{4} \text{ x } 17.5^2 \text{ X } 100 \quad = 24.05 \text{ x } 10^3 N = 24.05 KN$$

$P_b = \text{D x t X } \sigma_{bf}$

$P_b = 17.5 \text{ x } 6 \text{ X } 300 = 31.5 \text{ x } 10^3 N = 31.5 KN$

Rivet value= least of Ps and Pb

Rivet value= 24.05 KN

$$N = \frac{\textit{Total} \text{ Load}}{\textit{Rivet} \text{ Value}} = \frac{P}{Rv} = \frac{60}{24.05} = 2.49 \cong 3$$

Minimum Pitch = 2.5 X d = 2.5 X16= 40 mm (P no:96, C No:8.10)

Maximum Pitch = 16 t or 200 mm (Which is less) (16t for tension member) (P no:96, C No:8.10)

= 16 x 6= 96 or 200

= 96 mm

Assuming pitch = P= 50 mm

Edge Distance= 30 mm (P no 97, T No:8.2)

Force in member OA=100 KN (T)
Design member OA as tension member

Step 1: Find net effective sectional area

$$A_{net} = \frac{P}{\sigma_{at}}$$

$$A_{net} = \frac{100 X\ 10^3}{150} = 666.67 \text{ mm}^2$$

Step 2: Find gross area assuming 30 %

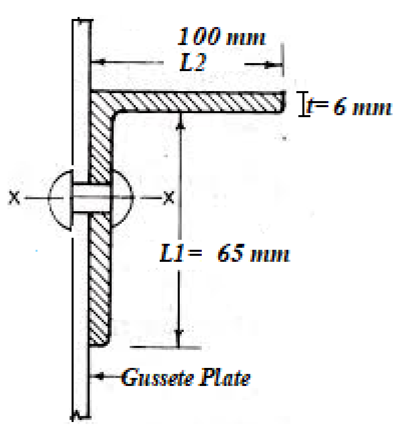
Assuming gross area 30% more

A_{gross}= 1.3 X 666.66 = 866.67 mm^2

Step 3: Try the section from steel table

From Steel Section , try section

ISA 100 X 65 X 6 mm



$A_{net} = A_1 + K\ A_2$

$$\lambda = \frac{l_{eff}}{r_{\min}}$$

A_1 = Net area of connected leg

$$A_1 = \left[\mathrm{L_1 - D} - \frac{t}{2}\right]t = \left[65 - 21.5 - \frac{6}{2}\right]6 = 243\ \mathrm{mm}^2$$

A_2 = Net area of outstanding leg

$$A_2 = \left[\mathrm{L_2} - \frac{t}{2}\right]t = \left[100 - \frac{6}{2}\right]6 = 582\ \mathrm{mm}^2$$

$$K = \frac{3A_1}{3A_1 + A_2} = \frac{3 \text{ x } 243}{3 \text{ x } 243 + 582} = 0.556$$

$$A_{net} = \mathrm{A_1 + A_2 K}$$

$$A_{net} = 243 + (582 \text{ x } 0.556) = 566.69\ \mathrm{mm}^2$$

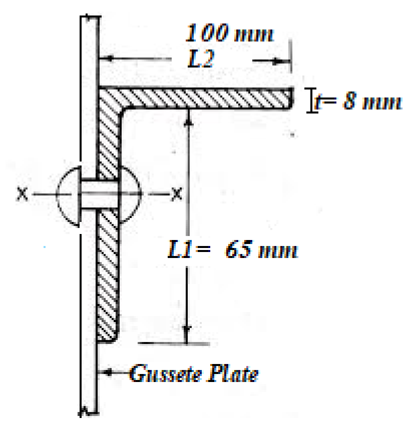
<u>Step 4</u>: Find load carrying capacity of trial section

$$P_t = A_{net} \text{ x } \sigma_{\mathrm{at}} = 648.55 \text{ x } 150 = 97.28 \text{ x } 10^3 \text{ N} = 97.28 \text{ KN} \prec 100 \text{ KN}$$

Unsafe, Try another section

ISA 100 X 65 X 8 mm

$A_{net} = A_1 + K\ A_2$



A_1 = Net area of connected leg

$$A_1 = \left[\mathrm{L_1 - D} - \frac{t}{2}\right]t = \left[65 - 21.5 - \frac{8}{2}\right]8 = 316\ \mathrm{mm}^2$$

A_2 = Net area of outstanding leg

$$A_2 = \left[\mathrm{L_2} - \frac{t}{2}\right]t = \left[100 - \frac{8}{2}\right]8 = 768\ \mathrm{mm}^2$$

$$K = \frac{3A_1}{3A_1 + A_2} = \frac{3 \text{ x } 316}{3 \text{ x } 316 + 768} = 0.55$$

$$A_{net} = \mathrm{A_1 + A_2 K}$$

$$A_{net} = 316 + (768 \text{ x } 0.550) = 738.4\ \mathrm{mm}^2$$

Step 4: Find load carrying capacity of trial section

$P_t = A_{net} \text{ x } \sigma_{at} = 738.4 \text{ x } 150 = 110.76 \text{ x } 10^3 \text{ N} = 110.76 \text{ KN} \succ 100 \text{ KN}$

Safe

Design of connection (Riveted)

Nominal Diameter=d=16 mm

Gross Diameter of rivet =D= 16 +1.5 = 17.5 mm

Assuming power driven shop rivets (PDS) (P No:95 Tno:8.1)

τ_{VF}= 100 N/mm^2

σ_{bf}= 300 N/mm^2

$$P_S = \frac{\Pi}{4} \text{ x } D^2 \text{ X } \tau_{Vf}$$

$$P_S = \frac{\Pi}{4} \text{ x } 17.5^2 \text{ X } 100 \quad = 24.05 \text{ x } 10^3 N = 24.05 KN$$

$$P_b = D \text{ x } t \text{ X } \sigma_{bf}$$

$$P_b = 17.5 \text{ x } 6 \text{ X } 300 = 31.5 \text{ x } 10^3 N = 31.5 KN$$

Rivet value= least of Ps and Pb

Rivet value= 24.05 KN

$$N = \frac{Total \text{ Load}}{Rivet \text{ Value}} = \frac{P}{Rv} = \frac{100}{24.05} = 4.15 \cong 5$$

Minimum Pitch = 2.5 X d = 2.5 X16= 40 mm (P no:96, C No:8.10)

Maximum Pitch = 16 t or 200 mm (Which is less) (16t for tension member) (P no:96, C No:8.10)

= 16 x 6= 96 or 200

= 96 mm

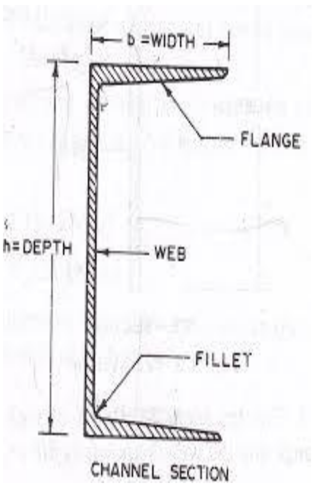
Assuming pitch = P= 50 mm

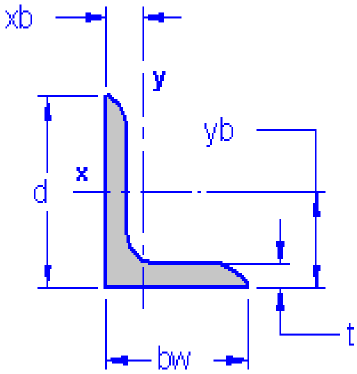
Edge Distance= 30 mm (P no 97, T No:8.2)

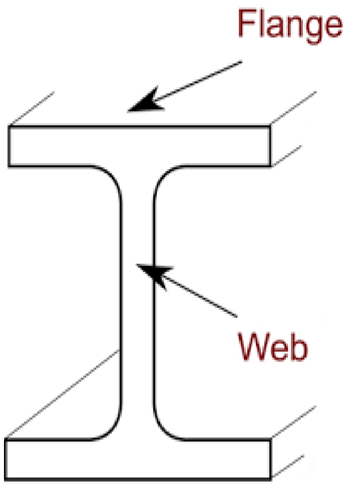
Design of Beam

Beam is defined as structural member subjected to transverse load. Transverse load produced bending moment and shear force in beam.

1. Joist: It is used for light weight
2. Girder: Any major beam in a structure is known as girder.
3. Purlin: In roof truss the horizontal beam spanning between two supports.
4. Lintel: The beam spanning over door, window and other opening of wall is known as Lintel.
5. Stringer Beam: The beam supported stair is known as Stringer beam.
   Beam may simply support, cantilever, continuous or fixed in nature.
   Different Section of Beam







Channel Section Angle Section I Section

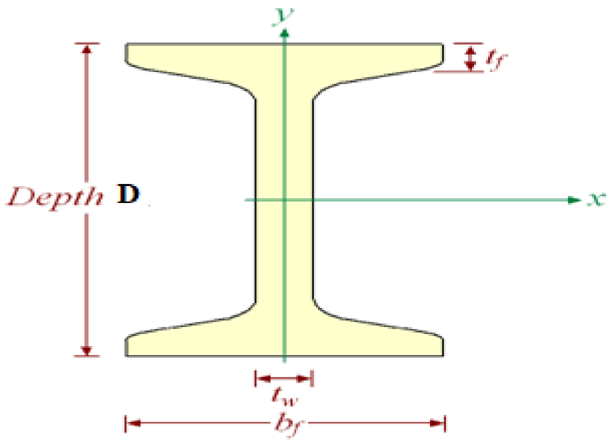
CASE I : Design of Main and Secondary Beam

1. Permissible bending Stress

   $\sigma_{bc} = \sigma_{bt} = 0.66 \text{ x } F_y$

   Fy= Yield Stress
2. Permissible shear stress

   $\tau_v = 0.40 \text{ x } F_y$



The calculated shear stresses

$$\tau_{v\ (cal)} = \frac{F}{t_w D}$$

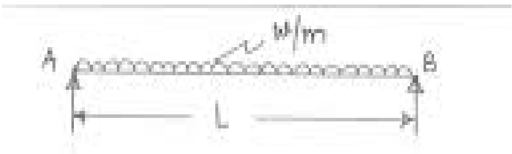
F= Maximum Shear Force

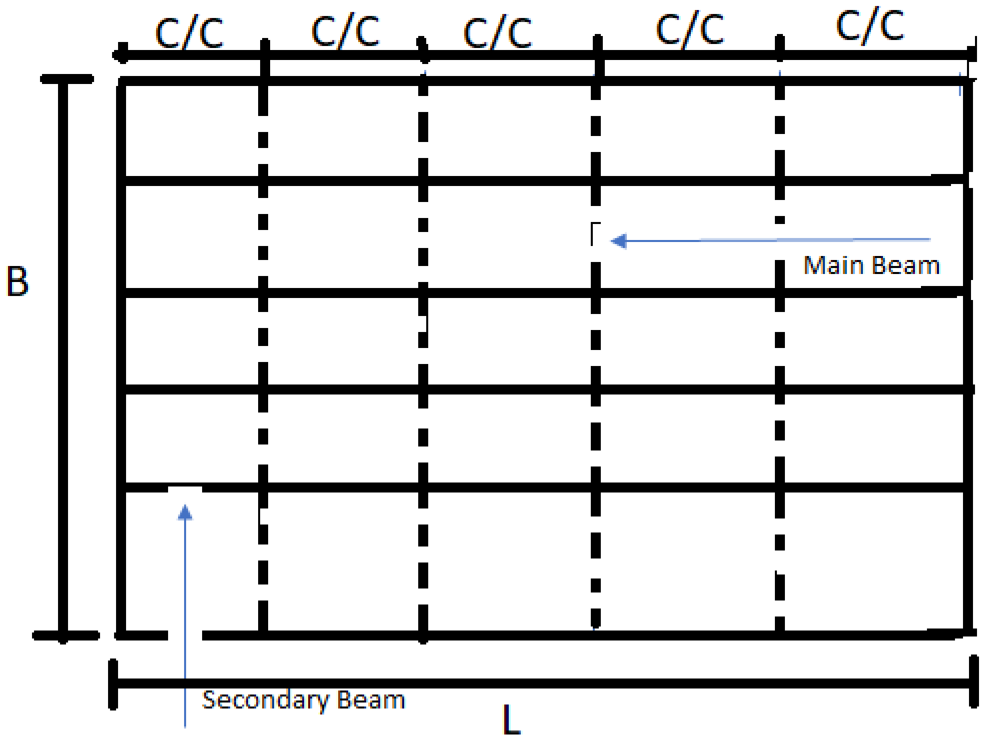
t_w= Thickness of web

D= Overall depth

3. Permissible Deflection $= \frac{Span}{325}$

Actual Deflection for Udl = $= \frac{5}{384}\frac{WL^4}{EI_{XX}}$





Main beam is parallel to width

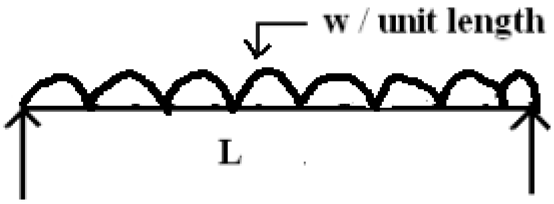
Secondary beam parallel to length

C/C = Centre to centre distance

Design procedure for secondary beam

1. Loads
   a) Weight of slab = Width X Thickness of Slab X Density of Concrete
      Density of Reinforced Concrete = 25 KN/m^3
      Density of Plain Concrete = 24 KN/m^3
   b) Live Load= Width X Intensity of Live Load
   c) Floor Finish = Width X Intensity of Floor Finish
   d) Self weight of secondary beam= 1 KN/m
      Total Load= a+b+c+d
2. Effective Span=L= Centre to centre distance between Support
   L= 0.7 X l (Compressive flange is restrain)
3. Maximum Bending moment and Shear Force

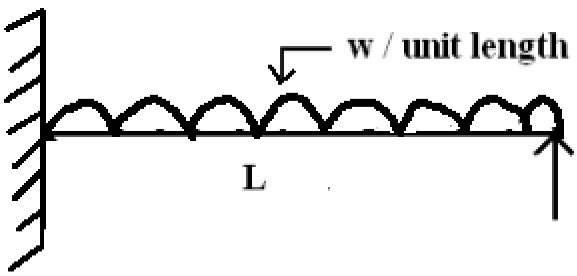
   a) For simply supported beam carrying udl over entire span



$$\textit{Maximum}\text{ Bending Moment } = M = \frac{WL^2}{8}$$

$$Maximum \text{ Shear Force} = F = \frac{WL}{2}$$

b) For cantilever beam carrying udl over entire span



$$Maximum \text{ Bending Moment} = M = \frac{WL^2}{2}$$

$$Maximum \text{ Shear Force} = F = WL$$

4. Find section modulus required

$$\sigma_b = \frac{M}{Z_{req}}$$

$$Z_{req} = \frac{M}{\sigma_b}$$

$$\sigma_b = 0.66Fy$$

5. Try Section from steel table
   For secondary beam ISLB
   For main beam ISHB, ISMB
   Z_{XX}= Section modulus (Given in steel table)
   A= Area of section (Given in steel table)
   I_{XX} = Moment of Inertia (Given in steel table)
6. Check for section for bending stress

   $$\sigma_{b\ (cal)} = \frac{M}{I_{xx}} \text{ Y} \prec 0.66 \text{ Fy}$$

7. Check for section for shear stress

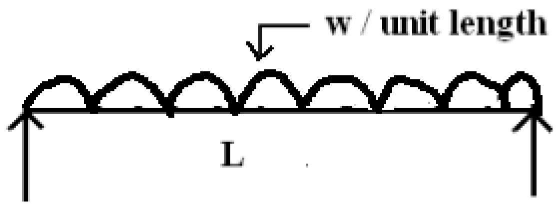
   $$\tau_{v\ (cal)} = \frac{F}{t_w D} \prec 0.4 \text{ Fy}$$

8. Check for deflection

   Permissible deflection = $\frac{Span}{325}$

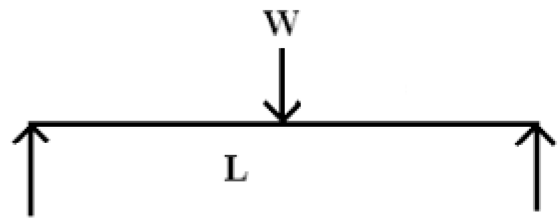
   Actual Deflection

   a) For simply supported beam carrying udl over entire span



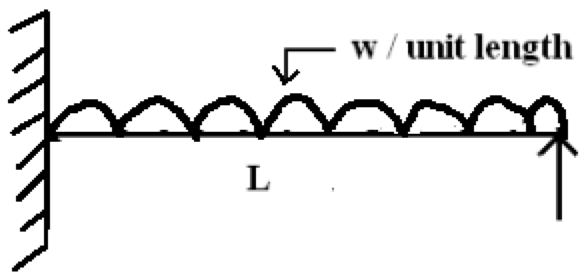
$$\delta_{Actual} = \frac{5}{384}\frac{WL^4}{EI_{XX}}$$

b) For simply supported beam with central point load



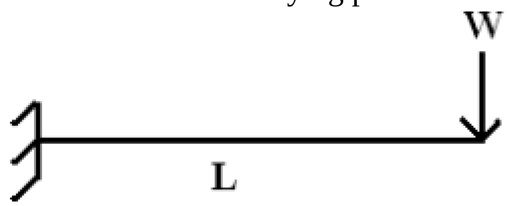
$$\delta_{Actual} = \frac{1}{48}\frac{WL^3}{EI_{XX}}$$

c) For cantilever beam carrying udl over entire span



$$\delta_{Actual} = \frac{1}{8}\frac{WL^4}{EI_{XX}}$$

d) For cantilever beam carrying point load at end



$$\delta_{Actual} = \frac{1}{3}\frac{WL^3}{EI_{XX}}$$

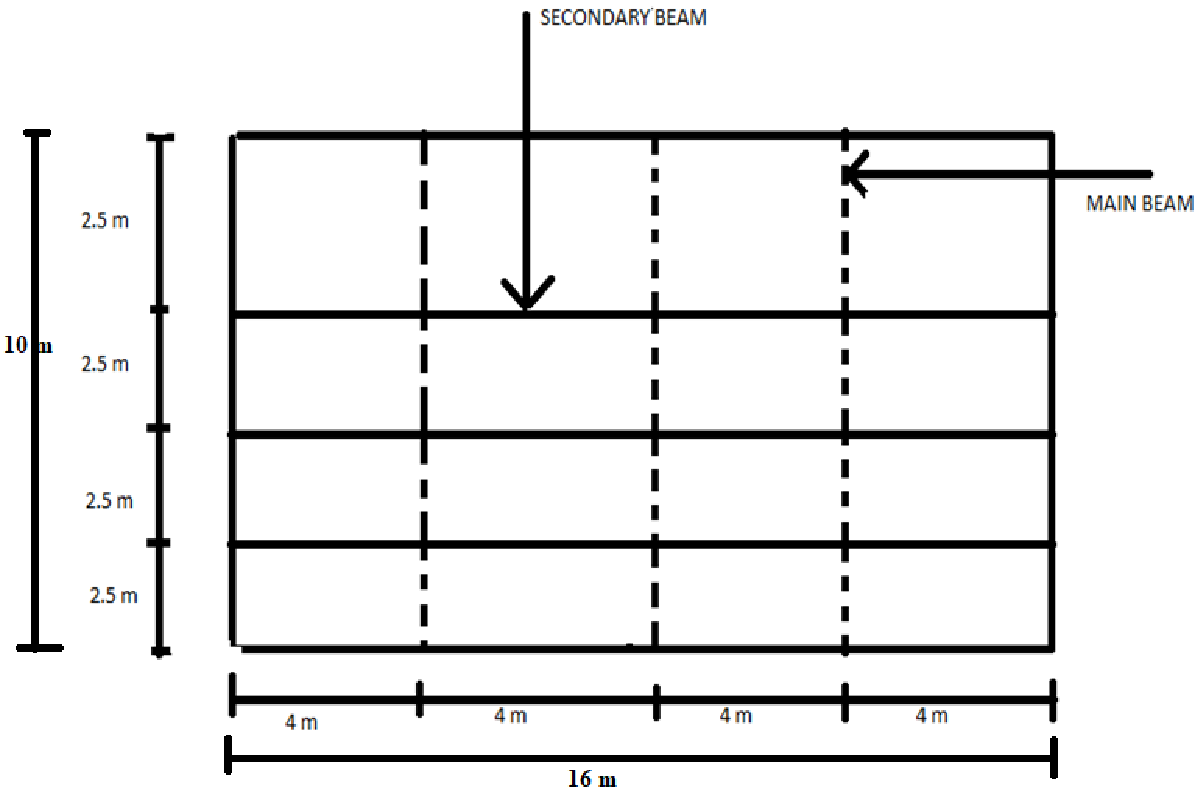
$$\delta_{permissible} > \delta_{Actual}$$

1. A concrete slab of 10 m X 16 m is having RC floor 120 mm thick is supported by main beam. The main beam having span of 10 m are spaced at 4 m centre to centre. The secondary beam having span of 4 m and spaced 2.5 m centre to centre. The secondary beam will be connected to the web of main beam. The density of concrete

25 KN/m^3. Live load is 5 KN/m^2, Floor finish is 0.75KN/m^2. Design any secondary beam and Main Beam. Take Fy= 250 N/mm^2

Solution :

<u>Design of Secondary Beam</u>:

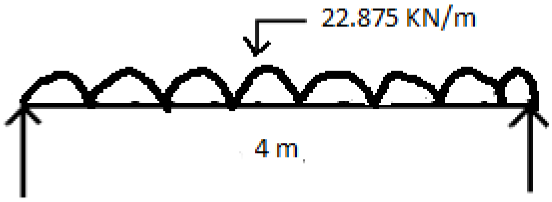


1. Loads:
    a) Weight of slab = Width X Thickness of Slab X Density of Concrete
       = 2.5 X 0.12 X 25 = 7.50 KN/m
    b) Live Load = Width X Intensity of Live Load
       = 2.5 X 5 = 12.50 KN/m
    c) Floor Finish = Width X Intensity of Floor Finish
       =2.5 X 0.75 = 1.875 KN/m
    d) Self weight of secondary beam= 1 KN/m

    Total Load = 7.5+ 12.5+1.875+1= 22.875 KN/m
2. Effective Span=L= Centre to centre distance between Support
   L = 4 m

3.Maximum Bending moment and Shear Force

For simply supported beam carrying udl over entire span



$$\textit{Maximum}\ \text{Bending Moment} = M = \frac{WL^2}{8} = \frac{22.875X4^2}{8} = 45.75KNm = 45.75X10^6\,Nmm$$

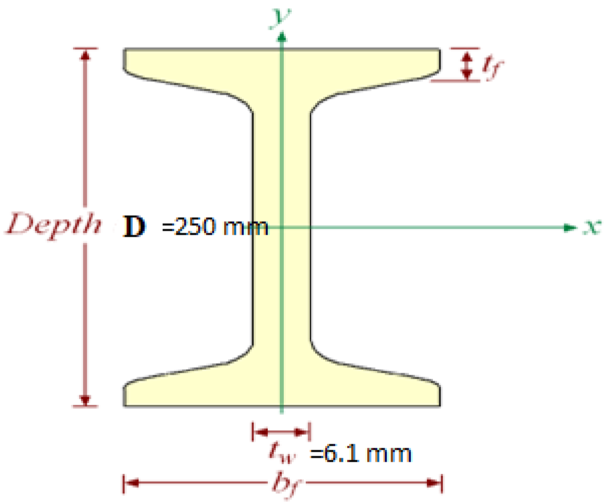
$$\text{Maximum Shear Force} = F = \frac{WL}{2} = \frac{22.875x4}{2} = 45.75KN = 45.75X10^3\,N$$

4. Section modulus required

$$\sigma_b = \frac{M}{Z_{req}}, Z_{req} = \frac{M}{\sigma_b}, \sigma_b = 0.66Fy = 0.66x250 = 165N/mm^2$$

$$Z_{req} = \frac{M}{\sigma_b} = \frac{45.75x10^6}{165} = 277.27x10^3\,mm^3$$

5. Try section from steel table
   ISLB 250@27.9 Kg/m
   Z_{XX}= 297.40 X 10^3 mm^3
   Ixx= 3717.8 x 10^4 mm^4
   t_w= 6.1 mm
   D=250 mm



6. Check for section for bending stress

$$\sigma_{b\ (cal)} = \frac{M}{I_{xx}}\ Y \prec 0.66\ Fy$$

$$\sigma_{b\ (cal)} = \frac{45.75x10^6}{3717.8x10^4} \frac{250}{2} \prec 0.66\text{ Fy}$$

$$\sigma_{b\ (cal)} = 153.82 N/mm^2 \prec 165 N/mm^2 \text{ ok}$$

7. Check for section for shear stress

$$\tau_{v\ (cal)} = \frac{F}{t_w D} \prec 0.4\text{ Fy}$$

$$\tau_{v\ (cal)} = \frac{45.75x10^3}{6.1x250} \prec 0.4\text{ x250=100}$$

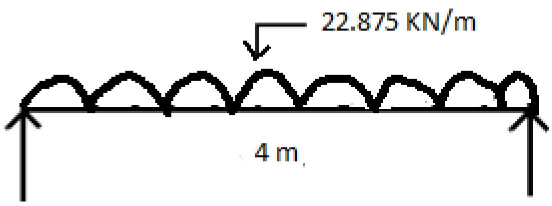
$$\tau_{v\ (cal)} = 30 N/mm^2 \prec 100 N/mm \text{ ok}$$

8. Check for deflection

Permissible deflection = $\frac{Span}{325} = \frac{4000}{325} = 12.30mm$

Actual Deflection

For simply supported beam carrying udl over entire span



$$\delta_{Actual} = \frac{5}{384}\frac{WL^4}{EI_{XX}} = \frac{5}{384}\frac{22.875x4000^4}{3717.8x10^4 x2x10^5}$$

$$\delta_{Actual} = 10.25mm \prec 12.30mm \text{ (ok)}$$

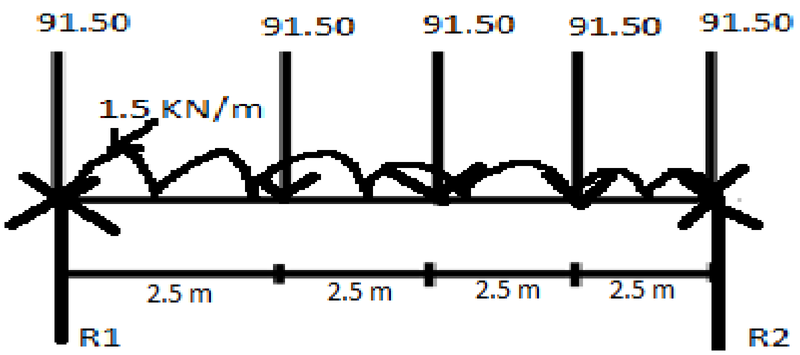
Design of Main Beam

1. Loading

Reaction of one secondary beam = $\frac{WL}{2} = \frac{22.875X4}{2} = 45.75KN$

Reaction of two secondary beam= $2x\frac{WL}{2} = 2x\frac{22.875X4}{2} = 91.50KN$

Assuming self weight of main beam = 1.5 KN/m

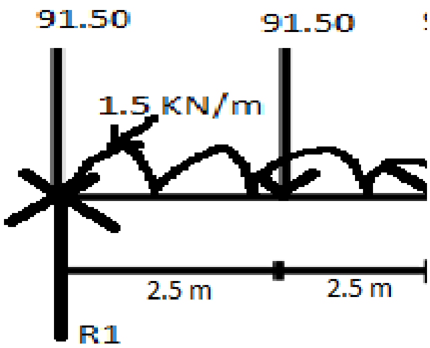


$$R_1 = R_2 = \frac{(5x91.50) + (1.5x10)}{2} = 236.25KN$$

$$\sum Fy = 0$$
$$R_1 + R_2 - 91.50 - 91.50 - 91.50 - 91.50 - 91.50 - (1.5x10) = 0$$
$$R_1 + R_2 = 472.50KN$$

$$\sum M @ R_1 = 0$$
$$(-R_2 x10) + (91.50x10) + (91.50x7.5) + (91.50x5) + (91.50x2.5) = 0$$
$$R_2 = 236.25KN$$
$$R_1 = 236.25KN$$

2. For maximum bending moment
   Maximum BM at centre



M= (236.25 x 5)-(91.50 x5)-(91.50 x 2.5)- (1.5 x 5x 2.5)

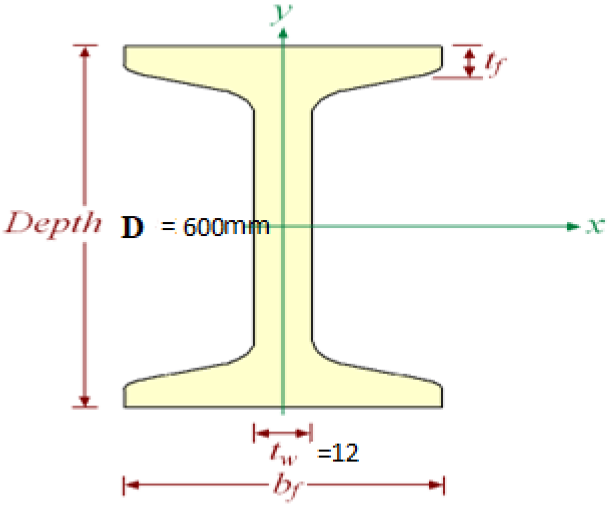
M= 476.25 KNm =476.25 x 10^6 Nmm

3. Section modulus required

$$\sigma_b = \frac{M}{Z_{req}}, Z_{req} = \frac{M}{\sigma_b}, \sigma_b = 0.66Fy = 0.66x250 = 165N/mm^2$$

$$Z_{req} = \frac{M}{\sigma_b} = \frac{476.25x10^6}{165} = 2.886x10^6 mm^3 = 2886.36x10^3 mm^3$$

4. Try section from steel table
   ISMB 600@122.60 Kg/m
   Z_{XX}= 3060.40 X 10^3 mm^3
   Ixx= 91813 x 10^4 mm^4
   t_w= 12 mm
   D=600 mm



5. Check for section for bending stress

$$\sigma_{b\ (cal)} = \frac{M}{I_{xx}}\ \text{Y} \prec 0.66\ \text{Fy}$$

$$\sigma_{b\ (cal)} = \frac{476.25x10^6}{91813x10^4}\ \frac{600}{2} \prec 0.66\ \text{Fy}$$

$$\sigma_{b\ (cal)} = 155.615 N/mm^2 \prec 165 N/mm^2\ \text{ok}$$

6. Check for shear stress

$$\tau_{v\ (cal)} = \frac{F}{t_w D} \prec 0.4\ \text{Fy}$$

Maximum Shear Force =F =R_1 OR R_2 Whichever is greater
Maximum Shear Force =F =236.25 KN

$$\tau_{v\ (cal)} = \frac{236.25x10^3}{12x600} \prec 0.4\ \text{x250=100}$$

$$\tau_{v\ (cal)} = 32.81 N/mm^2 \prec 100 N/mm\ \text{ ok}$$

7. Check for deflection

Permissible deflection = $\frac{Span}{325} = \frac{10000}{325} = 30.76mm$

Total Actual Deflection = Deflection due to udl +equally spaced point load

For simply supported beam carrying udl over entire span

$$\delta_{1Actual} = \frac{5}{384}\frac{WL^4}{EI_{XX}} = \frac{5}{384}\frac{1.5x10000^4}{91813x10^4 x2x10^5}$$

$$\delta_{1Actual} = 1.064mm$$

Deflection due equally spaced point load

$$\delta_{2Actual} = \frac{1}{192}\frac{WL^3}{EI_{XX}}n\left[3-\frac{1}{2}\left(1+\frac{4}{n^2}\right)\right]$$

$$\delta_{2Actual} = \frac{1}{192}\frac{91.50x10^3x10000^3}{2x10^5x91813x10^4}4\left[3-\frac{1}{2}\left(1+\frac{4}{4^2}\right)\right]$$

$W = 91.50KN$

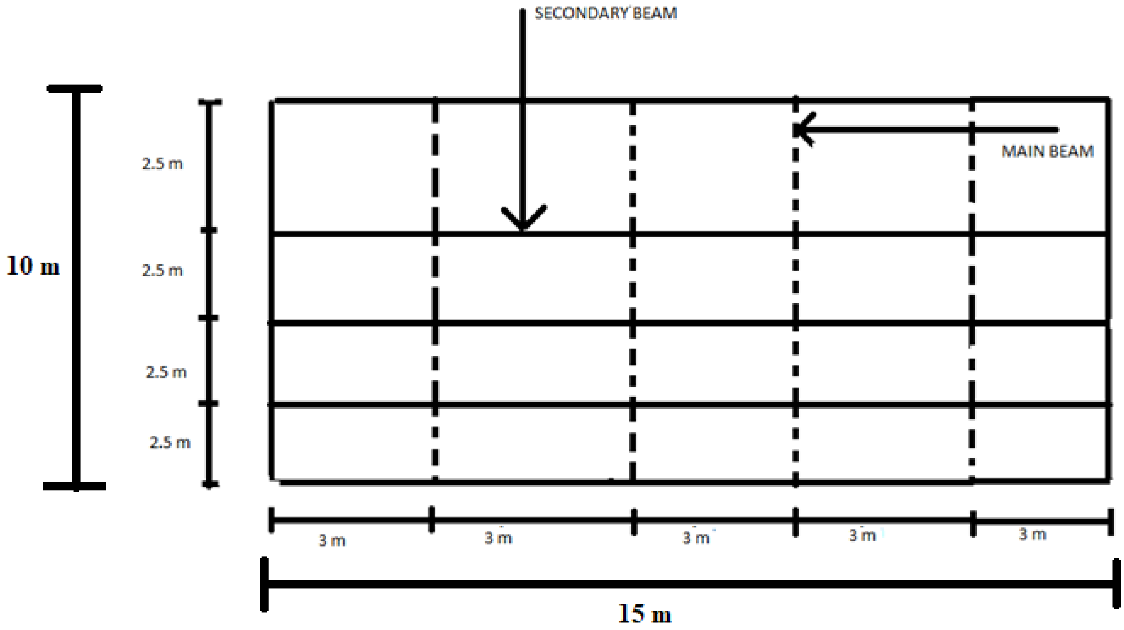
$n = 4$ (Number of spacing between loads)

$\delta_{2Actual} = 24.65mm$

Actual Deflection = $\delta = \delta_{1Actual} + \delta_{2Actual} = 1.064 + 24.65 = 25.719mm$

$$25.719 < 30.76 \text{ (ok)}$$

2. Figure shows building plan of RC Floor slab120 mm thick carries live load of 4 KN/mm^2, Floor finish is 1 KN/mm^2. Design secondary and main beam . Use Fy= 250 N/mm^2



1.Loads:

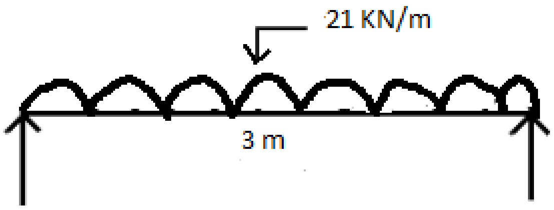
e) Weight of slab = Width X Thickness of Slab X Density of Concrete
 = 2.5 X 0.12 X 25 = 7.50 KN/m
f) Live Load = Width X Intensity of Live Load
 = 2.5 X 4 = 10 KN/m
g) Floor Finish = Width X Intensity of Floor Finish
 =2.5 X 1 = 1.875 KN/m
h) Self weight of secondary beam= 1 KN/m
 Total Load = 7.5+ 10+1.875+1= 21 KN/m

2.Effective Span=L= Centre to centre distance between Support

L = 3 m

3.Maximum Bending moment and Shear Force

For simply supported beam carrying udl over entire span



$$\textit{Maximum}\ \text{Bending Moment} = M = \frac{WL^2}{8} = \frac{21X3^2}{8} = 23.625KNm = 23.625X10^6\,Nmm$$

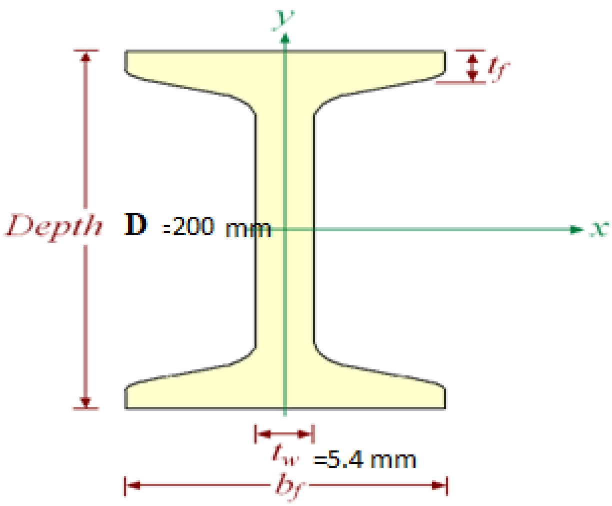
$$\text{Maximum Shear Force} = F = \frac{WL}{2} = \frac{21x3}{2} = 31.50KN = 31.50X10^3\,N$$

4.Section modulus required

$$\sigma_b = \frac{M}{Z_{req}}, Z_{req} = \frac{M}{\sigma_b}, \sigma_b = 0.66Fy = 0.66x250 = 165N/mm^2$$

$$Z_{req} = \frac{M}{\sigma_b} = \frac{23.625x10^6}{165} = 143.18x10^3\,mm^3$$

5.Try section from steel table
ISLB 200@19.8 Kg/m
Z_{XX}= 169.7 X 10^3 mm^3
Ixx= 1696.6 x 10^4 mm^4
t_w= 5.4 mm
D= 200 mm



6. Check for section for bending stress

$$\sigma_{b\ (cal)} = \frac{M}{I_{xx}}\ \text{Y} \prec 0.66\ \text{Fy}$$

$$\sigma_{b\ (cal)} = \frac{23.625x10^6}{1696.6x10^4}\ \frac{200}{2} \prec 0.66\ \text{Fy}$$

$$\sigma_{b\ (cal)} = 139.25N/mm^2 \prec 165N/mm^2\ \text{ok}$$

7. Check for section for shear stress

$$\tau_{v\ (cal)} = \frac{F}{t_w D} \prec 0.4\ \text{Fy}$$

$$\tau_{v\ (cal)} = \frac{31.50x10^3}{5.4x200} \prec 0.4\ \text{x250=100}$$

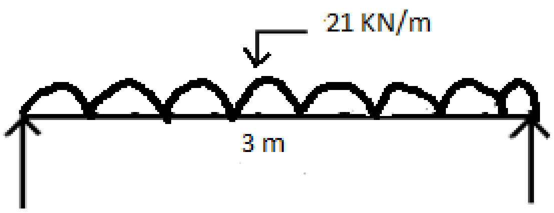
$$\tau_{v\ (cal)} = 29.17N/mm^2 \prec 100N/mm\ \text{ok}$$

