MODULE 1 - INTRODUCTION

Structural Design

Definition: Determination of overall proportions and dimensions of the supporting framework and the selection of individual members.

Responsibility: The structural engineer, within the constraints imposed by the architect (number of stories, floor plan,..) is responsible for structural design
Safety (the structure doesn't fall down)
Serviceability (how well the structure performs in term of appearance and deflection)
Economy (an efficient use of materials and labor)

Alternatives

Several alternative designs should be prepared and their costs compared

Types of Load

Dead Loads (permanent; including self-weight, floor covering, suspended ceiling, partitions,..)
Live Loads (not permanent; the location is not fixed; including furniture, equipment, and occupants of buildings)
Wind Load (exerts a pressure or suction on the exterior of a building)

Types of Load Continued

Earthquake Loads (the effects of ground motion are simulated by a system of horizontal forces)
Snow Load (varies with geographical location and drift)
Other Loads (hydrostatic pressure, soil pressure)

Types of Load Continued

If the load is applied suddenly, the effects of IMPACT must be accounted for.

If the load is applied and removed many times over the life of the structure, FATIGUE stress must be accounted for

Design Specifications

Provide guidance for the design of structural members and their connections.

They have no legal standing on their own, but they can easily be adopted, by reference, as part of a building code.

American Concrete Institute (ACI 318-99) Building Code Requirements for Structural Concrete

National Design Specifications for Wood Construction by American Forest and Paper Association.

Structural **Steel**

Steel is an alloy of primarily iron, carbon (1 to 2%) and small amount of other components (manganese, nickel, …)
Carbon contributes to strength but reduces ductility.

Steel Properties

The important characteristics of **steel** for **design** purposes are:

- yield stress (F^y)
- ultimate stress (F^u)
- modulus of elasticity (E)
- percent elongation (ε)
- coefficient of thermal expansion (α)

Standard Cross-Sectional Shapes

Refer steel table

Design Philosophies

Allowable Stress **Design** Method (ASD)
Load and Resistance Factor **Design** (LRFD)

A member is selected such that the max stress due to working loads does not exceed an allowable stress.

It is also called elastic **design** or working stress **design**.

- allowable stress=yield stress/factor of safety
- actual stress $\subseteq$ allowable stress

LRFD –Load and Resistance Factor Design

A member is selected such that its factored strength is more than the factored loads.

- Σ(loads x L factors) $\subseteq$ resistance x R factor

Each load effect (DL, LL, ..)has a different load factor which its value depends on the combination of loads under consideration.

Load Factors

The values are based on extensive statistical studies

- DL only 1.4D
- DL+LL+SL (LL domin.) 1.2D+1.6L+0.5S
- DL+LL+SL (SL domin.) 1.2D+0.5L+1.6S
- In each combination, one of the effects is considered to be at its "lifetime" max value and the others at their "arbitrary point in time " values.

Resistance Factor

The resistance factors range in value from 0.75 to 1.0 depending on the type of resistance (tension, bending, compression, ..)

These factors account for uncertainties in material properties, **design** theory, and fabrication and construction practices.

History

ASD has been the primary method used for **steel design** since the first AISC specifications was issued in 1923.
In 1986, AISC issued the first specification for LRFD.
The trend today is toward LRFD method, but ASD is still in use.

Advantages of LRFD

It provides a more uniform reliability in all structures subjected to many types of loading conditions. It does not treat DL and LL as equivalent, thereby leading to a more rational approach.

It provides better economy as the DL make up a greater percentage on a given structure. Because DLs are less variable by nature than live loads, a lower load factor is used.

This may lead to a reduction in member size and therefore better economy

STEEL AS A STRUCTURAL MATERIAL

1.1General

Structural steel is a material used for steel construction, which is formed with a specific shape following certain standards of chemical composition and strength. They can also be defined as hot rolled products, with a cross section of special form like angles, channels and beams/joints. There has been an increasing demand for structural steel for construction purposes in the United States and India.

Measures are been taken by the structural steel authority for ready availability of structural steel on time for the various projects. The people at every level are working hard to realize the purpose of producing steel on time, like, service centers, producers, fabricators and erectors along with the general contractors, engineers and architects are all working hand in hand. Steel has always been more preferred to concrete because steel offers better tension and compression thus resulting in lighter construction. Usually structural steel uses three dimensional trusses hence making it larger than its concrete counterpart. There are different new techniques which enable the production of a wide range of structures and shapes, the procedures being the following:

High-precision stress analysis

Computerized stress analysis

Innovative jointing

The structural steel all over the world pre-dominates the construction scenario. This material has been exhaustively used in various constructions all over the world because of its various specific characteristics that are very much ideally suited for construction. Structural steel is durable and can be well molded to give the desired shape to give an ultimate look to the structure that has been constructed. There is a mention of The Super dome situated in the United States and The Fukuoka Dome of Japan; both speak the unique language of the unique capabilities of the structural steel.

1.2 Types of structural steel:

Various types of structural steel sections and their technical specifications are as follows:

Beams

Channels

Angles

Flats

1.2.1 Steel Beams

Steel Beams is considered to be a structural element which mainly carries load in flexure meaning bending. Usually beams carry vertical gravitational force but are also capable of carrying horizontal loads generally in the case of an earthquake. The mechanism of carrying load in a beam is very unique, like; the load carried by a beam is transferred to walls, columns or girders which in turn transfer the force to the adjacent structural compression members. The joists rest on the beam in light frame constructions.

The beams are known by their profile meaning:

The length of the beam
The shape of the cross section
The material used

The most commonly found steel beam is the. I beam or the wide flanged beam also known by the name of universal beam or stouter sections as the universal column. Such beams are commonly used in the construction of bridges and steel frame buildings. The most commonly found types of steel beams are varied and they are mentioned below:

I beams
Wide flange beams
HP shape beams

Typical characteristics of beams

Beams experience tensile, sheer and compressive stresses internally due to the loads applied to them. Generally under gravity loads there is a slight reduction in the original length of the beam. This results in a smaller radius arc enclosure at the top of the beam thus showing compression. While the same beam at the bottom is slightly stretched enclosing a larger radius arc due to tension. The length of the beam midway and at the bends is the same as it is not under tension or compression and is defined as the neutral axis. The beam is completely exposed to shear stress above the support. There are some reinforced concrete beams that are completely under compression, these beams are called pre-stressed concrete beams and are built in such a manner to produce a compression more than the expected tension under loading conditions. The pre-stressed concrete steel beams have the manufacturing process like, first the high strength steel tendons are stretched and then the beam is cast over them. Then as the concrete begins to cure the tendons are released thus the beam is immediately under eccentric axial loads. An internal moment is created due to the eccentric axial load which in turn increases the moment carrying capacity of the beam. Such beams are generally used in highway and bridges.

Materials Used

In today's modern construction the beams are generally made up of materials like:

Steel

Wood

Reinforced concrete

1.2.2 Steel Channels:

Steel channels are used ideally as supports and guide rails. These are roll-formed products. The main metal used for making channels is steel along with aluminum. There are certain variations that are available in the channels category, the categorization is mainly on the shape of the channel, the varieties are mentioned below:

J channels: This kind of channel has two legs and a web. One leg is longer. This channel resembles the letter-J.

Hat channels: This channel has legs that are folded in the outward direction resembling an old fashioned man's hat.

U channels: This most common and basic channel variety. It has a base known as a web and two equal length legs.

C channels: In this channel the legs are folded back in the channel and resemble the letter-C. C channels are known as rests.

Hemmed channels: In this kind of channel the top of the leg is folded hence forming double thickness.

There are other variations of channels that are available, which are customized according to the customer's needs.

Application

Steel channels are subjected to a wide array of applications. The application fields are:

Construction

Appliances

Transportation

Used in making Signposts

Used in wood flooring for athletic purposes

Used in installing and making windows and doors

A major variant of the channel is the mild steel channel. Such channels are generally used in heavy industries. They are used in the heavy machinery industry and automotive industry too.

1.2.3 Steel Angle:

A steel angle is long steel with mutually vertical sides. The steel angles are the most basic type of roll-formed steel. The most commonly found steel angles are formed at a 90 degree angle and has two legs of equal length. The sides are either equal or of different sizes.

There are certain variations in the steel angles depending on it's basic construction. The variations are like; if one leg is longer than the other then it is known as L angle. If the steel angle is something different from 90 degrees then it is known as V angle. In some steel angles, double thickness is achieved by folding the legs inward. If the steel angle has same sides then it means that it has identical width. The steel angles are made according to the strength that is required for the different structures for construction purposes.

Applications

the steel angle finds an application in a number of things, they are mentioned below:

Used in framing

Used in trims

For reinforcement

In brackets

Used in transmission towers

Bridges

Lifting and transporting machinery

Reactors

Vessels

Warehouses

Industrial boilers

Structural steel angles are used in rolling shutters for fabricating guides for strength and durability.

1.2.4 Steel Flats:

Flats are actually thin strips of mild steel having the thickness of the strip commonly varying from 12mm to 10mm but thicker flats than this are also available. Steel flats are produced by the utilization of relatively smooth, cylindrical rolls on rolling mills. Generally the width to thickness ratio of flat rolled products is fairly large. The steel flat bars are manufactured using advanced thickness control technology for controlled thicknesses. The hi-tech machineries enable the production of top grade steel flat bars with superlative flatness and controlled thickness. This product is highly customized and the specific sizes according to the client's requirement are produced. After production the flat steels are subjected to a variety of finishes like, painting and galvanizing. The flat carbon steel is a hot or cold rolled strip product also known as a plate product. These plate products have a size variation between 10mm to 200mm and the thin flat rolled flat rolled product's size varies from 1 mm to 10 mm.

Applications

The steel flats are used in a wide array of applications. The varied applications are listed below:

Railway parts

Ordinance factories

Hand tools

Engineering industries

Auto components- two-wheeler, four-wheeler, commercial vehicles

Domestic white goods products

Office furniture's

Heart pacemakers

Tin cans

Press working

1.3 Advantages of steel as a structural material:

Structural steel sections are usually used for construction of buildings, buildings, and transmission line towers (TLT), industrial sheds and structures etc. They also find in manufacturing of automotive vehicles, ships etc.