8. Check for deflection

Permissible deflection = $\frac{Span}{325} = \frac{3000}{325} = 6.52mm$

Actual Deflection

For simply supported beam carrying udl over entire span



$$\delta_{Actual} = \frac{5}{384}\frac{WL^4}{EI_{XX}} = \frac{5}{384}\frac{21x3000^4}{1696.6x10^4 x2x10^5}$$

$$\delta_{Actual} = 6.52mm \prec 9.23mm\ \text{(ok)}$$

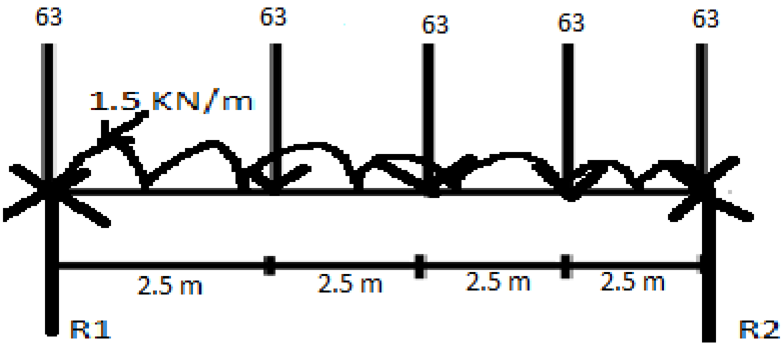
<u>Design of Main Beam</u>

1. Loading

Reaction of one secondary beam = $\frac{WL}{2} = \frac{21X3}{2} = 31.5KN$

Reaction of two secondary beam= $2x\frac{WL}{2} = 2x\frac{21X3}{2} = 63KN$

Assuming self weight of main beam = 1.5 KN/m

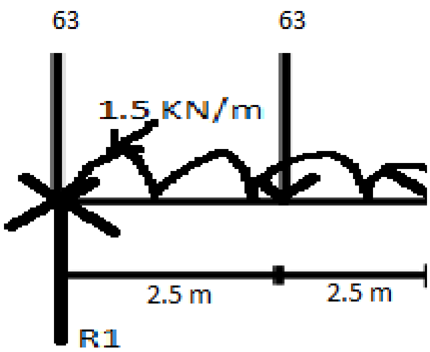


$$R_1 = R_2 = \frac{(5x63)+(1.5x10)}{2} = 165KN$$

$$\sum Fy = 0$$
$$R_1 + R_2 - 63 - 63 - 63 - 63 - 63 - (1.5x10) = 0$$
$$R_1 + R_2 = 315KN$$

$$\sum M @ R_1 = 0$$
$$(-R_2 x10) + (63x10) + (63x7.5) + (63x5) + (63x2.5) = 0$$
$$R_2 = 165KN$$
$$R_1 = 165KN$$

2. .For maximum bending moment
   Maximum BM at centre



M= (165 x 5)-(63 x5)-(63 x 2.5)- (1.5 x 5x 2.5)

M= 333.75 KNm =333.75 x 10^6 Nmm

3. Section modulus required

$$\sigma_b = \frac{M}{Z_{req}}, Z_{req} = \frac{M}{\sigma_b}, \sigma_b = 0.66Fy = 0.66x250 = 165N/mm^2$$

$$Z_{req} = \frac{M}{\sigma_b} = \frac{333.76x10^6}{165} = 2.023x10^6 mm^3 = 2023x10^3 mm^3$$

4. Try section from steel table

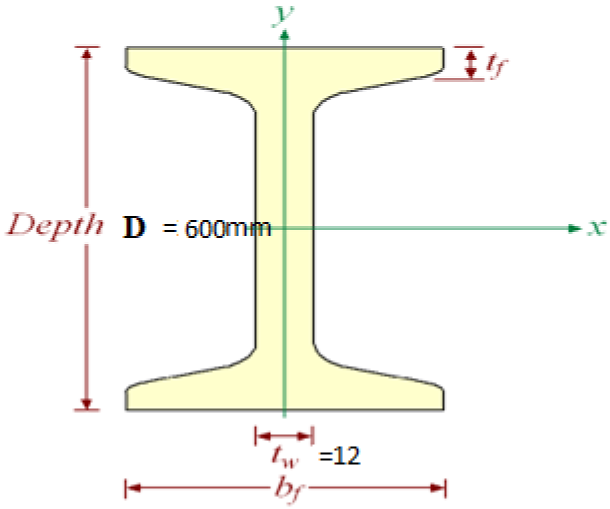
   ISMB 600@122.60 Kg/m

   Z_{XX}= 3060.40 X 10^3 mm^3

   Ixx= 91813 x 10^4 mm^4

   t_W= 12 mm

   D=600 mm



5. Check for section for bending stress

$$\sigma_{b\ (cal)} = \frac{M}{I_{xx}} \text{ Y} \prec 0.66 \text{ Fy}$$

$$\sigma_{b\ (cal)} = \frac{333.75x10^6}{91813x10^4} \frac{600}{2} \prec 0.66 \text{ Fy}$$

$$\sigma_{b\ (cal)} = 109.04N/mm^2 \prec 165N/mm^2 \text{ ok}$$

6. Check shear stress

$$\tau_{v\ (cal)} = \frac{F}{t_w D} \prec 0.4 \text{ Fy}$$

Maximum Shear Force =F =R_1 OR R_2 Whichever is greater

Maximum Shear Force =F =236.25 KN

$$\tau_{v\ (cal)} = \frac{165x10^3}{12x600} \prec 0.4 \text{ x250=100}$$

$$\tau_{v\ (cal)} = 22.916N/mm^2 \prec 100N/mm \text{ ok}$$

7. Check for deflection

Permissible deflection = $\frac{Span}{325} = \frac{10000}{325} = 30.76mm$

Total Actual Deflection = Deflection due to udl +equally spaced point load

For simply supported beam carrying udl over entire span

$$\delta_{1Actual} = \frac{5}{384}\frac{WL^4}{EI_{XX}} = \frac{5}{384}\frac{1.5x10000^4}{91813x10^4\,x2x10^5}$$

$$\delta_{1Actual} = 1.064mm$$

Deflection due equally spaced point load

$$\delta_{2Actual} = \frac{1}{192}\frac{WL^3}{EI_{XX}} n\left[3-\frac{1}{2}\left(1+\frac{4}{n^2}\right)\right]$$

$$\delta_{2Actual} = \frac{1}{192}\frac{63x10^3 x10000^3}{2x10^5 x91813x10^4} 4\left[3-\frac{1}{2}\left(1+\frac{4}{4^2}\right)\right]$$

$W = 63KN$

$n = 4$ (Number of spacing between loads)

$\delta_{2Actual} = 17.81mm$

Actual Deflection = $\delta = \delta_{1Actual} + \delta_{2Actual} = 1.064 + 17.81 = 18.877mm$

$18.877 < 30.76$ (ok)

CASE II : Design of laterally supported Beam (Simple Beam)

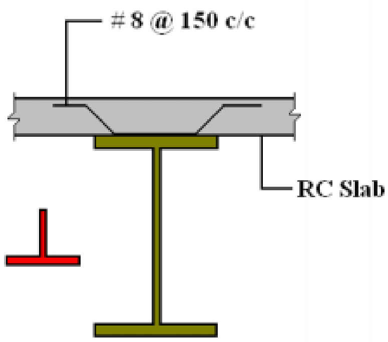
Laterally Supported (Restrained) beams

Beams subjected to BM develop compressive and tensile forces and the flange subjected to compressive forces has the tendency to deflect laterally. This out of plane bending is called lateral bending or buckling of beams. The lateral bending of beams depends on the effective span between the restraints, minimum moment of inertia (I_{YY}) and its presence reduces the plastic moment capacity of the section.

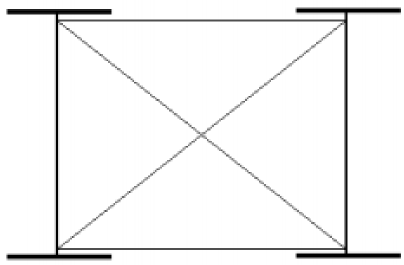
Beams where lateral buckling of the compression flange are prevented are called laterally restrained beams. Such continuous lateral supports are provided in two ways

i) The compression flange is connected to an RC slab throughout by shear connectors.

ii) External lateral supports are provided at closer intervals to the compression flange so that it is as good continuous lateral support.

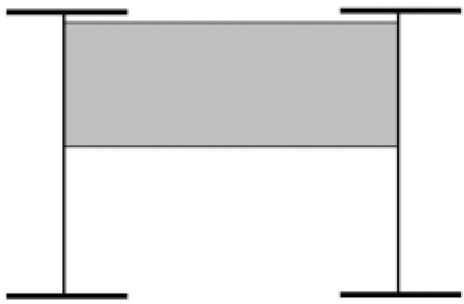


I section with rebars or studs



I section with cross frame

Typical lateral supports are shown in the figure.



I section with diaphragm

Typical lateral supports

Procedure:

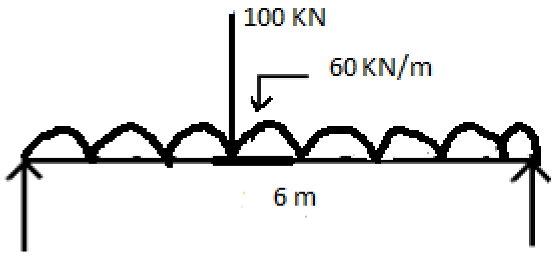
1. Calculate self weight of beam , assuming
   $$\frac{W}{300} \, to \, \frac{W}{350} \, KN/m$$
   W= Total Load
2. Calculate total load = Self weight + Live Load (Imposed Load) (If W is not given)
3. Calculate Bending Moment and
4. Shear force in the beam

   Take $\sigma_{bc} = \sigma_{bt} = 0.66 \text{ x } F_y = 0.66x250 = 165 N/mm^2$
5. Find section modulus (Z) of the beam
   $$Z_{req} = \frac{M}{\sigma_{bc}}$$

6. Increase Zreq by 20% to 40%
7. Select the value of Z (Section Modulus) more than Zreq
8. Check for Bending Stress
9. Check for shear Stress
10. Check for deflection

1. A simply supported beam has an effective span of 6m. It carries a udl of 60 KN/m and concentrated load of 100 KN at mid span. Assume E= 2 X 10^5 N/mm^2 and Fy= 250 N/mm^2. Design the beam for flexure, shear and deflection, If its laterally supported. Apply the usual check.

   Solution:



1. Calculate self weight of beam , assuming

$$\frac{W}{300} to \frac{W}{350} KN/m$$

W= (60X6)+100 = 460 KN

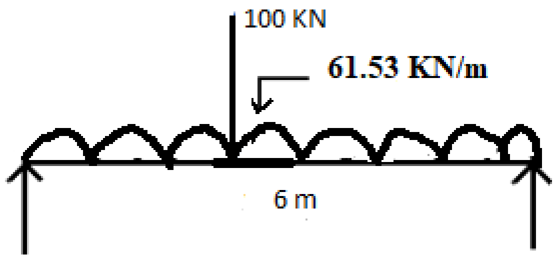
Self weight of beam $= \frac{W}{300} = \frac{460}{300} = 1.53 KN/m$

Total UDL =Wu=60+1.53 =61.53 KN/m

Point Load =Wp= 100 KN

2. Maximum Bending Moment

   Maximum bending moment at centre



$$Max\ \text{BM}=\frac{W_u L^2}{8}+\frac{W_p L}{4}$$

$$Max\ \text{BM}=\frac{61.53x6^2}{8}+\frac{100x6}{4}=426.885KNm=426.885x10^6\ Nmm$$

3. Maximum Shear Force

$$Max\ \text{SF=F=}\frac{W_u L}{2}+\frac{W_p}{2}$$

$$Max\ \text{SF=}\frac{61.53x6}{2}+\frac{100}{2}=234.59KN=234.59x10^3 N$$

$$\sigma_{bc}=\sigma_{bt}=0.66\text{ x F}_\text{y}=0.66x250=165N/mm^2$$

4. Find section modulus (Z) of the beam

$$Z_{req}=\frac{M}{\sigma_{bc}} \qquad Z_{req}=\frac{426.88x10^6}{165}=2587.15x10^3\ \text{mm}^3$$

5Increase Zreq by 30%

Zreq = 1.3 x 2587.15 x 10^3 =3363.30 x 10^3 mm^3

6.Select the value of Z (Section Modulus) more than Zreq

Try ISWB 600@133.70 kg/m
A= 17038 mm^2
tw=11.20mm
Zxx= 3540 x 10^3 mm^3
Ixx= 106198.5 x 10^4 mm^4
D=600 mm

7. Check for bending stress

$$\sigma_{b\ (cal)}=\frac{M}{I_{xx}}\text{ Y}\prec 0.66\text{ Fy}$$

$$\sigma_{b\ (cal)}=\frac{426.88x10^6}{106198.5x10^4}\ \frac{600}{2}\prec 0.66\text{ x }250$$

$$\sigma_{b\ (cal)}=120.58N/mm^2\prec 165\ N/mm^2\ \text{(OK)}$$

8.Check for shear stress

$$\tau_{v\ (cal)}=\frac{F}{t_w D}\ \prec 0.4\text{ Fy}$$

$$\tau_{v\ (cal)}=\frac{234.59x10^3}{11.20x600}\ \prec 0.4\text{ x }250$$

$$\tau_{v\ (cal)}=34.90N/mm^2<100N/mm^2$$

9. Check for deflection

Permissible deflection = $\frac{Span}{325}=\frac{6000}{325}=18.46mm$

Total Actual Deflection = Deflection due to udl + Deflection due to point load

$$\delta_{Actual} = \frac{5}{384}\frac{WuL^4}{EI_{XX}} + \frac{1}{48}\frac{WpL^3}{EI_{XX}}$$

$$= \frac{5}{384}\frac{61.53x6000^4}{106198.5x10^4x2x10^5} + \frac{1}{48}\frac{100x10^3x6000^3}{106198.5x10^4x2x10^5}$$

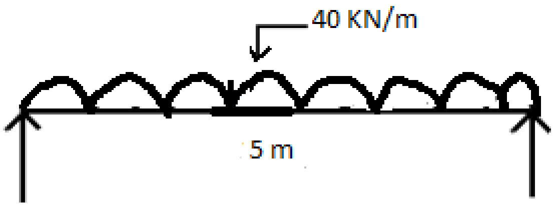
$$\delta_{Actual} = 4.88 + 2.118 = 7mm$$

$$\delta_{Actual} < \delta_{Permisible}$$

$$7\ mm < 18.46\ mm\ (ok)$$

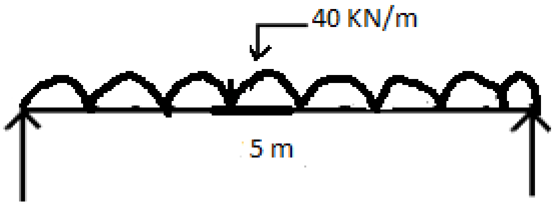
2. A simply supported beam steel joist with a 5 m effective span carries a udl of 40 KN/m over its span inclusive of self weight. The beam is supported throughout laterally. Select a suitable section and check its safety. Take Fy= 250 N/mm^2

   Solution:



   1. Maximum Bending Moment

      Maximum bending moment at centre



$$Max\ \text{BM}=\frac{W_uL^2}{8}$$

$$Max\ \text{BM}=\frac{40x5^2}{8} = 125KNm = 125x10^6\ Nmm$$

   2. Maximum Shear Force

$$Max\ \text{SF=F=}\frac{W_uL}{2}$$

$$Max\ \text{SF=}\frac{40x5}{2} = 100KN = 100x10^3\ N$$

$$\sigma_{bc} = \sigma_{bt} = 0.66\ \text{x}\ \text{F}_{\text{y}} = 0.66x250 = 165N/mm^2$$

4. Find section modulus (Z) of the beam

$$Z_{req} = \frac{M}{\sigma_{bc}} \qquad Z_{req} = \frac{125x10^6}{165} = 757.57x10^3 \text{ mm}^3$$

5.Increase Zreq by 30%

Zreq = 1.3 x 757.57 x 10^3 =984.85x 10^3 mm^3

6.Select the value of Z (Section Modulus) more than Zreq

Try ISMB 400@61.60 kg/m
A= 7846 mm^2
tw=8.90mm
Zxx= 1022.9 x 10^3 mm^3
Ixx= 20458.4 x 10^4 mm^4
D=400 mm

7. Check for bending stress

$$\sigma_{b\ (cal)} = \frac{M}{I_{xx}} \text{ Y} \prec 0.66 \text{ Fy}$$

$$\sigma_{b\ (cal)} = \frac{125x10^6}{20458.4x10^4} \ \frac{400}{2} \prec 0.66 \text{ x } 250$$

$$\sigma_{b\ (cal)} = 122.22 N/mm^2 \prec 165\ N/mm^2 \text{ (OK)}$$

8.Check for shear stress

$$\tau_{v\ (cal)} = \frac{F}{t_w D} \prec 0.4 \text{ Fy}$$

$$\tau_{v\ (cal)} = \frac{100x10^3}{8.9x400} \prec 0.4 \text{ x } 250$$

$$\tau_{v\ (cal)} = 28.09 N/mm^2 < 100 N/mm^2$$

9. Check for deflection

Permissible deflection = $\frac{Span}{325} = \frac{5000}{325} = 15.38mm$

Total Actual Deflection = Deflection due to udl

$$\delta_{Actual} = \frac{5}{384} \frac{WuL^4}{EI_{XX}}$$

$$= \frac{5}{384} \frac{40x5000^4}{20458.4x10^4 x2x10^5}$$

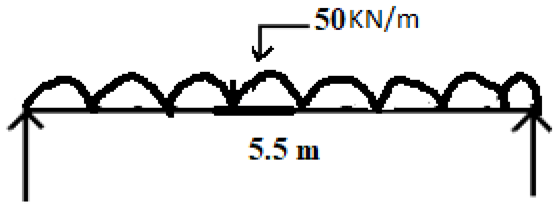
$$\delta_{Actual} = 7.96mm$$

$$\delta_{Actual} < \delta_{Permisible}$$

$$7.96\ mm < 15.38\ mm\ (ok)$$

3. A simply supported beam steel joist with a 5.5 m effective span carries a udl of 50 KN/m over its span exclusive of self weight . The beam is supported throughout laterally. Select a suitable section and check its safety. Take Fy= 250 N/mm^2

Solution:



1. Calculate self weight of beam , assuming

$$\frac{W}{300} to \frac{W}{350} KN/m$$

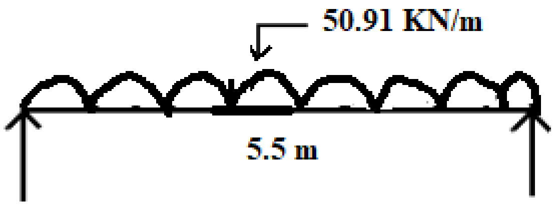
W= (50X5.5) = 275 KN

Self weight of beam $= \frac{W}{300} = \frac{275}{300} = 0.91 KN/m$

Total UDL =Wu=50+0.91 =50.91 KN/m

2. Maximum Bending Moment

   Maximum bending moment at centre



$$Max \text{ BM}= \frac{W_u L^2}{8}$$

$$Max \text{ BM}= \frac{50.91 x 5.5^2}{8} = 192.50 KNm = 192.50 x 10^6 Nmm$$

3. Maximum Shear Force

$$Max \text{ SF=F}= \frac{W_u L}{2}$$

$$Max \text{ SF}= \frac{50.91 x 5.5}{2} = 140 KN = 140 x 10^3 N$$

$$\sigma_{bc} = \sigma_{bt} = 0.66 \text{ x F}_\text{y} = 0.66 x 250 = 165 N/mm^2$$

4. Find section modulus (Z) of the beam

$$Z_{req} = \frac{M}{\sigma_{bc}} \qquad Z_{req} = \frac{192.50x10^6}{165} = 1166.66x10^3 \text{ mm}^3$$

5Increase Zreq by 30%

Zreq = 1.3 x 1166.66 x 10^3 =1516.66 x 10^3 mm^3

6.Select the value of Z (Section Modulus) more than Zreq

Try ISHB 450 @ 87.2 kg/m
A= 11114 mm^2
tw=9.8 mm
Zxx= 1742.7 x 10^3 mm^3
Ixx= 39210.8 x 10^4 mm^4
D=450 mm

7. Check for bending stress

$$\sigma_{b\ (cal)} = \frac{M}{I_{xx}} \text{ Y} \prec 0.66 \text{ Fy}$$

$$\sigma_{b\ (cal)} = \frac{192.5x10^6}{39210.8x10^4} \; \frac{450}{2} \prec 0.66 \text{ x } 250$$

$$\sigma_{b\ (cal)} = 110.48N / mm^2 \prec 165 \; N / mm^2 \text{ (OK)}$$

8.Check for shear stress

$$\tau_{v\ (cal)} = \frac{F}{t_w D} \prec 0.4 \text{ Fy}$$

$$\tau_{v\ (cal)} = \frac{140x10^3}{9.8x450} \prec 0.4 \text{ x } 250$$

$$\tau_{v\ (cal)} = 31.74N / mm^2 < 100N / mm^2 \text{ (OK)}$$

9. Check for deflection

Permissible deflection = $\frac{Span}{325} = \frac{5500}{325} = 16.92mm$

Total Actual Deflection = Deflection due to udl

$$\delta_{Actual} = \frac{5}{384} \frac{WuL^4}{EI_{XX}}$$

$$= \frac{5}{384} \frac{50.91x5500^4}{39210.8x10^4 x2x10^5}$$

$$\delta_{Actual} = 7.73mm$$

$$\delta_{Actual} < \delta_{Permisible}$$

$$7.73 \; mm < 16.92 \; mm \; (ok)$$

CASE III : Design of Laterally Unsupported Beam

Procedure:

1. Calculate Effective span
2. Calculate self weight of beam
3. Calculate total load
4. Calculate effective length of compressive flange from IS 800:1984, P. No: 76
5. Calculate BM and
6. Calculate shear force
7. Calculate section modulus

   $$Z = \frac{M}{\sigma_{bc}}$$

8. Increase section modulus by 50% to 80%
   If Leff=0.70 L Then Zreq=1.5 Z
   Leff=L Then Zreq=1.8 Z
9. Select the section having Section modulus (Z) more than Zreq
10. For beams and channels with equal flanges

    Find i) $\frac{T}{t}$ Ratio

    ii) $\frac{d_1}{t}$ Ratio

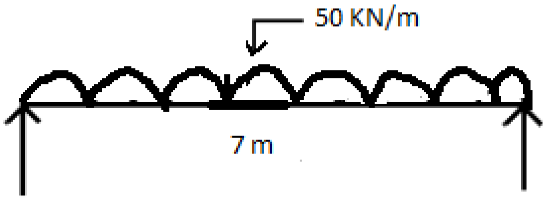
    iii) $\frac{D}{t}$ Ratio

    iv) $\frac{l_{eff}}{r_{yy}}$ Ratio

    For the trial section and find maximum permissible bending stress from table 6.10 A , P No:57 and 6.10 B , P No:58
11. Calculate $Z_{req} = \frac{M}{\sigma_{bc(act)}} > Z$ Trial
12. Check for Shear Stress
13. Check for Deflection

1. A simply supported beam has effective span of 7 m and carries a udl of 50 KN/m. Design the beam if the beam is laterally unsupported. Each end point of the beam is restrained against torsion and ends of the compressive flanges are fully restrained against lateral bending. Use Fy= 250 N/mm^2, E= 2 X 10^5 N/mm^2
   Solution : Given Data
   L=7 m
   L_{eff} =7 m
   Fy= 250 N/mm^2

E= 2 X 10^5 N/mm^2

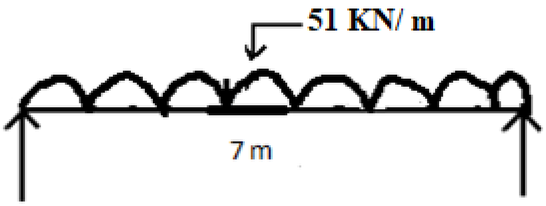


1. Assuming self weight of beam

   Self weight $= \frac{W}{350} = \frac{50x7}{350} = 1KN/m$

   Total Load =Wu= 50+1= 51 KN/m
2. Maximum Bending Moment

   Maximum bending moment at centre



$$Max\ \text{BM}= \frac{W_u L^2}{8} = \frac{51x7^2}{8} = 312.375KNm = 312.375x10^6\ Nmm$$

3.Maximum Shear Force

$$Max\ \text{SF}= \frac{W_u L}{2} = \frac{51x7}{2} = 178.50KN = 178.50x10^3\ N$$

$$\sigma_{bc} = \sigma_{bt} = 0.66 \text{ x } F_y = 0.66x250 = 165N/mm^2$$

4. Find section modulus (Z) of the beam

$$Z = \frac{M}{\sigma_{bc}} \qquad Z = \frac{312.375x10^6}{165} = 1.893x10^6\ \text{mm}^3 = 1893x10^3\ \text{mm}^3$$

5.Increase Zreq by 80%

Zreq= 1.8 X Z =1.80 X 1.893 X 10^6 =3.407 X 10^6 mm^3=3407 X10^3 mm^3

6. Try ISWB 600@133.7 kg/m

D=600 mm

t= t_w=11.20 mm

$T = t_f = 21.30$ mm

$Ixx = 106198.5 \times 10^4$ mm^4

$ryy = 52.50$ mm

$h_1 = 514.20$ mm

$Zxx = 3540 \times 10^3$ mm^3

$$\text{Ratio}\,\frac{T}{t} = \frac{t_f}{t_w} = \frac{21.30}{11.20} = 1.90 < 02$$

$$d_1 = D - 2t_f = 600 - (2x21.30) = 557.40$$

$$\text{Ratio}\,\frac{d_1}{t} = \frac{557.40}{11.2} = 49.76 < 85$$

Use Table 6.1 B , P. No:58 C No:6.2.2 from IS 800 because $\frac{T}{t} < 2 \; and \; \frac{d_1}{t} < 85$

$$\frac{D}{T} = \frac{600}{21.30} = 28.16$$

$$\frac{l}{r_{yy}} = \frac{7000}{52.30} = 133.33$$

$\frac{D}{T}$ for 25 to 30

For $\frac{D}{T} = 25$

l/ryy	6_{bc}
130	108
133.33	?
140	103

6_{bc} For $\left(\frac{D}{T}\right)_{25} = 108 + \left[\frac{103-108}{140-130} x(133.33-130)\right] = 106.335 N/mm^2$

For $\frac{D}{T} = 30$

l/ryy	6_{bc}

130	103
133.33	?
140	97

6bc For $\left(\frac{D}{T}\right)_{30} = 103 + \left[\frac{97-103}{140-130} x(133.33-130)\right] = 101.002 N/mm^2$

For $\frac{D}{T} = 28.17$

$\frac{D}{T}$	6bc
25	106.35
28.17	?
30	101.002

6bc For $\left(\frac{D}{T}\right)_{28.17} = 106.335 + \left[\frac{101.002-106.335}{30-20} x(28.17-25)\right] = 102.959 N/mm^2$

7.Section Modulus

$$Z = \frac{M}{\sigma_{bc}} = \frac{312.375x10^6}{102.959} = 3034.23x10^3 \text{ mm}^3$$

$$Z < Z_{req} \text{ (ok)}$$

8. Check for shear stress

$$\tau_{v\ (cal)} = \frac{F}{t_w D} \prec 0.4 \text{ Fy}$$

$$\tau_{v\ (cal)} = \frac{178.50x10^3}{11.2x600} \prec 0.4 \text{ x250}$$

$$\tau_{v\ (cal)} = 26.56 N/mm^2 \prec 100\ N/mm^2 \text{ (ok)}$$

9.Check for deflection

Permissible deflection = $\frac{Span}{325} = \frac{7000}{325} = 21.53mm$

Total Actual Deflection = Deflection due to udl

$$\delta_{Actual} = \frac{5}{384}\frac{WuL^4}{EI_{XX}}$$

$$= \frac{5}{384}\frac{51x7000^4}{106198.5x10^4 x2x10^5}$$

$$\delta_{Actual} = 7.50mm$$

$$\delta_{Actual} < \delta_{Permisible}$$

$$7.50\ mm < 21.53\ mm\ (ok)$$

2.Design a beam of 4.5 m effective span carrying a udl of 25 KN/m, if the compression flange is laterally unsupported. Use Fy= 250 N/mm^2, E= 2 X 10^5 N/mm^2

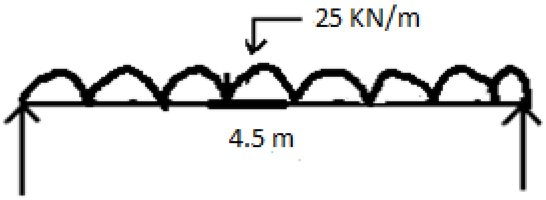
Solution : Given Data

L=4.5m

L_{eff} =4.5 m

Fy= 250 N/mm^2

E= 2 X 10^5 N/mm^2



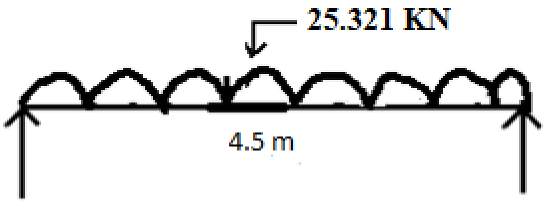
1.Assuming self weight of beam

Self weight $= \frac{W}{350} = \frac{25x4.5}{350} = 0.321KN / m$

Total Load =Wu= 25+0.321= 25.321 KN/m

2.Maximum Bending Moment

Maximum bending moment at centre



$$Max\ \text{BM}= \frac{W_u L^2}{8} = \frac{25.321x4.5^2}{8} = 64.09KNm = 64.09x10^6\ Nmm$$

3.Maximum Shear Force

$$Max\ \text{SF=F=}\frac{W_u L}{2}=\frac{25.321x4.5}{2}=56.97KN=56.97x10^3\,N$$

$$\sigma_{bc}=\sigma_{bt}=0.66\text{ x }F_y=0.66x250=165N/mm^2$$

4. Find section modulus (Z) of the beam

$$Z=\frac{M}{\sigma_{bc}} \qquad Z=\frac{64.09x10^6}{165}=388.424x10^3\text{ mm}^3$$

5.Increase Zreq by 80%

Zreq= 1.8 X Z =1.80 X 388.424 X 10^6 =699.163 X 10^3 mm^3

6. Try ISMB 350@52.4 kg/m

D=350 mm

t= t_w=8.10 mm

T=t_f= 14.20 mm

Ixx= 13630.3 x 10^4 mm^4

ryy= 28.4 mm

h_1= 288 mm

Zxx= 778.90 x 10^3 mm^3

$$\text{Ratio}\frac{T}{t}=\frac{t_f}{t_w}=\frac{14.20}{8.10}=1.75<02$$

$$d_1=D-2t_f=350-(2x14.20)=321.60mm$$

$$\text{Ratio}\frac{d_1}{t}=\frac{321.60}{8.10}=39.70<85$$

Use Table 6.1 B , P. No:58 C No:6.2.2 from IS 800 because $\frac{T}{t}<2\ and\ \frac{d_1}{t}<85$

$$\frac{D}{T}=\frac{350}{14.20}=24.647$$

$$\frac{l}{r_{yy}}=\frac{4500}{28.40}=158.45$$

$\frac{D}{T}$ for 20 to 25

For $\frac{D}{T} = 20$

l/ryy	6_{bc}
150	105
158.45	?
160	101

$$6_{bc}\text{ For }\left(\frac{D}{T}\right)_{20} = 105 + \left[\frac{101-105}{160-140}x(158.45-150)\right] = 101.62N/mm^2$$

For $\frac{D}{T} = 25$

l/ryy	6_{bc}
150	98
158.45	?
160	93

$$6_{bc}\text{ For }\left(\frac{D}{T}\right)_{30} = 98 + \left[\frac{93-98}{160-150}x(158.45-150)\right] = 93.775N/mm^2$$

For $\frac{D}{T} = 24.647$

$\frac{D}{T}$	6_{bc}
20	101.62
24.647	?
25	93.775

$$6_{bc}\text{ For }\left(\frac{D}{T}\right)_{24.647} = 101.62 + \left[\frac{93.775-101.62}{25-20}x(24.647-20)\right] = 94.328N/mm^2$$

7.Section Modulus

$$Z = \frac{M}{\sigma_{bc}} = \frac{64.09x10^6}{94.328} = 679.437x10^3 \text{ mm}^3$$

$$Z < Z_{req} \text{ (ok)}$$

8. Check for shear stress

$$\tau_{v\ (cal)} = \frac{F}{t_w D} \prec 0.4 \text{ Fy}$$

$$\tau_{v\ (cal)} = \frac{57.09x10^3}{8.10x350} \prec 0.4 \text{ x250}$$

$$\tau_{v\ (cal)} = 20.09 N/mm^2 \prec 100\ N/mm^2 \text{ (ok)}$$

9.Check for deflection

Permissible deflection = $\frac{Span}{325} = \frac{4500}{325} = 13.85mm$

Total Actual Deflection = Deflection due to udl

$$\delta_{Actual} = \frac{5}{384}\frac{WuL^4}{EI_{XX}}$$

$$= \frac{5}{384}\frac{25.321x4500^4}{13630.3x10^4 x2x10^5}$$

$$\delta_{Actual} = 4.97mm$$

$$\delta_{Actual} < \delta_{Permisible}$$

$$4.97\ mm < 13.85\ mm\ (ok)$$

3.Design a steel I section for a simply supported beam with clear span of 4.5 m. The beam carries a udl of 25 KN/m inclusive of self weight of beam and a point load of 20 KN at the centre of span. The beam is laterally unsupported. Show necessary check.

Use Fy= 250 N/mm^2, E= 2 X 10^5 N/mm^2

Roof Truss

The roof trusses are used when

1. The span is very large and beam construction is not economical.
2. The building is in area of heavy rainfall

   Type of roof trusses

   1. Howe Truss: Span 6 m to 9 m
   2. Pratt Truss: Span 6 m to 80 m
   3. Simple fink roof Truss: Span 6 m to 9 m
   4. Compound fink roof Truss: Span 20 m to 30 m
   5. Compound French roof Truss: Span 20 m to 30 m
   6. Simple fan Truss: Span 10 m to 15 m
   7. North light Roof Truss: Span 8 m to 10 m

1. A 20 m high building is to be constructed in Delhi for a 50 Year life. The size of building over 50 m. The topography of the site is an open terrain with very few obstructions and is classified under category I. Determine wind pressure of the site.

   Solution:

   1.Basic wind speed for Delhi=V_b=47 m/s

   2. The building must design for a 50 year

   Risk Coefficient/ Probability Factor K_1=1

   3.The size of building is over 50 m

   The building is classified as Class C and Category 1

   Terrain, Height and Structure Size Factor=K_2=1.06

   4.Topography Factor=K_3=1

   5. Design Wind Speed = $V_z = K_1 K_2 K_3 V_b$

   $$= V_z = 1 \text{ x } 1.06 \text{ x } 1 \text{ x } 47$$

   $$= 49.82 \text{ m/s}$$

   6. Design Wind Pressure = $P_d = 0.6 \text{ x } V^2_z$

   $$= 0.6 \text{ x } 49.82^2 = 1489.219 \text{ N/m}^2$$

2. It is proposed to design an industrial building 12 m high in Lacknow for 50-year life. The building size is range between 20 m to 50 m. The topography of the site is plain, and terrain is in city (Industrial Area). Determine the design wind pressure at site.

Solution:

1.Basic wind speed for Lucknow =V_b=47 m/s

2. The building must design for a 50 year

Risk Coefficient/ Probability Factor K_1=1

3.The size of building between 20 m to 50 m.