Steel exhibits desirable physical properties that make it one of the most versatile structural materials in use.

Its great strength, uniformity, light weight, easy of use, and many other desirable properties makes it the material of choice for numerous structures such as steel bridges, high rise buildings, towers, and other structure.

Elasticity: steel follows hooks law very accurately.

Ductility: A very desirable of property of steel, in which steel can withstand extensive deformation without failure under high tensile stresses, i:e., it gives warning before failure takes place.

Toughness: Steel has both strength and ductility.

Additions to existing structures: Example: new bays or even entire new wings can be added to existing frame buildings, and steel bridges may easily be widened.

1.4 Disadvantages of steel as a structural material:

Although steel has all this advantages as structural material, it also has many disadvantages that make reinforced concrete as a replacement for construction purposes.

For example steel columns sometimes cannot provide the necessary strength because of buckling, where as RCC columns generally sturdy and massive, i:e., no buckling problem occurs.

Many disadvantages of steel can be summarized below:

Maintenance cost: Steel structures are susceptible to corrosion when exposed to air.

Fire proofing cost: steel is an incombustible material; however, its strength is reduced tremendously at high temperature due to common fires.

Fatigue: The strength of structural steel member can be reduced if this member is subjected to cyclic loading.

Brittle fracture: under certain conditions steel lose its ductility, and brittle fracture may occur at places of stress concentration. Fatigue type loadings and very low temperature trigger the situation.

Limit state design:

Refer IS:800-2009 in detail and other text bookd

Bolted Connections: Introduction, Types of Bolts, Behaviour of bolted joints, Design of High Strength friction Grip (HSFG) bolts, Design of Simple bolted Connections (Lap and Butt joints)

Welded Connections: Introduction, Types and properties of welds, Effective areas of welds, Weld Defects, Simple welded joints for truss member, Advantages and Disadvantages of Bolted and Welded Connections.

10 Hours L1,L2,L3

Introduction:

Various components of any structure need to be connected by means of fasteners so as to enable them to behave as single composite units. Connections are also required for extending the lengths of members, for connecting columns to footings and for joining two parts of a structure during erection.

Based on tests results, past performance and the ductile behaviour of steel, many approximations and assumptions are made in the design of bolted and welded connection. Following are the requirements of a good connection in steelwork:

1) It should be rigid, to avoid fluctuating stresses which may cause fatigue failure.
2) It should be such that there is the least possible weakening of the parts to be joined.
3) It should be such that it can be easily installed, inspected and maintained.

In general following types of connections are adopted:

(a) Riveted connections (b) Welded Connections
(c) Bolted Connections (d) Pin Connections

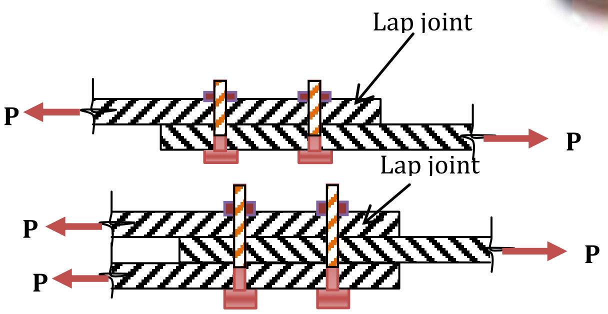
Simple connections:

In many cases, a connection is required to transmit a force only and there may not be any moment acting on the group of connectors, even though the connection may be capable of transmitting some amount of moment. Such a connection is referred to as simple, force, pinned or flexible connection

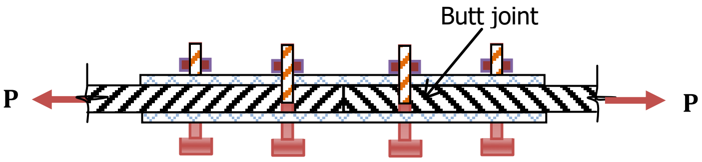
The different types of simple connections found in steel structures may be classified as follows:

- Lap and Butt joints
- Truss joint connections
- Connections at beam column junctions
- Seat angle connection
- Web angle connection
- Stiffened seat angle connection
- Tension and flange splices.

Lap and butt joints:

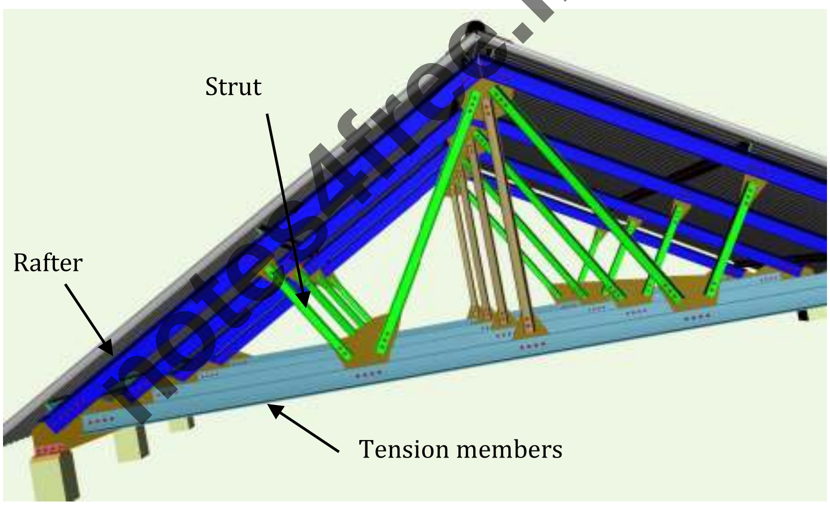


Lap joint is the one in the plates are connected with overlap with each other. The lap joint may have single – row, staggered or chain bolting connections. Though lap joints are the simplest, they result in eccentricity of the applied loads.

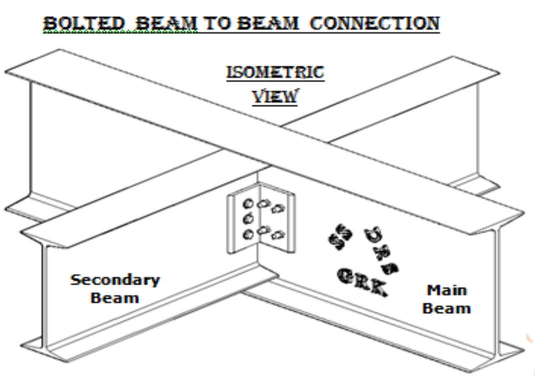


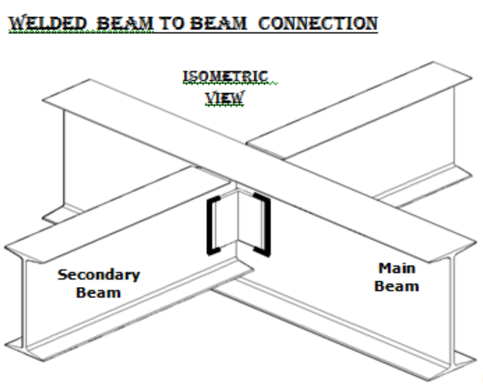
Butt joints connection is the one in the plates are connected butt against each other and connection is made by providing a cover plate on one or both sides of the joint. The Butt joint may have single – row, staggered or chain bolting connections. Butt joints on the other hand eliminate eccentricity at the connection.

Truss joint connections:



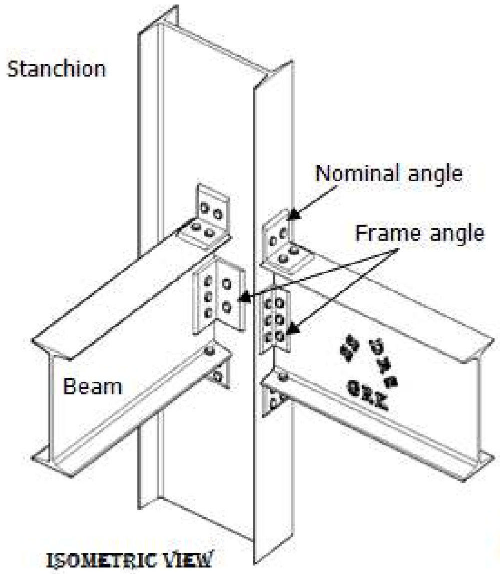
Beam to beam connection: (Web angle connection)





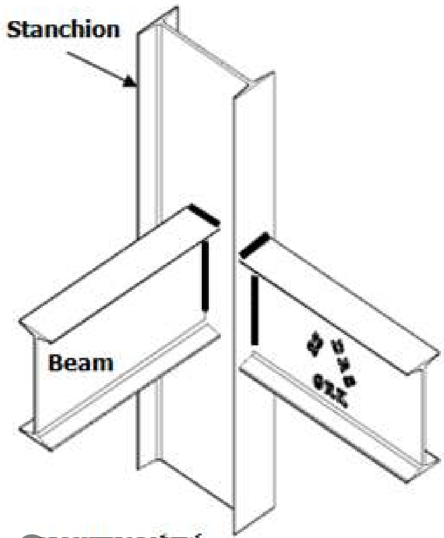
Connections at beam column junctions

BOLTED BEAM TO COLUMN FRAMED CONNECTION



ISOMETRIC VIEW

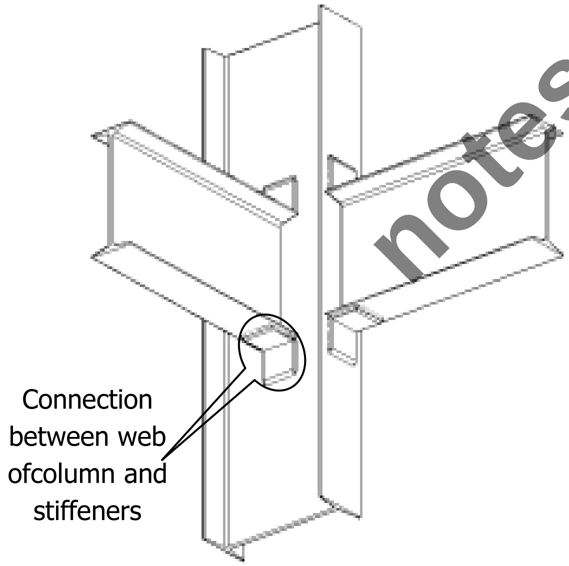
WELDED BEAM TO COLUMN FRAMED CONNECTION

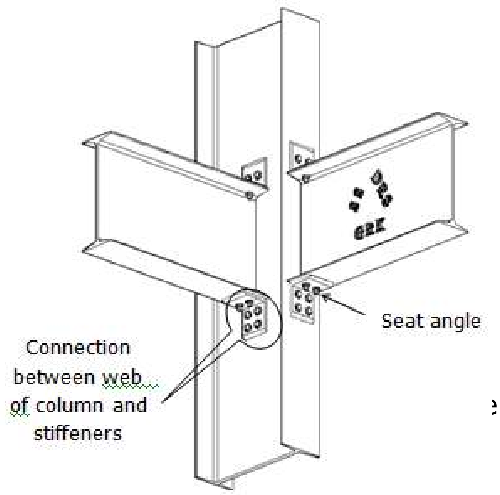


ISOMETRIC VIEW

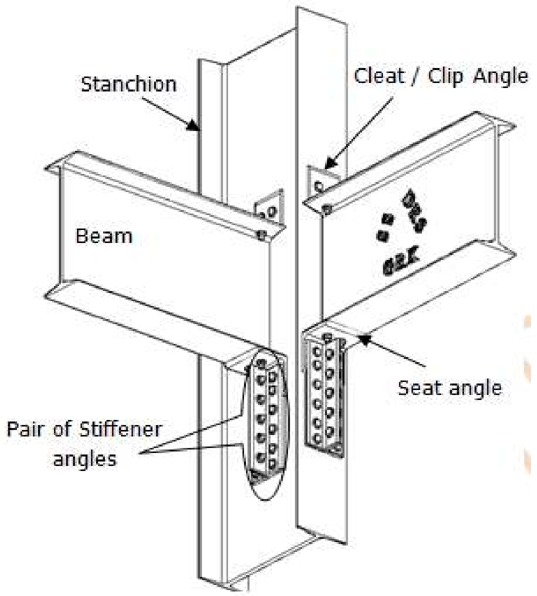
Seat angle connection:

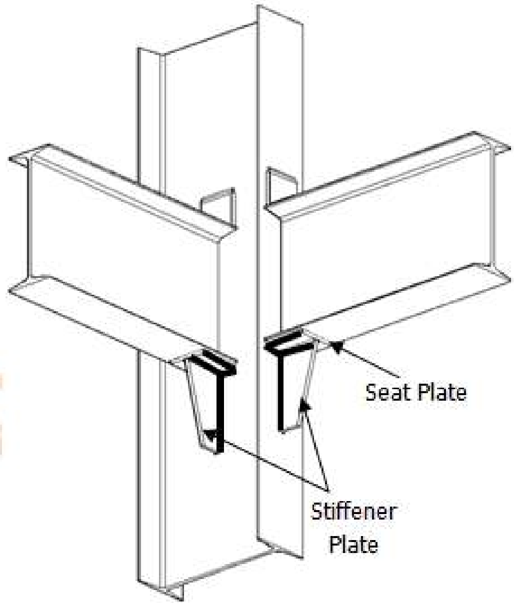
i) Unstiffened seated connection: (Bolted and welded type)





ii) Stiffened seated connection: (Bolted and welded type)

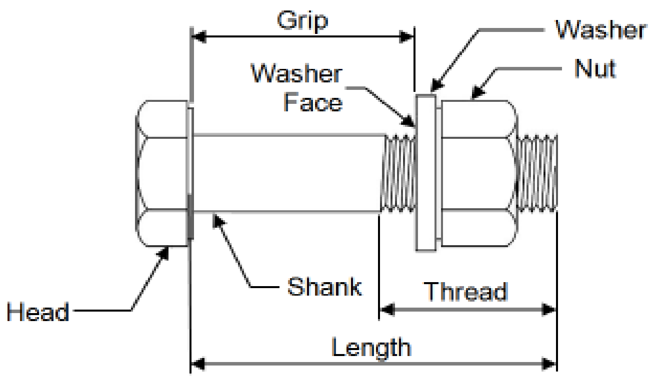




Bolted joint Connections:

Bolts may be used for structure not subjected to vibrations. The cost of bolts is more but it takes less time to fabricate structure with bolted connections. The fabrication work with bolts is noise less and less skilled workers can also handle it.

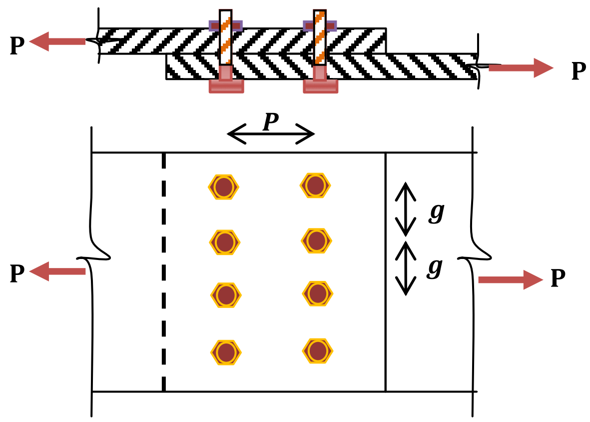
Parts of the bolts assembly:



- Grip is the distance from behind the bolt head to the back of a nut or washer
- It is the sum of the thickness of all the parts being joined exclusive of washers.
- Thread length is the threaded portion of the bolt.
- Bolt length is the distance from

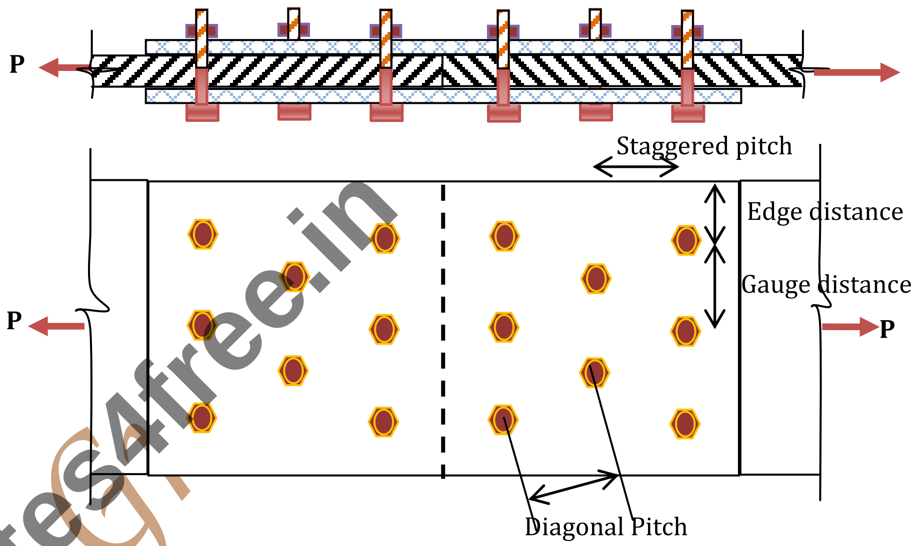
behind the bolt head to the end of the bolt.

Gauge distance (g): The gauge distance is the transverse distance between two consecutive bolts of adjacent chains and is measured at right angles to the direction of the force in the structural member.



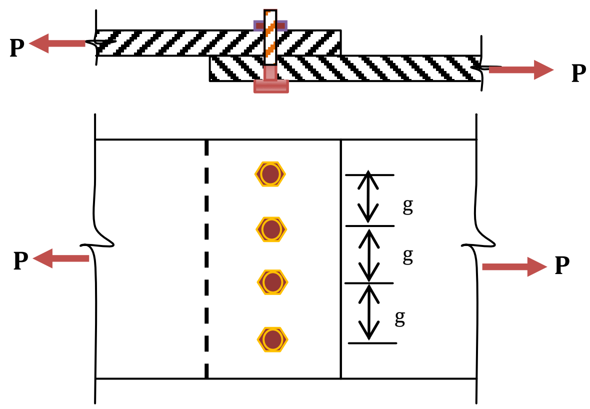
Pitch of bolts (p): It is the distance between centres of two adjacent bolts in a row measured parallel to the direction of the force.

(**Note:** There is a lot of confusion in the available literature about the nomenclatures 'Pitch' and 'Gauge')



Diagonal Pitch: The distance between centres of any two adjacent bolts in the diagonal direction is called diagonal pitch.

Staggered pitch: The distance between centres of any two consecutive bolts in a zig – zag bolting, measured parallel to the direction of stress in the member is called staggered pitch

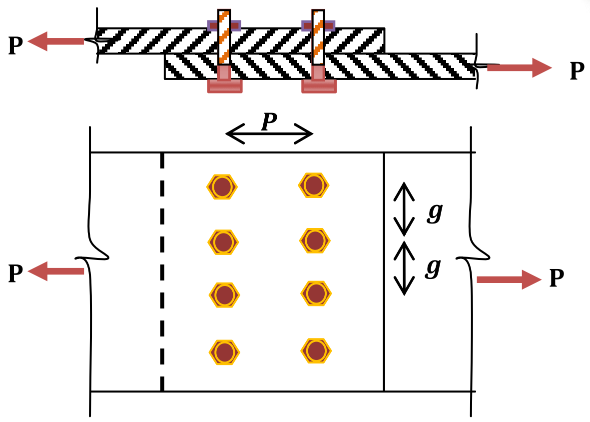


TYPES OF BOLTED JOINTS:

There are two types of bolted joints

(i) **Lap joint:** when two members which are to be connected are simply overlapped and connected together by means of bolts or welds, the joint is called a lap joint. The lap joint may have single-row, staggered or chain bolting as shown in Fig.