The building is classified as Class B and Category 4

Terrain, Height and Structure Size Factor=K_2=0.76

4.Topography Factor=K_3=1

5. Design Wind Speed = $V_z = K_1K_2K_3\ V_b$

$= V_z = 1 \times 0.76 \times 1 \times 47$

$= 35.72$ m/s

6. Design Wind Pressure = $P_d = 0.6 \times V^2_z$

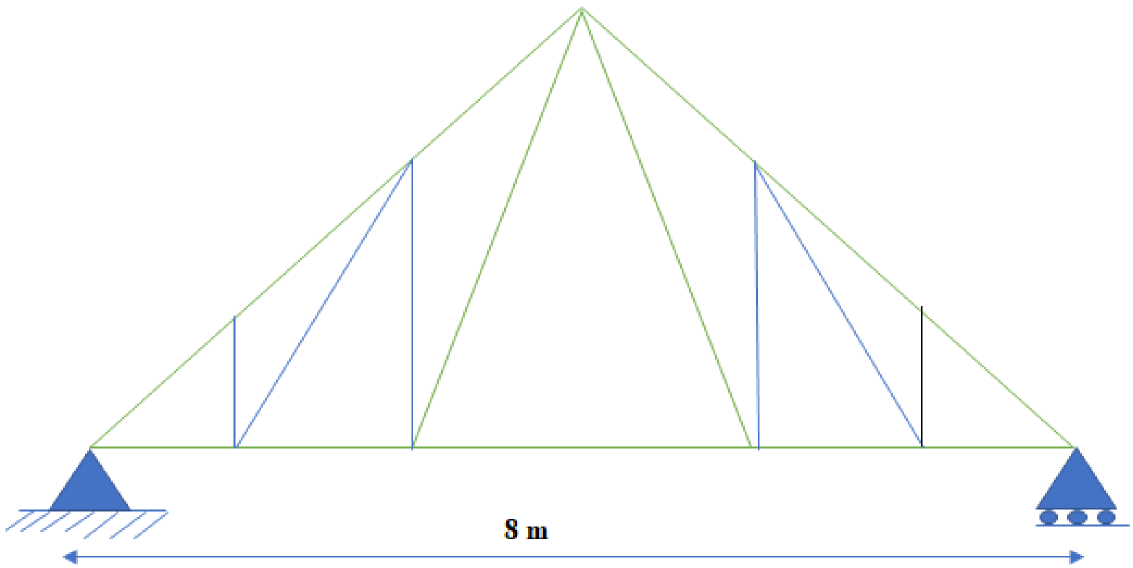
$= 0.6 \times 35.72^2 = 765.55$ N/m^2

3. Calculate Dead Load, Live Load, Wind load for following truss Aurangabad City

Span= 8 m

Spacing centre to centre = 4 m

Eave height = 6.20 m



Solution:

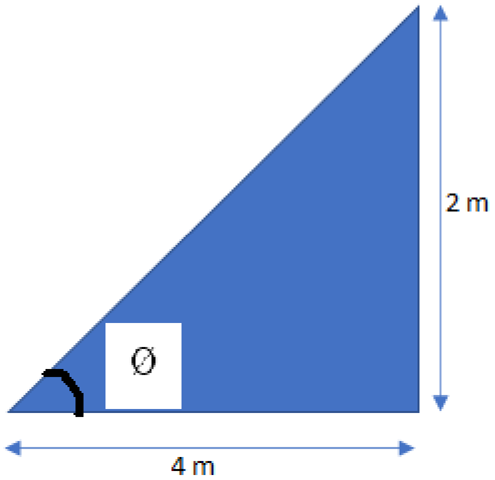
Assuming Pitch = (1/4)

Pitch = Rise/Span

¼ = Rise/Span

Rise= ¼ X Span

Rise= ¼ X 8= 2 m



$$\tan\theta = \frac{2}{4}$$

$$\theta = \tan^{-1}\frac{2}{4}$$

$$\theta = 26.56^0$$

Length of Principle Rafter $= \sqrt{2^2 + 4^2}$

$= 4.47m$

Panel point length = $\frac{4.47}{3} = 1.49m$

Dead Load

1. Self-weight of Covering Sheet (GI) = 120 N/m^2
2. Self-weight of Purlin =80 N/m^2
3. Self-weight of Bracing=15 N/m^2
4. Self-weight of Truss = $\left(\frac{Span}{3} + 5\right) x$ 10

$= \left(\frac{8}{3} + 5\right) x$ 10=76.67 N/m^2

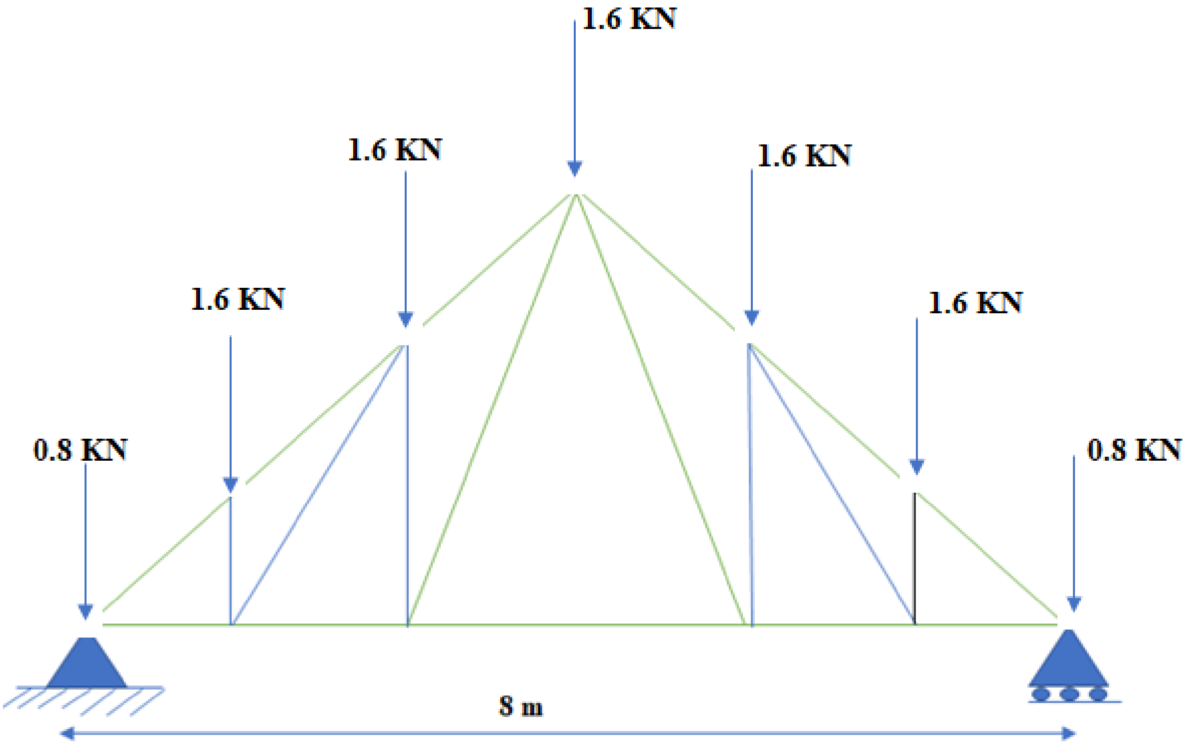
Total Dead Load= 120+80+15+76.67= 291.67 N/m^2

Total Dead Load $\cong$ 300 N/m^2

Total Dead Load = Span X Spacing X Intensity of Dead Load

Total Dead Load= 8 X 4 X 300= 9600 N= 9.6 KN

L



e

Panel = 9600/6= 1600 N= 1.6 KN

Load on End Panel= 1.6/2= 0.8 KN

Live Load

As slope of roof = ϴ= 26.56^0

Intensity of Live Load= 750-[20(ϴ-10)]

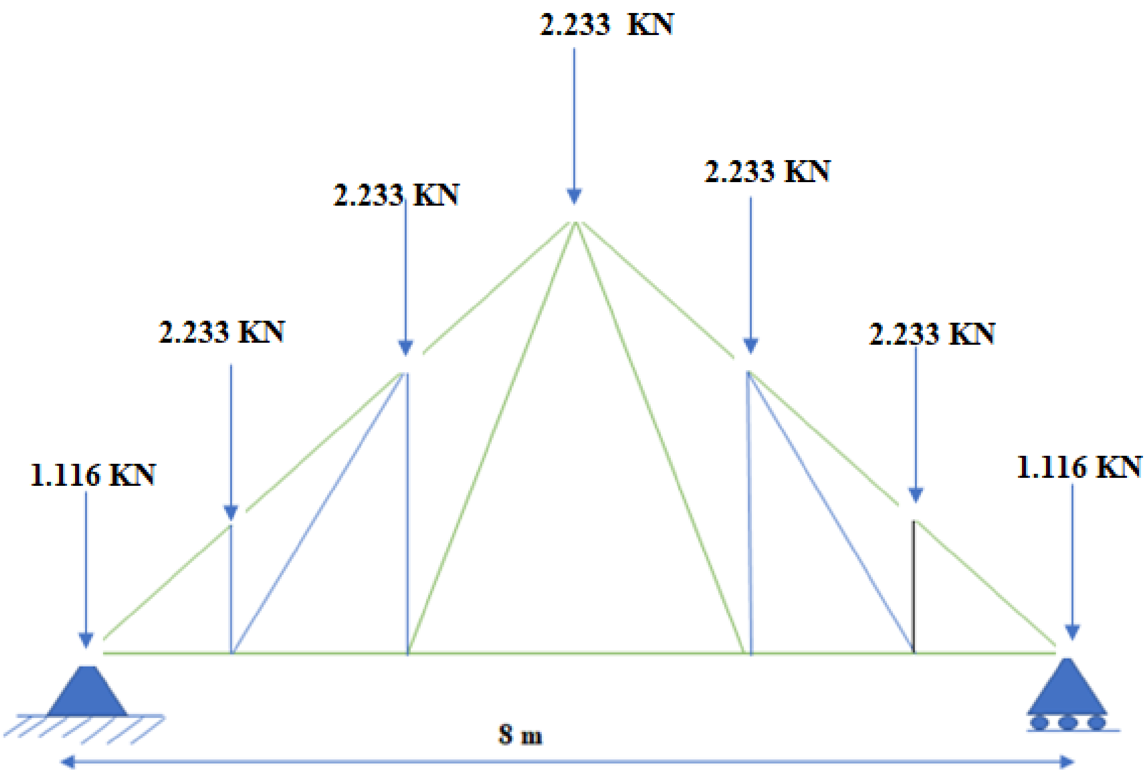
Intensity of Live Load=750-[20(26.56-10)]

Intensity of Live Load=418.80 N/m^2

Total Live Load = Span X Spacing X Intensity of Live Load

Total Live Load = 8 X 4 X 418.80= 13401.6 N=13.4016 KN

Load on Intermediate Panel = 13.4016/6= 2.233 KN



Load on End Panel= 2.233/2= 1.116 KN

Wind Load

Intensity of Wind Load = F

$$F = (C_{Pe} - C_{Pi})\, x \text{ A } x \text{ P}_d$$

Where

C_{pe}=External air pressure coefficient

C_{pi}= Internal air pressure coefficient

A = Surface area under consideration

P_d= Design wind pressure

1.Basic wind speed for Aurangabad=V_b=39m/s

2. The building must design for a 50 year

Risk Coefficient/ Probability Factor K_1=1

3.The size of building is less than 20 m

The building is classified as Class A and Category 4

Terrain, Height and Structure Size Factor=K_2=0.8

4.Topography Factor=K_3=1

5. Design Wind Speed = V_z =$K_1K_2K_3$ V_b

$= V_z = 1 \times 0.8 \times 1 \times 39$

$= 31.20$ m/s

6. Design Wind Pressure = $P_d = 0.6 \times V_z^2$

$= 0.6 \times 31.20^2 = 584.064$ N/m^2

Ratio $\frac{h}{w} = \frac{Eave\ Height}{\text{Span}} = \frac{6.2}{8} = 0.775$

$\frac{1}{2} < \frac{h}{w} \leq \frac{3}{2}$

As slope of roof = ϴ= 26.56^0

Wind normal to ridge

ϴ= 0^0

Roof Angle	EF	GH
20^0	-0.7	-0.5
26.56^0	? (-0.372)	? (-0.5)
30	-0.20	-0.5

$$C_{pe}\ for\ \text{EF (Windward)} = -0.7 + \left[\frac{(-0.2 - (-0.7))}{(30 - 20)}\ \text{X (26.56-20)} \right] = -0.372$$

$$C_{pe}\ for\ \text{GH (Leedward)} = -0.5$$

Wind parallel to ridge

ϴ= 90^0

Roof Angle	EG	FH
20^0	-0.8	-0.6
26.56^0	? (-0.8)	? (-0.73)
30	-0.8	-0.8

C_{pe} *for* EF (Windward) $= -0.8$

C_{pe} *for* GH (Leedward) $= -0.6 + \left[\frac{(-0.8-(-0.6))}{(30-20)} \text{ X } (26.56\text{-}20) \right] = -0.73$

Assume normal permeability

$C_{pi} = \pm 0.2$

Intensity of Wind Load = F

$F = \left(C_{Pe} - C_{Pi}\right) x \text{ A } x \text{ P}_{\text{d}}$

$F_1 = \left(-0.372 - 0.2\right) x \ 1 \ x \ 584.064 = -334.685 \text{ N/m}^2$

$F_2 = \left(-0.372 - (-0.2)\right) x \ 1 \ x \ 584.064 = -100.45 \text{ N/m}^2$

$F_3 = \left(-0.5 - 0.2\right) x \ 1 \ x \ 584.064 = -408.84 \text{ N/m}^2$

$F_4 = \left(-0.5 - (-0.2)\right) x \ 1 \ x \ 584.064 = -175.22 \text{ N/m}^2$

$F_5 = \left(-0.8 - 0.2\right) x \ 1 \ x \ 584.064 = -584.064 \text{ N/m}^2$

$F_6 = \left(-0.8 - (-0.2)\right) x \ 1 \ x \ 584.064 = -350.44 \text{ N/m}^2$

$F_7 = \left(-0.73 - 0.2\right) x \ 1 \ x \ 584.064 = -543.18 \text{ N/m}^2$

$F_8 = \left(-0.73 - (-0.2)\right) x \ 1 \ x \ 584.064 = -309.55 \text{ N/m}^2$

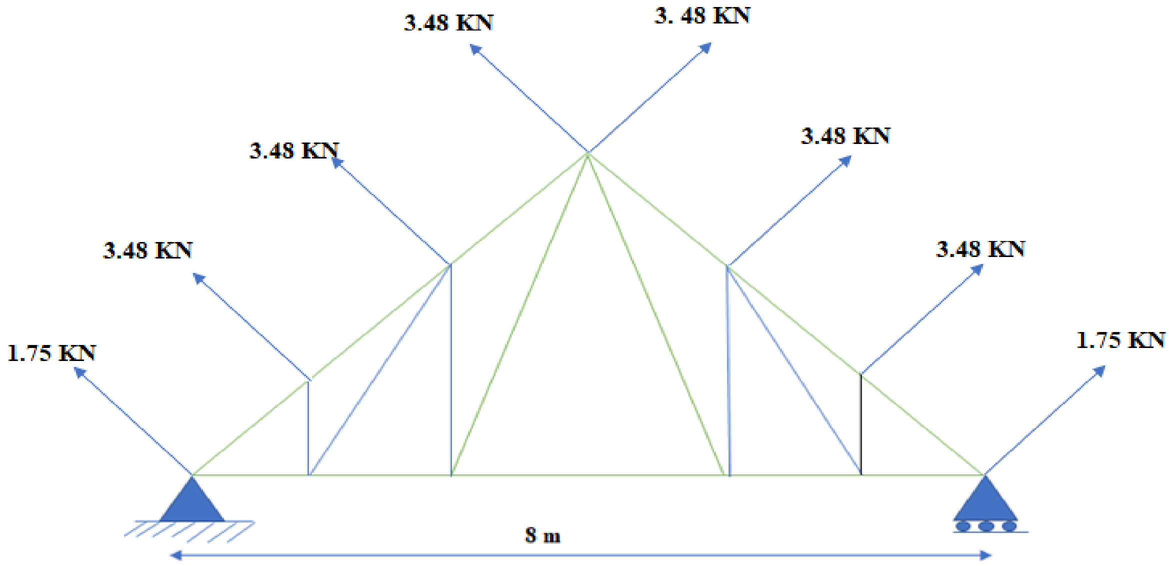
Maximum wind pressure intensity =-584.064 N/m^2

Total Wind Load = Length of Principle rafter X Spacing X Intensity of wind load

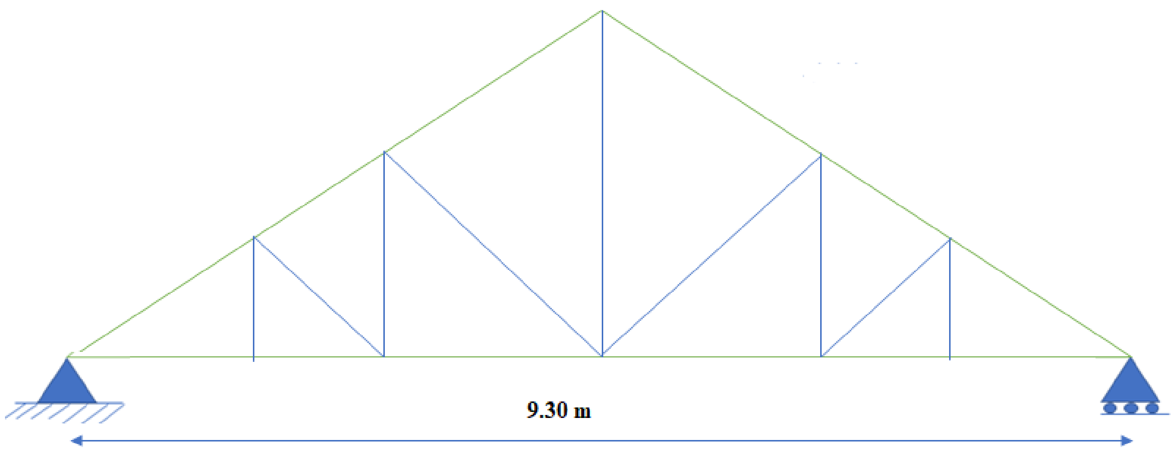
Total Wind Load = -4.47 X 4 X 584.064 = -10.44 x 10^3 N =-10.44 KN

Load on each intermediate panel = -10.44/3= -3.48 KN

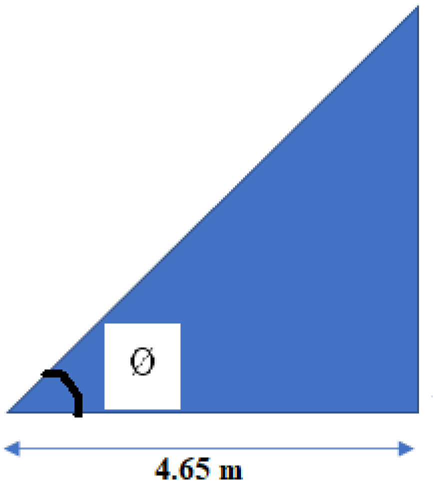
Load on each end panel = -3.48/2=-1.75 KN



4.Design a roof truss of a span 9.30 m at a spacing of 4 m for an industrial shed. The height of eave is 6.5 m. It is situated near Delhi. The roof truss is supported on 40 cm thick brick masonary. The roof angle is 30^0.



Solution:
Span= 9.30 m
Spacing= 4 m
Eave Height=6.50 m
Θ= 30^0 (Roof Angle)



$$\cos 60 = \frac{4.65}{l}$$

$$l = 5.36m$$

Panel point length = $\frac{5.36}{3} = 1.78m$

Dead Load

1.Self-weight of Covering Sheet (AC Sheet) = 125 N/m^2

2.Self-weight of Purlin =80 N/m^2

3.Self-weight of Bracing=15 N/m^2

4.Self-weight of Truss = $\left(\frac{Span}{3}+5\right)x\ 10$

$=\left(\frac{9.3}{3}+5\right)x\ 10$=81 N/m^2

Total intensity of Dead Load= 125+80+15+81= 301 N/m^2

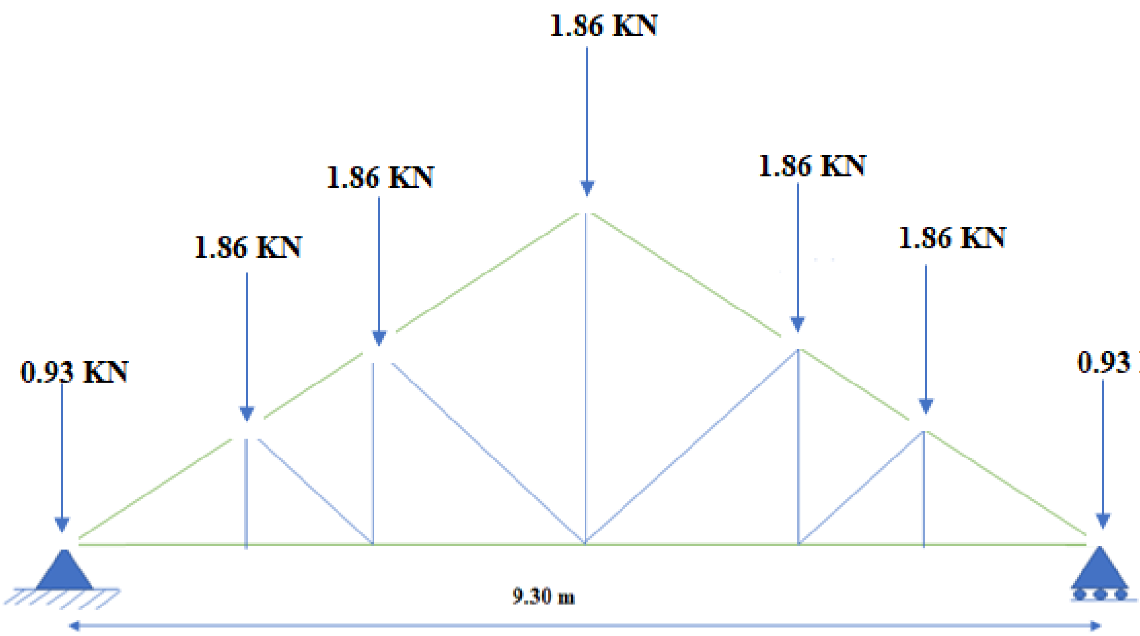
Total Dead Load $\cong$ 301 N/m^2

Total Dead Load = Span X Spacing X Intensity of Dead Load

Total Dead Load= 9.3 X 4 X 301= 11197. 20 N= 11.20 KN

Load on Intermediate Panel = 11.20/6=1.86 KN

Load on End Panel= 1.86/2= 0.92 KN



Live Load

As slope of roof = ϴ= 30^0

Intensity of Live Load= 750-[20(ϴ-10)]

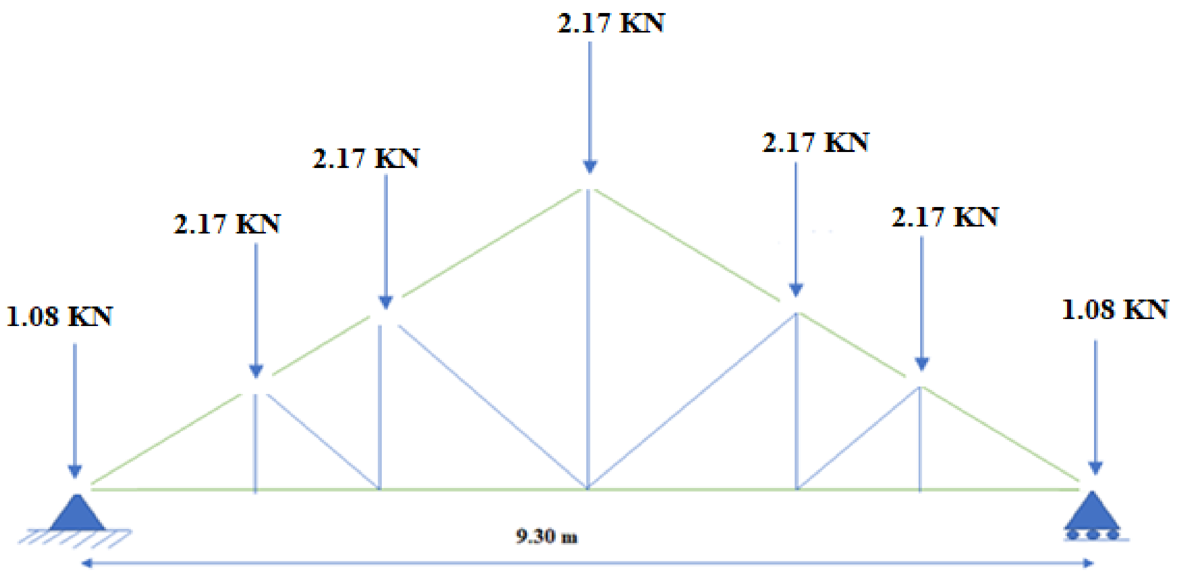
Intensity of Live Load=750-[20(30-10)]

Intensity of Live Load=350 N/m^2

Total Live Load = Span X Spacing X Intensity of Live Load

Total Live Load = 9.3 X 4 X 350= 13020 N=13.020 KN

Load on Intermediate Panel = 13.020/6= 2.17 KN



Load on End Panel= 2.17/2= 1.08 KN

Wind Load

Intensity of Wind Load = F

$F = (C_{Pe} - C_{Pi})\, x\ A\ x\ P_d$

Where

C_{pe}=External air pressure coefficient

C_{pi}= Internal air pressure coefficient

A = Surface area under consideration

P_d= Design wind pressure

1.Basic wind speed for Delhi=V_b=47m/s

2. The building must design for a 50 year

Risk Coefficient/ Probability Factor K_1=1

3.The size of building is less than 20 m

The building is classified as Class A and Category 4

Terrain, Height and Structure Size Factor=K_2=0.8

4.Topography Factor=K_3=1

5. Design Wind Speed = $V_z = K_1 K_2 K_3\ V_b$

$= V_z = 1 \times 0.8 \times 1 \times 47$

$= 37.60$ m/s

6. Design Wind Pressure = $P_d = 0.6 \times V^2_z$

$= 0.6 \times 37.60^2 = 848.256\ N/m^2$

Ratio $\frac{h}{w} = \frac{Eave\ Height}{Span} = \frac{6.5}{9.3} = 0.70$

$\frac{1}{2} < \frac{h}{w} \leq \frac{3}{2}$

As slope of roof = ϴ= 30^0

Wind normal to ridge

ϴ= 0^0

Roof Angle	EF	GH
30^0	-0.2	-0.5

C_{pe} *for* EF (Windward) = –0.2

C_{pe} *for* GH (Leedward) = –0.5

Wind parallel to ridge

ϴ= 90^0

Roof Angle	EG	FH
30^0	-0.8	-0.8

C_{pe} *for* EF (Windward) = –0.8

C_{pe} *for* GH (Leedward) = –0.8

Assume normal permeability

$C_{pi} = \pm 0.2$

Intensity of Wind Load = F

$F = (C_{Pe} - C_{Pi})\ x\ \text{A}\ x\ \text{P}_\text{d}$

$F_1 = (-0.2 - 0.2)\ x\ 1\ x\ 848.256$=-339.30 N/m^2

$F_2 = (-0.2 - (-0.2))\ x\ 1\ x\ 848.256$=0 N/m^2

$F_3 = (-0.5 - 0.2)\ x\ 1\ x\ 848.256$=-593.78 N/m^2

$F_4 = (-0.5 - (-0.2))\ x\ 1\ x\ 848.256$=-254.48 N/m^2

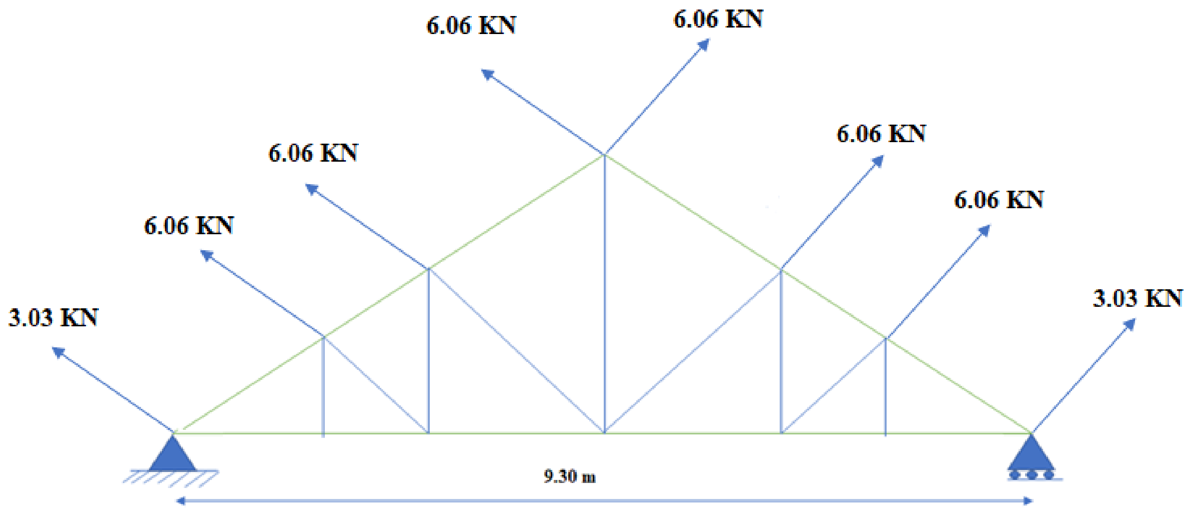
$F_5 = (-0.8 - 0.2)\ x\ 1\ x\ 848.256$=-848.256 N/m^2

$F_6 = (-0.8-(-0.2))\, x\ 1\ x\ 848.256$=-508.95 N/m^2

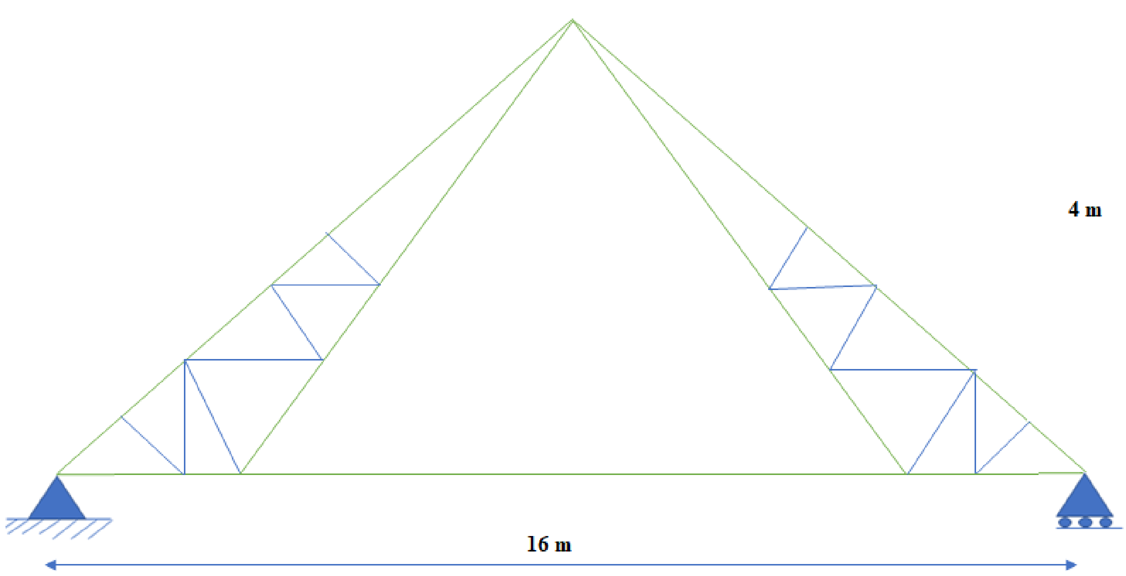
$F_7 = (-0.73-0.2)\, x\ 1\ x\ 848.256$=-848.256 N/m^2

$F_8 = (-0.73-(-0.2))\, x\ 1\ x\ 848.256$=-508.95 N/m^2

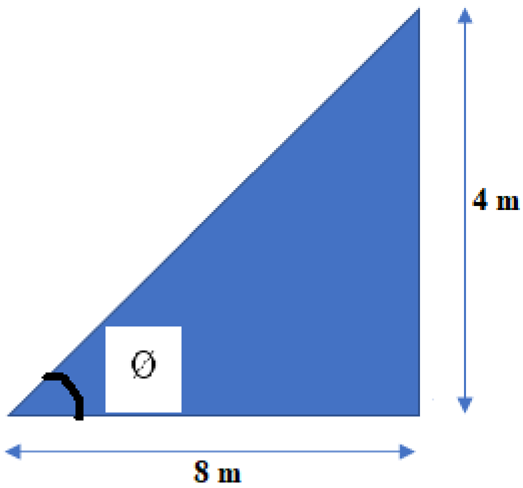
Maximum wind pressure intensity =-848.256 N/m^2
Total Wind Load = Length of Principle rafter X Spacing X Intensity of wind load
Total Wind Load = -5.36 X 4 X 848.256 = -18.18 x 10^3 N =-18.18 KN
Load on each intermediate panel = -18.18/3= -6.06 KN
Load on each end panel = -6.06/2=-3.03 KN



5. Design a roof truss to the following particulars
1.Span of truss= 16 m
2. Rise of truss= 4 m
3. Height of eaves= 8m
4. Spacing of truss=4
Roofing shall be of GI Sheet
The truss is supported on 400 mm thick brickwall. The building is located in Pandicherry.
Take Risk coefficient= K_1= 1
Terrain Factor = K_2= 0.82
Topography Factor= K_3= 1
Also design purlin



Solution:
Span= 16 m
Rise= 4 m
Spacing= 4 m
Eave Height=6.50 m



$$\tan\theta = \frac{4}{8}$$

$$\theta = \tan^{-1}\frac{4}{8}$$

$$\theta = 26.56^0$$

Length of Principle Rafter $= \sqrt{4^2 + 8^2}$
$= 8.94m$

Panel point length = $\frac{8.94}{5} = 1.78m$

Dead Load

Self-weight of Covering Sheet (GI Sheet) = 85 N/m^2
Self-weight of Purlin =80 N/m^2
Self-weight of Bracing=15 N/m^2

Self-weight of Truss = $\left(\frac{Span}{3}+5\right)x\ 10$

$=\left(\frac{16}{3}+5\right)x\ 10$=103.33 N/$m^2$

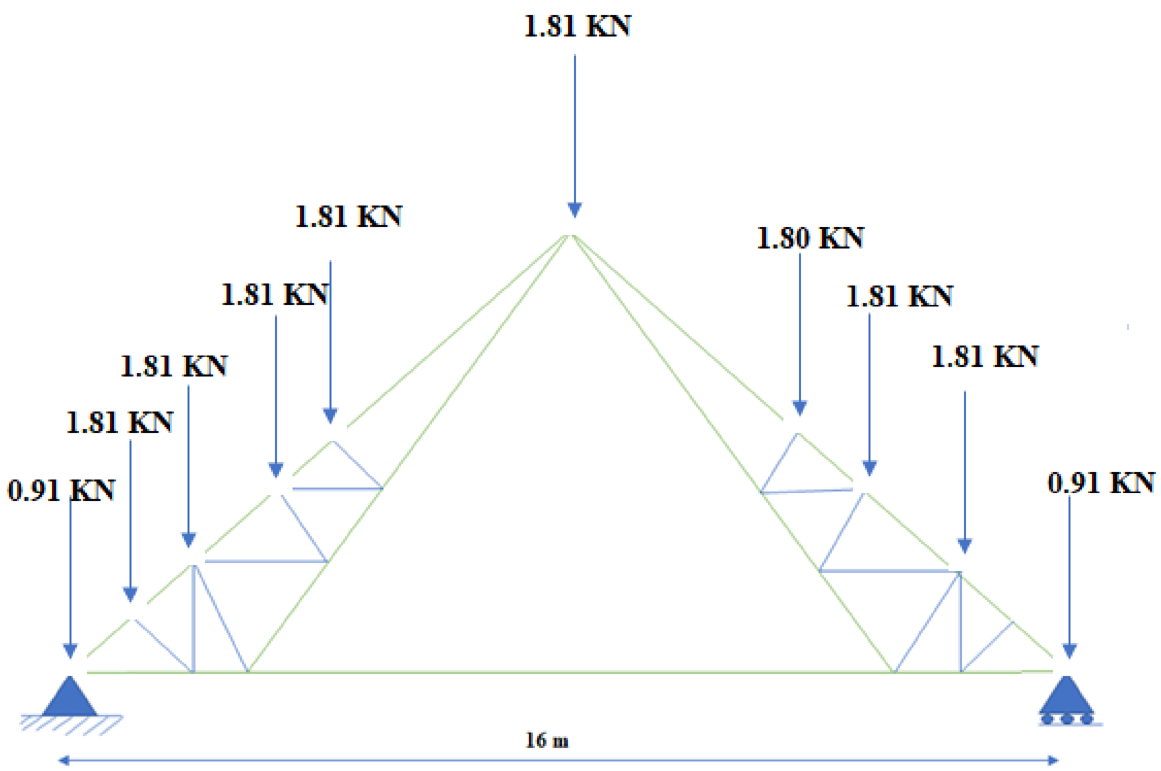
Total Dead Load= 85+80+15+103.33= 283.33 N/m^2

Total Dead Load = Span X Spacing X Intensity of Dead Load

Total Dead Load= 16 X 4 X 283.33= 18133.12 N= 18.13 KN

Load on Intermediate Panel = 18.13/10=1.81 KN

Load on End Panel= 1.81/2= 0.91 KN



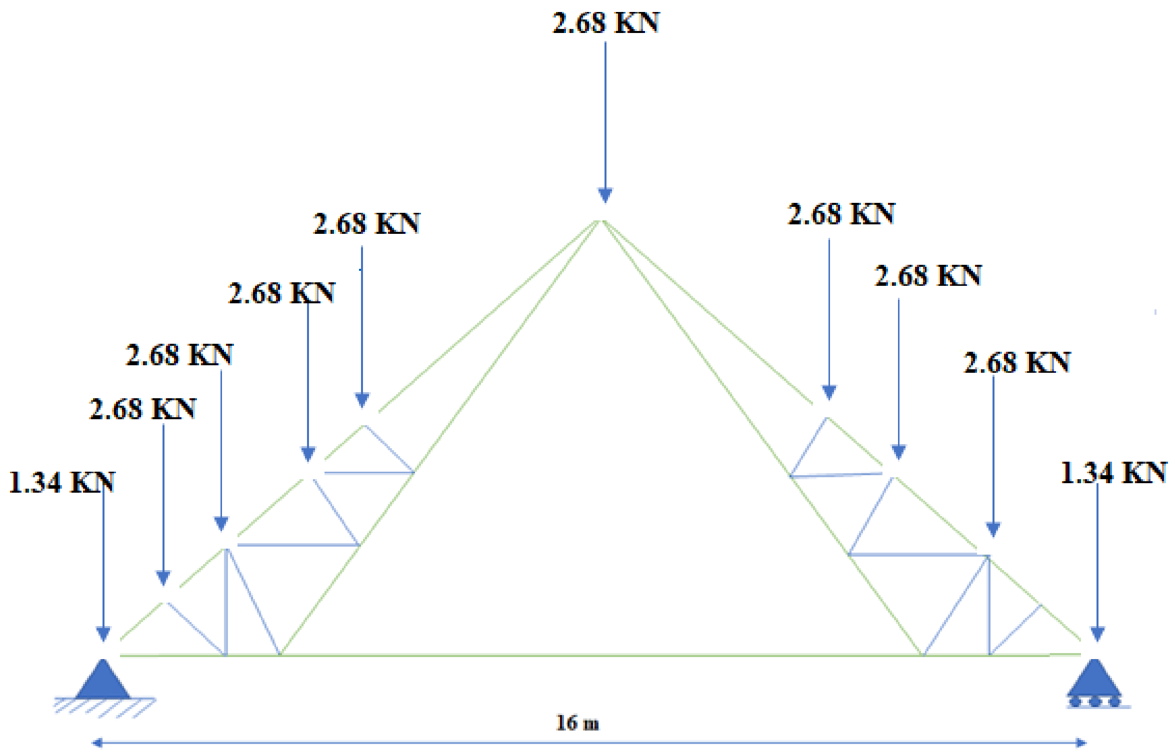
Live Load

As slope of roof = ϴ= 26.56^0
Intensity of Live Load= 750-[20(ϴ-10)]
Intensity of Live Load=750-[20(26.56-10)]

Intensity of Live Load=418.80 N/m^2
Total Live Load = Span X Spacing X Intensity of Live Load
Total Live Load = 16 X 4 X 418.80= 26800 N=26.80 KN