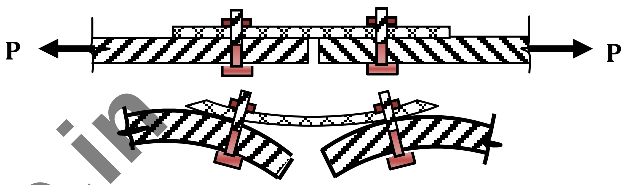
Double bolted Lap joint



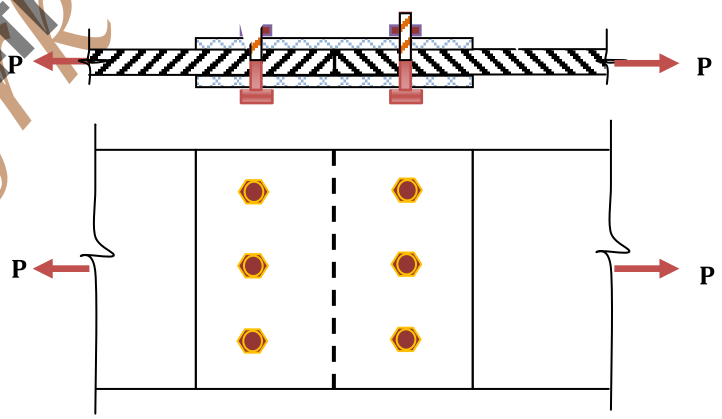
(ii) Butt joint: In butt joint the plates are connected against each other and the connection is made by providing a cover plate on one or both sides of the joint.

(a) Single cover single bolted butt joint:

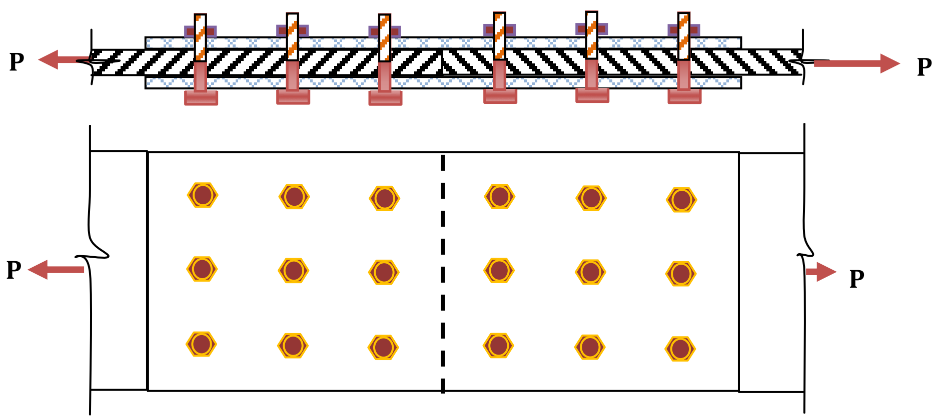
In single cover butt joint, bending stresses may develop, tending to distortion the joint. This possibility is completely eliminated by using a double cover butt joint.



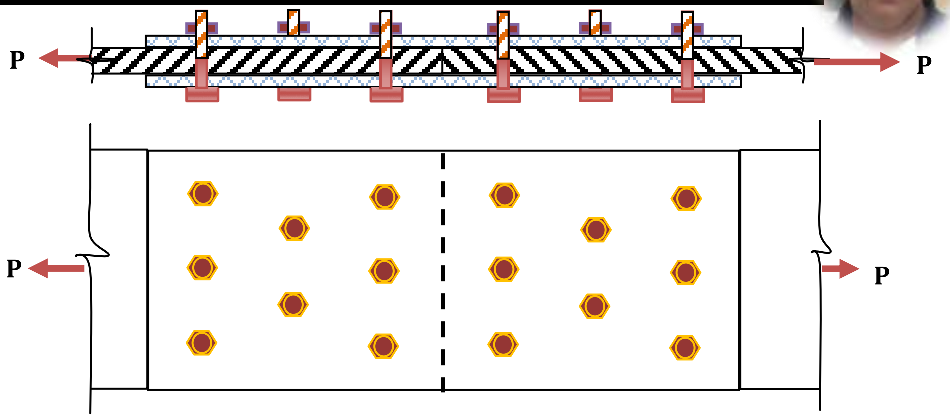
(b)Double cover single bolted butt joint:



(c) Double cover chain bolted butt joint:



d) Double cover zig – zagbolted butt joint:



FAILURE OF A BOLTED JOINT

Loads are transferred from one member to another by means of the connections between them. The possible "Limit states" or failure modes that may control the strength of a bolted connection are in any of the following ways:

1. **Shear failure of bolts**: The plates bolted together and subjected to tensile loads may results in the shear of the bolts. The bolts are sheared across their cross-sectional areas. Single shear occurring in a lap joint has been show in Fig. (a) and double shear occurring in butt joint has been show in Fig (b)

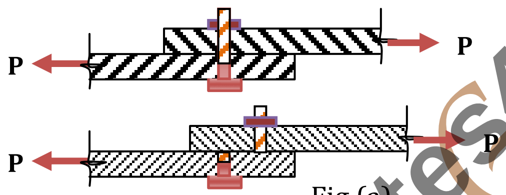


Fig (a)

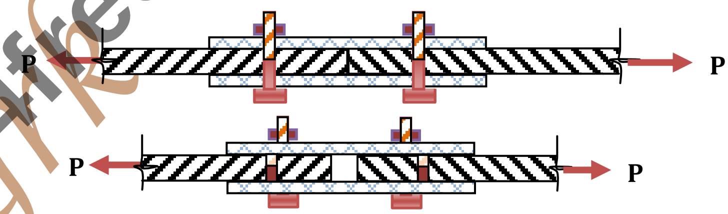
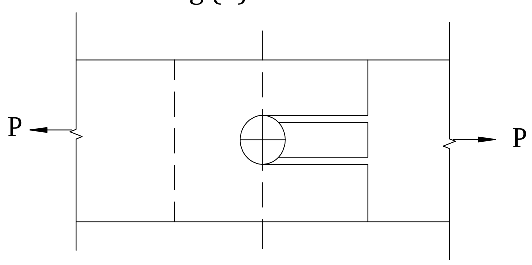
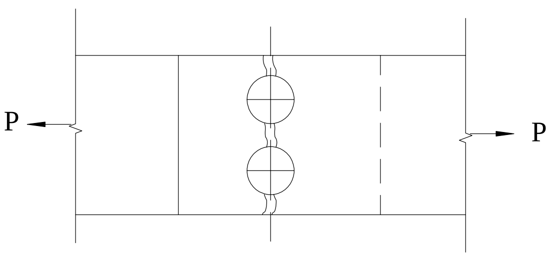


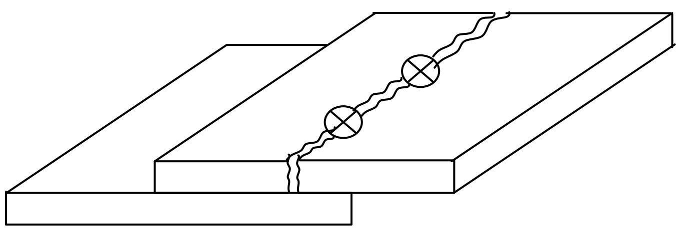
Fig (b)

2. **Shear failure of plates**: A plate may fail in shear along two lines as shown in Fig. This may occur when minimum proper edge distance is not provided.

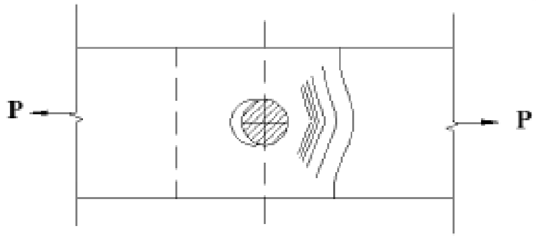


3. **Tearing failure of plates:** When plates bolted together are carrying tensile load, tearing of plate may occur, when strength of the plate of less than that of bolts. The tearing failure occurs at the net sectional area of plate as shown in Fig.

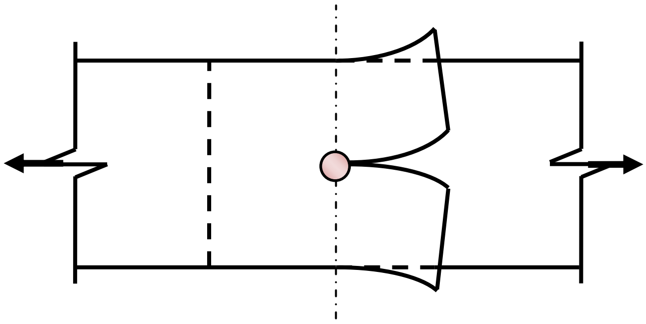




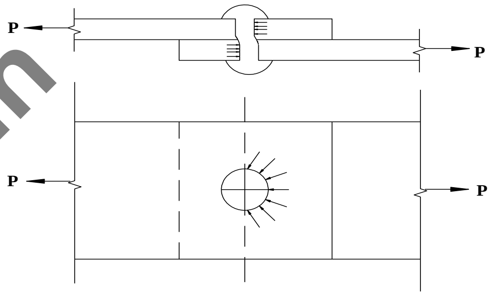
4. Bearing failure of plates: The bearing failure of a plate may occur because of insufficient edge distance in the bolted joint. The crushing of plate against the bearing of bolt as shown in Fig takes a place in such failure.



5. Splitting failure of plates: The splitting failure of a plate may occur because of insufficient edge distance in the bolted joint. The splitting (cracking) of plate as shown in Fig takes place in such failure.