Load on Intermediate Panel = 26.80/10= 2.68KN

Load on End Panel= 2.68/2= 1.34 KN



Wind Load

Intensity of Wind Load = F

$F = (C_{Pe} - C_{Pi})\, x \text{ A } x \text{ P}_d$

Where

C_{pe}=External air pressure coefficient

C_{pi}= Internal air pressure coefficient

A = Surface area under consideration

P_d= Design wind pressure

1.Basic wind speed for Pandicherry=V_b=50m/s

2. Risk Coefficient/ Probability Factor K_1=1

3.Terrain, Height and Structure Size Factor=K_2=0.82

4.Topography Factor=K_3=1

5. Design Wind Speed = $V_z = K_1 K_2 K_3 V_b$

$$= V_z = 1 \text{ x } 0.82 \text{ x } 1 \text{ x } 50$$

$$= 41 \text{ m/s}$$

6. Design Wind Pressure = $P_d = 0.6 \text{ x } V^2_z$

$$= 0.6 \text{ x } 41^2 = 1008.60 \text{ N/m}^2$$

Ratio $\frac{h}{w} = \frac{Eave\ Height}{\text{Span}} = \frac{8}{16} = 0.5$

$\frac{h}{w} \leq \frac{1}{2}$

As slope of roof = ϴ= 26.56^0

Wind normal to ridge

ϴ= 0^0

Roof Angle	EF	GH
20^0	-0.4	-0.4
26.56^0	? (-0.14)	? (-0.4)
30^0	0	-0.4

$$C_{pe}\ for\ \text{EF (Windward)} = -0.4 + \left[\frac{(0-(-0.4))}{(30-20)}\ \text{X (26.56-20)}\right] = -0.1376 \cong -0.14$$

$$C_{pe}\ for\ \text{GH (Leedward)} = -0.4$$

Wind parallel to ridge

$\theta = 90^0$

Roof Angle	EG	FH
20^0	-0.7	-0.6
26.56^0	? (-0.7)	? (-0.6)
30^0	-0.7	-0.6

$$C_{pe}\ for\ \text{EF (Windward)} = -0.7$$

$$C_{pe}\ for\ \text{GH (Leedward)} = -0.6$$

Assume normal permeability

$C_{pi} = \pm 0.2$

Intensity of Wind Load = F

$F = \left(C_{Pe} - C_{Pi}\right) x\ \text{A}\ x\ \text{P}_d$

$F_1 = \left(-0.14 - 0.2\right) x\ 1\ x\ 1008.6\text{=-342.924 N/m}^2$

$F_2 = \left(-0.14 - (-0.2)\right) x\ 1\ x\ 1008.6\text{=60.516 N/m}^2$

$F_3 = \left(-0.4 - 0.2\right) x\ 1\ x\ 1008.6\text{=-605.16 N/m}^2$

$F_4 = \left(-0.4 - (-0.2)\right) x\ 1\ x\ 1008.6\text{=-201.72 N/m}^2$

$F_5 = \left(-0.7 - 0.2\right) x\ 1\ x\ 1008.6\text{=-907.74 N/m}^2$

$F_6 = \left(-0.7 - (-0.2)\right) x\ 1\ x 1008.6\text{=-504.30 N/m}^2$

$F_7 = \left(-0.6 - 0.2\right) x\ 1\ x\ 1008.6\text{=-806.88 N/m}^2$

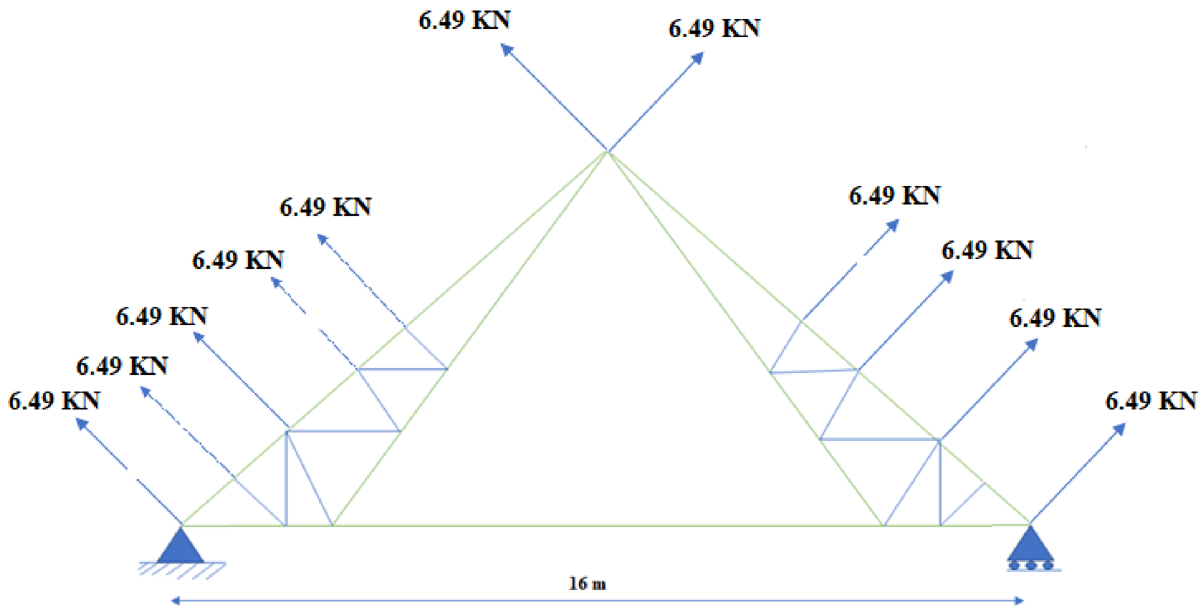
$F_8 = \left(-0.6 - (-0.2)\right) x\ 1\ x\ 1008.6\text{=-403.44 N/m}^2$

Maximum wind pressure intensity =-907.74 N/m^2

Total Wind Load = Length of Principle rafter X Spacing X Intensity of wind load

Total Wind Load = -8.94 X 4 X 907.74 = -32.460 x 10^3 N =-32.46 KN

Load on each intermediate panel = -32.46/5= -6.49 KN
Load on each end panel = -6.49/2=-3.25 KN



1. Dead load of purlin
   a) Self weight of purlin= 80 N/m^2
   b) Self weight of GI Sheet= 85N/m^2
   Total Load = 80+85= 165 N/m^2

Dead load on purlin= Span X Spacing X Intensity of Live Load
=4 x 16 x 165 = 10560 N

Dead load per purlin= 10560/10=1056 N

2. Live load per purlin=26803/10=2680 N
3. Wind load per purlin=-907.74 x 4 x 1.78=-6463.10 N
   Load Combination
   1. DL+LL= 1056+2680=3736 N
   2. (DL+WL)/1.33= (1056-6463.10)/1.33=-4065.48 N
   Maximum Load= -4065.48 N

Maximum Bending Moment = M= (WL/10) = (4065.48 X 4)/10= 1.626195 KN m
M= 1626.195 X 10^3 Nmm

Section Modulus

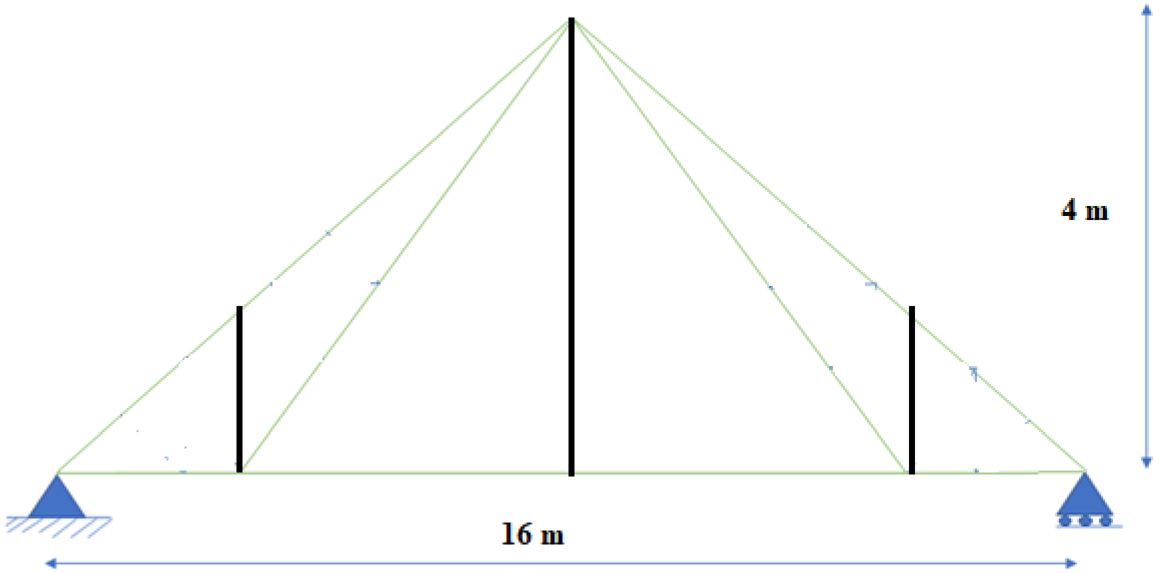
$$Z_{req} = \frac{M}{\sigma_{bc}}$$

$$\sigma_{bc} = \sigma_{bt} = 0.66 \text{ x } F_y = 0.66x250 = 165N/mm^2$$

$$Z_{req} = \frac{1626.20x10^3}{165} = 9.856x10^3 \text{ mm}^3$$

Approximate depth of angle section for purlin= L/45=4000/45=88.89 $\cong$ 90 mm
Approximate width of angle section for purlin= L/60=4000/60=66.67 $\cong$ 70 mm
Provide ISA 100 X 75 X 6
Zxx= 14.40 X 10^3 mm^3 > 9.859 X 10^3 mm^3

6. Design a roof truss to the following data
   Span of truss= 16 m
   Rise of truss= 4 m
   Spacing of truss= 4
   Roofing shall be of GI Sheet
   Live load = 5000 N/m^2
   Wind pressure acting normally on the windward side =1200 N/m^2



Length of Principle Rafter $= \sqrt{4^2 + 8^2}$
$= 8.94m$

Dead Load

Self-weight of Covering Sheet (GI Sheet) = 85 N/m^2
Self-weight of Purlin =80 N/m^2
Self-weight of Bracing=15 N/m^2
Self-weight of Truss = $\left(\frac{Span}{3} + 5\right) x\ 10$

$$=\left(\frac{16}{3}+5\right)x\ 10=103.33\ N/m^2$$

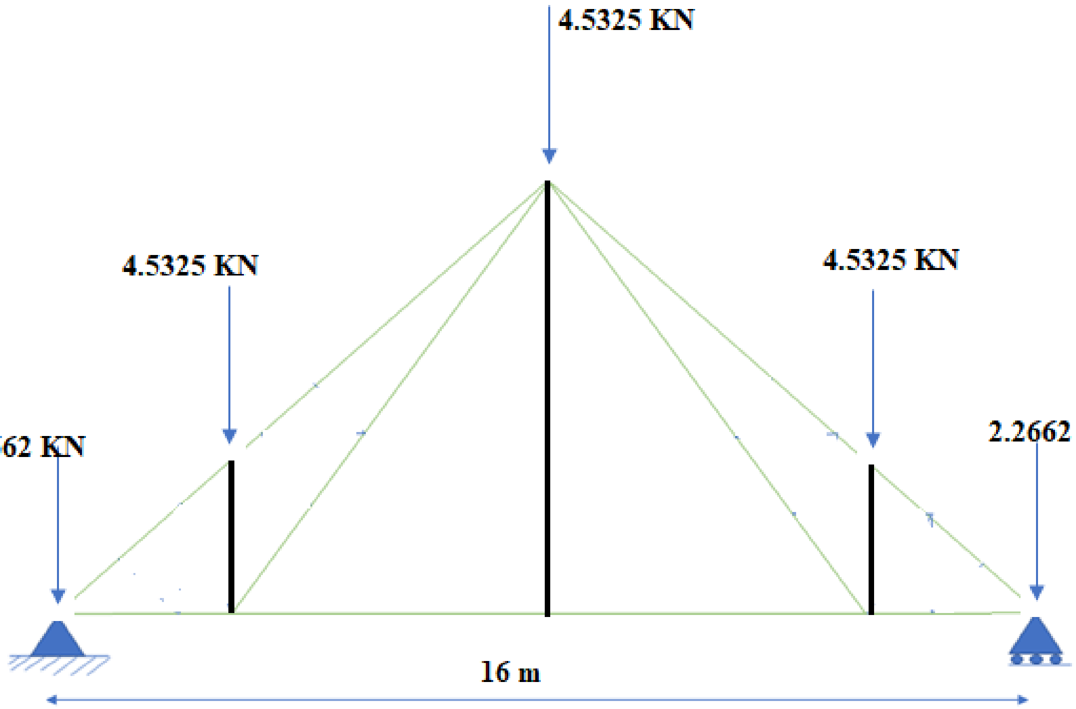
Total Dead Load= 85+80+15+103.33= 283.33 N/m^2

Total Dead Load = Span X Spacing X Intensity of Dead Load

Total Dead Load= 16 X 4 X 283.33= 18133.12 N= 18.13 KN

Load on Intermediate Panel = 18.13/4=4.5325 KN

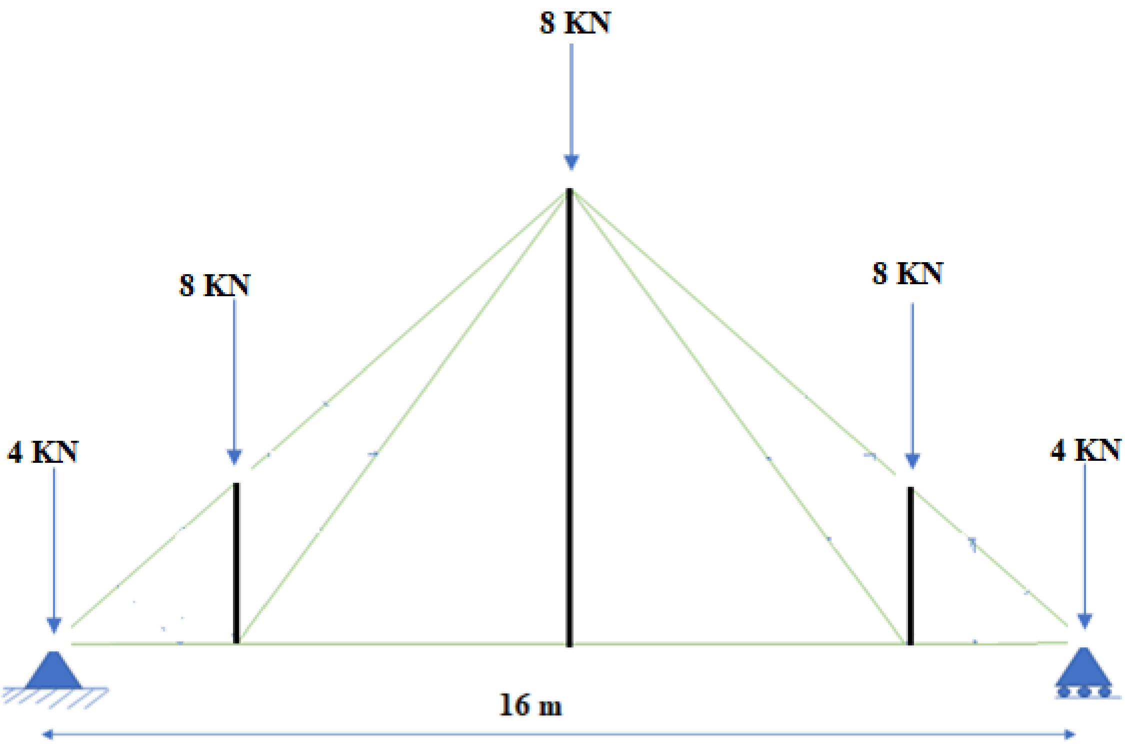
Load on End Panel= 4.5325/2= 2.2662 KN



Live Load

Intensity of Live Load=500 N/m^2
Total Live Load = Span X Spacing X Intensity of Live Load
Total Live Load = 16 X 4 X 500= 32000 N=32 KN

Load on Intermediate Panel = 32/4= 8KN



Load on End Panel= 8/2= 4 KN

Wind Load

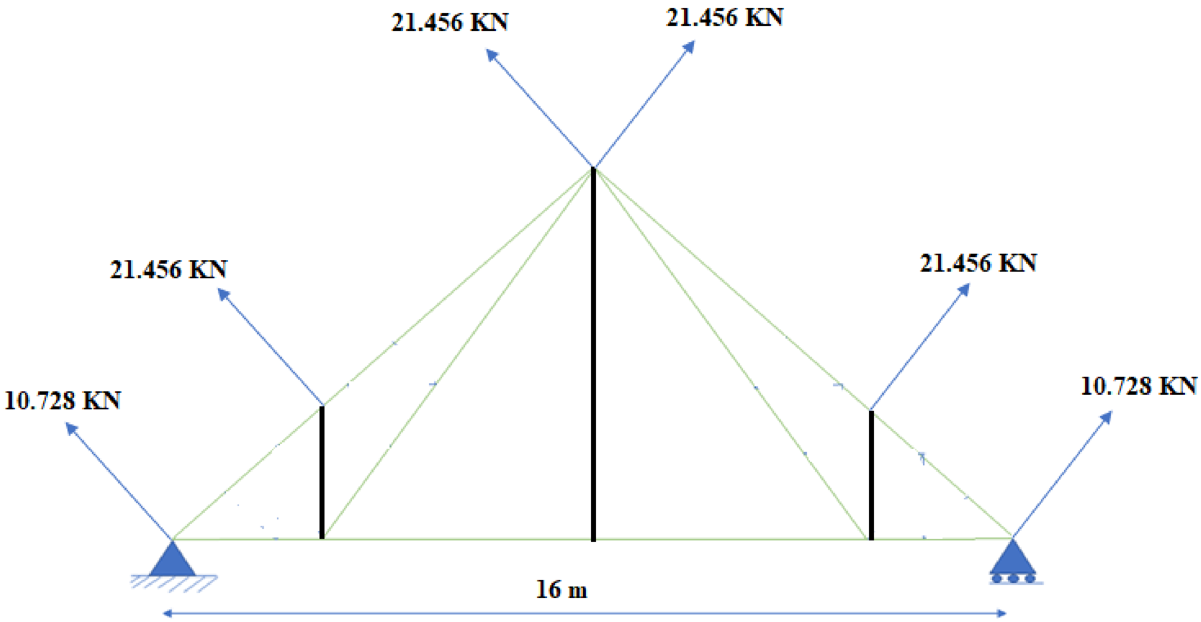
Maximum wind pressure intensity =-1200 N/m^2

Total Wind Load = Length of Principle rafter X Spacing X Intensity of wind load

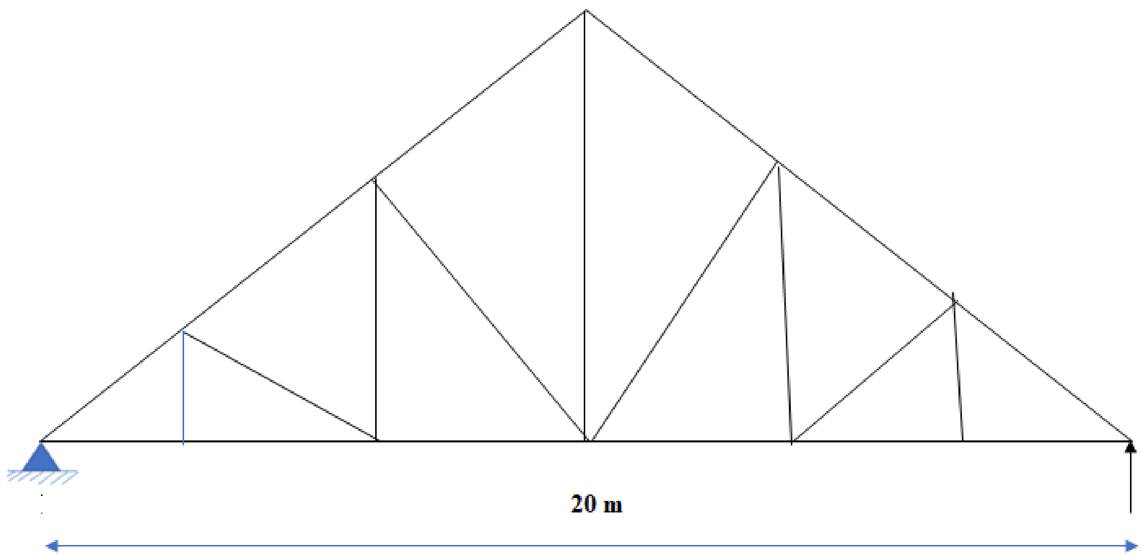
Total Wind Load = -8.94 X 4 X 1200 = -42912 x 10^3 N =-42.912KN

Load on each intermediate panel = -42.912/2= -21.456 KN

Load on each end panel = -21.456/2=-10.728KN



7. Determine the dead load, live load and wind load per nodal point for a factory building , roof truss for a span of 20 m and pitch is 1/5, the height of truss at eave



is 4.5 m, Spacing 4 m, the factory building which is 36 long is located at Delhi

Dead Load:

1. Self wt of Covering Sheet (GI) = 85 N/m^2
2. Self wt of Purlin = 80 N/m^2
3. Self wt of Bracing = 15 N/m^2
4. Self wt of Truss ((Span/ 3)+5) X 10 = 116.67 N/m^2

Total Intensity of Dead Load = 296.67 N/ m^2

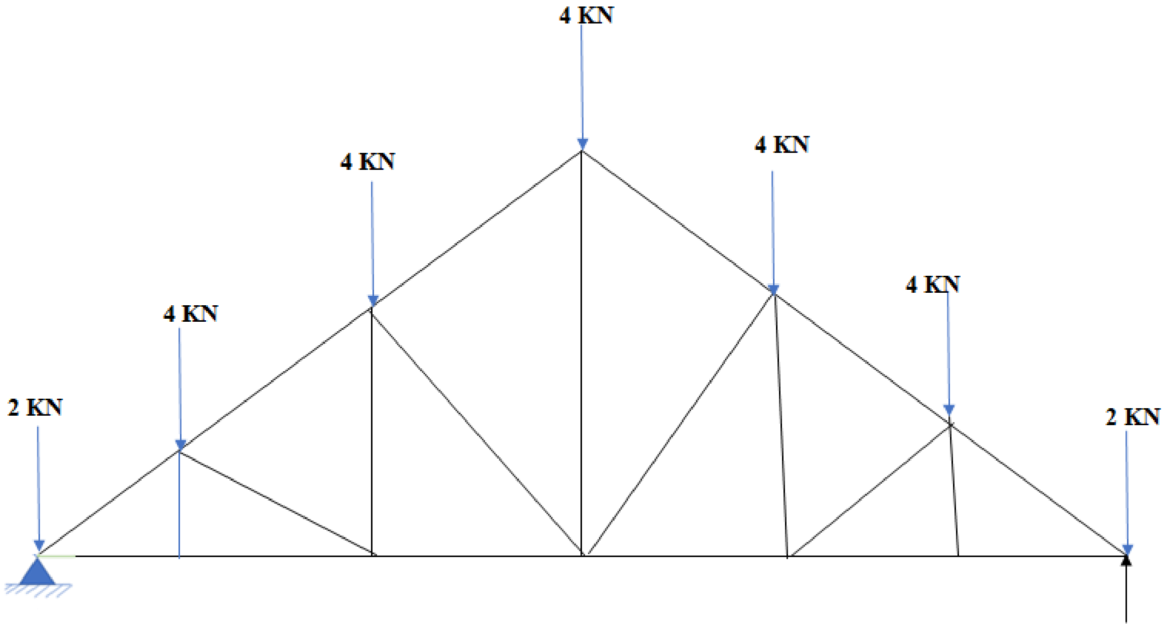
Total Intensity of Dead Load = 300 N/ m^2

Total Dead Load = Span X Spacing X Intensity of Dead Load

= 20 X 4X 300 = 24000 N= 24 KN

Load on Intermediate Panel =24/6 = 4 KN

Load on End Panel =4/2 = 2 KN



DEAD LOAD

Live Load:

Roof Angle = Θ= 21.80^0

Intensity of Live Load = 750-[20(Θ-10)]

Intensity of Live Load = 750-[20(21.80-10)]

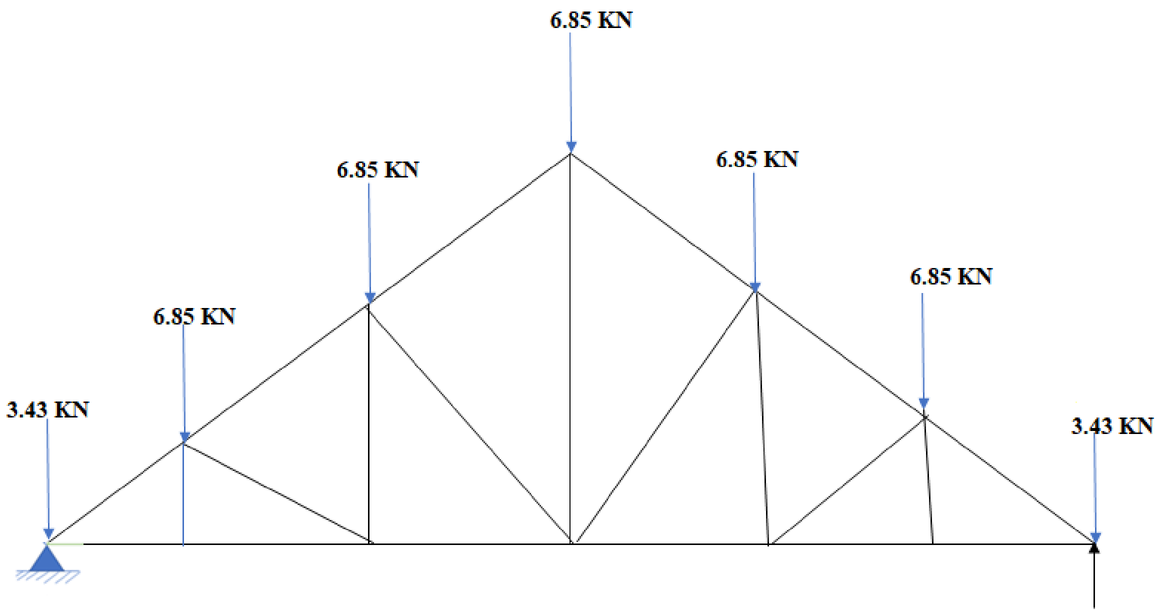
= 514 N/m^2

Total Live Load = Span X Spacing X Intensity of Live Load

= 20 X 4X 514 = 41120 N= 41.120 KN

Load on Intermediate Panel =41.12/6 = 6.85 KN

Load on End Panel =6.85/2 = 3.43 KN



LIVE LOAD

WIND LOAD

Assume life of Structures is 50 Years, Risk Coefficient/ Probability Factor K_1=1

Width of Truss = 36 m , Lies between 20m to 50 m , Class B

Category = 4 for Developed Area

Terrain, Height and Structure Size Factor=K_2=0.76

Topography Factor=K_3=1

Basic Wind Speed = V_b = **47 m/s**

Design Wind Speed = $V_z = K_1K_2K_3\, V_b$

V_z **=1 x0.76x1x 47=35.72 m/s**

Design Wind Pressure = $P_d = 0.6 \times V_z^2$

$$=0.6x\ 35.72^2 = 765.55\ N/m^2$$

Ratio $\frac{h}{w} = \frac{Eave\ Height}{Span} = \frac{4.5}{20} = 0.225$

$\frac{h}{w} \leq \frac{1}{2}$, $0.225 \leq 0.5$ ok

Ɵ=21.80^0

To find external pressure Coefficient C_{pe}

Wind Normal to ridge, Ɵ= 0^0

Roof Angle	EF	GH
20	-0.4	-0.4
21.80	?(-0.328)	?(-0.4)
30	0	-0.4

Wind Normal to ridge, Ɵ= 90^0

Roof Angle	EG	FH
20	-0.7	-0.6
21.80	?(-0.7)	?(-0.6)
30	-0.7	-0.6

To find Internal pressure Coefficient C_{pi}

Upto 5% of wall area Internal pressure Coefficient =C_{pi}== ± 0.2

Intensity of Wind Load

F= (C_{pe}- C_{pi}) x A X P_d

A= 1 (Always)

F_1= (-0.328- (+0.2)) x 1 X 765.55 = -404.21 N/m^2

F_2= (-0.4- (+0.2)) x 1 X 765.55 = -459.33 N/m^2

F_3= (-0.7- (+0.2)) x 1 X 765.55 = -688.995 N/m^2

F_4= (-0.6- (+0.2)) x 1 X 765.55 = -612.44 N/m^2

F_5= (-0.328- (-0.2)) x 1 X 765.55 = -97.99 N/m^2

F_6= (-0.4- (-0.2)) x 1 X 765.55 = -153.11 N/m^2

F_7= (-0.7- (-0.2)) x 1 X 765.55 = -382.775 N/m^2

F_8= (-0.6- (-0.2)) x 1 X 765.55 = -306.22 N/m^2

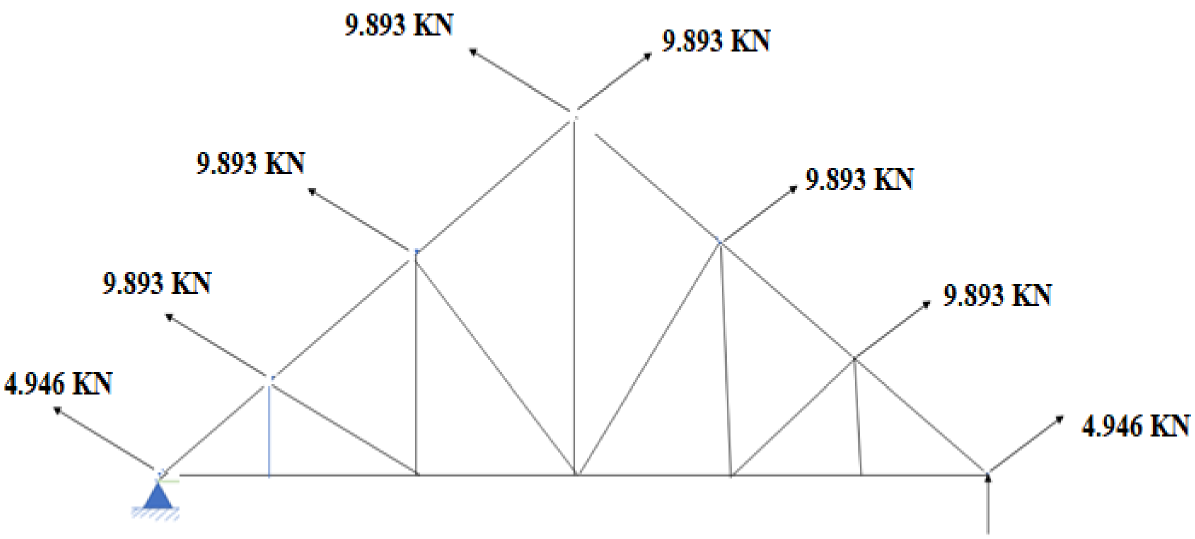
Take maximum value F= 688.995 N/m^2

Total Wind Load = Length of Principle rafter X Spacing X Intensity of wind load

Total Wind Load = 10.77 X 4 X 688. 995 = 29.68 X 1000 N =29.68 KN

Load on each intermediate panel = 29.68/3= 9.893 KN

Load on each end panel = 9.893 /2=4.946 KN



Simple and built up section

A column made by two channels section or two I sections is called built up column.

1. A column of effective length 5.5 m has to carry axial load of 1100 KN , consisting of two channels placed back to back at a suitable distance. Design a column section. Fy= 250 N/mm^2

Solution: Given Data

P= 1100 KN = 1100 x 10^3 N
Effective length = L_{eff}=5.5 m=5500 mm
Assuming λ = 80 for channel section
σac = 101 N/mm^2 (From IS CODE) Page No: 39 Table No:5.1
Fy = 250 N/mm^2

1. Find required area

$$A = \frac{P}{\sigma_{ac}} = \frac{1100X10^3}{101} = 10891.08\ mm^2$$

2. Select Suitable section from steel table (P. No 98)
   Try 2 ISMC 350 @ 84.2 kg/m
   A=10732 mm^2
   rxx=136.6 mm
   To carry maximum load
   $rxx \cong ryy$
   ryy= 137.4 mm (Back to back spacing 220 mm)
   rmin= 136.6 mm
3. To find slenderness ratio

$$\lambda = \frac{l_{eff}}{r_{\min}} = \frac{5500}{136.6} = 40.26$$

4. From IS 800-1984, Page No: 39 Table No:5.1, Fy= 250 N/mm^2

λ	σac
40	139
40.26	?
50	132

By interpolation

$$\sigma ac = 139 + \left[\frac{(132-139)}{(50-40)} \text{ X } (40.26\text{-}40)\right] = 138.81 \text{ N/mm}^2$$

5. Load carrying capacity

$$P_c = \text{A x } \sigma_{ac}$$

P_c =10732 x 138.8= 1489.60x 10^3 N
P_c = 1489.60 KN > 1100 KN , SAFE (OK)

2. Design a built up column to carry axial load of 1200 KN , effective length of column is 3.2 m. Use two channels placed back to back. $F_y = 250\ N/mm^2$

Solution: Given Data

P= 1200 KN = 1200 x 10^3 N
Effective length = L_{eff}=3.2 m=3200 mm
Assuming λ = 80 for channel section
σac = 101 N/mm^2 (From IS CODE) Page No: 39 Table No:5.1
Fy = 250 N/mm^2

1..Find required area

$$A = \frac{P}{\sigma_{ac}} = \frac{1200X10^3}{101} = 11881.18\,m$$

2. Select Suitable section from steel table (P. No 98)
   Try 2 ISMC 350 @ 84.2 kg/m
   A=10732 mm^2
   rxx=136.6 mm
   To carry maximum load
   $rxx \cong ryy$
   ryy= 137.4 mm (Back to back spacing 220 mm)
   rmin= 136.6 mm
3. To find slenderness ratio

$$\lambda = \frac{l_{eff}}{r_{\min}} = \frac{3200}{136.6} = 23.42$$

4. From IS 800-1984, Page No: 39 Table No:5.1, Fy= 250 N/mm^2

λ	σac
20	148
23.42	?
30	145

By interpolation

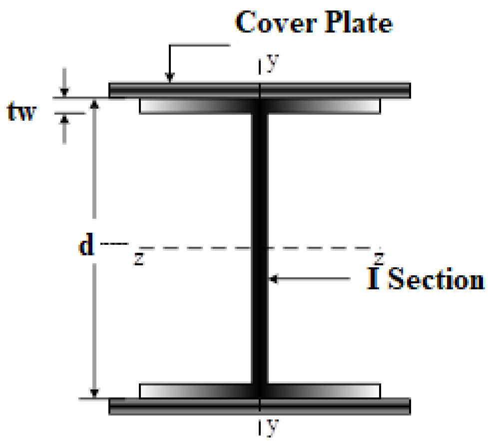
$$\sigma ac\ = 148 + \left[\frac{(145-148)}{(30-20)}\ X\ (23.42\text{-}20)\right] = 146.97\ N/mm^2$$

5. Load carrying capacity

$$P_c = \text{ A x } \sigma_{ac}$$

P_c =10732 x 146.97= 1577.28 x 10^3 N
P_c = 1577.28 KN > 1200 KN , SAFE (OK)

I Section with cover plate



Cross section of column

3. A column 5.6 m long is to support a load of 2500 KN, the ends of column are effectively held in position and direction. Design a rolled steel beam 16 mm thick plate are available.

Solution: Given Data

P= 2500 KN = 2500 x 10^3 N

length = L=5.6 m=5600 mm
Thickness of plate = 16 mm
(Both end fixed) the ends of column are effectively held in position and direction
Effective length = L_{eff}= 0.65 L=0.65 X 5600 = 3640 mm
Assuming λ = 40 (for heavy load)
σac = 139 N/mm^2 (From IS CODE) Page No: 39 Table No:5.1
Fy = 250 N/mm^2

1.Find required area

$$A = \frac{P}{\sigma_{ac}} = \frac{2500 X 10^3}{139} = 17985\ mm^2$$

2. Select Suitable section from steel table
   Try ISHB 450 @ 92.5 kg/m
   A=11789 mm^2
   Ixx=40349.9 x 10^4 mm^4
   Iyy=3045 x 10^4 mm^4
   rxx=185 mm
   ryy=50.8 mm
   b_f= Width of flange= 250 mm
   To carry maximum load
   $rxx \cong ryy$ =185 mm
   rmin= 185 mm
3. To find slenderness ratio

$$\lambda = \frac{l}{r_{\min}} = \frac{3640}{185} = 19.68$$

4. From IS 800-1984, Page No: 39 Table No:5.1, Fy= 250 N/mm^2

λ	σac
10	150
19.68	?
20	148

By interpolation

$$\sigma ac \ = 150 + \left[\frac{(148-150)}{(20-10)} \text{ X (19.68-10)} \right] = 148.06 \text{ N/mm}^2$$

Effective sectional area

$$A = \frac{P}{\sigma_{ac}} = \frac{2500 X 10^3}{148.06} = 16885 \ mm^2$$

Area of I- Section provided= 117.89 x 10^2 mm^2

Area to be provided by cover plate= 168.85 x 10^2 -117.89 x 10^2

Area to be provided by cover plate= 5096 mm^2

Assuming two cover plate (Each at top and bottom)

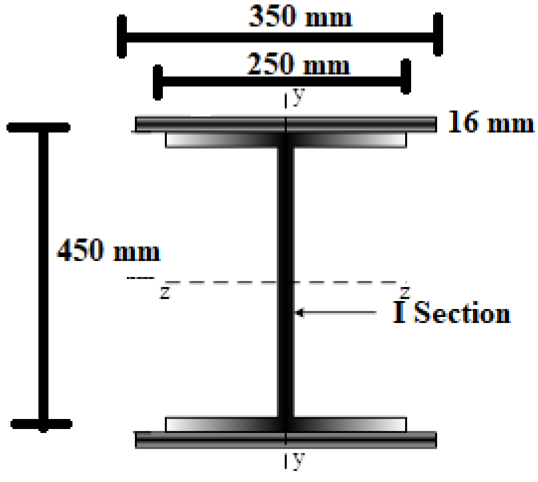
Area to be provided by one cover plate=5096/2=2548 mm^2

Thickness of cover plate = 16 mm (Given)

Width of cover plate= 2548/16= 159.25 mm

Assuming width of plate = 350 mm

(Assume width of plate greater than (b_f) Width of flange= 250 mm)



Cross section of column

Check for outstanding width = (outstanding width / Thickness) <16

Outstanding width= (350-250)/16 = 3.125 <16 (OK)

Providing cover plate of 350 x 16 mm at top and bottom flange

Properties of compound section

A= Area of I-Section + MI of plates

A= [11789]+[(350 x16) x2] = 22989 mm^2

Ixx= [MI of Section] + [MI of plates]

$$I_{xx} = [Ixx] + \left[\frac{bd^3}{12} + Ah^2\right] x2$$

$$I_{xx} = [40349.9 \text{ x } 10^4] + \left[\frac{350x16^3}{12} + 350x16x\left(\frac{16}{2} + \frac{450}{2}\right)^2\right] x2$$

$$I_{xx} = 1.09x10^9 mm^4$$

$$I_{yy} = [I_{yy}] + \left[\frac{db^3}{12} + Ah^2\right] x2 \text{ (h=0)}$$

$$I_{yy} = [3045 \text{ x } 10^4] + \left[\frac{16x350^3}{12} + 0\right] x2$$

$$I_{yy} = 0.144x10^9 mm^4$$

I_{min}=0.144 X 10^9 mm^4

$$r_{\min} = \sqrt{\frac{I_{min}}{A}} = \sqrt{\frac{0.144x10^9}{22989}} = 79.35mm$$

$$\lambda = \frac{l}{r_{\min}} = \frac{3640}{79.35} = 45.872$$

λ	σac
40	139
45.872	?
50	132

By interpolation

$$\sigma ac = 139 + \left[\frac{(132-139)}{(50-40)} \text{ X (45.872-40)}\right] = 134.88 \text{ N/mm}^2$$

5. Load carrying capacity

$$P_c = \text{ A x } \sigma_{ac}$$

P_c =22989 x 134.8 = 3098 x 10^3 N

P_c = 3098 KN > 2500 KN , SAFE (OK)

ISHB 450 @ 92.5 kg/m with the cover plates of size 350 mm X 16 mm

4. A column 5 m long is to support a load of 4500 KN, the ends of column are effectively held in position and direction (Both end fixed). Design a rolled steel beam 18 mm thick plate are available.