6. Bearing failure of bolts: The bearing failure of a bolt occurs when the bolt is crushed by the plate as show in Fig.



The bearing, shearing and splitting failure of plates may be avoided by providing adequate edge distance. To safeguard a bolted joint against other modes of failure, the joint should be designed properly.

Strength of bolted joint:

P-75, 10.3.3

1. Strength of bolted joint against shearing of the bolts (V_{dsb}):

The strength of bolted joint against the shearing of bolts is equal to the product of strength of one bolt inshear and the number of bolts on each side of the joint.

Strength of one bolt in single shear $V_{dsb} = \dfrac{V_{nsb}}{\gamma_{mb}}$

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

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

Where,

f_u = ultimate tensile stress of a bolt → depends on grade of bolts

n_n = Number shear planes with treads intercepting the shear plane;

n_s = Number of shear planes without treads intercepting the shear plane;

A_{sb} = Nominal plain shank area of the bolt; and

A_{nb} = Net area of the bolt at threads, may be taken as the area corresponding to root diameter at the thread.

γ_{mb} = partial safety factor for materials – bolt bearing, P- 30, Table – 5.

GRADE OF BOLT:

i) Property class 4.6 (black bolt/ ordinary bolts/unfinished bolts)

Number before decimal indicates 1/100th of the nominal ultimate strength and the number after decimal indicate the ratio of yield stress to ultimate stress, expressed as a percentage. i.e.,

Ultimate tensile stress = 400 N/mm^2

Yield stress = 0.6 x 400 = 240 N/mm^2

ii) Property class 5.6

Ultimate tensile stress = 500 N/mm^2

Yield stress = 0.6 x 500 = 300 N/mm^2

iii) Property class 8.8

Ultimate tensile stress = 800 N/mm^2

Yield stress = 0.8 x 800 = 640 N/mm^2

Grade of Plate or Material grade = f_u = 410 N/mm^2

As per IS 1367 (Part -1)

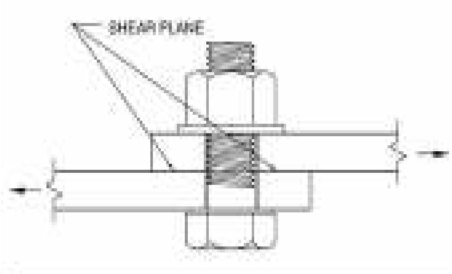
$$A_{sb} = \frac{\pi}{4} \times d^2$$

$A_{nb} = \frac{\pi}{4} \times (d - 0.9382p)^2$ p = Course pitch of threads = 2, 2.5, 3 and 3.5 mm for 16, 20, 24 and 30 mm dia bolts respectively.

OR

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

Threads in the shear plane



Threads Included In The Shear Plane

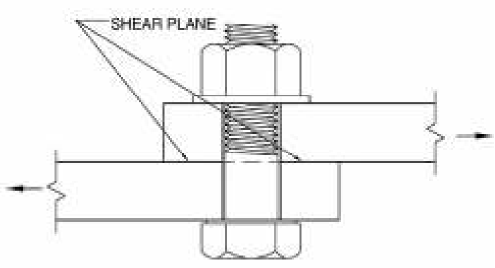
- The shear plane is the plane between two or more pieces under two or more pieces under load where the pieces tend to move parallel from each other, but in opposite directions.
- The threads of a bolt may either be included in the shear plane or excluded from the shear plane.
- The capacity of a bolt is greater with the threads excluded from the shear plane.

Note:

- If thread is interfering the shear plane in case of lap joint

$$n_n = 1, \quad \& \quad n_s = 0.$$

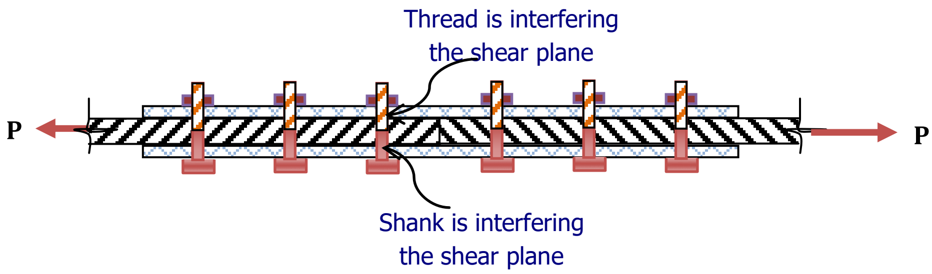
If Shank is interfering the shear plane in case of lap joint



Threads Excluded From The Shear Plane

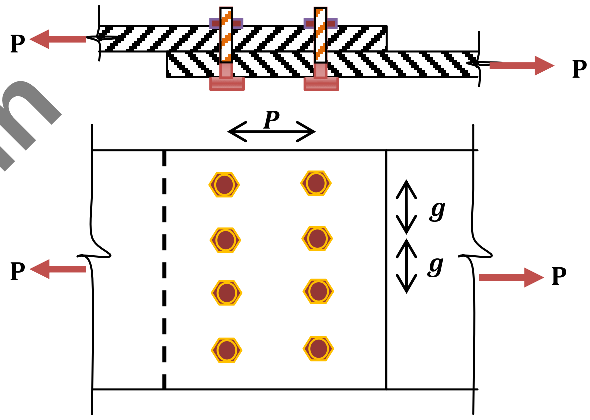
$$n_n = 0, \quad \& \quad n_s = 1.$$

- If Thread and Shank are interfering the shear plane as in case of butt joint



$$n_n = 1, \quad \& \quad n_s = 1.$$

LAP JOINT:

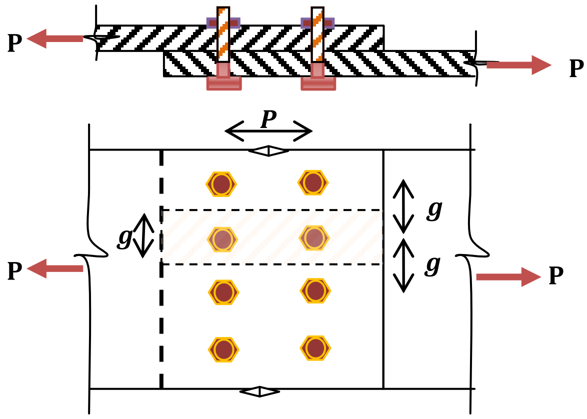


Strength of bolted joint against shearing of bolts :

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

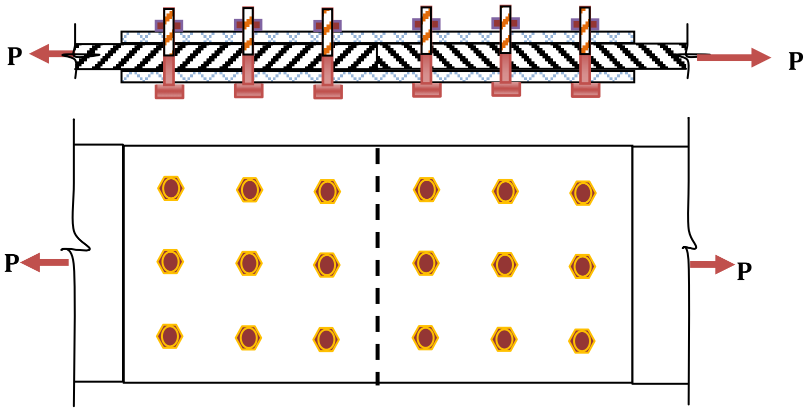
When the strength of bolted joint against the shearing of the bolts is determined per gauge width of the plate, then, the number of bolts, '*n*' per gauge is taken into consideration. Therefore

Strength of bolted joint against shearing of bolts per gauge width



$$V_{dsb-1} = n \times \left(\frac{f_u}{\sqrt{3}}\right) \times \left(\frac{n_n A_{nb} + n_s A_{sb}}{\gamma_{mb}}\right)$$

BUTT JOINT:



Strength of bolted joint against shearing of bolts

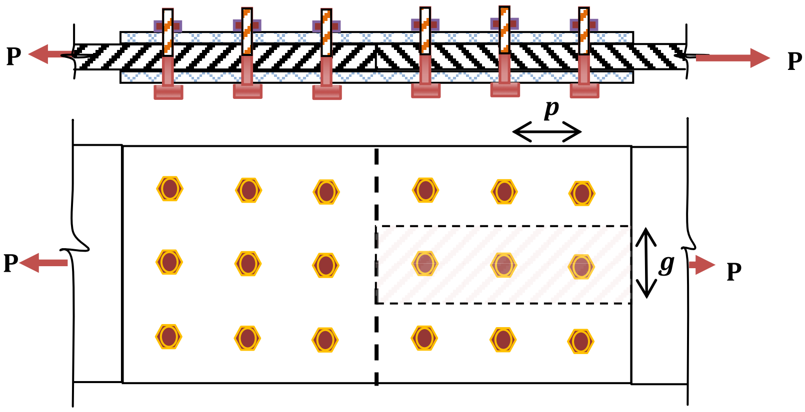
$$V_{dsb} = N \times \left[\left(\frac{f_u}{\sqrt{3}}\right) \times \left(\frac{n_n A_{nb} + n_s A_{sb}}{\gamma_{mb}}\right)\right]$$

Where,

N = Number of bolts on each side of the joint

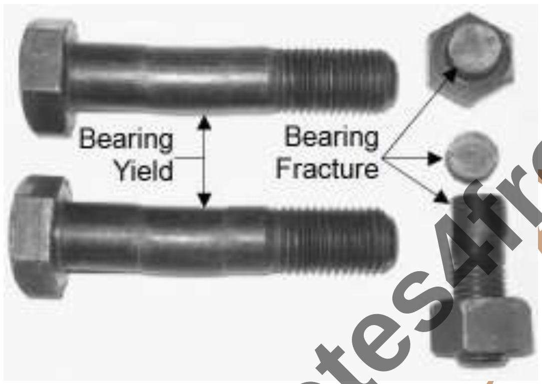
Strength of the bolt in Double shear per gauge width:

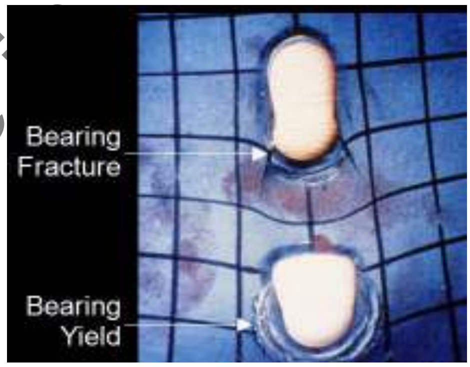
$$V_{dsb-1} = n \times \left[\left(\frac{f_u}{\sqrt{3}}\right) \times \left(\frac{n_n A_{nb} + n_s A_{sb}}{\gamma_{mb}}\right)\right]$$



2. Strength of bolted joint against the bearing of the bolts V_{dpb}(Cl 10.3.4, P- 75)

Bolted joint failure mode





- Bolts in bearing joints are designed to meet two limit states:
  1. Yielding, which is an inelastic deformation.
  2. Fracture, which is a failure of the joint.
- The material the bolt bears against is also subjected to yielding or fracture if it is undersized for the load.
- Tension connection act similarly to bearing connections.
- Many times, connection in direction tension are reconfigured so that the bolts act in shear.

Strength of one bolt in bearing $V_{dpb} = \frac{V_{npb}}{\gamma_{mb}}$

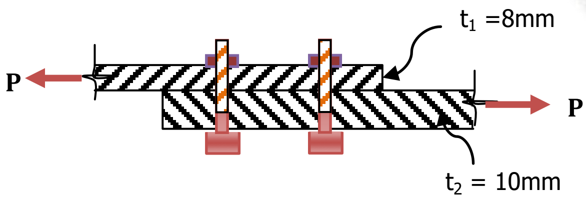
$$V_{dpb} = \frac{2.5\, k_b \times d \times t^* \times f_u}{\gamma_{mb}}$$

Where,

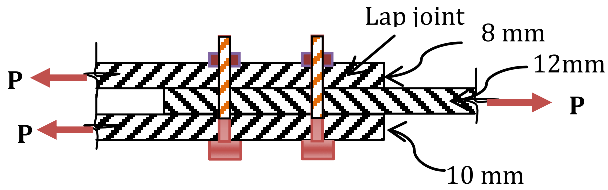
f_u = Ultimate stress of the bolt

t^* = Thickness of the thinnest plate

In case of lap joint:



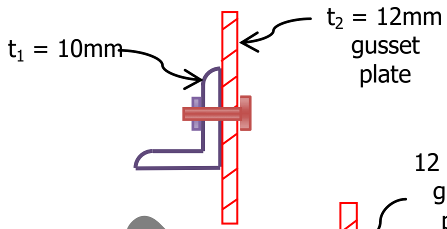
- Min of t_1, t_2.
  i.e., $t^* = t_1$=8 mm



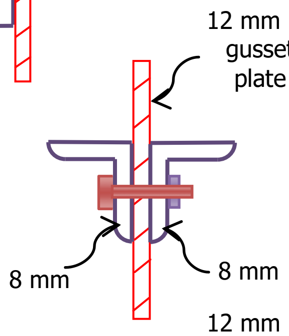
- t^* = Min thickness of

a) Thickness of mainplate = 12 mm

b) Sum of the thickness of cover plate = 8 +10= 18 mm   i.e., t^* = 12mm



- Min of t_1, t_2.
  i.e., $t^* = t_1$=10 mm

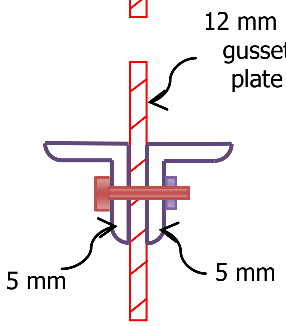


- t^* = Min thickness of

a) Thickness of gusset plate = 12 mm

b) Sum of the thickness of angles = 8+8= 16 mm

i.e., t^* = 12mm



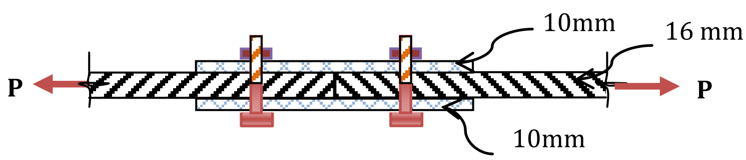
- t^* = Min thickness of

a) Thickness of gusset plate = 12 mm

b) Sum of the thickness of angles = 5 + 5 = 10 mm

i.e., t^* = 10 mm

In case of butt joint with double cover:

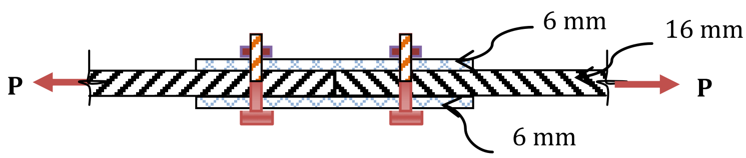


t^* = Min thickness of

a) Thickness of main plate = 16 mm

b) Sum of the thickness of cover plate = 10 + 10 = 20mm

i.e., t^* = 16mm



t^* = Min thickness of

a) Thickness of main plate = 16 mm

b) Sum of the thickness of cover plate = 6 + 6 = 12 mm

i.e., t^* = 12 mm

d = Nominal diameter of the bolt

k_b = Smaller of the following

$$\left(\frac{e}{3d_o}\right),\quad \left(\frac{p}{3d_o}\right) - 0.25,\quad \left(\frac{f_{ub}}{f_u}\right),\quad \text{and} \quad 1$$

e = edge distance

p = pitch of the fasterner along the bearing direction

d_o = dia of the bolt hole

Note: f_{ub} = Ultimate stress of the bolt {400 N/mm^2 (grade 4.6)/ 500 N/mm^2 (grade 5.6)/ 800 N/mm^2(grade 8.8)/}

f_u = Ultimate stress of the plate in MPa. (410 N/mm^2)

The strength of bolted joint against the bearing of bolts is equal to the product of strength of one bolt in bearing and the number of bolts on each side of the joint.

Strength of bolted joint against the bearing of bolts

$$V_{dpb} = N \times \left(\frac{2.5k_b \times d \times t^* \times f_u}{\gamma_{mb}}\right)$$

When the strength of bolted joint against the bearing of bolts ***per gauge*** width of the plate is taken into consideration, then, the number of bolts, ***n*** per gauge. Therefore

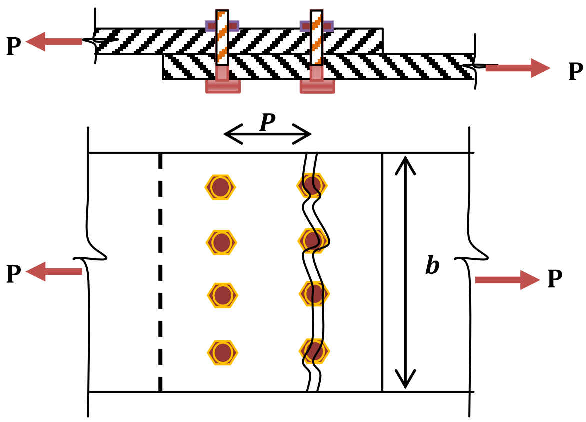
$$V_{dpb-1} = n \times \frac{\left(2.5k_b \times d \times t^* \times f_u\right)}{\gamma_{mb}}$$

3. Strength of plate in tearing (T_{dn}) P-32, Cl: 6.3.1

The strength of plate in tearing depends upon the resisting section of the plate.

The strength of the plate in tearing

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



Where,

f_u = ultimate stress of the material (plate) = 410 N/mm^2.

γ_{m1} = Partial safety factor for failure at ultimate stress = 1.25, P-30.

A_n = Net area of the member.

The strength of the plate in tearing for full width of the plate

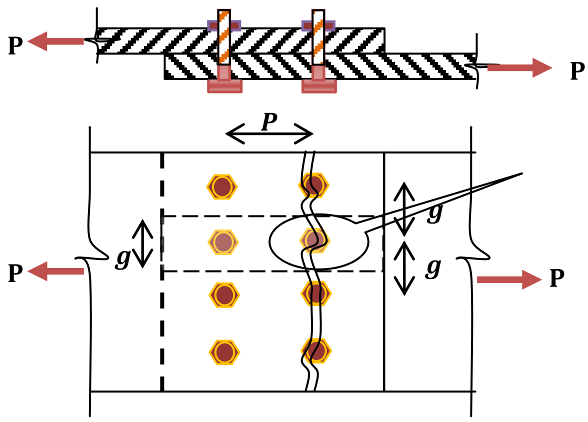
$$T_{dn} = \frac{0.9(b - nd_0) \times t \times f_u}{\gamma_{m1}}$$

n = No of holes along the tearing line.

Strength of plate in tearing ***per gauge width*** of the plate

$$T_{dn-1} = \frac{0.9(p - d_0) \times t \times f_u}{\gamma_{m1}} \quad \text{or}$$

$$T_{dn-1} = \frac{0.9(g - d_0) \times t \times f_u}{\gamma_{m1}}$$



The strength of bolted joint per gauge width of plate is the least of V_{dsb-1}, V_{dpb-1}, and T_{dn-1}

EFFICIENCY OF JOINTS (η)

Holes are drilled in the plates for the connection with bolts, hence the original strength of the full section is reduced. The joint which causes minimum in strength is said to be more efficient. Thus, for better efficiency the section should have the least no of holes at the critical section. The efficiency, expressed in percentage, is the ratio of actual strength of the connection to the gross strength of the connected members.