Solution: Given Data

P= 4500 KN = 2500 x 10^3 N

length = L=5 m=5000 mm

Thickness of plate = 18 mm
(Both end fixed) the ends of column are effectively held in position and direction
Effective length = L_{eff}= 0.65 L=0.65 X 5000 = 3250 mm
Assuming λ = 40 (for heavy load)
σac = 139 N/mm^2 (From IS CODE) Page No: 39 Table No:5.1
Fy = 250 N/mm^2

1.Find required area

$$A = \frac{P}{\sigma_{ac}} = \frac{4500 X 10^3}{139} = 32374.10\ mm^2$$

2.Select Suitable section from steel table

Try ISHB 600 @ 145.1 kg/m
A=18486 mm^2
Ixx=115626.6 x 10^4 mm^4
Iyy=5298.3x 10^4 mm^4
rxx=250.1mm
ryy=53.5 mm
b_f= Width of flange= 250 mm
To carry maximum load
$rxx \cong ryy$ =250.1 mm
rmin= 250.1 mm

3.To find slenderness ratio

$$\lambda = \frac{l}{r_{\min}} = \frac{3250}{250.1} = 12.99$$

4.From IS 800-1984, Page No: 39 Table No:5.1, Fy= 250 N/mm^2

λ	σac
10	150
19.68	?
20	142

By interpolation

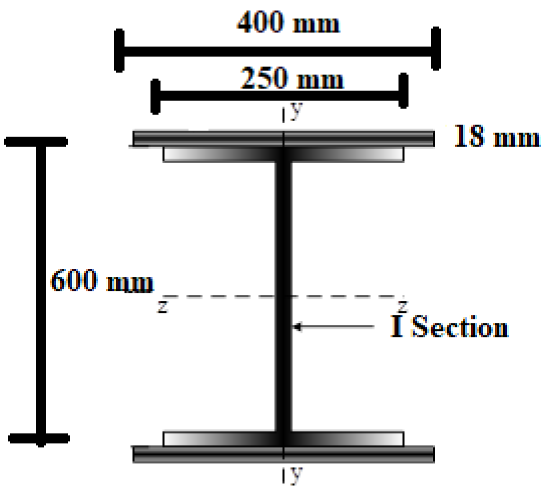
$$\sigma ac\ = 150 + \left[\frac{(148-150)}{(20-10)} \text{ X (12.99-10)} \right] = 149.40 \text{ N/mm}^2$$

Effective sectional area

$$A = \frac{P}{\sigma_{ac}} = \frac{4500 X 10^3}{149.40} = 30120.48\ mm^2$$

Area of I- Section provided= 184.86 x 10^2 mm^2
Area to be provided by cover plate= 301.20 x 10^2 -184.7.86 x 10^2
Area to be provided by cover plate= 116.34 x 10^2 mm^2

Assuming two cover plate (Each at top and bottom)
Area to be provided by one cover plate=11634/2=5817 mm^2
Thickness of cover plate = 18 mm (Given)
Width of cover plate= 5817/18= 323.16 mm
Assuming width of plate = 400 mm
(Assume width of plate greater than (b_f) Width of flange= 250 mm)



Cross section of column

Check for outstanding width = (outstanding width / Thickness) <16
Outstanding width= (400-250)/18 = 4.16 <16 (OK)
Providing cover plate of 400 x 18 mm at top and bottom flange
Properties of compound section
A= Area of I-Section + MI of plates
A= [18486]+[(400 x18) x2] = 32886 mm^2
Ixx= [MI of Section] + [MI of plates]

$$I_{xx} = [Ixx] + \left[\frac{bd^3}{12} + Ah^2\right] \text{x2}$$

$$I_{xx} = [115626.6 \text{ x } 10^4] + \left[\frac{400x18^3}{12} + 400x18x\left(\frac{18}{2} + \frac{600}{2}\right)^2\right] \text{x2}$$

$$I_{xx} = 2.53x10^9 mm^4$$

$$I_{yy} = [I_{yy}] + \left[\frac{db^3}{12} + Ah^2\right] \text{x2 (h=0)}$$

$$I_{xx} = [5298.3 \text{ x } 10^4] + \left[\frac{18x400^3}{12} + 0\right] \text{x2}$$

$$I_{xx} = 0.244x10^9 mm^4$$

I_{min}=0.244 X 10^9 mm^4

$$r_{\min} = \sqrt{\frac{I_{\min}}{A}} = \sqrt{\frac{0.244x10^9}{32886}} = 86.13mm$$

$$\lambda = \frac{l}{r_{\min}} = \frac{3250}{86.13} = 37.73$$

λ	σac
30	145
37.73	?
40	139

By interpolation

$$\sigma ac \; = 145 + \left[\frac{(139-145)}{(40-30)} \text{ X (37.73-30)} \right] = 140.36 \text{ N/mm}^2$$

5.Load carrying capacity

$$P_c = \text{ A x } \sigma_{ac}$$

P_c =32886 x 140.36 = 4615.94x 10^3 N

P_c = 4615.94 KN > 4500 KN , SAFE (OK)

ISHB 600 @ 145 kg/m with the cover plates of size 400mm X 18 mm

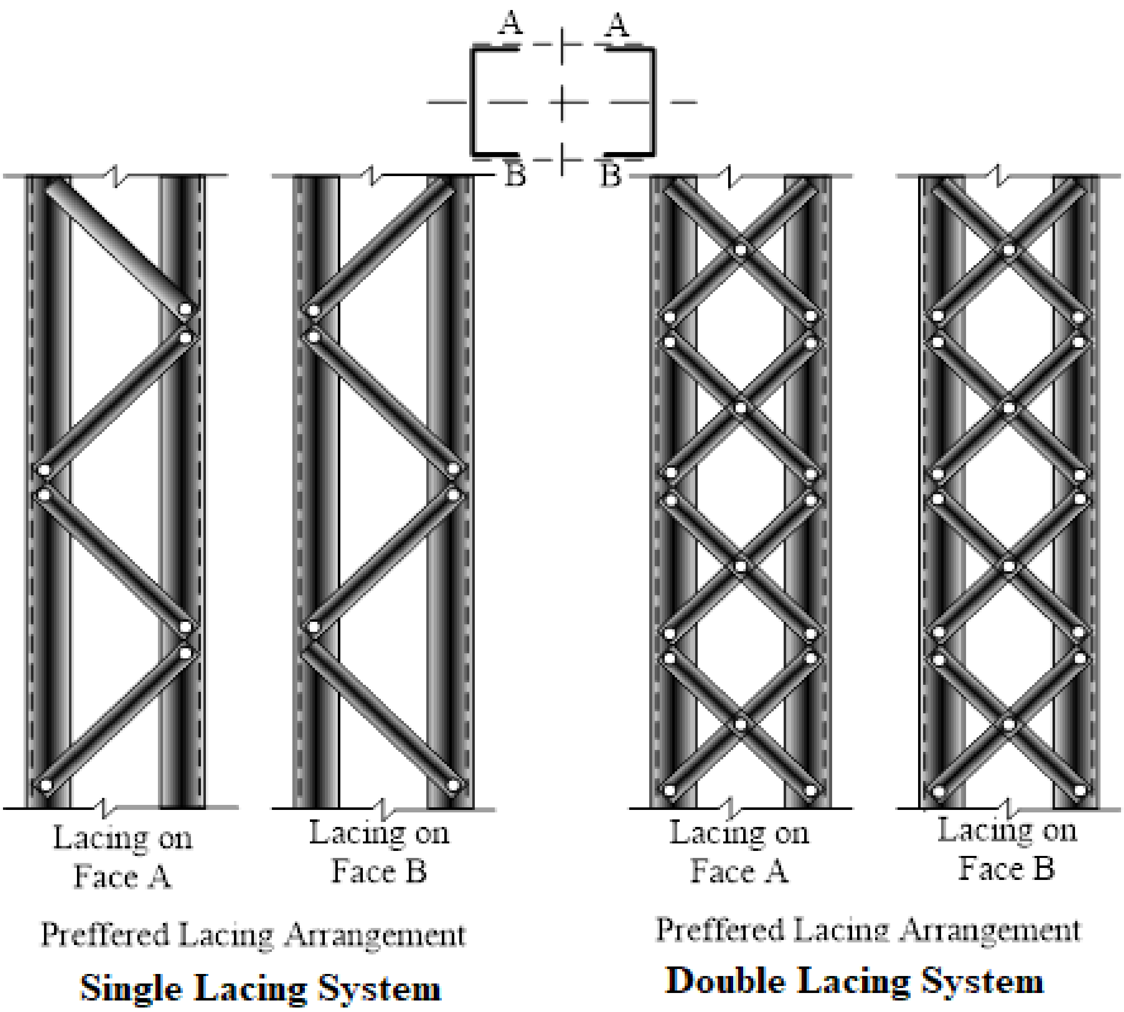
5. A column 5.2 m long is to support a load of 3600 KN, the ends of column are fixed. Design a rolled steel beam 16 mm thick plate are available.

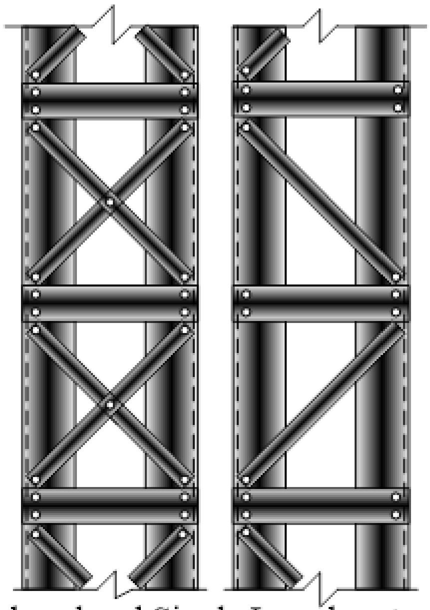
Design of Lacing

In case of built up column a suitable lateral system is needed to connect rolled steel section to hold column in position. It does not share the axial load, the main object is to carry transverse shear, the rolled steel section, flats are used of lacing system. Compression members comprising two main components laced and tied, its radius of gyration about the axis perpendicular the plane of lacing not less than the radius of gyration about the axis parallel to the plane of lacing. As far as practicable, the lacing system shall be uniform throughout the length of the column

There are two types of lacing system

a) Single lacing system
b) Double lacing system



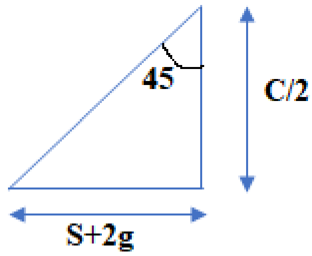


Double laced and Single Laced system combined with Cross Numbers

Design Procedure for lacing

1. Assuming single or double lacing:
   Lacing bars, whether in double or single system, shall be inclined at an angle not less than 40^0 nor more than 70^0 to the axis of the built up members.
   Assume $\Theta = 45^0$

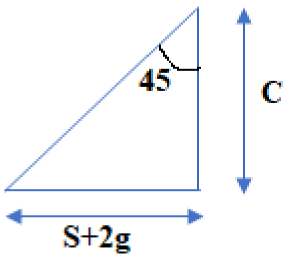
   For Single lacing



$$\tan\theta = \frac{S+2g}{C/2}$$

$$C = \frac{2(S+2g)}{\tan\theta}$$

For Double lacing



$$\tan\theta = \frac{S+2g}{C}$$

$$C = \frac{(S+2g)}{\tan\theta}$$

$$C = Spacing\ of\ lacing\ bar$$

2. Check for Spacing
   IS Code 800-1984, Page Number 51, Clause 5.7.6

   $$\lambda = \frac{C}{r_{\min}}\ Not \succ\ 0.7\lambda \text{ of whole column}$$

   or

   50 (Which is less)

3. Width of Lacing Bars (IS 800-1984, Page Number 50)
   In bolted/riveted connection, the minimum width of lacing bars shall be three times the nominal diameter (d) of the end bolt/rivet.
   Width of Lacing Bars = 3d

4. **Thickness of Lacing Bars**.
   The thickness of flat lacing bars shall not be less than one-fortieth of its effective length for single lacings and one-sixtieth of the effective length for double lacings.
   For single lacing

   $$t = \frac{1}{40} \text{ x } length\ of\ lacing\ bar\ between\ inner\ rivet$$

   For double lacing

   $$t = \frac{1}{60} \text{ x } length\ of\ lacing\ bar\ between\ inner\ rivet$$

5. Check for lacing bar
   For single lacing

   $$\lambda = \frac{l}{r_{\min}} < 145$$

   For double lacing

$$\lambda = \frac{0.7l}{r_{\min}} < 145$$

l=L--------------For single lacing

l=0.7L--------------For double lacing

$$r_{\min} = \frac{t}{\sqrt{12}}$$ where t = thickness of lacing bar

6. Transverse shear (IS 800-1984, Page Number 50 , Clause No 5.7.2.1
   The lacing shall be proportioned to resist a total transverse shear, V at any point in the member, equal to at least 2.5 % of the axial force in the member and shall be divided equally among all transverse lacing system in parallel planes.
   V=2.5 % of total load
   $$V = \frac{2.5}{100} \text{ X P}$$
7. Force in lacing bar
   For single lacing
   $$F = \frac{V}{n} Co\sec\theta$$
   For double lacing
   $$F = \frac{V}{2n} Co\sec\theta$$
   n= number of parallel systems
8. Force in rivet
   Force on rivet = 2F sin Θ
   Find rivet value
   Least of P_s and P_b
   Rivet value > Force on rivet (ok)
9. Check the strength of lacing bar in compression and tension
1. A column of effective length 2.8 m must carry an axial load of 1200 KN. Design a column section consisting of two channels placed back to back at suitable distance. Design also lacing for column.

6. A column 5.8 m long is to support a load of 1200 KN, the ends of column are fixed. Design column section consisting of two channels placed back to back at a suitable distance. Also Design lacing for column.

Solution: Given Data

P= 1200 KN = 1200 x 10^3 N

Effective length = L_{eff}=5.8 m=5800 mm

Assuming λ = 80 for channel section

σac = 101 N/mm^2 (From IS CODE) Page No: 39 Table No:5.1

Fy = 250 N/mm^2

1.Find required area

$$A = \frac{P}{\sigma_{ac}} = \frac{1200 X 10^3}{101} = 11881.18\ mm^2$$

2.Select Suitable section from steel table (P. No 98)

Try 2 ISMC 400@ 98.8 kg/m
A=12586 mm^2
rxx=154.8 mm
ryy= 156.8 mm (Back to back spacing 260 mm)
NOTE: Select suitable ryy closer to rxx and from Steel table P. No 98, get spacing
rmin= 154.8 mm
g=60 mm

3.To find slenderness ratio

$$\lambda = \frac{l_{eff}}{r_{\min}} = \frac{5800}{154.8} = 37.46$$

4. From IS 800-1984, Page No: 39 Table No:5.1, Fy= 250 N/mm^2

λ	σac
30	145
37.46	?
40	139

By interpolation

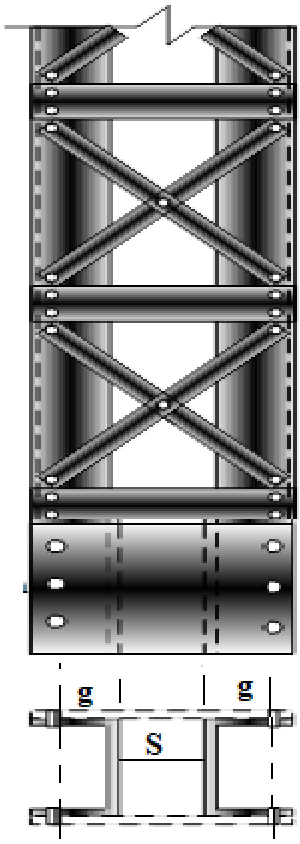
$$\sigma ac\ = 145 + \left[\frac{(139-145)}{(40-30)} \text{ X } (37.46\text{-}30) \right] = 140.52 \text{ N/mm}^2$$

5. Load carrying capacity

$$P_c = \text{ A x } \sigma_{ac}$$

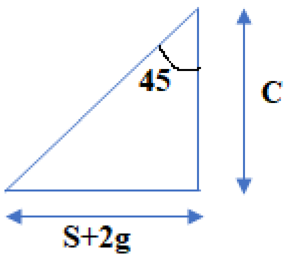
P_c =12586 x 140.52= 1768.58 x 10^3 N
P_c = 1768.58 KN > 1200 KN , SAFE (OK)

2) Design of lacing system
1. Assuming double lacing



Assuming angle of lacing bar i.e Assume ϴ = 45^0
S+2g=260+ (2x60)= 380 mm

For Double lacing



$$\tan\theta = \frac{S+2g}{C}$$

$$C = \frac{(260+2x60)}{\tan 45^0}$$

$$C = 380\ mm$$

2. Check for Spacing
   IS Code 800-1984, Page Number 51, Clause 5.7.6

$$\lambda = \frac{C}{r_{\min}} \ Not \succ \ 0.7\lambda \text{ of whole column}$$

or

50 (Which is less)

For Single channel section

ISMC 400@ 24.7 kg/m

rxx=154.8 mm

ryy= 28.3 mm

rmin= 28.3 mm

$$\frac{C}{r_{\min}} \prec 0.7x\ 37.46 = 26.3 \text{ or } 50$$

$$\frac{380}{28.3} \prec 26.3$$

$13.42 \prec 26.3$ (ok)

Providing C= 380mm

3.Width of Lacing Bars (IS 800-1984, Page Number 50)

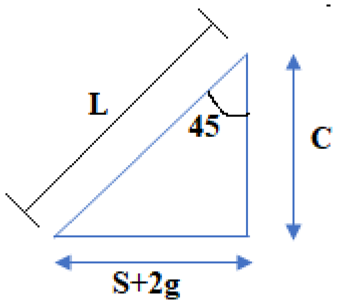
Assuming Nominal diameter (d) of the end rivet = 22 mm

Width of Lacing Bars = 3d=3x 22= 66 $\cong 70mm$

4. **Thickness of Lacing Bars**.

For double lacing

$$t = \frac{1}{60} \text{ x } length\ of\ lacing\ bar\ between\ inner\ rivet$$



$$\sin 45^0 = \frac{380}{L}$$

L= 537.40 mm

$$t = \frac{1}{60}\ x\ 537.40$$

$t = 8.96mm$

$Assuming\ t = 10\ mm$

5. Check for lacing bar

For double lacing

$$\lambda = \frac{0.7l}{r_{\min}} \; < \; 145$$

$$\lambda = \frac{0.7x537.4}{r_{\min}} \; < \; 145$$

$$r_{\min} = \frac{t}{\sqrt{12}} = \frac{10}{\sqrt{12}} = 2.88mm$$

$$\lambda = \frac{0.7x537.4}{2.88} \; < \; 145$$

$$\lambda = 130.6 \; < \; 145 \text{ (ok)}$$

Providing lacing bar of width 70 mm and thickness 10 mm

6. Transverse shear (IS 800-1984, Page Number 50 , Clause No 5.7.2.1

V=2.5 % of total load

$$V = \frac{2.5}{100} \text{ X P= } \frac{2.5}{100} \text{ X 1200= 30 KN}$$

7. Force in lacing bar

For double lacing

Assuming n=2

$$F = \frac{V}{2n} Co\sec\theta = \frac{30}{2x2} Co\sec 45^0 = 10.6KN$$

n= number of parallel systems

8. Force in rivet

Force on rivet = 2F sin Θ = 2x10.6 x sin 45^0=15 KN

Find rivet value

Least of P_s and P_b

Nominal Diameter=d=22 mm
Gross Diameter of rivet =D= 22 +1.5 = 23.5 mm
Assuming power driven shop rivets (PDS) (P No:95 Tno:8.1)
τ_{VF}= 100 N/mm^2
σ_{bf}= 300 N/mm^2

$$P_S = \frac{\Pi}{4} \text{ x } D^2 \text{ X } \tau_{Vf}$$

$$P_S = \frac{\Pi}{4} \text{ x } 23.5^2 \text{ X 100} \qquad = 43.373 \text{ x } 10^3 N = 43.373KN$$

$$P_b = \text{D x t X } \sigma_{bf}$$

$$P_b = 23.5\text{x 10 X 300} = 70.5 \text{ x } 10^3 N = 70.5KN$$

Rivet value= least of Ps and Pb
Rivet value= 43.373 KN

Rivet value > Force on rivet (ok)

43.373 KN > 15 KN

9. Check the strength of lacing bar in compression (For lacing bar)

$\lambda = 130.6$

λ	σac
130	57
130.6	?
140	51

By interpolation

$$\sigma ac \ = 57 + \left[\frac{(51-57)}{(140-130)} \text{ X } (130.6\text{-}130) \right] = 56.72 \text{ N/mm}^2$$

$$P_c = \text{ A x } \sigma_{ac}$$

P_c =(70 x 10) x 56.72 = 39.704 x 10^3 N

P_c = 39.704 KN > 10.6KN , SAFE (OK)

Check for tension

$P_t = (b - \text{ D) X tx } \sigma_{at}$

$P_t = (70 - \text{ 23.5) X 10x 150}$

$P_t = 69.75x10^3 N = 69.75KN > 10.6$ KN (OK)

7. A steel column has to support a load of 1000 KN, length of column is 6m. Design a built up column with two channels placed back to back. Design lacing system also.

Solution: Given Data

P= 1000 KN = 1000 x 10^3 N

Effective length = L_{eff}=6 m=6000 mm

Assuming λ = 80 for channel section

σac = 101 N/mm^2 (From IS CODE) Page No: 39 Table No:5.1

Fy = 250 N/mm^2

1.Find required area

$$A = \frac{P}{\sigma_{ac}} = \frac{1000 X 10^3}{101} = 9900.9 \ mm^2$$

2.Select Suitable section from steel table (P. No 98)

Try 2 ISMC 350@ 84.2 kg/m

A=10732 mm^2

rxx=136.6 mm

ryy= 137.4 mm (Back to back spacing 220 mm)

NOTE: Select suitable ryy closer to rxx and from Steel table P. No 98, get spacing

rmin= 136.6 mm

g=60 mm

3.To find slenderness ratio

$$\lambda = \frac{l_{eff}}{r_{\min}} = \frac{6000}{136.6} = 43.92$$

6. From IS 800-1984, Page No: 39 Table No:5.1, Fy= 250 N/mm^2

λ	σac
40	189
43.92	?
50	132

By interpolation

$$\sigma ac \; = 139 + \left[\frac{(132-139)}{(50-40)} \text{ X } (43.92\text{-}40) \right] = 136.25 \text{ N/mm}^2$$

7. Load carrying capacity

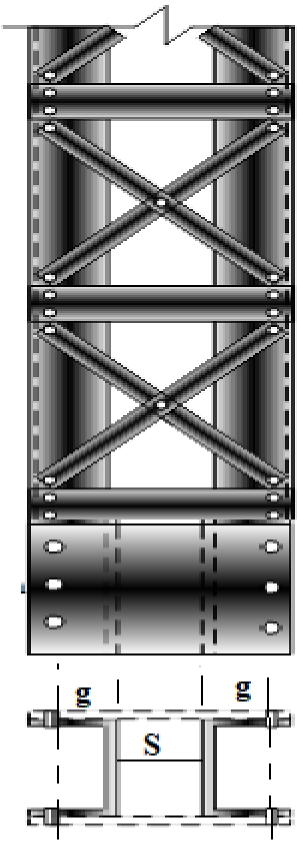
$$P_c = \text{ A x } \sigma_{ac}$$

P_c =10732 x 136.25= 1462.23 x 10^3 N

P_c = 1462.23 KN > 1000 KN , SAFE (OK)

2) Design of lacing system

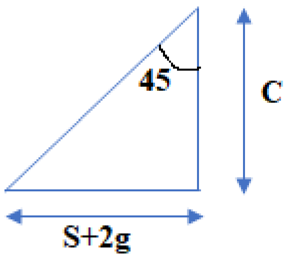
1. Assuming double lacing



Assuming angle of lacing bar i.e Assume ϴ = 45^0

S+2g=220+ (2x60)= 340 mm

For Double lacing



$$\tan\theta = \frac{S+2g}{C}$$

$$C = \frac{(220+2x60)}{\tan 45^0}$$

$$C = 340\ mm$$

2.Check for Spacing

IS Code 800-1984, Page Number 51, Clause 5.7.6

$$\lambda = \frac{C}{r_{\min}}\ Not \succ\ 0.7\lambda \text{ of whole column}$$

or

50 (Which is less)

For Single channel section

ISMC 350@ 42.1kg/m

rxx=136.6 mm
ryy= 28.3 mm
rmin= 28.3 mm

$$\frac{C}{r_{\min}} \prec 0.7x\ 43.92 = 30.744 \text{ or } 50$$

$$\frac{340}{28.3} \prec 30.744$$

$$12.01 \prec 30.744 \text{ (ok)}$$

Providing C= 340mm

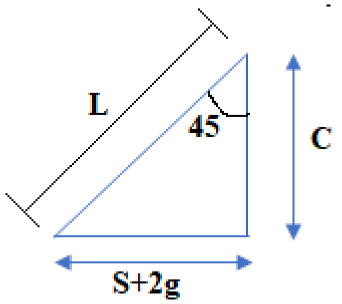
3.Width of Lacing Bars (IS 800-1984, Page Number 50)

Assuming Nominal diameter (d) of the end rivet = 22 mm
Width of Lacing Bars = 3d=3x 22= 66 $\cong 70mm$

4.Thickness of Lacing Bars.

For double lacing

$t = \frac{1}{60}$ x *length of lacing bar between inner rivet*



$\sin 45^0 = \frac{340}{L}$

L= 480.83 mm

$t = \frac{1}{60}\ x\ 480.83$

$t = 8.01mm$

$Assu\min g\ t = 10\ mm$

5. Check for lacing bar

For double lacing

$\lambda = \frac{0.7l}{r_{\min}} < 145$

$\lambda = \frac{0.7x480.86}{r_{\min}} < 145$

$r_{\min} = \frac{t}{\sqrt{12}} = \frac{10}{\sqrt{12}} = 2.88mm$

$\lambda = \frac{0.7x480.83}{2.88} < 145$

$\lambda = 116.86 < 145$ (ok)

Providing lacing bar of width 70 mm and thickness 10 mm

6.Transverse shear (IS 800-1984, Page Number 50 , Clause No 5.7.2.1

V=2.5 % of total load

$V = \frac{2.5}{100}$ X P= $\frac{2.5}{100}$ X 1000= 25 KN

7.Force in lacing bar

For double lacing

Assuming n=2

$F = \frac{V}{2n} Co\sec\theta = \frac{25}{2x2} Co\sec 45^0 = 8.8KN$

n= number of parallel systems

8.Force in rivet

Force on rivet = 2F sin ϴ = 2x8.8 x sin 45^0= 12.44 KN
Find rivet value
Least of P_s and P_b

Nominal Diameter=d=22 mm
Gross Diameter of rivet =D= 22 +1.5 = 23.5 mm
Assuming power driven shop rivets (PDS) (P No:95 Tno:8.1)
τ_{VF}= 100 N/mm²
σ_{bf}= 300 N/mm²

$$P_S = \frac{\Pi}{4} \text{ x } D^2 \text{ X } \tau_{Vf}$$

$$P_S = \frac{\Pi}{4} \text{ x } 23.5^2 \text{ X } 100 \quad = 43.373 \text{ x } 10^3 N = 43.373KN$$

$$P_b = \text{D x t X } \sigma_{bf}$$

$$P_b = 23.5\text{x } 10 \text{ X } 300 = 70.5 \text{ x } 10^3 N = 70.5KN$$

Rivet value= least of Ps and Pb
Rivet value= 43.373 KN

Rivet value > Force on rivet (ok)
43.373 > 12.44 KN (OK)

9.Check the strength of lacing bar in compression (For lacing bar)

$\lambda = 116.86$

λ	σac
110	72
116.86	?
120	64

By interpolation

$$\sigma ac \;= 72 + \left[\frac{(64-72)}{(120-110)} \text{ X (116.86-110)} \right] = 66.54 \text{ N/mm}^2$$

$$P_c = \text{ A x } \sigma_{ac}$$

P_c =(70 x 10) x 66.54 = 46.578 x 10^3 N
P_c = 46.578 KN > 8.8KN , SAFE (OK)
Check for tension

$$P_t = (b - \text{ D) X tx } \sigma_{at}$$

$$P_t = (70 - \text{ 23.5) X 10x 150}$$

$$P_t = 69.75x10^3 N = 69.75KN > 8.8 \text{ KN (OK)}$$

3. A steel column 12 m long carries axial load of 1000 KN. The column is hinged at both ends. Design the channel section placed back to back. Also design single lacing system.

Solution: Given Data

P= 1000 KN = 1000 x 10^3 N

Effective length = L_{eff}=12 m=12000 mm
Assuming λ = 80 for channel section
σac = 101 N/mm^2 (From IS CODE) Page No: 39 Table No:5.1
Fy = 250 N/mm^2

1.Find required area

$$A = \frac{P}{\sigma_{ac}} = \frac{1000X10^3}{101} = 9900.9\ mm^2$$

2.Select Suitable section from steel table (P. No 98)

Try 2 ISMC 350@ 84.2 kg/m
A=10732 mm^2
rxx=136.6 mm
ryy= 137.4 mm (Back to back spacing 220 mm)
NOTE: Select suitable ryy closer to rxx and from Steel table P. No 98, get spacing
rmin= 136.6 mm
g=60 mm

3.To find slenderness ratio

$$\lambda = \frac{l_{eff}}{r_{\min}} = \frac{12000}{136.6} = 87.8$$

4.From IS 800-1984, Page No: 39 Table No:5.1, Fy= 250 N/mm^2

λ	σac
80	101
87.8	?
90	90

By interpolation

$$\sigma ac\ = 101 + \left[\frac{(90-101)}{(90-80)}\text{ X (87.8-80)}\right] = 92.42\text{N/mm}^2$$

5.Load carrying capacity

$$P_c = \text{A x } \sigma_{\text{ac}}$$

P_c =10732 x 92.42= 991.85 x 10^3 N
P_c = 991.85 KN <1000 KN , UNSAFE
Try another section
Try 2 ISMC 350@ 84.2 kg/m
A=10732 mm^2
rxx=136.6 mm
ryy= 137.4 mm (Back to back spacing 220 mm)
NOTE: Select suitable ryy closer to rxx and from Steel table P. No 98, get spacing
rmin= 136.6 mm

g=60 mm

3.To find slenderness ratio

$$\lambda = \frac{l_{eff}}{r_{\min}} = \frac{12000}{136.6} = 87.8$$

4.From IS 800-1984, Page No: 39 Table No:5.1, Fy= 250 N/mm^2

λ	σac
80	101
87.8	?
90	90

By interpolation

$$\sigma ac \ = 101 + \left[\frac{(90-101)}{(90-80)} \text{ X } (87.8\text{-}80) \right] = 92.42 \text{N/mm}^2$$

5.Load carrying capacity

$$P_c = \text{ A x } \sigma_{ac}$$

P_c =10732 x 92.42= 991.85 x 10^3 N

P_c = 991.85 KN <1000 KN , UNSAFE

Try another section

Try 2 ISMC 400@ 98.8 kg/m

A=12586 mm^2

rxx=154.8 mm

ryy= 156.8 mm (Back to back spacing 260 mm)

NOTE: Select suitable ryy closer to rxx and from Steel table P. No 98, get spacing

rmin= 154.8 mm

g=60 mm

$$\lambda = \frac{l_{eff}}{r_{\min}} = \frac{12000}{154.8} = 77.5$$

From IS 800-1984, Page No: 39 Table No:5.1, Fy= 250 N/mm^2

λ	σac
70	112
77.5	?
80	101

By interpolation

$$\sigma ac \ = 112 + \left[\frac{(101-112)}{(80-70)} \text{ X } (77.5\text{-}70) \right] = 103.75 \text{N/mm}^2$$

Load carrying capacity

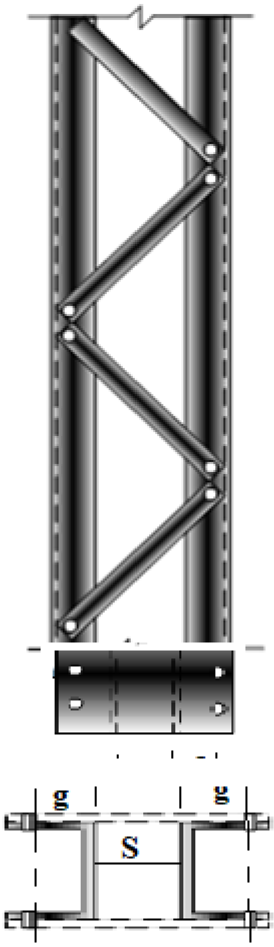
$$P_c = \text{ A x } \sigma_{ac}$$

P_c =12586 x 103.75= 1305.6 x 10^3 N

P_c = 1305.61 KN >1000 KN , SAFE (ok)

2) Design of lacing system

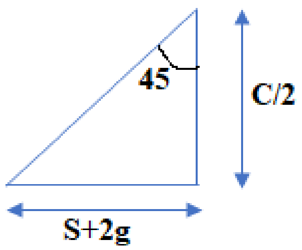
1. Single lacing



Assuming angle of lacing bar i.e Assume Θ = 45^0

S+2g=260+ (2x60)= 380 mm

For Single lacing



$$\tan\theta = \frac{S+2g}{C/2}$$

$$\frac{C}{2} = \frac{(260+2x60)}{\tan 45^0}$$

$$C = 760\ mm$$

2.Check for Spacing

IS Code 800-1984, Page Number 51, Clause 5.7.6

$\lambda = \dfrac{C}{r_{\min}}$ $Not \succ$ 0.7λ of whole column

or

50 (Which is less)

For Single channel section

ISMC 400 @ 49.4kg/m

rxx=154.8 mm
ryy= 28.3 mm
rmin= 28.3 mm

$\dfrac{C}{r_{\min}} \prec 0.7x\ 77.51 = 54.257\text{or } 50$

$\dfrac{760}{28.3} \prec 50$

$26.85 \prec 50$ (ok)

Providing C= 760mm

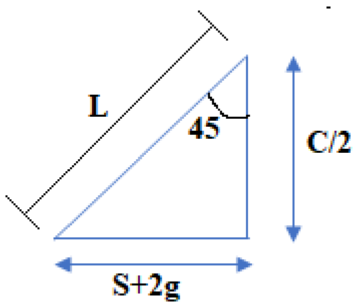
3.Width of Lacing Bars (IS 800-1984, Page Number 50)

Assuming Nominal diameter (d) of the end rivet = 22 mm
Width of Lacing Bars = 3d=3x 22= 66 $\cong 70mm$

4.Thickness of Lacing Bars.

For single lacing

$t = \dfrac{1}{40}$ x *length of lacing bar between inner rivet*



$\sin 45^0 = \dfrac{380}{L}$

L= 537.4 mm

$$t = \frac{1}{40} \ x \ 537.80$$

$$t = 13.43mm$$

$$Assu\min g \ t = \ 14 \ mm$$

6. Check for lacing bar

For single lacing

$$\lambda = \frac{l}{r_{\min}} \ < \ 145$$

$$\lambda = \frac{537.4}{r_{\min}} \ < \ 145$$

$$r_{\min} = \frac{t}{\sqrt{12}} = \frac{14}{\sqrt{12}} = 4.04mm$$

$$\lambda = \frac{537.4}{4.04} \ < \ 145$$

$$\lambda = 132.97 \ < \ 145 \text{ (ok)}$$

Providing lacing bar of width 70 mm and thickness 14 mm

6.Transverse shear (IS 800-1984, Page Number 50 , Clause No 5.7.2.1

V=2.5 % of total load

$$V = \frac{2.5}{100} \text{ X P=} \frac{2.5}{100} \text{ X 1000= 25 KN}$$

7.Force in lacing bar

For single lacing

Assuming n=2

$$F = \frac{V}{n} Co\sec\theta = \frac{25}{2} Co\sec 45^0 = 17.67KN$$

n= number of parallel systems

8.Force in rivet

Force on rivet = 2F sin ϴ = 2x17.67 x sin 45^0= 24.98 KN

Find rivet value

Least of P_s and P_b

Nominal Diameter=d=22 mm
Gross Diameter of rivet =D= 22 +1.5 = 23.5 mm
Assuming power driven shop rivets (PDS) (P No:95 Tno:8.1)
τ_{VF}= 100 N/mm^2
σ_{bf}= 300 N/mm^2

$P_S = \frac{\Pi}{4}$ x D^2 X τ_{Vf}

$P_S = \frac{\Pi}{4}$ x 23.5^2 X 100 $= 43.373 \text{ x } 10^3 N = 43.373 KN$

$P_b = D \text{ x t X } \sigma_{bf}$

$P_b = 23.5\text{x } 14 \text{ X } 300 = 98.7 \text{ x } 10^3 N = 98.7 KN$

Rivet value= least of Ps and Pb

Rivet value= 43.373 KN

Rivet value > Force on rivet (ok)

43.374 > 24.98 KN (OK)

9.Check the strength of lacing bar in compression (For lacing bar)

$\lambda = 132.97$

λ		σac
130	57	
132.97	?	
140	51	

By interpolation

$$\sigma ac = 57 + \left[\frac{(51-57)}{(140-130)} \text{ X } (132.97\text{-}130) \right] = 55.21 \text{ N/mm}^2$$

$P_c =$ A x σ_{ac}

P_c =(70 x 14) x 55.21 = 54.105 x 10^3 N

P_c = 54.105 KN > 17.67 KN , SAFE (OK)

Check for tension

$P_t = (b - D) \text{ X tx } \sigma_{at}$

$P_t = (70 - 23.5) \text{ X } 14\text{x } 150$

$P_t = 97.65x10^3 N = 97.65KN > 17.67$ KN (OK)

4. A column has length of 6 m an axial load of 2000 KN. Design the compound section for column consisting of two I Sections. Also design lacing system

Solution: Given Data

P= 2000 KN = 2000 x 10^3 N

Effective length = L_{eff}=6 m=6000 mm

Assuming λ = 80 for I Section section

σac = 101 N/mm^2 (From IS CODE) Page No: 39 Table No:5.1

Fy = 250 N/mm^2

1.Find required area

$$A = \frac{P}{\sigma_{ac}} = \frac{2000 X 10^3}{101} = 19801.98\ mm^2$$

2.Select Suitable section from steel table (P. No 98)

Try 2 ISHB 350@ 72.4 kg/m
$A=18442mm^2$
rxx=146.5mm
ryy= 147.1mm (Back to back spacing 275 mm)
NOTE: Select suitable ryy closer to rxx and from Steel table P. No 98, get spacing
rmin= 146.5 mm
g=140 mm

3.To find slenderness ratio

$$\lambda = \frac{l_{eff}}{r_{\min}} = \frac{6000}{146.8} = 40.96$$

8. From IS 800-1984, Page No: 39 Table No:5.1, Fy= 250 N/mm^2

λ	σac
40	139
40.96	?
50	132

By interpolation

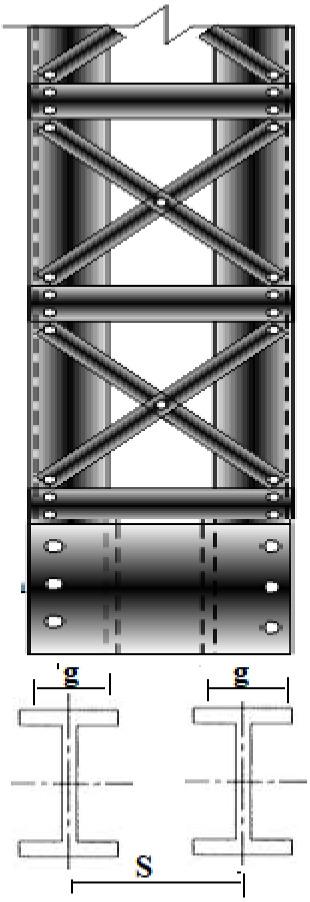
$$\sigma ac \ = 139 + \left[\frac{(132 - 139)}{(50 - 40)} \text{ X (40.96-30)} \right] = 138.33 \text{ N/mm}^2$$

9. Load carrying capacity

$$P_c = \text{ A x } \sigma_{ac}$$

P_c =18442 x 138.33= 2551.08 x 10^3 N
P_c = 2551.08 KN > 2000 KN , SAFE (OK)

2) Design of lacing system
1. Assuming double lacing

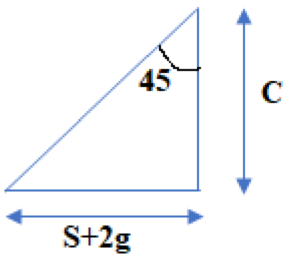


Distance between inner rivets= $= s - \frac{g}{2} - \frac{g}{2}$

$= 275 - \frac{140}{2} - \frac{140}{2} = 135mm$

Assuming angle of lacing bar i.e Assume ϴ = 45^0

For Double lacing



$$\tan\theta = \frac{Distance\ between\ inner\ rivet}{C}$$

$$C = \frac{135}{\tan 45^0}$$

$$C = 135\ mm$$

2.Check for Spacing

IS Code 800-1984, Page Number 51, Clause 5.7.6

$\lambda = \frac{C}{r_{\min}}$ $Not \succ$ 0.7λ of whole column

or

50 (Which is less)

For Single I section

ISHB 350@ 72.4 kg/m

rxx=146.5 mm

ryy= 52.2 mm

rmin= 52.2 mm

$\frac{C}{r_{\min}} \prec 0.7x\ 40.95 = 28.66$ or 50

$\frac{135}{52.2} \prec 28.66$

$2.58 \prec 28.66$ (ok)

Providing C= 185mm

3.Width of Lacing Bars (IS 800-1984, Page Number 50)

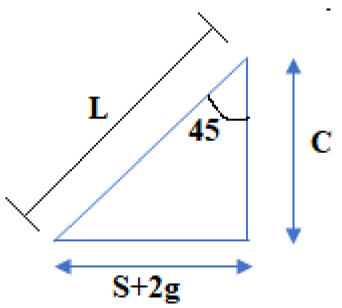
Assuming Nominal diameter (d) of the end rivet = 22 mm

Width of Lacing Bars = 3d=3x 22= 66 $\cong 70mm$

4.Thickness of Lacing Bars.

For double lacing

$t = \frac{1}{60}$ x *length of lacing bar between inner rivet*



$\sin 45^0 = \frac{135}{L}$

L= 190.91 mm

$$t = \frac{1}{60} \ x \ 190.91$$

$$t = 3.18mm$$

$$Assu\min g \ t = \ 6 \ mm$$

5.Check for lacing bar

For double lacing

$$\lambda = \frac{0.7l}{r_{\min}} \ < \ 145$$

$$\lambda = \frac{0.7x190.91}{r_{\min}} \ < \ 145$$

$$r_{\min} = \frac{t}{\sqrt{12}} = \frac{6}{\sqrt{12}} = 1.73mm$$

$$\lambda = \frac{0.7x537.4}{2.88} \ < \ 145$$

$$\lambda = 130.6 \ < \ 145 \text{ (ok)}$$

Providing lacing bar of width 70 mm and thickness 6mm

6.Transverse shear (IS 800-1984, Page Number 50 , Clause No 5.7.2.1

V=2.5 % of total load

$$V = \frac{2.5}{100} \text{ X P=} \frac{2.5}{100} \text{ X 2000= 50 KN}$$

7.Force in lacing bar

For double lacing

Assuming n=2

$$F = \frac{V}{2n} Co\sec\theta = \frac{50}{2x2} Co\sec 45^0 = 17.67KN$$

n= number of parallel systems

8.Force in rivet

Force on rivet = 2F sin ϴ = 2x17.67x sin 45^0=24.97 KN

Find rivet value

Least of P_s and P_b

Nominal Diameter=d=22 mm

Gross Diameter of rivet =D= 22 +1.5 = 23.5 mm

Assuming power driven shop rivets (PDS) (P No:95 Tno:8.1)

τ_{VF}= 100 N/mm^2

σ_{bf}= 300 N/mm^2

$P_S = \frac{\Pi}{4}$ x D^2 X τ_{Vf}

$P_S = \frac{\Pi}{4}$ x 23.5^2 X 100 = 43.373 x $10^3 N = 43.373 KN$

P_b = D x t X σ_{bf}

P_b = 23.5x 10 X 300 = 70.5 x $10^3 N = 70.5 KN$

Rivet value= least of Ps and Pb

Rivet value= 43.373 KN

Rivet value > Force on rivet (ok)

43.373 > 24.97 KN

9.Check the strength of lacing bar in compression (For lacing bar)

$\lambda = 77.25$

λ	σac
70	112
77.25	?
80	101

By interpolation

$$\sigma ac = 112 + \left[\frac{(101-112)}{(80-70)} \text{ X (77.25-70)} \right] = 104.02 \text{ N/mm}^2$$

$P_c = $ A x σ_{ac}

P_c =(70 x 6) x 101.02 = 42.428 x 10^3 N

P_c = 42.428 KN > 24.97KN , SAFE (OK)

Check for tension

P_t = (b – D) X tx σ_{at}

P_t = (70 – 23.5) X 6x 150

P_t = $71.85x10^3 N$ =41.85KN > 24.97 KN (OK)

5. Design a built up column to carry an axial load of 1200 KN having effective length is 3.2 m. Use two channels facing each other connected by lacing also. Design the lacing.

Solution: Given Data

P= 1200 KN = 1200 x 10^3 N

Effective length = L_{eff}=3.2 m=3200 mm

Assuming λ = 80 for channel section

σac = 101 N/mm^2 (From IS CODE) Page No: 39 Table No:5.1

Fy = 250 N/mm^2

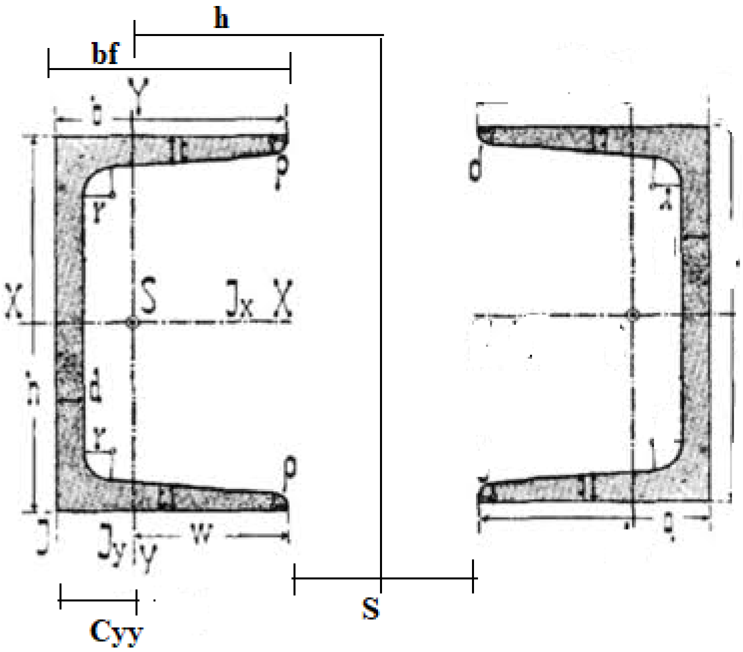
1.Find required area

$$A = \frac{P}{\sigma_{ac}} = \frac{1200 X 10^3}{101} = 11881.18\ mm^2$$

Area of one channel =11881.18/2=5940.49 mm^2

2.Select Suitable section from steel table (P. No 98)

Try ISMC 350@ 42.1 kg/m
A=5366 mm^2
Ixx=10008 x 10^4 mm^4
Iyy=430.6 x 10^4 mm^4
Cyy=24.4 mm
b_f=100 mm
g=60 mm



For compound section
Ixx= 2 X [MI of one channel about XX axis]
Ixx=Iyy= 2 x[10008 x 10^4]
Iyy= 2 X [MI of one channel about YY axis]

$$Iyy = 2 \text{ x } \left[Iyy + Ah^2 \right]$$

$$Iyy = 2 \text{ x } \left[430.6 \text{ x } 10^4 + 5366 \left[\frac{s}{2} + 75.6 \right]^2 \right]$$

To carry maximum load

Ixx= Iyy

$$2 \text{ x}[10008 \text{ x } 10^4] = 2 \text{ x } \left[430.6 \text{ x } 10^4 + 5366 \left[\frac{s}{2} + 75.6 \right]^2 \right]$$

S= 115.9mm $\cong 115mm$

Imin= Ixx= Iyy= 2 x[10008 x 10^4]= 20016 x 10^4 mm^4

$$r_{\min} = \sqrt{\frac{\mathrm{I}_{\min}}{A}} = \sqrt{\frac{20016X10^4}{2X5366}} = 136.56mm$$

3.To find slenderness ratio

$$\lambda = \frac{l_{eff}}{r_{\min}} = \frac{3200}{136.56} = 23.53$$

10. From IS 800-1984, Page No: 39 Table No:5.1, Fy= 250 N/mm^2

λ	σac
20	148
23.43	?
30	145

By interpolation

$$\sigma ac \ = 148 + \left[\frac{(145-148)}{(30-20)}\ \mathrm{X}\ (23.43\text{-}20)\right] = 146.97\ \mathrm{N/mm^2}$$

11. Load carrying capacity

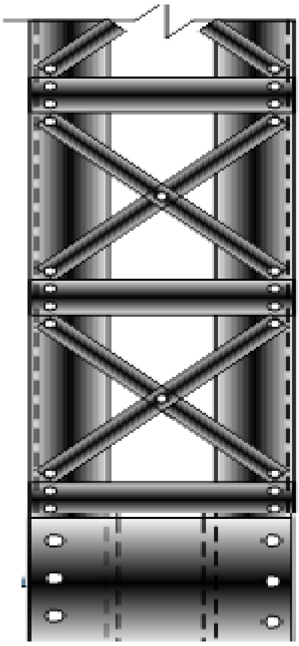
$$P_c = \ \mathrm{A\ x}\ \sigma_{\mathrm{ac}}$$

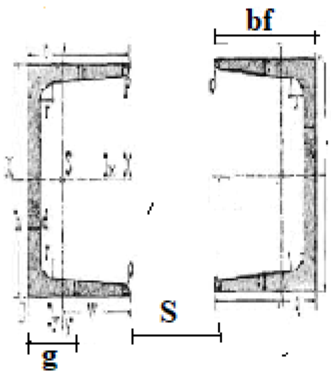
P_c =2 x 5366 x 146.97= 1577.28 x 10^3 N

P_c = 1577.28 KN > 1200 KN , SAFE (OK)

2) Design of lacing system

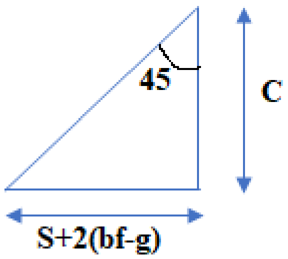
1. Assuming double lacing





Assuming angle of lacing bar i.e Assume ϴ = 45^0

For Double lacing



$$\tan\theta = \frac{S + 2(b_f - g)}{C}$$

$$C = \frac{115 + 2(100 - 60)}{\tan 45^0}$$

$$C = 195\ mm$$

6. Check for Spacing
   IS Code 800-1984, Page Number 51, Clause 5.7.6

$$\lambda = \frac{C}{r_{\min}} \; Not \succ \; 0.7\lambda \text{ of whole column}$$

or

50 (Which is less)

For Single channel section

ISMC 350@ 42.1 kg/m

rxx=136.6 mm

ryy= 28.3 mm

rmin= 28.3 mm

$$\frac{C}{r_{\min}} \prec \; 0.7x\; 23.43 = 16.40 \text{ or } 50$$

$$\frac{195}{28.3} \prec 16.40$$

$6.89 \prec 16.40$ (ok)

Providing C= 195mm

3.Width of Lacing Bars (IS 800-1984, Page Number 50)

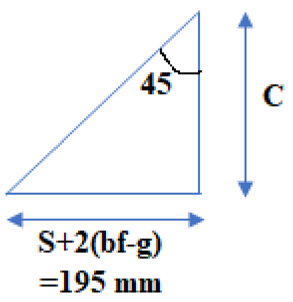
Assuming Nominal diameter (d) of the end rivet = 22 mm

Width of Lacing Bars = 3d=3x 22= 66 $\cong 70mm$

10. **Thickness of Lacing Bars**.

For double lacing

$$t = \frac{1}{60} \text{ x } length\ of\ lacing\ bar\ between\ inner\ rivet$$



$$\sin 45^0 = \frac{195}{L}$$

L= 275.77 mm

$$t = \frac{1}{60} \ x \ 275.77$$

$$t = 4.59 mm$$

$$Assu \min g \ t = \ 8 \ mm$$

11. Check for lacing bar

For double lacing

$$\lambda = \frac{0.7l}{r_{\min}} \ < \ 145$$

$$\lambda = \frac{0.7x275.77}{r_{\min}} \ < \ 145$$

$$r_{\min} = \frac{t}{\sqrt{12}} = \frac{8}{\sqrt{12}} = 2.30mm$$

$$\lambda = \frac{0.7x257.77}{2.30} \ < \ 145$$

$$\lambda = 83.93 \ < \ 145 \text{ (ok)}$$

Providing lacing bar of width 70 mm and thickness 8 mm

12. Transverse shear (IS 800-1984, Page Number 50 , Clause No 5.7.2.1

V=2.5 % of total load

$$V = \frac{2.5}{100} \text{ X P=} \frac{2.5}{100} \text{ X 1200= 30 KN}$$

13. Force in lacing bar

For double lacing

Assuming n=2

$$F = \frac{V}{2n} Co\sec\theta = \frac{30}{2x2} Co\sec 45^0 = 10.6KN$$

n= number of parallel systems

14. Force in rivet

Force on rivet = 2F sin ϴ = 2x10.6 x sin 45^0=15 KN

Find rivet value

Least of P_s and P_b

Nominal Diameter=d=22 mm
Gross Diameter of rivet =D= 22 +1.5 = 23.5 mm
Assuming power driven shop rivets (PDS) (P No:95 Tno:8.1)
τ_{VF}= 100 N/mm^2
σ_{bf}= 300 N/mm^2

$$P_S = \frac{\Pi}{4} \text{ x } D^2 \text{ X } \tau_{Vf}$$

$$P_S = \frac{\Pi}{4} \text{ x } 23.5^2 \text{ X } 100 \qquad = 43.373 \text{ x } 10^3 N = 43.373KN$$

$$P_b = \text{D x t X } \sigma_{bf}$$

$$P_b = 23.5\text{x } 10 \text{ X } 300 = 70.5 \text{ x } 10^3 N = 70.5KN$$

Rivet value= least of Ps and Pb
Rivet value= 43.373 KN

Rivet value > Force on rivet (ok)
43.373 KN > 15 KN

15. Check the strength of lacing bar in compression (For lacing bar)

$\lambda = 83.93$

λ	σac
80	101
83.93	?
90	90

By interpolation

$$\sigma ac = 101 + \left[\frac{(90-101)}{(90-80)} \text{ X } (83.93\text{-}80) \right] = 96.677 \text{ N/mm}^2$$

$$P_c = \text{ A x } \sigma_{ac}$$

P_c =(70 x 8) x 96.677 = 54.139 x 10^3 N

P_c = 54.139 KN > 10.6KN , SAFE (OK)

Check for tension

$P_t = (b - \text{ D}) \text{ X tx } \sigma_{at}$

$P_t = (70 - 23.5) \text{ X } 8\text{x } 150$

$P_t = 55.8x10^3 N = 55.8KN > 10.6 \text{ KN (OK)}$

Design of Batten

The batten plates are use in lateral system; the batten plates are also called tie plate. The column should not be battened when it is subjected to an eccentric load. Number of battens should be such that the column is divided not less than 3 bays within actual length from the centre to centre of connection

Compression members composed of two main components battened should be preferably have the individual members of the same cross section and symmetrically disposed about their major axis. Where practicable, the compression members should have a radius of gyration about the axis perpendicular to the plane of the batten not less than the radius of gyration about the axis parallel to the plane of batten (see Fig. 1).

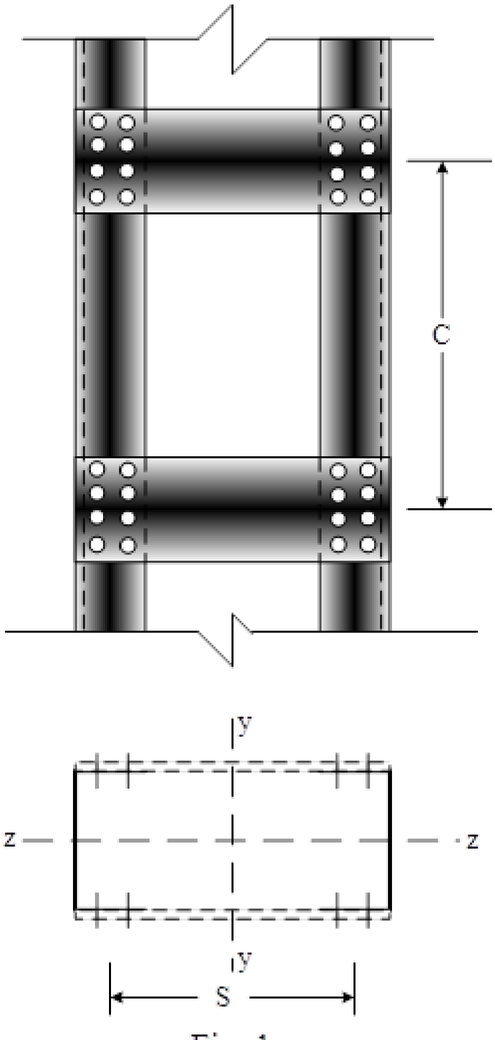


Fig. 1

Design Procedure:

1. In case of battened column effective length increased by 10%
   l= 1.1 X Effective length
2. Compression member having 2 components

$r_{xx} < r_{yy}$

3. Spacing of batten

   IS Code 800-1984, Page Number 51, Clause 5.7.6

   $\lambda = \frac{C}{r_{\min}} < 0.7\lambda$ of whole column

   or

   50 (Which is less)

4. Effective depth of end batten

   Effective depth= d= S+2Cyy > 2 X b_f

   Where S=Spacing of channel

   Cyy= Distance of CG from back of channel

   b_f= Width of flange

   Overall depth = d+ (2 X Edge Distance)

5. For intermediate batten

   Effective depth = (3/4) x Effective depth of end batten > 2 X bf

6. Thickness of batten plate

   $t = \frac{1}{50}$ x Distance *between innermost rivet*

7. Length of batten plate= S+2 bf

8. Transverse shear (IS 800-1984, Page Number 50 , Clause No 5.7.2.1)

   V=2.5 % of total load

   $V = \frac{2.5}{100}$ X P

9. Longitudinal shear: (IS 800-1984, Page Number 53 , Clause No 5.8.2.1)

   Battens shall be plates, angles, channels, or I-sections and at their ends shall be riveted, bolted, or welded to the main components to resist simultaneously a shear

   V_1 = VC/N S_1 along the column axis and a moment M = V1C/2N at each connection,

   Where

   V_1 = transverse shear force as defined above

   C = centre to centre longitudinal distance of battens

   N = number of parallel plane of battens

   S_1 = minimum transverse distance between the centroid of connection of the battens to the main members.

10. Moment: (IS 800-1984, Page Number 53, Clause No 5.8.2.1)

    M= (VC/2N)

11. Check for bending stress

    $\sigma_{cal} = \frac{6M}{tD^2} < 0.66Fy$

12. Check for shear stress

$$\tau_{cal} = \frac{V_1}{tD} < 0.4Fy$$

13. Design of connection
    Find rivet value and number of rivets

8. Design a built-up column having effective length of 10 m. Axial load 750 KN. Use 2 channel back to back. Design batten also

Solution:

Given Data

P= 750 KN = 750 x 10^3 N

Effective length = L_{eff}=10 m=10000 mm

For batten column l= 1.1 x 10000= 11000 mm

Assuming λ = 80 for channel section

σac = 101 N/mm^2 (From IS CODE) Page No: 39 Table No:5.1

Fy = 250 N/mm^2

1.Find required area

$$A = \frac{P}{\sigma_{ac}} = \frac{750X10^3}{101} = 7425.74\ mm^2$$

2.Select Suitable section from steel table (P. No 98)

Try 2 ISMC 300@ 71.6 kg/m

A=9128 mm^2

rxx=118.1 mm

ryy= 126.3 mm (Back to back spacing 200 mm)

NOTE: Select suitable ryy closer to rxx and from Steel table P. No 98, get spacing

rmin= 118.1 mm

3.To find slenderness ratio

$$\lambda = \frac{l_{eff}}{r_{\min}} = \frac{11000}{118.1} = 93.14$$

12. From IS 800-1984, Page No: 39 Table No:5.1, Fy= 250 N/mm^2

λ	σac
90	90
93.14	?
100	80

By interpolation

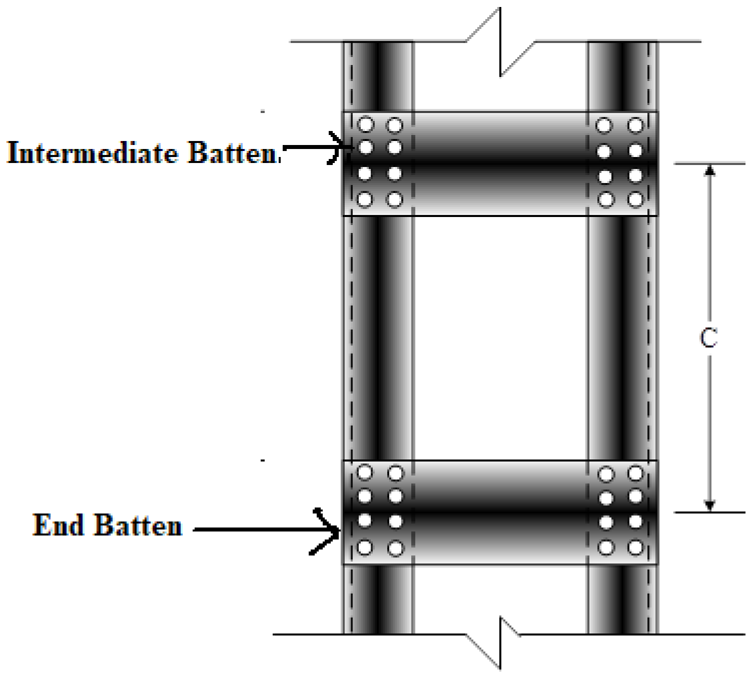
$$\sigma ac = 90 + \left[\frac{(80-90)}{(100-90)} \text{ X (93.14-90)} \right] = 86.86 \text{ N/mm}^2$$

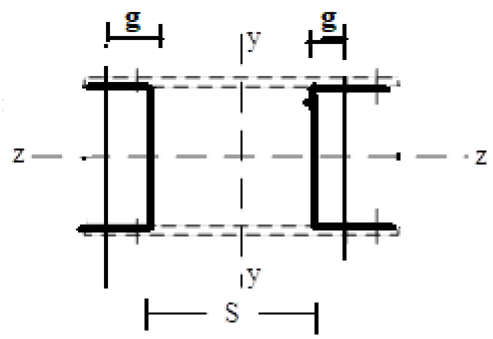
13. Load carrying capacity

$$P_c = \text{A x } \sigma_{ac}$$

P_c =9128 x 86.86= 792.85 x 10^3 N

P_c = 792.85 KN > 750 KN, SAFE (OK)





3. For single ISMC-300

g= 50 mm, rxx= 118.1 mm, ryy= 26.1 mm

r_{min}=26.1 mm

C_{yy}= 23.6 mm

bf=90 mm

Spacing of batten

$\frac{C}{r_{\min}} < 0.7\lambda$ of whole column

or

50 (Which is less)

$\frac{C}{26.1} < 0.7x93.14 = 65.198$ or 50 (Which is less)

$\frac{C}{26.1} < 50$

C=1305 mm

$Assu\min g = C = 1000mm$

$Number$ of batten plate $= \frac{length}{spacing} + 1$

$Number$ of batten plate $= \frac{10000}{1000} + 1 = 11 > 4(ok)$

4.Effective depth of end batten

Effective depth= d= S+2Cyy > 2 X b_f
Where S=Spacing of channel
Cyy= Distance of CG from back of channel
b_f= Width of flange
Effective depth= d= 200+2x23.6 > 2 X90=180 mm
d= 247.2 mm > 180 mm (ok)

Overall depth = d+ (2 X Edge Distance)
Assuming 22 mm diameter of rivet
Edge distance= 40 mm (P No:97)
Overall depth = 247.2+ (2 X 40) =327.2 mm $\cong 330mm$

Providing overall depth of end batten =330 mm

5.For intermediate batten
Effective depth = (3/4) x Effective depth of end batten > 2 X bf
Effective depth = (3/4) x 247.2> 2 X 90
Effective depth = 185.4 mm> 180 mm (ok)
Overall depth = d+ (2 X Edge Distance)
Overall depth = 185.4+ (2 X40) =265.5 mm $\cong 270mm$

Providing overall depth of intermediate batten =270 mm

6.Thickness of batten plate

$$t = \frac{1}{50} \text{ x Distance } between\ innermost\ rivet$$
$$t = \frac{1}{50} \text{ x } [200 + (2x50)]$$
$$t = 6mm$$

Assuming = t =8 mm

Providing thickness of end and intermediate batten = 8 mm

7. Length of batten plate= S+2 bf
   Length of batten plate= 200+(2x90)=380 mm

8.Transverse shear (IS 800-1984, Page Number 50, Clause No 5.7.2.1)

V=2.5 % of total load

$$V = \frac{2.5}{100} \text{ X P=} \frac{2.5}{100} \text{ X 750=18.75 KN}$$

9.Longitudinal shear: (IS 800-1984, Page Number 53 , Clause No 5.8.2.1)

$$V_1 = \frac{VC}{NS_1}$$

Where $S_1 =$ Distance between cg of rivet= 200+(2x50)=300 mm

N= Number of parallel system = 2

$$V_1 = \frac{VC}{NS_1} = \frac{18.75x1000}{2x300} = 31.25KN$$

10.Moment: (IS 800-1984, Page Number 53, Clause No 5.8.2.1)

M= (VC/2N)= (18.75 X1000)/(2X2)=4687.5 KNmm=4687.5 x 10^3 Nmm

11.Check for bending stress for intermediate batten

$$\sigma_{cal} = \frac{6M}{tD^2} < 0.66Fy$$

$$\sigma_{cal} = \frac{6x4687.5x10^3}{8x270^2} < 0.66x250$$
$$\sigma_{cal} = 48.2N/mm^2 < 165N/mm^2 (ok)$$

12.Check for shear stress

$$\tau_{cal} = \frac{V_1}{tD} < 0.4Fy$$

$$\tau_{cal} = \frac{31.25x10^3}{8x270} < 0.4 \text{ x } 250$$

$$\tau_{cal} = 14.6N / mm^2 < 100N / mm^2 (ok)$$

13.Design of connection

Nominal Diameter=d= = 22 mm
Gross Diameter of rivet =D= 22 +1.5 = 23.5 mm
Assuming power driven shop rivets (PDS)
τ_{VF}= 100 N/mm^2
σ_{bf}= 300 N/mm^2

$$P_S = \frac{\Pi}{4} \text{ x } D^2 \text{ X } \tau_{Vf}$$

$$P_S = \frac{\Pi}{4} \text{ x } 23.5^2 \text{ X } 100 = 43.37 \text{ x } 10^3 N = 43.37KN$$

$$P_b = D \text{ x t X } \sigma_{bf}$$

$$P_b = 23.5 \text{ x } 8 \text{ X } 300 = 56.4 \text{ x } 10^3 N = 56.4KN$$

Rivet value= least of Ps and Pb
Rivet value= 43.37 KN

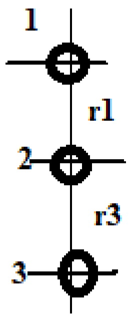
$$N = \frac{LongitudinalShear}{Rivet \text{ Value}} = \frac{31.25}{43.37} = 0.7 \cong 3$$

Poviding 3 rivet to account bending moment

Pitch =[270-(40x2)]/2=95 mm

<u>Step 1</u>: Direct Force $F_1 = \frac{Total \text{ Load}}{Number \text{ of } Rivet} = \frac{P}{N} = \frac{31.25\text{x } 10^3}{3} = 10.41KN$

<u>Step 2</u>: Bending Force



$$F_2 = \frac{M\ r_1}{\sum r^2}$$

r_1 = Distance of critical rivet from the C.G. of rivet=95 mm

r_2 =0 mm

$$\sum r^2 = r_1^2 + r_2^2 + r_3^2 = (95)^2 + (0)^2 + (95)^2 = 18050\ mm^2$$

$$F_2 = \frac{M\ r_1}{\sum r^2} = \frac{4687.5 \text{ x } 10^3 \text{ x } 95}{18.05 \text{ x } 10^3} = 24.67 \text{ x } 10^3 N$$

<u>Step 3</u>: Resultant Force:

$$F_R = \sqrt{F_1^2 + F_2^2 + 2F_1F_2 \cos\theta} = \sqrt{(10.41x10^3\ )^2 + (24.67x10^3)^2 + 2 \text{ x } 10.41\text{x}10^3 \text{ x } 24.67\text{x}10^3 \text{ x cos } 90^0}$$

$F_R =$ 26.7 x 10^3 N =26.7 KN

Rivet value > Fr

43.37>26.7 (ok)

9. A column has a length of 6 m. It support an axial load of 2000 KN. Design a compound section the column consisting of two I Section battened together and also design batten.