Efficiency of bolted joint per pitch length (η)

$$\eta = \frac{\text{Strength of bolted joint per pitch length}}{\text{Strength of solid plate per pitch length}} \times 100$$

Where, Strength of solid plate per pitch length $= \dfrac{0.9 \times p \times t \times f_u}{\gamma_{mb}}$

$$\eta = \frac{\text{Least of } V_{dsb-1}, V_{dpb-1} \text{ or } T_{dn-1}}{V}$$

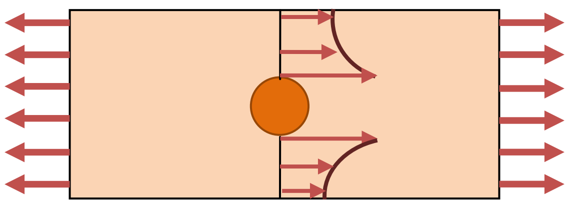
$$\text{Efficiency of bolted joint } (\eta) = \frac{\text{Strength of bolted joint}}{\text{Strength of solid plate}} \times 100$$

Strength of solid plate for full width $= \dfrac{0.9 \times b \times t \times f_u}{\gamma_{mb}}$

$$\eta = \frac{\text{Least of } V_{dsb}, V_{dpb} \text{ or } T_{dn}}{V}$$

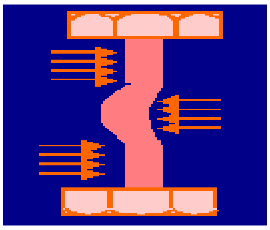
Bolt Value (BV):

The strength of a bolt in **shearing** and in **bearing** is computed and the **lesser** is called the **Bolt value (BV)** (i.e., Least of V_{nsb} and V_{npb})



$$\text{No of bolts} = \frac{\text{Force}}{\text{Bolt value}}$$

The assumptions made in the theory of bolted connections are:



1) Shear is uniform on the cross section of the bolt.
2) Distribution of tensile stress on the portions of the plate between the bolt holes is uniform.
3) Bolts in a group subjected to direct load through their centroid share the load equally.
4) Bending stresses in the bolt are neglected.

Advantages and disadvantages of Bolted connection:

The following are the **advantages** of bolted connections over riveted or welded connections:

1) Making joints is noiseless.
2) Do not need skilled labour
3) Needs less labour.
4) Connections can be made quickly.
5) Structures can be put to use immediately.
6) Accommodates minor discrepancies in dimensions.
7) Alterations, if any, can be done easily.
8) Working area required in the field is less.

The **disadvantages** of unfinished (black) bolt connections are:

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

Common definitions:

1) Nominal diameter: It is the diameter of the bolt.

The dia of bolt for a given plate thickness is chosen by the ***Unwins*** formula:

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

Where, t = thickness of plate in mm.

The bolts are available from 5 to 36 mm in diameter. The common ones are M16, M20, M24 to M30.

Bolt Hole (d_0):(Clause 10.2.1, P- 73).

Bolts may be located in standard size, over size, short slotted or long slotted hole.

Table 19 gives the details of clearances for fastener holes.

Size of the hole = Nominal Diameter of the fastener + Clearances

Size of the hole = Nominal Diameter of the fastener + Clearances

Table 19

Sl No	Nominal size of fastener, d mm	Size of the hole = Nominal Diameter of the fastener + clearances			
		Standards clearance in Diameter and width of slot	Over size clearance in Diameter	Clearance in the length of the slot	
				Short slot	Long Slot
i)	12 – 14	1.0	3.0	4.0	2.5 d
ii)	16 -22	2.0	4.0	6.0	2.5 d
iii)	24	2.0	6.0	8.0	2.5 d
iv)	Larger than 24	3.0	8.0	10.0	2.5 d

(P-73, IS 800 – 2007) Bolts and Bolting:

Pitch of the bolt:

Centre to centre distance between two adjacent bolts in a given row is called a pitch of a bolt.

Clause 10.2.2

a. ***Minimum Pitch*** – The distance between centres of bolts should be not less than 2.5 times the nominal diameter of the bolt.

b. ***Maximum Pitch*** **(Clause 10.2.3)**

Clause 10.2.3.1

i) The distance between centres of any two adjacent bolts (including tacking bolts) shall not exceed 32 t or 300 mm, whichever is less, where t is the thickness of the thinner outside plate.

Clause 10.2.3.2

ii) The distance between centres of two adjacent bolts, in a line lying in the direction of stress, shall not exceed 16 t or 200 mm, whichever is less in tension members and 12 t or 200 mm, whichever is less in compression members. In the case of compression members in which forces are transferred through butting faces, this distance shall not exceed 4.5 times the diameter of the bolts for a distance equal to 1.5 times the width of the member from the a butting faces.

Clause 10.2.3.3

iii) The distance between centres of any two consecutive bolts in a line adjacent and parallel to an edge of an outside plate shall not exceed (100 mm + 4t) or 200 mm, whichever is less in compression or tension members.

Clause 10.2.3.4

iv) When bolts are staggered at equal intervals and the gauge does not exceed 75 mm, the distances specified in (ii) and (iii) between centres of bolts, may be increased by 50 percent subjected maximum spacing specified in 10.2.3.1.

Clause 10.2.4.2 Edge and End Distance

The minimum edge and end distances from the centre of any hole to the nearest edge of a plate shall not be less than 1.7 times the hole diameter in case of sheared

or hand – flame cut edges; and 1.5 times the hole diameter in case of rolled, machine- flame cut, swan and planed edges.

Clause 10.2.5.2 Tacking Bolts -
Tacking bolts shall have spacing in line not exceeding 32 times the thickness of the thinner outside plate or 300 mm, whichever is less. Where the plates are exposed to the weather, the pitch in line shall not exceed 16 times, the thickness of the outside plate or 200 mm, whichever is less. In both cases, the lines of bolts shall not be apart at a distance greater than these pitches.

Clause 10.2.5.4: In tension members composed of two flats, angles, channels or less in contact back-to-back or separated back-to-back by a distance not exceeding the aggregate thickness of the connected parts, tacking bolts, with solid distance pieces where the parts are separated, shall be provided at pitch in line not exceeding 1000 mm.
Clause 10.2.5.5: For compression members, tacking bolts in a line shall be spaced at a distance not exceeding 600mm.

Design procedure for bolted joint:

Following are the steps for the design of a bolted joint:
1) The size of the bolt is determined by using ***Unwins*** formula:

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

Where, t = thickness of plate in mm, d = dia of bolt in mm.

2) Determine Bolt value (BV)
It is the least of the following
a) Strength of one bolt in single shear or double shear

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

or

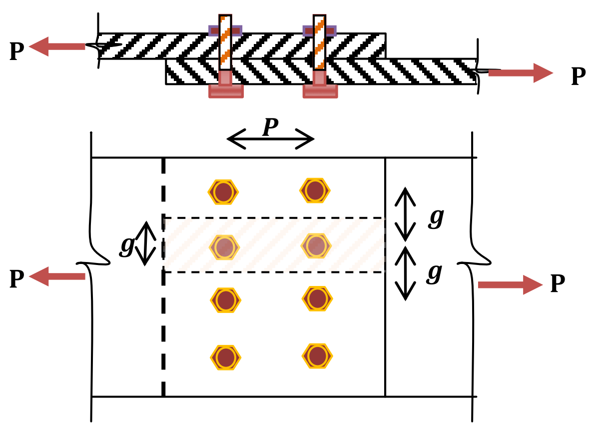
Strength of one bolt in Double shear $V_{dsb} = \left[\left(\frac{f_u}{\sqrt{3}}\right) \times \left(\frac{n_n A_{nb} + n_s A_{sb}}{\gamma_{mb}}\right)\right]$

b) Strength of one bolt in bearing

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

3) No of bolts $= \dfrac{\text{Force}}{\text{Bolt value}}$

The number of bolts so obtained is provided on one side of the joint and an equal number of bolts is provided on the other side of the joint also.



4) If gauge distance is to be determined for lap joint or butt joint, tearing strength of plate is determined which should be less than or equal to bolt value.

Strength of plate in tearing per gauge width of the plate

$$T_{dn-1} = \frac{0.9(g - d_0) \times t \times f_u}{\gamma_{m1}} \leq BV$$

From the above relation gauge distance can be determined

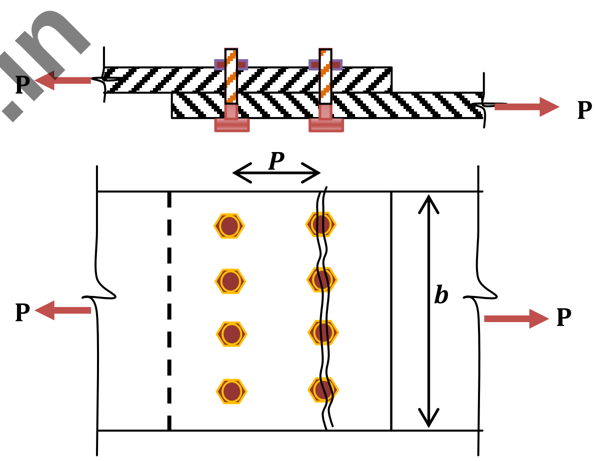
$$g = \frac{BV \times \gamma_{m1}}{0.9 \times t \times f_u} + d_o$$

If pitch is required:

Strength of plate in tearing per pitch

$$T_{dn-1} = \frac{0.9(p - d_0) \times t \times f_u}{\gamma_{m1}}$$

$$p = \frac{BV \times \gamma_{m1}}{0.9 \times t \times f_u} + d_o$$



5) If width of plate is to be determined, the strength of the plate in tearing for full width of the plate is equated to the force to be transmitted by the joint.

$$T_{dn} = \frac{0.9(b - nd_0) \times t \times f_u}{\gamma_{m1}} = \text{Pull/(Force)/(P)}$$

$$b = \frac{P \times \gamma_{m1}}{0.9 \times t \times f_u} + nd_0$$

n = no of holes along the failure section

6) Thickness of cover plate (in case of cover plate)

$t_c \not< \frac{5}{8}t$ for double cover butt joint

$t_c \not< \frac{9}{8}t$ for single cover butt joint

where, t_c = Thickness of cover plates

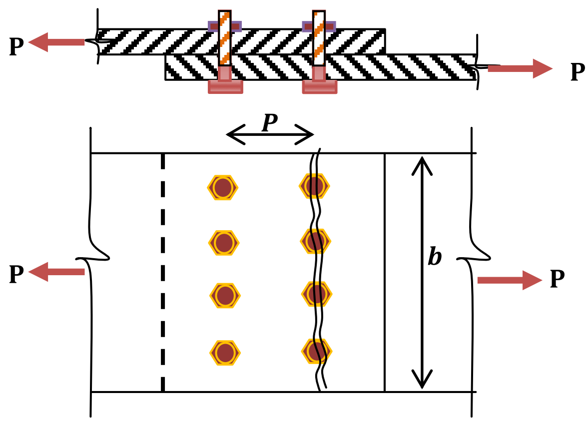
t = Thickness of main plate.