Solution:

Given Data

P= 2000 KN = 2000 x 10^3 N

Effective length = L_{eff}=6 m=6000 mm

For batten column l= 1.1 x 6000= 6600 mm

Assuming λ = 80 for I section

σac = 101 N/mm^2 (From IS CODE) Page No: 39 Table No:5.1

Fy = 250 N/mm^2

1.Find required area

$$A = \frac{P}{\sigma_{ac}} = \frac{2000X10^3}{101} = 19801.98\ mm^2$$

2.Select Suitable section from steel table (P. No 98)

Try 2 ISHB 350@ 144.8 kg/m

A=18442 mm^2

rxx=146.5 mm

ryy= 147.1 mm (Back to back spacing 275 mm)

NOTE: Select suitable ryy closer to rxx and from Steel table P. No 98, get spacing

rmin= 146.5 mm

3.To find slenderness ratio

$$\lambda = \frac{l_{eff}}{r_{\min}} = \frac{6600}{146.5} = 45.05$$

4.From IS 800-1984, Page No: 39 Table No:5.1, Fy= 250 N/mm^2

λ	σac
40	139
45.05	?
50	132

By interpolation

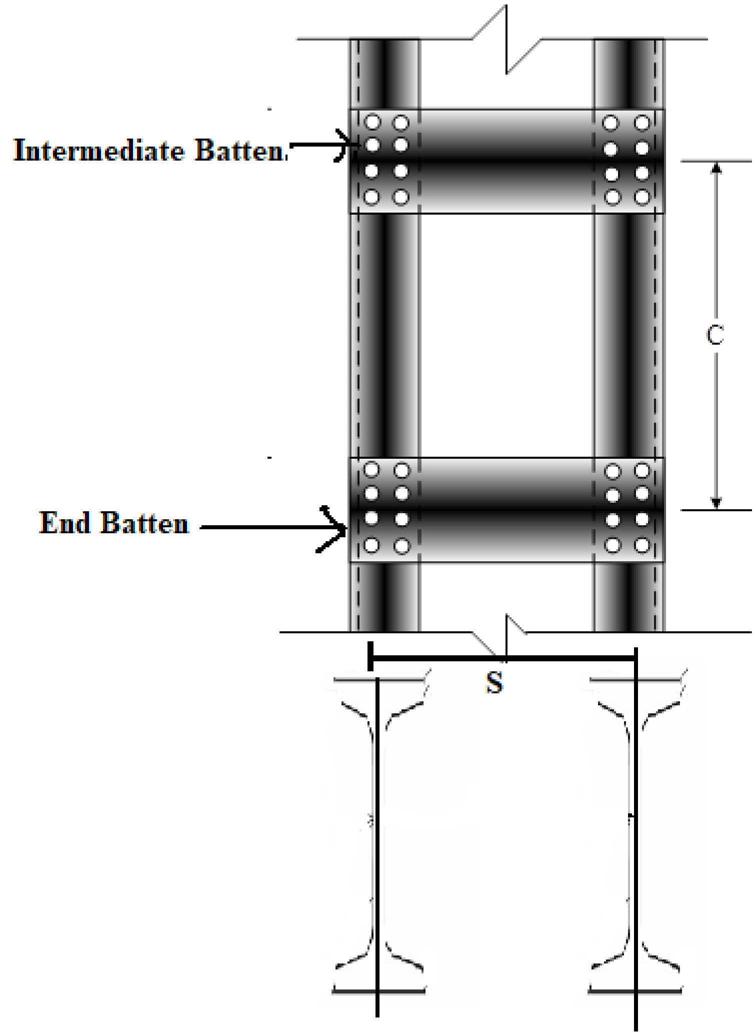
$$\sigma ac \; = 139 + \left[\frac{(132 - 139)}{(50 - 40)} \text{ X (45.05-40)} \right] = 135.46 \text{ N/mm}^2$$

5.Load carrying capacity

$$P_c = \text{ A x } \sigma_{ac}$$

P_c =18442 x 135.46= 2498.24 x 10^3 N

P_c = 2498.24 KN > 2000 KN, SAFE (OK)



3.For single ISHB-350

g= 140 mm

r_{min}=52.2 mm

bf=250 mm

Spacing of batten

$$\frac{C}{r_{\min}} < 0.7\lambda \text{ of whole column}$$

or

50 (Which is less)

$$\frac{C}{52.2} < 0.7x45.05 = 31.53 \text{ or } 50 \text{ (Which is less)}$$

$$\frac{C}{52.2} < 31.53$$

C=1646.12 mm

$Assu\min g = C = 1200mm$

$Number$ of batten plate $= \frac{length}{spacing} + 1$

$Number$ of batten plate $= \frac{6000}{1200} + 1 = 6 > 4(ok)$

4.Effective depth of end batten

Effective depth= d= S+2Cyy > 2 X b_f
Where S=Spacing of channel
Cyy= Distance of CG from back of channel
b_f= Width of flange
Effective depth= d= 275> 2 X250=500 mm
d= 275 mm ,< 500 mm (ok)
Providing Effective depth= d= 500 mm

Overall depth = d+ (2 X Edge Distance)
Assuming 22 mm diameter of rivet
Edge distance= 40 mm (P No:97)
Overall depth = 500+ (2 X 40) =580 mm

Providing overall depth of end batten =580 mm

5.For intermediate batten
Effective depth = (3/4) x Effective depth of end batten > 2 X bf
Effective depth = (3/4) x 500> 2 X 250
Effective depth = 375 mm<500 mm (Ok)
Provide Effective depth= 500 mm
Overall depth = d+ (2 X Edge Distance)
Overall depth = 500+ (2 X40) =580 mm

Providing overall depth of intermediate batten =580mm

6.Thickness of batten plate

$t = \frac{1}{50}$ x Distance $between\ innermost\ rivet$

$$t = \frac{1}{50} \text{ x } [275 - \frac{140}{2} - \frac{140}{2}]$$
$$t = 2.75mm$$

Assuming = t =6 mm

Providing thickness of end and intermediate batten = 6 mm

8. Length of batten plate= S+ bf/2+ bf/2
   Length of batten plate= 275+(250/2)+(250/2)= 525 mm

8.Transverse shear (IS 800-1984, Page Number 50 , Clause No 5.7.2.1)

V=2.5 % of total load

$$V = \frac{2.5}{100} \text{ X P=} \frac{2.5}{100} \text{ X 2000=50 KN}$$

9.Longitudinal shear: (IS 800-1984, Page Number 53 , Clause No 5.8.2.1)

$$V_1 = \frac{VC}{NS_1}$$

Where $S_1 =$ Distance between cg of rivet= 275 mm

N= Number of parallel system = 2

$$V_1 = \frac{VC}{NS_1} = \frac{50x1200}{2x275} = 109.09KN$$

10.Moment: (IS 800-1984, Page Number 53, Clause No 5.8.2.1)

M= (VC/2N)= (50 x 10^3 X1200)/(2X2)= 15000 x 10^3 Nmm

11.Check for bending stress for intermediate batten

$$\sigma_{cal} = \frac{6M}{tD^2} < 0.66Fy$$

$$\sigma_{cal} = \frac{6x15000x10^3}{6x580^2} < 0.66x250$$

$$\sigma_{cal} = 44.58N/mm^2 < 165N/mm^2 (ok)$$

12.Check for shear stress

$$\tau_{cal} = \frac{V_1}{tD} < 0.4Fy$$

$$\tau_{cal} = \frac{109.09x10^3}{6x580} < 0.4\,x\,250$$

$$\tau_{cal} = 31.35N/mm^2 < 100N/mm^2 (ok)$$

13.Design of connection

Nominal Diameter=d= = 22 mm
Gross Diameter of rivet =D= 22 +1.5 = 23.5 mm
Assuming power driven shop rivets (PDS)
τ_{VF}= 100 N/mm^2
σ_{bf}= 300 N/mm^2

$$P_S = \frac{\Pi}{4}\text{ x D}^2\text{ X }\tau_{Vf}$$

$$P_S = \frac{\Pi}{4}\text{ x }23.5^2\text{ X }100 = 43.37\text{ x }10^3 N = 43.37KN$$

$$P_b = \text{D x t X }\sigma_{bf}$$

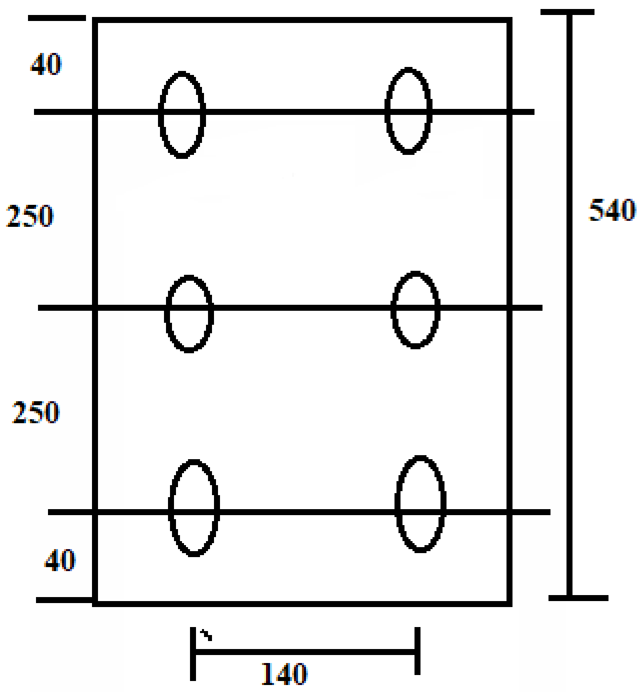
$$P_b = 23.5\text{ x }6\text{ X }300 = 42.31\text{ x }10^3 N = 42.31KN$$

Rivet value= least of Ps and Pb
Rivet value= 42.31 KN

$$N = \frac{LongitudinalShear}{Rivet\ \text{Value}} = \frac{109.09}{42.31} = 2.58 \cong 6$$

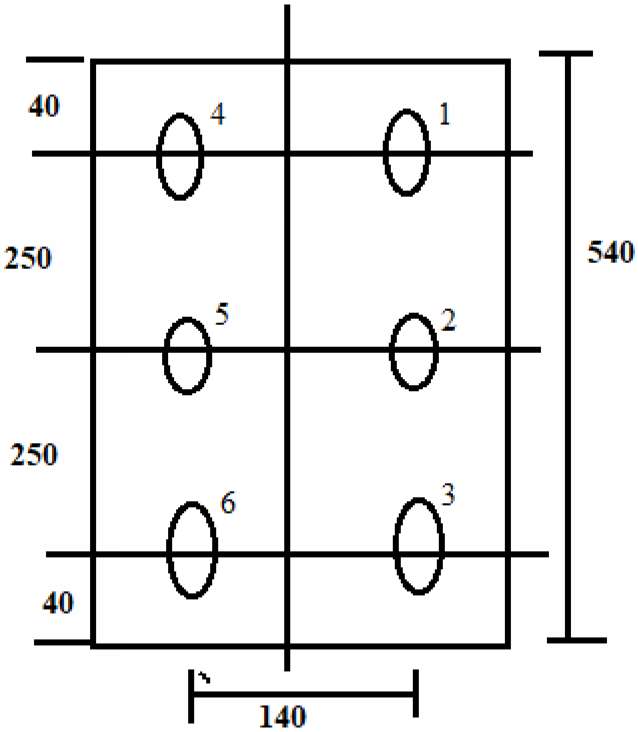
Providing 6 rivets, 3 rivet in each row

Pitch =[580-(40x2)]/2=250mm



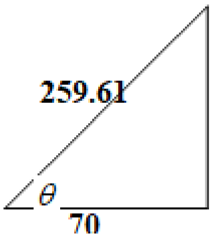
Step 1: Direct Force $F_1 = \dfrac{Total\ \text{Load}}{Number\ \text{of}\ Rivet} = \dfrac{P}{N} = \dfrac{109.09\text{x } 10^3}{3} = 18.18KN$

Step 2: Bending Force



$F_2 = \dfrac{M\ r_1}{\sum r^2}$

r_1 = Distance of critical rivet from the C.G. of rivet



$r_1 = \sqrt{70^2 + 250^2} = 259.61mm$

$r_1 = r_3 = r_4 = r_6 = 259.61\text{mm}$

$r_2 = r_5 = 70mm$

$\sum r^2 = 4xr_1^{\ 2} + 2r_2^{\ 2} = 4\text{x}(259.61)^2 + 2(70)^2 = 279.38 \text{ x } 10^3\ mm^2$

$F_2 = \dfrac{M\ r_1}{\sum r^2} = \dfrac{15000 \text{ x } 10^3 \text{ x}259.61}{279.38 \text{ x } 10^3} = 13.93 \text{ x } 10^3 N$

Cos0= 70/259.61=0.269

Step 3: Resultant Force:

$F_R = \sqrt{F_1^2 + F_2^2 + 2F_1F_2 \text{ Cos}\,\theta} = \sqrt{(18.18x10^3\ )^2 + (13.93\,x\,10^3)^2 + 2 \text{ x } 18.18 \text{ x } 13.93x0.269}$

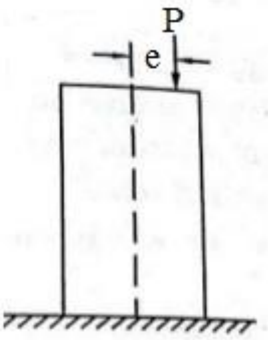
$F_R =$ 25.7 x 10^3 N =25.7 KN

Rivet value > Fr

42.31>25.7 (ok)

Eccentrically loaded column

When the load is applied at eccentric distance from centre of column, the column is called eccentrically loaded column. These column are subjected to bending moment along with axial load, then the strength of column is reduced.



Design procedure

10. Assuming λ = 80 for Fy= 250 N/mm^2
11. Find required area

$$A = \frac{P}{\sigma_{ac}}$$

12. Increase the area 1.5 to 2. Times
13. Try the section from steel table
14. To find slenderness ratio

$$\lambda = \frac{l_{eff}}{r_{\min}}$$

15. To find 6ac
16. Find load carrying capacity

$$P_c = \text{A x } \sigma_{\text{ac}} > P$$

17. Check

$$\frac{\sigma_{ac(cal)}}{\sigma_{ac}} + \frac{\sigma_{bcx(cal)}}{\sigma_{bc}} + \frac{\sigma_{bcy(cal)}}{\sigma_{bc}} \leq 1$$

$$Where\ \sigma_{ac(cal)} = \frac{Total\ \text{vertical load}}{Sectional\ \text{Area}}$$

$$\sigma_{bcx(cal)} = \frac{Mxx}{Zxx} = \frac{Pe_x}{Zxx}$$

$$\sigma_{bcy(cal)} = \frac{Myy}{Zyy} = \frac{Pe_y}{Zyy}$$

1. Design a column section to carry an axial load of 1000 KN with moment of 40 KNm about major axis, effective length of column is 3.5 m use Fy= 250 N/mm^2

Solution:

Given Data

P= 1000 KN = 1000x 10^3 N

Effective length = L_{eff}=3.5 m=3500 mm

Moment about major axis

M_{xx}= 40KNm=40x 10^6Nmm

1. Assuming λ = 80

σac = 101 N/mm^2 (From IS CODE) Page No: 39 Table No:5.1

Fy = 250 N/mm^2

2. Find required area

$$A = \frac{P}{\sigma_{ac}} = \frac{1000X10^3}{101} = 9900.99\ mm^2$$

3. Increase the area by 1.6 Times

   A=1.6 X 9900.99= 15841.58 mm^2

4 .Select Suitable section from steel table (P. No 98)

Try ISWB 600@ 133.7 kg/m

A=17038 mm^2

rxx=249.7 mm

ryy= 52.5 mm

rmin= 52.5 mm

Zxx=3540 x 10^3 mm^3

5.To find slenderness ratio

$$\lambda = \frac{l_{eff}}{r_{\min}} = \frac{3500}{52.5} = 66.67$$

6.From IS 800-1984, Page No: 39 Table No:5.1, Fy= 250 N/mm^2

λ	σac
60	122
66.67	?
70	112

By interpolation

$$\sigma ac\ = 122 + \left[\frac{(112-122)}{(70-60)} \text{ X (66.67-60)} \right] = 115.33 \text{ N/mm}^2$$

7.Load carrying capacity

$$P_c = \text{ A x } \sigma_{ac}$$

P_c =17038 x 115.33= 1964.6 x 10^3 N

P_c = 1964.6 KN > 1000 KN, SAFE (OK)

8.Check

$$\frac{\sigma_{ac(cal)}}{\sigma_{ac}}+\frac{\sigma_{bcx(cal)}}{\sigma_{bc}}\leq 1$$

$$Where\ \sigma_{ac(cal)}=\frac{Total\ \text{vertical load}}{Sectional\ \text{Area}}=\frac{1000x10^3}{17038}=58.69N/mm^2$$

$$\sigma_{bcx(cal)}=\frac{Mxx}{Zxx}=\frac{Pe_x}{Zxx}=\frac{40x10^6}{3540x10^3}=11.29N/mm^2$$

$$\sigma_{bc}=0.66fy=0.66x250=165N/mm^2$$

$$\frac{58.69}{115.33}+\frac{11.29}{165}\leq 1$$

$0.57\leq 1$ (ok)

2. Design a column section to carry an axial load of 800 KN with moment of 40 KNm about major axis, effective length of column is 4 m use Fy= 250 N/mm^2

Solution:

Given Data

P= 800 KN = 800x 10^3 N

Effective length = L_{eff}=4 m=4000 mm

Moment about major axis

Mxx= 40KNm=40x 10^6Nmm

1.Assuming λ = 80

σac = 101 N/mm^2 (From IS CODE) Page No: 39 Table No:5.1

Fy = 250 N/mm^2

2.Find required area

$$A=\frac{P}{\sigma_{ac}}=\frac{800X10^3}{101}=7920.79\ mm^2$$

3.Increase the area by 1.6 Times

A=1.6 X 7920.79= 12673.24 mm^2

4 .Select Suitable section from steel table (P. No 98)

Try ISWB 500@ 95.2 kg/m

A=12122 mm^2

rxx=207.7 mm

ryy= 49.6 mm

rmin= 49.6 mm

Zxx=2091.6 x 10^3 mm^3

5.To find slenderness ratio

$$\lambda = \frac{l_{eff}}{r_{\min}} = \frac{4000}{49.6} = 80.64$$

6.From IS 800-1984, Page No: 39 Table No:5.1, Fy= 250 N/mm^2

λ	σac
80	101
80.64	?
90	90

By interpolation

$$\sigma ac \; = 101 + \left[\frac{(90-101)}{(90-80)} \text{ X (80.64-80)} \right] = 100.29 \text{ N/mm}^2$$

7.Load carrying capacity

$$P_c = \text{ A x } \sigma_{\text{ac}}$$

P_c =12122 x 100.29= 1215.78x 10^3 N

P_c = 1215.78 KN > 800 KN, SAFE (OK)

8.Check

$$\frac{\sigma_{ac(cal)}}{\sigma_{ac}} + \frac{\sigma_{bcx(cal)}}{\sigma_{bc}} \le 1$$

$$Where \;\; \sigma_{ac(cal)} = \frac{Total \text{ vertical load}}{Sectional \text{ Area}} = \frac{800x10^3}{12122} = 65.99 N/mm^2$$

$$\sigma_{bcx(cal)} = \frac{Mxx}{Zxx} = \frac{Pe_x}{Zxx} = \frac{40x10^6}{2091.6x10^3} = 19.12 N/mm^2$$

$$\sigma_{bc} = 0.66 fy = 0.66x250 = 165 N/mm^2$$

$$\frac{65.99}{100.29} + \frac{19.12}{165} \le 1$$

$0.77 \le 1$ (ok)

3. A column is 4 m long carries a vertical load of 1000 KN at an eccentricity of 50 mm from its longitudinal axis acting on one of principle axis. Design a si=suitable section assuming both ends are hinged.

Solution:

Given Data

P= 1000 KN = 1000x 10^3 N

Effective length = L_{eff}=4 m=4000 mm (Both ends are hinged)

Eccentricty about X axis= ex= 50 mm

Moment about major axis

Mxx= 1000 x 10^3 x 50=50 x 10^{6}Nmm

1.Assuming λ = 80

σac = 101 N/mm^2 (From IS CODE) Page No: 39 Table No:5.1

Fy = 250 N/mm^2

2.Find required area

$$A = \frac{P}{\sigma_{ac}} = \frac{1000X10^3}{101} = 9900.99\ mm^2$$

3.Increase the area by 1.6 Times

A=1.6 X 9900.99= 15841.58 mm^2

4 .Select Suitable section from steel table (P. No 98)

Try ISWB 550@ 112.5 kg/m
A=14334 mm^2
rxx=228.6 mm
ryy= 51.5 mm
rmin= 51.5 mm
Zxx=2723.9 x 10^3 mm^3

5.To find slenderness ratio

$$\lambda = \frac{l_{eff}}{r_{\min}} = \frac{4000}{51.6} = 78.27$$

6.From IS 800-1984, Page No: 39 Table No:5.1, Fy= 250 N/mm^2

λ	σac
70	112
78.27	?
80	101

By interpolation

$$\sigma ac\ = 112 + \left[\frac{(101-112)}{(80-70)}\ \text{X (78.27-70)} \right] = 102.9\ \text{N/mm}^2$$

7.Load carrying capacity

$$P_c = \text{A x } \sigma_{\text{ac}}$$

P_c =14334 x 102.9= 1475x 10^3 N
P_c = 1475 KN > 1000 KN, SAFE (OK)

8.Check

$$\frac{\sigma_{ac(cal)}}{\sigma_{ac}} + \frac{\sigma_{bcx(cal)}}{\sigma_{bc}} \leq 1$$

$$Where\ \sigma_{ac(cal)} = \frac{Total\ \text{vertical load}}{Sectional\ \text{Area}} = \frac{1000x10^3}{14334} = 102.9 N / mm^2$$

$$\sigma_{bcx(cal)} = \frac{Mxx}{Zxx} = \frac{Pe_x}{Zxx} = \frac{50x10^6}{2723.9x10^3} = 18.36 N / mm^2$$

$$\sigma_{bc} = 0.66 fy = 0.66x250 = 165 N / mm^2$$

$$\frac{69.76}{102.9} + \frac{18.36}{165} \leq 1$$

$0.78 \leq 1$ (ok)

4. Design a column section to carry axial load 1000 KN. It is eccentric to major axis by 60 mm and to the minor axis 40 mm, length of column is 3.6m with both ends are hinged.
   Solution

P= 1000 KN = 1000x 10^3 N
Effective length = L_{eff}=3.6 m=3600 mm (Both ends are hinged)
Eccentricty about X axis= ex= 60 mm
Eccentricty about Y axis= ey= 40 mm
Moment about major axis
Mxx= 1000 x 10^3 x 60=60 x 10^6Nmm
Moment about manorr axis
Mxx= 1000 x 10^3 x 40=40 x 10^6Nmm

1.Assuming λ = 80

σac = 101 N/mm^2 (From IS CODE) Page No: 39 Table No:5.1
Fy = 250 N/mm^2

2.Find required area

$$A = \frac{P}{\sigma_{ac}} = \frac{1000X10^3}{101} = 9900.99\ mm^2$$

3.Increase the area by 1.6 Times

A=1.6 X 9900.99= 15841.58 mm^2

4 .Select Suitable section from steel table (P. No 98)

Try ISWB 600@ 133.7 kg/m
A=17038 mm^2
rxx=249.7 mm
ryy= 52.5 mm

rmin= 52.5 mm
Zxx=3540 x 10^3 mm^3
Zyy=376.2 x 10^3 mm^3

5.To find slenderness ratio

$$\lambda = \frac{l_{eff}}{r_{\min}} = \frac{3600}{52.5} = 68.57$$

6.From IS 800-1984, Page No: 39 Table No:5.1, Fy= 250 N/mm^2

λ	σac
60	122
68.57	?
70	112

By interpolation

$$\sigma ac = 122 + \left[\frac{(112-122)}{(70-60)} \text{ X } (68.57\text{-}60) \right] = 113.43 \text{ N/mm}^2$$

7.Load carrying capacity

$$P_c = \text{A x } \sigma_{ac}$$

P_c =17038 x 113.43= 1932.62x 10^3 N
P_c = 1932.62 KN > 1000 KN, SAFE (OK)

8.Check

$$\frac{\sigma_{ac(cal)}}{\sigma_{ac}} + \frac{\sigma_{bcx(cal)}}{\sigma_{bc}} + \frac{\sigma_{bcy(cal)}}{\sigma_{bc}} \leq 1$$

$$Where\ \sigma_{ac(cal)} = \frac{Total \text{ vertical load}}{Sectional \text{ Area}} = \frac{1000x10^3}{17038} = 58.69 N / mm^2$$

$$\sigma_{bcx(cal)} = \frac{Mxx}{Zxx} = \frac{Pe_x}{Zxx} = \frac{60x10^6}{3540x10^3} = 16.95 N / mm^2$$

$$\sigma_{bcy(cal)} = \frac{Myy}{Zyy} = \frac{Pe_y}{Zyy} = \frac{40x10^6}{3762x10^3} = 106.3 N / mm^2$$

$$\sigma_{bc} = 0.66 fy = 0.66x250 = 165 N / mm^2$$

$$\frac{58.69}{113.43} + \frac{16.95}{165} + \frac{106.3}{165} \leq 1$$

$1.26 > 1$ (unsafe)

Use ISHB-300 (With cover plate 12 mm thick) Page No: 76

A=17625 mm^2
rxx=143.6 mm
ryy= 92.4 mm

rmin= 92.4 mm
Zxx=2242.2 x 10^3 mm^3
Zyy=752.3 x 10^3 mm^3

5.To find slenderness ratio

$$\lambda = \frac{l_{eff}}{r_{\min}} = \frac{3600}{92.4} = 38.96$$

6.From IS 800-1984, Page No: 39 Table No:5.1, Fy= 250 N/mm^2

λ	σac
30	145
38.96	?
40	139

By interpolation

$$\sigma ac \ = 145 + \left[\frac{(139-145)}{(40-30)} \text{ X (38.96-30)} \right] = 139.62 \text{ N/mm}^2$$

7.Load carrying capacity

$$P_c = \text{ A x } \sigma_{ac}$$

P_c =17625 x 139.62= 2460.8x 10^3 N
P_c = 2460.8 KN > 1000 KN, SAFE (OK)

8.Check

$$\frac{\sigma_{ac(cal)}}{\sigma_{ac}} + \frac{\sigma_{bcx(cal)}}{\sigma_{bc}} + \frac{\sigma_{bcy(cal)}}{\sigma_{bc}} \leq 1$$

$$Where \ \sigma_{ac(cal)} = \frac{Total \text{ vertical load}}{Sectional \text{ Area}} = \frac{1000x10^3}{17625} = 56.74 N/mm^2$$

$$\sigma_{bcx(cal)} = \frac{Mxx}{Zxx} = \frac{Pe_x}{Zxx} = \frac{60x10^6}{2242.2x10^3} = 26.76 N/mm^2$$

$$\sigma_{bcy(cal)} = \frac{Myy}{Zyy} = \frac{Pe_y}{Zyy} = \frac{40x10^6}{752.3x10^3} = 53.17 N/mm^2$$

$$\sigma_{bc} = 0.66 fy = 0.66x250 = 165 N/mm^2$$

$$\frac{56.74}{139.6} + \frac{26.76}{165} + \frac{53.17}{165} \leq 1$$

$0.89 \leq 1$ (ok)

5. A column section ISHB-400@77.4 Kg/m effective length is 5.2 m, it carries a load of 250 KN which is eccentric to major axis by 50 mm and minor axis 30 mm. Check safety of section.
   Solution
   ISHB-400@77.4 Kg/m

P= 250 KN = 250x 10^3 N
A=9866 mm^2
Effective length = L_{eff}=5.2 m=5200 mm
Eccentricity about X axis= ex= 50 mm
Eccentricity about Y axis= ey= 30 mm
Moment about major axis
Mxx= 250 x 10^3 x 50=12.5 x 10^6 Nmm
Moment about minor axis
Myy= 250x 10^3 x 30=7.5 x 10^6 Nmm
Zxx=1404.2 x 10^3 mm^3
Zyy=218.3 x 10^3 mm^3
rxx=168.7mm
ryy= 52.6mm

1.To find slenderness ratio

$$\lambda = \frac{l_{eff}}{r_{min}} = \frac{5200}{52.6} = 98.86$$

2.From IS 800-1984, Page No: 39 Table No:5.1, Fy= 250 N/mm^2

λ	σac
90	90
98.86	?
100	80

By interpolation

$$\sigma ac \ = 90 + \left[\frac{(80-90)}{(100-90)} \text{ X (98.86-90)} \right] = 81.15 \text{ N/mm}^2$$

3.Load carrying capacity

$$P_c = \text{ A x } \sigma_{ac}$$

P_c =9866 x 81.15= 800.62x 10^3 N
P_c = 800.62 KN > 250 KN, SAFE (OK)

4.Check

$$\frac{\sigma_{ac(cal)}}{\sigma_{ac}}+\frac{\sigma_{bcx(cal)}}{\sigma_{bc}}+\frac{\sigma_{bcy(cal)}}{\sigma_{bc}}\leq 1$$

$$Where\ \sigma_{ac(cal)}=\frac{Total\ \text{vertical load}}{Sectional\ \text{Area}}=\frac{250x10^3}{9866}=25.34N/mm^2$$

$$\sigma_{bcx(cal)}=\frac{Mxx}{Zxx}=\frac{Pe_x}{Zxx}=\frac{12.5x10^6}{1404.2x10^3}=8.90N/mm^2$$

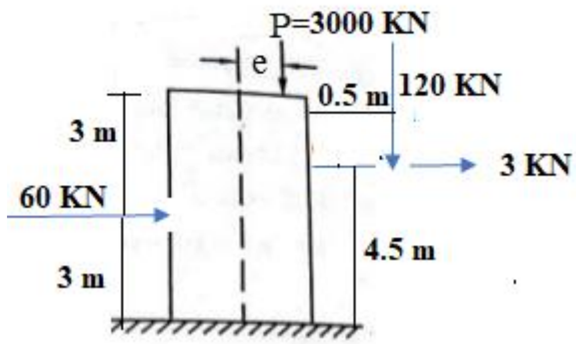
$$\sigma_{bcy(cal)}=\frac{Myy}{Zyy}=\frac{Pe_y}{Zyy}=\frac{7.5x10^6}{218.3x10^3}=34.36N/mm^2$$

$$\sigma_{bc}=0.66fy=0.66x250=165N/mm^2$$

$$\frac{25.34}{81.15}+\frac{8.90}{165}+\frac{34.36}{165}\leq 1$$

$0.57\leq 1$ (safe)

6. Design a section for column supporting loads as shown in figure assuming one end fixed and other hinged.



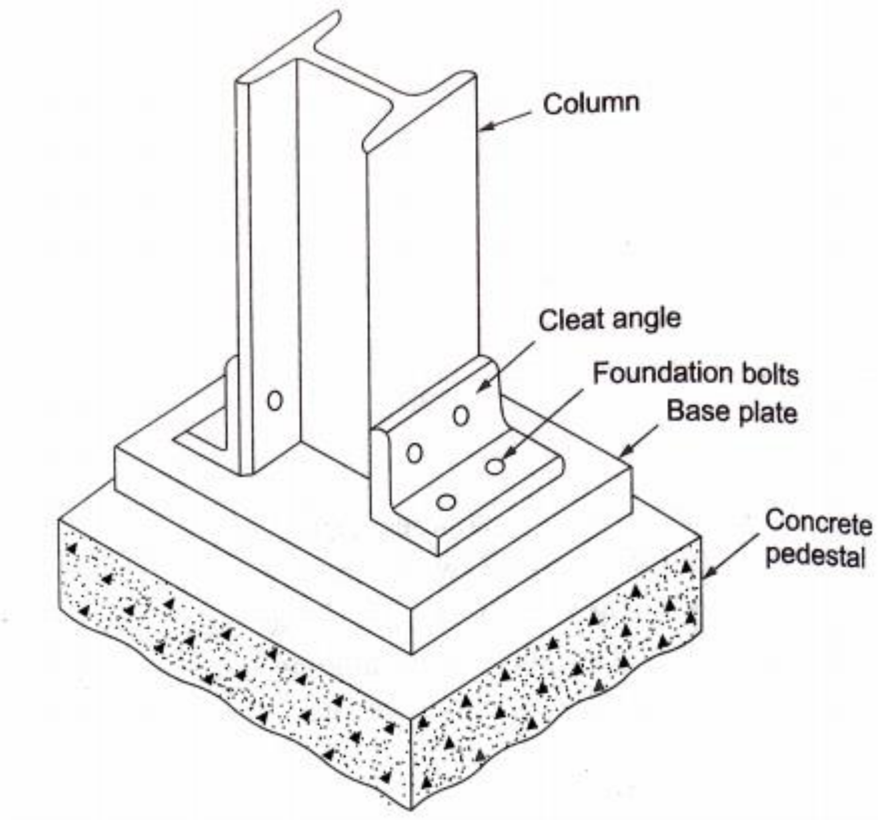
Design of column Base

The column base used to rest the column on concrete base. The function of providing a column base is transfer the load from the column and distribute it uniformally on concrete bed. The column based on concrete which transfers load on soil. The stress in concrete should be within permissible limit.

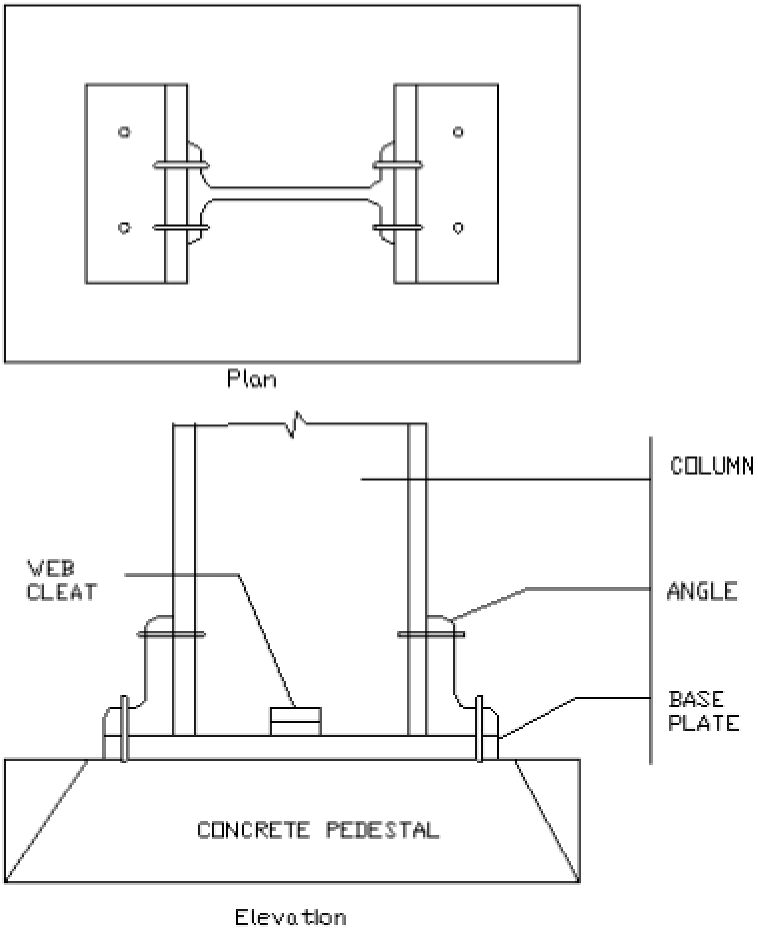
There are two types of column bases

1) Slab Base
2) Gusset Plate

Slab Base: It consist of a base plate under a column so as to have complete bearing on the plate, the column is properly secured to base plate by means of fastening. The fastening is not to designed, these are simply used to secure column with base plate.



Slab Base



Design Procedure

Given Data

P= Axial load on the column

6c= Permissible stress in the concrete

For M_{15} then 6c= 4 N/mm^2

M_{20} then 6c= 5 N/mm^2

1. Area of base plate

$$A = \frac{P}{\sigma_c} = \frac{Load \text{ on column}}{Permisible \text{ stress in concrete}}$$

For rectangular plate = A= Lx B

For square =A= LxL

For Circular plate=A=$\frac{\pi}{4}D^2$

2. Actual bearing pressure on concrete

$$w = \frac{Load \text{ on column}}{Area \text{ of palte provided}} < Permisible \text{ stress in concrete}$$

3. Thickness of the plate (IS 800 Page no 44)

$$t = \sqrt{\frac{3w}{\sigma_{bs}}\left(a^2 - \frac{b^2}{4}\right)}$$

t = thickness of base plate

w= actual pressure on concrete

a=Greater projection of plate beyond column

b=Lesser projection of plate beyond column

$\sigma_{bs} = Permissible$ bending stress in slab base

4. Design of concrete block
   Load on column= P
   Assuming self weight = 10% to 15% of P
   Total load = P+ Self weight
5. Area of concrete block required

$$Area \text{ of concrete block required } A_1 = \frac{\text{Total } Load}{SBC \text{ of soil}}$$

SBC = Safe bearing capacity

For square concrete block $= A_1 = L_1 \text{ X } L_1 \quad (B_1 = L_1)$

For rectangular concrete block $= A_1 = L_1 \text{ X } B_1$

$$\frac{L_1}{B_1} = \frac{L}{B}$$

6. Depth of concrete block
   Assuming angle of dispersion 45^0
   Depth of concrete block = $\frac{L_1 - L}{2}$ or $\frac{B_1 - B}{2}$ (Which is greater)
7. Cleat angle
   Providing nominal size of cleat angle ISA 90X90X6 mm connected with 20 mm diameter of rivet

1.Design a slab base for column consisting of ISHB-300@58.8 kg/m carrying axial load of 800 KN. Assume M15grade of concrete and safe bearing capacity of soil is 150 KN/m^2

Solution
Given Data
ISHB-300@58.8 kg/m
b_f=250 mm
D=300 mm

P=800 KN = 800 x 10^3 N
For M15grade of concrete=6c= 4 N/mm^2
Safe bearing capacity of soil =150 KN/m^2

1.Area of base plate

$$A = \frac{Load \text{ on column}}{Permisible \text{ stress in concrete}} = \frac{P}{\sigma_c} = \frac{800x10^3}{4} = 200x10^3 mm^2$$

$Assu\min g$ square plate

L=B=$\sqrt{200x10^3} = 447.21mm$

Pr$oviding$ L=B=450 mm

2.Actual bearing pressure on concrete

$$w = \frac{Load \text{ on column}}{Area \text{ of plate provided}} < Permisible \text{ stress in concrete}$$

$$w = \frac{800x10^3}{450x450} < 4$$

$$w = 3.95 \text{ N/mm}^2 < 4\text{N/mm}^2 \text{ (OK)}$$

3.Thickness of the plate (IS 800 Page no 44)

$$t = \sqrt{\frac{3w}{\sigma_{bs}}\left(a^2 - \frac{b^2}{4}\right)}$$

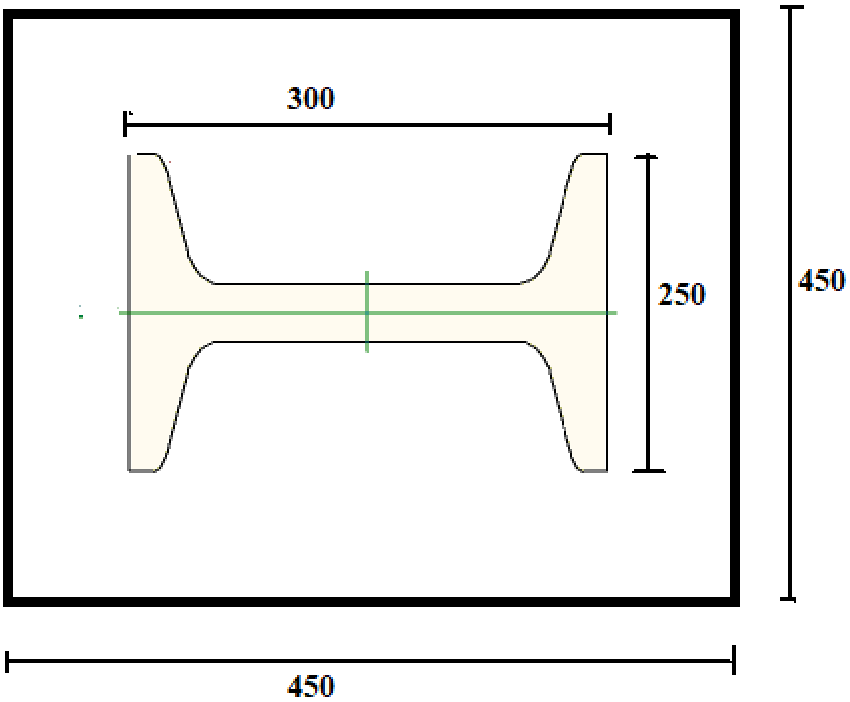
t = thickness of base plate

w= actual pressure on concrete=3.95 N/mm^2

a=Greater projection of plate beyond column

b=Lesser projection of plate beyond column

$\sigma_{bs} = Permissible$ bending stress in slab base=185 N/mm^2



Projection

$$\frac{450-300}{2}=75mm$$

$$\frac{450-250}{2}=100mm$$

Take a = 100 mm (Greater projection)

b = 75 mm (Smaller projection)

$$t=\sqrt{\frac{3x3.95}{185}\left(100^2-\frac{75^2}{4}\right)}=23.46mm\cong 25mm$$

Providing size of base plate = 450 mm X 450 mm X 25 mm

4.Design of concrete block

Load on column= P=800 KN
Assuming self weight = 10% of P
Assuming self weight =(10x800)/100=80 KN
Total load = P+ Self weight
Total load =800+80=880 KN

5.Area of concrete block required

$Area$ of concrete block required $A_1 = \frac{\text{Total } Load}{SBC \text{ of soil}} = \frac{880}{150} = 5.86m^2$

For square concrete block $= A_1 = L_1 \text{ X } L_1 \quad (B_1 = L_1)$

$L_1 = B_1 = \sqrt{5.86} = 2.45m \cong 2.5m$

$L_1 = B_1 = 2500mm$

6.Depth of concrete block

Assuming angle of dispersion 45^0

Depth of concrete block = $\frac{L_1 - L}{2}$ or $\frac{B_1 - B}{2}$ (Which is greater)

$$\frac{2500 - 450}{2} \text{ or } \frac{2500 - 450}{2}$$

Depth of concrete block = 1025 mm or 1025 mm

$\cong$ 1050 mm

Providing Depth of concrete block = 1050 mm

Provided concrete block of size= 2500 mm X 2500 mm X 1050 mm

7.Cleat angle

Providing nominal size of cleat angle ISA 90X90X6 mm connected with 20 mm diameter of rivet

2. A column section ISHB-250 with cover plate 12 mm thick carries load of 700 KN. Design slab base for column, the permissible bearing pressure on concrete is 4 N/mm^2 and safe bearing capacity of soil is 150 KN/m^2

Solution

Given Data

ISHB-250

Thickness of cover plate 12 mm

b_f=250 mm

D=250 mm

P=700 KN = 700 x 10^3 N

6c= 4 N/mm^2

Safe bearing capacity of soil =150 KN/m^2

1.Area of base plate

$$A = \frac{Load \text{ on column}}{Permisible \text{ stress in concrete}} = \frac{P}{\sigma_c} = \frac{700x10^3}{4} = 175x10^3 mm^2$$

$Assu \min g$ square plate

L=B=$\sqrt{175x10^3} = 418.38mm$

$\Pr oviding$ L=B=430 mm

2.Actual bearing pressure on concrete

$$w = \frac{Load \text{ on column}}{Area \text{ of palte provided}} < Permisible \text{ stress in concrete}$$

$$w = \frac{700x10^3}{430x430} < 4$$

$$w = 3.78 \text{ N/mm}^2 < 4\text{N/mm}^2 \text{ (OK)}$$

3.Thickness of the plate (IS 800 Page no 44)

$$t = \sqrt{\frac{3w}{\sigma_{bs}}\left(a^2 - \frac{b^2}{4}\right)}$$

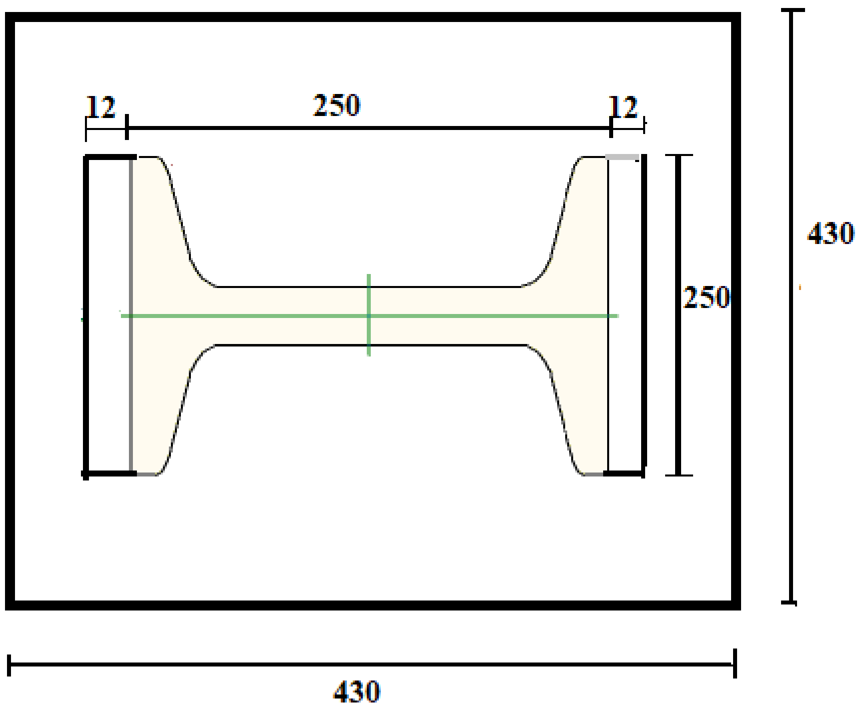
t = thickness of base plate

w= actual pressure on concrete=3.95 N/mm^2

a=Greater projection of plate beyond column

b=Lesser projection of plate beyond column

σ_{bs} = *Permissible* bending stress in slab base=185 N/mm^2



Projection

$$\frac{430-274}{2} = 78mm$$

$$\frac{430-250}{2} = 90mm$$

Take a = 90 mm (Greater projection)

b = 78 mm (Smaller projection)

$$t=\sqrt{\frac{3x3.78}{185}\left(90^2-\frac{78^2}{4}\right)}=20.08mm\cong 25mm$$

Providing size of base plate = 430 mm X 430 mm X 25 mm

4.Design of concrete block

Load on column= P=700 KN
Assuming self weight = 10% of P
Assuming self weight =(10x700)/100=70 KN
Total load = P+ Self weight
Total load =700+70=770 KN

5.Area of concrete block required

$$Area \text{ of concrete block required } \mathrm{A}_1 = \frac{\text{Total } Load}{SBC \text{ of soil}} = \frac{770}{150} = 5.13m^2$$

For square concrete block $=\mathrm{A}_1 = L_1 \text{ X } \mathrm{L}_1 \quad (\mathrm{B}_1 = \mathrm{L}_1)$

$L_1 = \mathrm{B}_1 = \sqrt{5.13} = 2.26m \cong 2.3m$

$L_1 = \mathrm{B}_1 = 2300mm$

6.Depth of concrete block

Assuming angle of dispersion 45^0

Depth of concrete block = $\frac{L_1 \text{ - L}}{2}$ or $\frac{B_1 \text{ - B}}{2}$ (Which is greater)

$\frac{2300 - 430}{2}$ or $\frac{2300 - 430}{2}$

Depth of concrete block = 935 mm or 935 mm

$\cong$ 950 mm

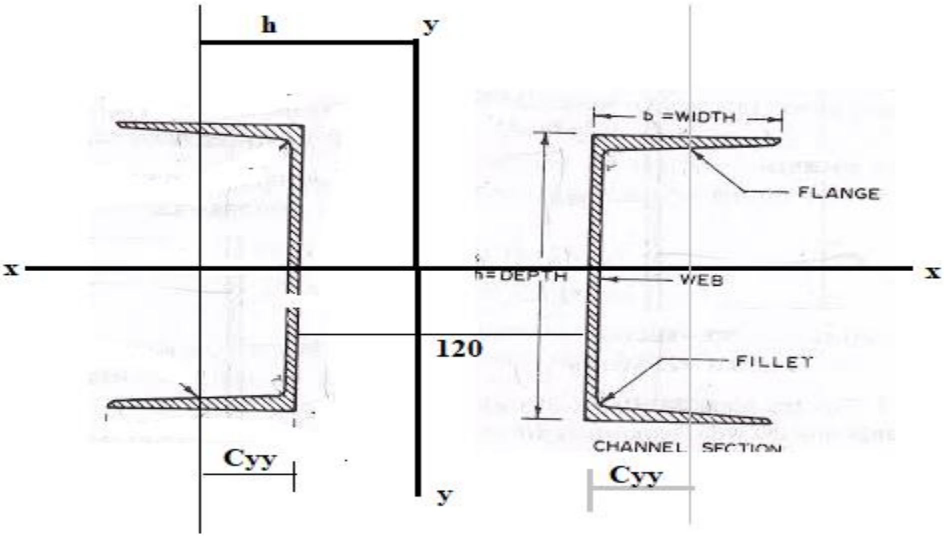
Providing Depth of concrete block = 950mm
Provided concrete block of size= 2300 mm X 2300 mm X 950 mm

7.Cleat angle

Providing nominal size of cleat angle ISA 90X90X6 mm connected with 20 mm diameter of rivet

3. A column consist of two channel ISLC-150 kept back to back at a clear distance of 120 mm and suitably batten, it carries the maximum load over a effective length of 5 m. Design the slab base for the column. Assume the grade of concrete is M15 and safe bearing capacity of soil is 150 KN/m^2

Solution: Given Data



Properties of one ISLC-150

A= 1836 mm^2

Ixx=697.2 X 10^4 mm^4

Iyy=103.2 X 10^4 mm^4

Cyy=23.8 mm

bf= 75 mm

Ixx= 2x [Ixx+Ah^2]   h=0

Ixx= 2 X 697.2 X 10^4=13.94 X 10^4 mm^4

Iyy= 2x [Iyy+Ah^2]   h=[120/2]+23.8=83.8 mm

Iyy= 2x [103.2 X 10^4 +1836 x 83.8 ^{2}]

Iyy= 27.85 x 10^6 mm^4

Imin=13.94 X 10^4 mm^4

$$r_{\min} = \sqrt{\frac{\mathrm{I}_{\mathrm{m}in}}{A}} = \sqrt{\frac{13.94x10^6}{3676}} = 61.61 \text{ mm} \qquad A = 2 \text{ X } 1836 = 3672 \text{ mm}^2$$

$$\lambda = \frac{l_{eff}}{r_{\min}} = \frac{5000}{61.61} = 81.15$$

From IS 800-1984, Page No: 39 Table No:5.1, Fy= 250 N/mm^2

λ	σac
80	101

81.15 ?

90 90

By interpolation

$$\sigma ac = 101 + \left[\frac{(90-101)}{(90-80)} \text{ X } (81.15\text{-}80) \right] = 99.73 \text{ N/mm}^2$$

Load carrying capacity

$$P = A \text{ x } \sigma_{ac}$$

P_c =3672 x 99.73= 366.2 x 10^3 N

P_c = 366.2 KN

Design of Slab base

ISLC-150

D=150 mm

P=366.2 KN = 366.2 x 10^3 N

6c= 4 N/mm^2

Safe bearing capacity of soil =150 KN/m^2

1.Area of base plate

$$A = \frac{Load \text{ on column}}{Permisible \text{ stress in concrete}} = \frac{P}{\sigma_c} = \frac{366.2x10^3}{4} = 91556.73mm^2$$

$Assu\min g$ B=250mm

$$\text{L=} \frac{91556.73}{250} = 366.22mm$$

$\Pr oviding$ L=400 mm

Providing rectangular plate of size

L=400 mm

B=250 mm

2.Actual bearing pressure on concrete

$$w = \frac{\textit{Load} \text{ on column}}{\textit{Area} \text{ of palte provided}} < \textit{Permisible} \text{ stress in concrete}$$

$$w = \frac{366.2x10^3}{250x400} < 4$$

$$w = 3.66 \text{ N/mm}^2 < 4\text{N/mm}^2 \text{ (OK)}$$

3.Thickness of the plate (IS 800 Page no 44)

$$t = \sqrt{\frac{3w}{\sigma_{bs}}\left(a^2 - \frac{b^2}{4}\right)}$$

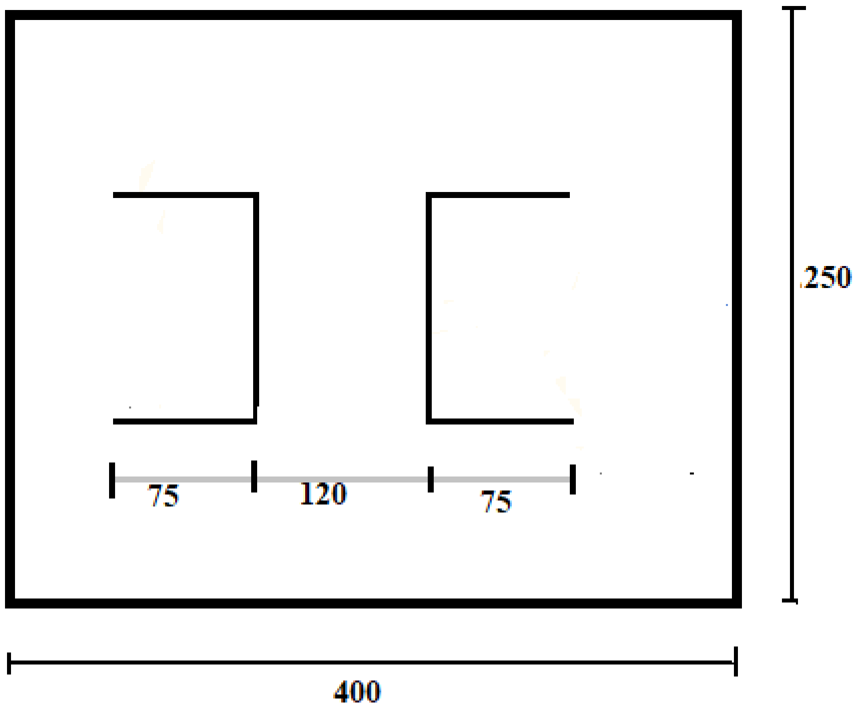
t = thickness of base plate

w= actual pressure on concrete=3.95 N/mm^2

a=Greater projection of plate beyond column

b=Lesser projection of plate beyond column

σ_{bs} = *Permissible* bending stress in slab base=185 N/mm^2



Projection

$$\frac{400-120-75-75}{2} = 65mm$$

$$\frac{250-150}{2} = 50mm$$

Take a = 65 mm (Greater projection)

b = 50 mm (Smaller projection)

$$t=\sqrt{\frac{3x3.66}{185}\left(65^2-\frac{50^2}{4}\right)}=14.61mm\cong 16mm$$

Providing size of base plate = 400 mm X 250 mm X 16 mm

4.Design of concrete block

Load on column= P=366.2 KN
Assuming self weight = 10% of P
Assuming self weight =(10x366.2)/100=36.62 KN
Total load = P+ Self weight
Total load =366.2+36.62=402.82 KN

5.Area of concrete block required

$$Area \text{ of concrete block required } A_1 = \frac{\text{Total } Load}{SBC \text{ of soil}} = \frac{402.82}{150} = 2.68m^2$$

For rectangular concrete block $=A_1 = L_1 \text{ X } B_1$

$$\frac{L_1}{B_1}=\frac{L}{B}=\frac{400}{250}=1.6$$

$L_1 = 1.6B_1$

$L_1 \text{ X } B_1 = 2.68$

$1.6B_1 \text{X } B_1 = 2.68$

$B_1 = 1.29m \cong 1.3m$

$L_1 = 1.6B_1 = 1.6x1.3 = 2.08m \cong 2.1m$

6.Depth of concrete block

Assuming angle of dispersion 45^0

Depth of concrete block = $\frac{L_1 - L}{2}$ or $\frac{B_1 - B}{2}$ (Which is greater)

$\frac{2100 - 400}{2}$ or $\frac{1300 - 250}{2}$

Depth of concrete block = =850 mm or 525 mm
=850
$\cong$ 900 mm

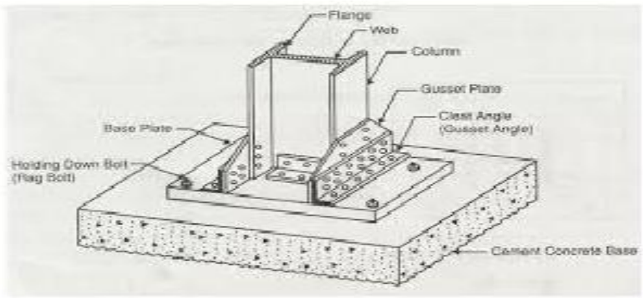
Providing Depth of concrete block = 900mm
Provided concrete block of size= 2100 mm X 1300 mm X 900 mm

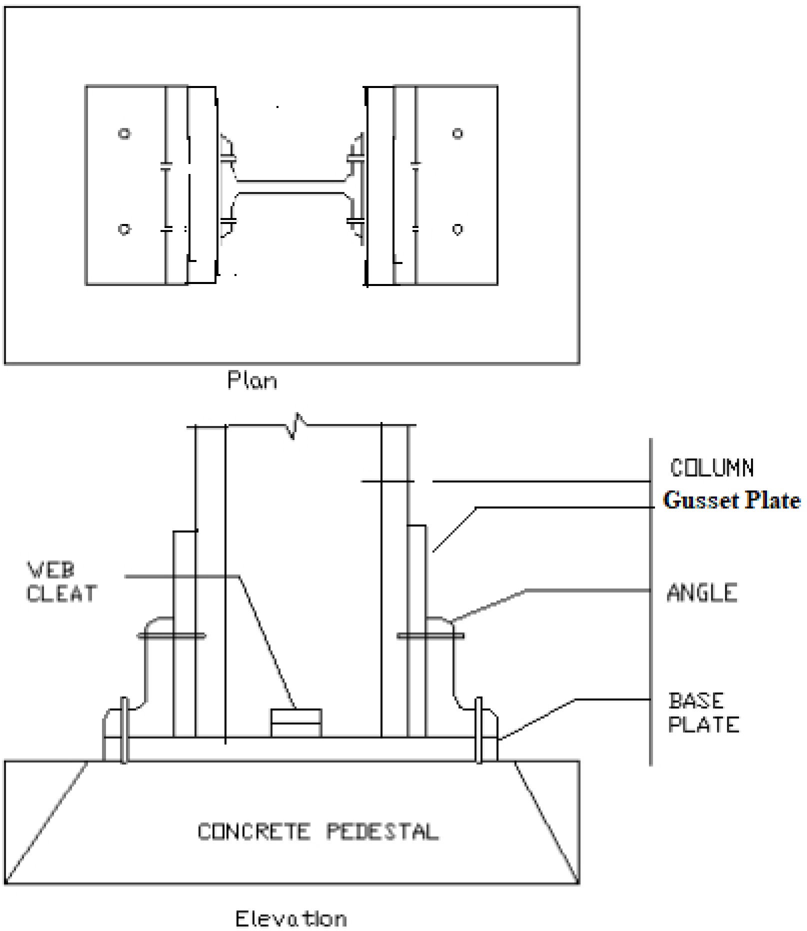
7.Cleat angle

Providing nominal size of cleat angle ISA 90X90X6 mm connected with 20 mm diameter of rivet

Gusseted Base

Gusseted base plates are used in the columns carrying heavy load. In this case fastening are used to connect base plates and column in the form of gusset plate and angle. In case the end of column is properly machined so as to provide full bearing on the base plate along half the column load is transferred to base plate by directly bearing and half load is transferred to gusset plate, if columns are not properly machined , total load is transferred to gusset plate.





Gusset Base

Design Procedure

Given Data

Given Data

P= Axial load on the column

6c= Permissible stress in the concrete

For M_{15} then 6c= 4 N/mm^2

M_{20} then 6c= 5 N/mm^2

1. Area of base plate

$$A = \frac{P}{\sigma_c} = \frac{Load \text{ on column}}{Permisible \text{ stress in concrete}}$$

2. Minimum length of base plate

L= Depth of column section+ (2 X Thickness of gusset plate) +(2 X Leg of angle)

3. Width of base plate

$$B = \frac{A}{L}$$

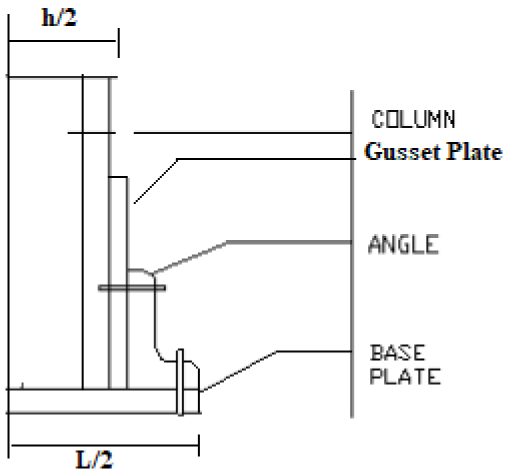
Size of base plate = L X B

4. Actual bearing pressure on concrete

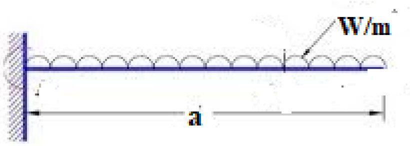
$$w = \frac{Load \text{ on column}}{Area \text{ of palte provided}} < Permisible \text{ stress in concrete}$$

5. Thickness of base plate

Assuming base plate is fixed at vertical leg of angle



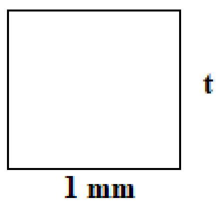
a= L/2 -h/2- thickness of gusset plate- thickness of angle



$$M = \frac{wa^2}{2}$$

Moment of resistance =Mr= $6_{bs} X \ Z$

Consider 1 mm width of base plate



$$M_r = 6_{bs} \text{ x } 1 \text{ x } \frac{t^2}{6}$$

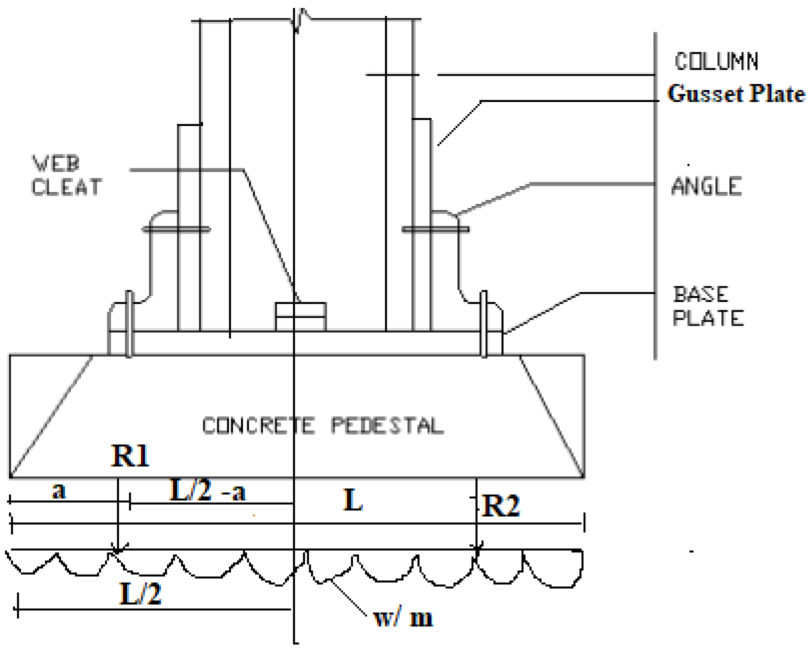
Max BM = Mr

$$\frac{wa^2}{2} = 6_{bs} \text{ x } 1 \text{ x } \frac{t^2}{6}$$

$t = ?$

Thickness of base plate = T =t- Thickness of angle

Assuming base plate is simply supported at the vertical leg of angle



Maximum bending moment at centre

$$Max \text{ BM } = \left(w \text{ x } \frac{L}{2} x \frac{L}{4} \right) - \left(R_1 x \left(\frac{L}{2} - a \right) \right)$$

$$Moment \text{ of resistance =Mr= } 6_{bs} \, X \; Z$$

$$Max \text{ BM} = 6_{bs} \text{ x } 1 \text{ x } \frac{t^2}{6}$$

$t = ?$

6. Design of connection
7. Design of concrete block

1. Design gusseted base for ISHB-350@22.4 Kg/m, the axial load on column is 1000 KN. The safe bearing capacity of soil is 160 KN/m^2. Assume grade of concrete is M_{15}.
Solution : Given Data
ISHB-350@22.4 Kg/m
Axial Load = P= 1000 KN=1000 x 10^3 N
Grade of concrete is M_{15}

For M_{15} then 6c= 4 N/mm^2

SBC Of soil = 160 KN/m^2

1. Area of base plate

$$A=\frac{P}{\sigma_c}=\frac{Load\text{ on column}}{Permisible\text{ stress in concrete}}=\frac{1000\text{ x }10^3}{4}=250x10^3 mm^2$$

2. Minimum length of base plate
L= Depth of column section+ (2 X Thickness of gusset plate) +(2 X Leg of angle)
Assuming thickness of gusseted plate = 16 mm
angle section ISA 150 X 115 X 15 mm
L= Depth of column section+ (2 X Thickness of gusset plate) +(2 X Leg of angle)
L= 350 +(2 x 16) +(2 x 115) = 612 mm
Providing =L =620 mm
3. Width of base plate

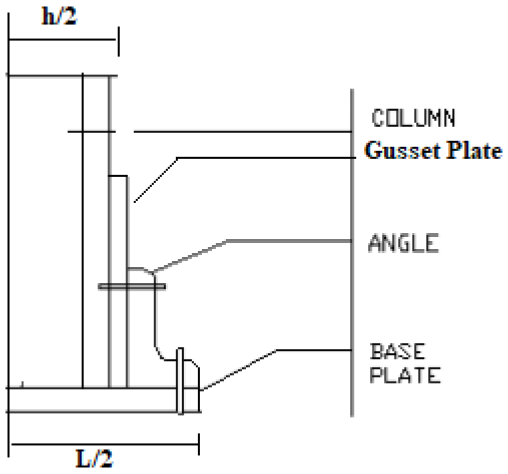
$$B=\frac{A}{L}=\frac{250x10^3}{620}=403.22mm\cong 410mm$$

Providing Size of base plate = 620 mm X 410 mm
4. Actual bearing pressure on concrete

$$w=\frac{Load\text{ on column}}{Area\text{ of palte provided}}<Permisible\text{ stress in concrete}$$

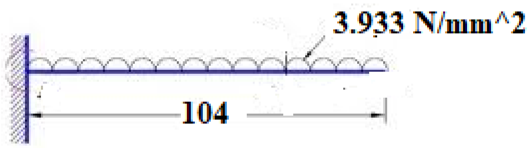
$$w=\frac{P}{LxB}=\frac{1000x10^3}{620x410}=3.933N/mm^2<4N/mm^2 \quad \text{(ok)}$$

5. Thickness of base plate
Assuming base plate is fixed at vertical leg of angle



a= L/2 -h/2- thickness of gusset plate- thickness of angle

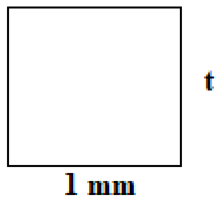
a= 620/2 – 350/2- 16-15 = 104 mm



$$M = \frac{wa^2}{2} = \frac{3.933x104^2}{2} = 21.269x10^3 Nmm$$

Moment of resistance =Mr= $6_{bs} X \; Z$

Consider 1 mm width of base plate



$$M_r = 6_{bs} \text{ x 1 x } \frac{t^2}{6} = 185 \text{ x 1 x } \frac{t^2}{6}$$

Max BM = Mr

$$\frac{wa^2}{2} = 185 \text{ x 1 x } \frac{t^2}{6}$$

$$21.269x10^3 = 185 \text{ x 1 x } \frac{t^2}{6}$$

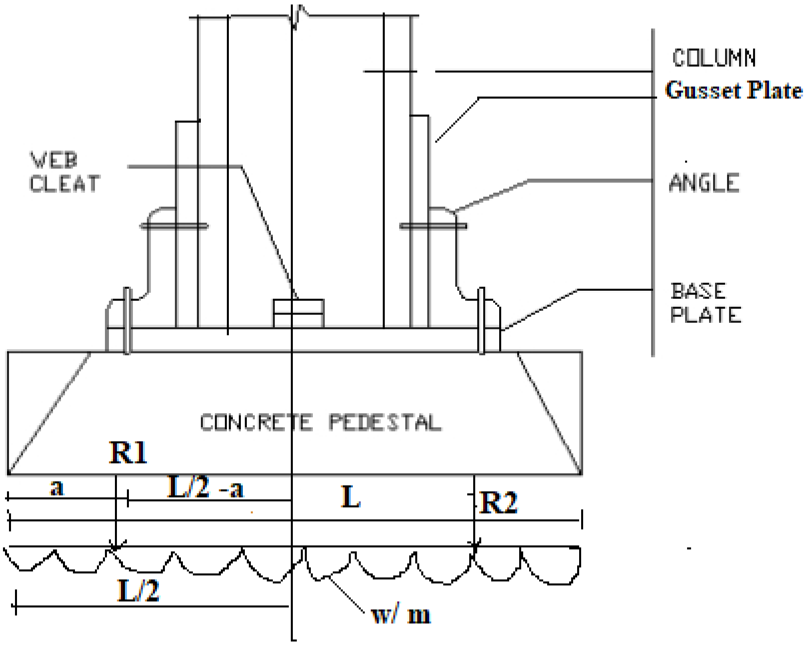
$$t^2 = 689.82$$

$$t = 26.264mm$$

Thickness of base plate = T =t- Thickness of angle

Thickness of base plate = T= 26.264-15=11.264 mm (1)

Assuming base plate is simply supported at the vertical leg of angle



R_1=R_2=WL/2= (3.933 X620) /2=1219.23 N

Maximum bending moment at centre

$$Max\ BM = \left(w \text{ x } \frac{L}{2} x \frac{L}{4}\right) - \left(R_1 x\left(\frac{L}{2} - a\right)\right)$$

$$Max\ BM = \left(3.933 \text{ x } \frac{620}{2} x \frac{620}{4}\right) - \left(1219.23x\left(\frac{620}{2} - 104\right)\right)$$

$$Max\ BM = -62.180x10^3\,Nmm \text{ (Hogging)}$$

$$Moment \text{ of resistance =Mr= } 6_{bs}\,X\ Z$$

$$Max\ BM = 6_{bs} \text{ x 1 x } \frac{t^2}{6}$$

$$Max\ BM = 185\text{x 1 x } \frac{t^2}{6}$$

$$Max\ BM = Mr$$

$$62.180x10^3 = 185\text{x 1 x } \frac{t^2}{6}$$

t= 44.90 mm $\cong$ 45 mm

Taking Greater value of t =45 mm

Providing Size of base plate = 620 mm X 410 mm X 45 mm

6. Design of connection :

Nominal diameter of rivet = $6.04\sqrt{t} = 6.04\sqrt{15} = 23.39mm \cong 24mm$

Providing = d= 24 mm

Gross Diameter of rivet= D =d+1.5 = 24+1.5=25.5 mm

Assuming power driven shop rivets (PDS)

τ_{VF}= 100 N/mm^2

σbf= 300 N/mm^2

$$P_S = 2\frac{\Pi}{4} \text{ x } D^2 \text{ X } \tau_{Vf}$$

$$P_S = 2\frac{\Pi}{4} \text{ x } 25.5^2 \text{ X } 100 = 102.14 \text{ x } 10^3 N = 102.14 KN$$

$$P_b = D \text{ x } t \text{ X } \sigma_{bf}$$

$$P_b = 25.5 \text{ x } 15 \text{ X } 300 = 114.75\text{x } 10^3 N = 114.75 KN$$

Rivet value= least of Ps and Pb

Rivet value= 104.14 KN

$$N = \frac{Load \text{ on one gusset plate}}{Rivet \text{ Value}}$$

Assuming column is not perfectly machined, total load transfer through two gusset plate, load on one gusset plate= P/2= 1000/2= 500 KN

$$N = \frac{Load \text{ on one gusset plate}}{Rivet \text{ Value}} = \frac{500}{102.14} = 4.89 \cong 5$$

7. Design of concrete block

   Load on column= P=1000 KN

   Assuming self weight = 10% of P

   Assuming self weight =(10x1000)/100=100 KN

   Total load = P+ Self weight

   Total load =1000+100=1100 KN

.Area of concrete block required

$$Area \text{ of concrete block required } A_1 = \frac{\text{Total } Load}{SBC \text{ of soil}} = \frac{1100}{160} = 6.875 m^2$$

For rectangular concrete block =$A_1 = L_1 \text{ X } B_1$

$$\frac{L_1}{B_1} = \frac{L}{B} = \frac{620}{410} = 1.512$$

$$L_1 = 1.512 B_1$$

$$L_1 \text{ X } B_1 = 6.875$$

$$1.512 B_1 \text{X } B_1 = 6.875$$

$$B_1 = 2.14 m \cong 2.15 m$$

$$L_1 = 1.512 B_1 = 1.512 x 2.15 = 3.220 m \cong 3.3 m$$

8.Depth of concrete block

Assuming angle of dispersion 45^0

Depth of concrete block = $\frac{L_1 - L}{2}$ or $\frac{B_1 - B}{2}$ (Which is greater)

$$= \frac{3300 - 620}{2} \text{ or } \frac{2150 - 410}{2}$$

Depth of concrete block = $=1340$ mm or 870 mm

$=1340 \cong 1350$ mm

Providing Depth of concrete block = 1350mm

Provided concrete block of size= 3300 mm X 2150 mm X 1350 mm

2. Design the gusseted base for a column consisting ISHB-350 with flange plate 300 x 12 mm on each flange. The column carries load of 1000 KN and

Module 6

Basics and Principles of Plastic Analysis

Definition:

Plastic analysis is defined as the analysis in which the criterion for the design of structures is the ultimate load. We can define it as the analysis inelastic material is studied beyond the elastic limit (which can be observed in stress strain diagram). Plastic analysis derives from a simple mode failure in which plastic hinges form. Actually the ultimate load is found from the strength of steel in plastic range. This method of analysis is quite rapid and has rational approach for analysis of structure. It controls the economy regarding to weight of steel since the sections required by this method are smaller than those required by the method of elastic analysis. Plastic analysis has its application in the analysis and design of indeterminate structures.

Basics of Plastic analysis:

Plastic analysis is usually based on the idealization of stress strain curve as perfectly plastic. In this analysis it is assumed that width thickness ratio of plate elements is small so the local buckling does not occur. Broadly speaking the section will be declared as perfectly plastic. Keeping in mind these assumptions, it can be said that section will reach its plastic moment capacity and after that will be subjected to considerable moment at applied moments.

Principles of Plastic analysis:

There are following conditions for plastic analysis

1. Mechanism condition
2. Equilibrium condition
3. Plastic moment condition

Mechanism condition:

When the ultimate load is reached collapse mechanism usually formed.

Equilibrium condition:

$$\Sigma FX=0, \quad \Sigma FY=0, \quad \Sigma Mxy=0$$

Plastic moment condition:

The bending moment at any section in the structure should not be more than the full plastic moment (moment at which plastic hinges form and structure moves to failure) of the section.

Plastic moment:

If we consider the case of simply supported beam, when the load is gradually applied on it, bending moment and stresses increases. As the load is increased, the stresses in fibers of beam reach to yield stress. At this stage the moment which has converted the stresses into the yield stress is said to be as Plastic moment. it is usually denoted by Mp at this stage the beam member cannot take up any additional moment but may maintain this moment for some amount of rotation and acts like a plastic hinge(hinge means having no capacity to resist moment). Plastic hinge behaves like an ordinary hinge allowing free rotation about itself. The yield moment and plastic moment has relationship which can be described by help of following relation:

My = 2/3 Mp

In calculation of plastic moments the term shape factor has its own importance. Shape factor can be defined as the ratio of plastic moment to yield moment is said to be as the shape factor. Shape factor depend usually on shape of the cross section.

For rectangular cross section the plastic moment can be calculated as:

Yield stress x   ($bh^2/4$)

When the load is applied on the body which is elastic (return to its shape after the load is removed), it will show resistance against deformation, such a body is called to be as structure. On the other hand if no resistance is shown against the body, then it is known as mechanism. when plastic hinges equal to n+1 form in the structure, then the structure will collapse(where n is degree of indeterminacy of structure). It means if the plastic hinges in structures increases in number than the their degree of indeterminacy, structures move towards collapse.

Plastic hinge and degree of indeterminacy:

Whenever plastic hinge forms in the structure, equilibrium is obtained. As the result the degree of static indeterminacy reduces by one with the formation of one plastic hinge. We can say that if the structure has 'n' number of degree of indeterminacy, its degree of indeterminacy reduces and it becomes determinate structure if '*n*' number of plastic hinges forms in it.

Plastic hinge formulation: In the structural engineering beam theory term, plastic hinge, is used to describe the deformation of a section of a beam where plastic bending occurs. In earthquake engineering plastic hinge is also a type of energy damping device allowing plastic rotation [deformation] of an otherwise rigid column connection

Plastic Behaviour:

In plastic limit analysis of structural members subjected to bending, it is assumed that an abrupt transition from elastic to ideally plastic behaviour occurs at a certain value of moment, known as plastic moment (M_p). Member behaviour between M_{yp} and M_p is considered to be elastic. When M_p is reached, a plastic hinge is formed in the member. In contrast to a frictionless hinge permitting free rotation, it is postulated that the plastic hinge allows large rotations to occur at constant plastic moment M_p.

Plastic hinges extend along short lengths of beams. Actual values of these lengths depend on cross-sections and load distributions. But detailed analyses have shown that it is sufficiently accurate to consider beams rigid-plastic, with plasticity confined to plastic hinges at points. While this assumption is sufficient for limit state analysis, finite element formulations are available to account for the spread of plasticity along plastic hinge lengths.

By inserting a plastic hinge at a plastic limit load into a statically determinate beam, a kinematic mechanism permitting an unbounded displacement of the system can be formed. It is known as the collapse mechanism. For each degree of static indeterminacy of the beam, an additional plastic hinge must be added to form a collapse mechanism

Sufficient number of plastic hinges(N) required to make a collapse mechanism (unstable structure):

N=Degree of static indeterminacy + 1

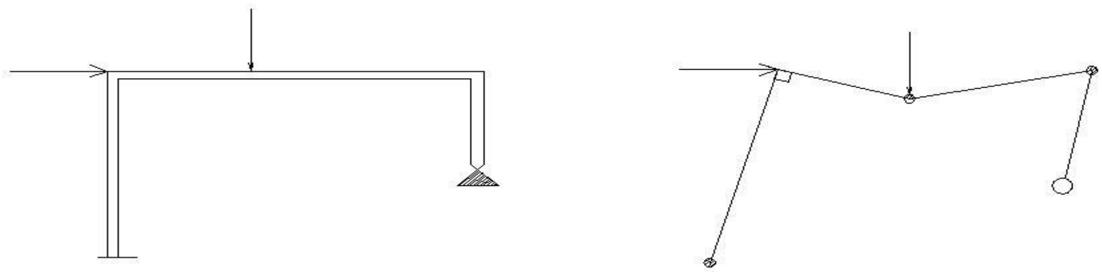


Diagram of a structure featuring plastic hinges

Collapse mechanism

The formation of a single plastic hinge gives a collapse mechanism for a simply supported beam. Collapse occurs when there is no more remaining stable element that can carry the additional load.

Plastics Design :- The design of steel or reinforced-concrete structural frames which is based on the assumption that plastic hinges form at points of utmost bending moment.

Elastic Design :- Design of a structure based on working stresses which are about 1/2 to 2/3 of the elastic limit of the material.

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Shape Factor: The ratio of the plastic moment to the yield moment is known as the Shape factor. Mp/My is known as shape factor. It may be remembered that shape factor is the property of a section which depends only upon the geometry of the cross section.

Plastic-collapse load : The load at which sufficient number of plastic hinges are formed in a structure such that a collapse mechanism is created is called plastic-collapse load or plastic-limit load

The elastic section modulus is defined as S = I / y, where I is the second moment of area (or moment of inertia) and y is the distance from the neutral axis to any given fiber. The Plastic section modulus is used for materials where (irreversible) plastic behavior is dominant.

Recent approaches in Steel Structure design based on Limit State Approach

Limit state design can therefore be defined as the process of designing a structure so that it doesn't break and remains fit for its designed use. The Working Stress Method assumes that all material used in the design behaves in a linear elastic manner and calculations are based on service conditions. The two principal types of limit state are the ultimate limit state and the serviceability limit state. This requires that the structure must be able to withstand, with an adequate factor of safety against collapse, the loads for which it is designed. Serviceability limit state (SLS)

The servicability limit state is the design to ensure a structure is comfortable and useable. These are the conditions that are not strength-based but still may render the structure unsuitable for its intended use, for example, it may cause occupant discomfort under routine conditions.

Limit state method uses multiple safety factor format that helps to provide adequate safety at ultimate loads and adequate serviceability at service loads, by considering all possible limit states.

Introduction to Provisions in IS 800-2007 :

Philosophy of limit state design for strength: Limit state of strength using connected factor of safety are those connected with failure , under the action of probable and mostly unfavorable combination of load on the structure which may danger the safety of life and property it includes. The limit state of strength includes

a) Loss of equilibrium of the structures as a whole or any of its parts or components.

b) Loss of stability of the structure (including the effect of the sway where appropriate and overturning) or any of its parts including supports and foundation.

c) Failure by excessive deformation , rupture of the structure or any of its parts or components.

d) Fracture due to fatigue

e) Brittle fracture

Philosophy of limit state design for serviceability:

Limit state of serviceability is related to the satisfactory performance of the structure and working load. There are four major type of serviceability , limit state applicable to steel structure, they are

a) Deflection: Excessive deflection posses number of problems i.e. feeling lack of safety ,imparing strength of structures or its component and damage to finishing.

b) Durability: Durability is defined as ability of structures to maintain its level of reliability and performing the desired in working environment under anticipated exposure condition without detoration of cross sectional area and loss of strength due to corrosion during its life span.

c) Vibration: Suitable provision shall be made for vibration set in due to machinery operating loads.

d) Fire resistance: Temperature causes vibration of mechanical properties of steel such as vibration shall be made to resist fire.