Example

Strength of bolted joint against shearing of bolts

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

Strength of bolted joint against shearing of bolts



$$V_{dsb} = 8 \times \left(\frac{f_u}{\sqrt{3}}\right) \times \left(\frac{n_n A_{nb} + n_s A_{sb}}{\gamma_{mb}}\right)$$

Strength of bolted joint against the bearing of bolts

$$V_{dpb} = N \times \left(\frac{2.5k_b \times d \times t \times f_u}{\gamma_{mb}}\right)$$

Strength of bolted joint against the bearing of bolts

$$V_{dpb} = 8 \times \left(\frac{2.5k_b \times d \times t \times f_u}{\gamma_{mb}}\right)$$

The strength of the plate in tearing

$$T_{dn} = \frac{0.9(b - nd_0) \times t \times f_u}{\gamma_{m1}}$$

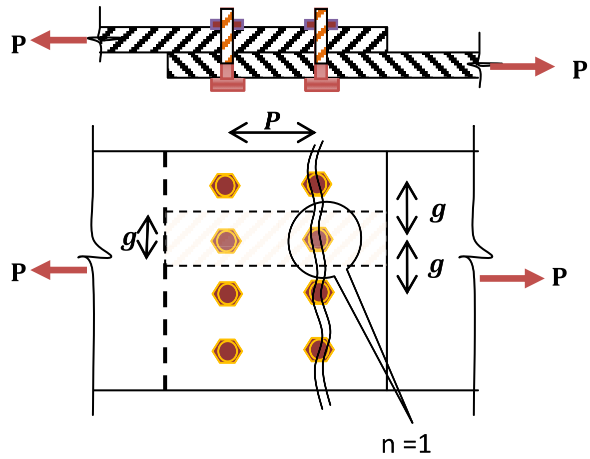
n = Number of holes along the tearing line =4

The strength of the plate in tearing $T_{dn} = \dfrac{0.9(b - 4d_0) \times t \times f_u}{\gamma_{m1}}$

The strength of a bolted joint is the least of V_{dsb}, V_{dpb} and T_{dn}.

Strength of bolted joint per gauge width of plate

Strength of bolt in single shear per gauge width



$$V_{dsb-1} = n \times \left(\frac{f_u}{\sqrt{3}}\right) \times \left(\frac{n_n A_{nb} + n_s A_{sb}}{\gamma_{mb}}\right)$$

Strength of bolt in single shear per gauge width

$$V_{dsb-1} = 2 \times \left(\frac{f_u}{\sqrt{3}}\right) \times \left(\frac{n_n A_{nb} + n_s A_{sb}}{\gamma_{mb}}\right)$$

Strength of bolt in bearing per gauge width

$$V_{dpb-1} = \frac{n \times (2.5k_b \times d \times t \times f_u)}{\gamma_{mb}}$$

Strength of bolt in bearing per gauge width

$$V_{dpb-1} = \frac{2 \times (2.5k_b \times d \times t \times f_u)}{\gamma_{mb}}$$

Strength of plate in tearing per gauge width of the plate

$$T_{dn-1} = \frac{0.9(g - nd_0) \times t \times f_u}{\Upsilon_{m1}} = \frac{0.9(g - 1 \times d_0) \times t \times f_u}{\Upsilon_{m1}}$$

n = Number of holes along the tearing line per gauge length =1

Strength of plate in tearing per pitch

$$T_{dn-1} = \frac{0.9(p - d_0) \times t \times f_u}{\gamma_{m1}}$$

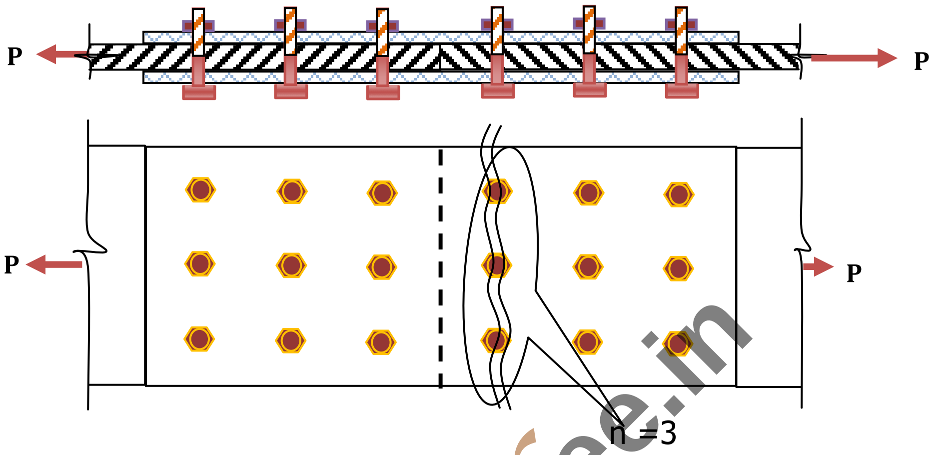
The strength of bolted joint per gauge width of plate is the least of V_{dsb-1}, V_{dpb-1} and T_{dn-1}.

Strength of bolted Butt Joint

Strength of bolted joint against shearing of bolts

$$V_{dsb-2} = N \times \left[\left(\frac{f_u}{\sqrt{3}}\right) \times \left(\frac{n_n A_{nb} + n_s A_{sb}}{\gamma_{mb}}\right)\right]$$

Where, N = Number of bolts on each side of the joint



Strength of bolted joint against shearing of bolts

$$V_{dsb-2} = 9 \times \left[\left(\frac{f_u}{\sqrt{3}}\right) \times \left(\frac{n_n A_{nb} + n_s A_{sb}}{\gamma_{mb}}\right)\right]$$

Strength of bolted joint against the bearing

$$V_{dpb-2} = N \times \left(\frac{2.5 \times k_b \times d \times t \times f_u}{\gamma_{mb}}\right)$$

Strength of bolted joint against the bearing

$$V_{dpb-2} = 9 \times \left(\frac{2.5 \times k_b \times d \times t \times f_u}{\gamma_{mb}}\right)$$

Strength of the plate in tearing

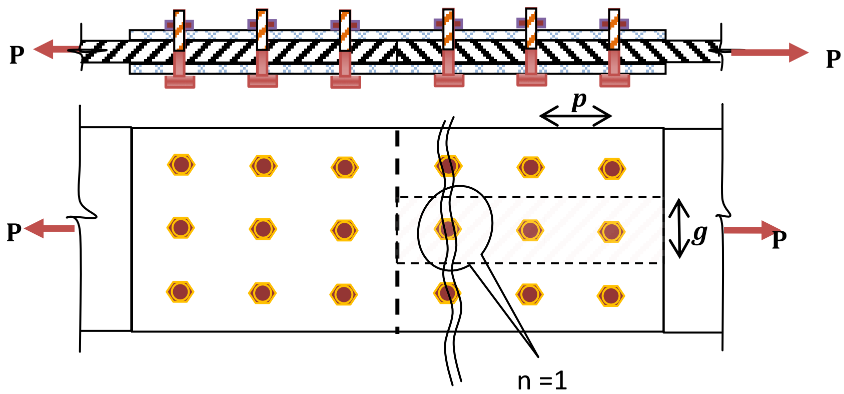
$$T_{dn-2} = \frac{0.9(b - nd_0) \times t \times f_u}{\gamma_{m1}}$$

Strength of the plate in tearing

$$T_{dn-2} = \frac{0.9(b - 3d_0) \times t \times f_u}{\gamma_{m1}}$$

The strength of bolted butt joint per gauge width of plate is the least of V_{dsb-2}, V_{dpb-2} and T_{dn-2}.

Strength of the bolt in double shear per gauge width



Strength of bolt in Double shear per gauge width

$$V_{dsb-1} = n \times \left[\left(\frac{f_u}{\sqrt{3}} \right) \times \left(\frac{n_n A_{nb} + n_s A_{sb}}{\gamma_{mb}} \right) \right]$$

Strength of bolt inDouble shear

$$V_{dsb-1} = 3 \times \left[\left(\frac{f_u}{\sqrt{3}} \right) \times \left(\frac{n_n A_{nb} + n_s A_{sb}}{\gamma_{mb}} \right) \right]$$

Strength of bolt in Bearing per gauge width

$$V_{dpb-1} = \frac{n \times (2.5 k_b \times d \times t \times f_u)}{\gamma_{mb}}$$

Strength of bolt in Bearing per gauge width

$$V_{dpb-1} = \frac{3 \times (2.5 k_b \times d \times t \times f_u)}{\gamma_{mb}}$$

Strength of plate in tearing per gauge width of the plate

$$T_{dn-1} = \frac{0.9(g - d_0) \times t \times f_u}{\gamma_{m1}}$$

Strength of plate in tearing per pitch

$$T_{dn-1} = \frac{0.9(p - d_0) \times t \times f_u}{\gamma_{m1}}$$

The strength of bolted butt joint per gauge width of plate is the least of V_{dsb-1}, V_{dpb-1}, and T_{dn-1}.

High Strength Friction Grip Bolts: (HSFG)

The HSFG bolts are made from high strength steel rods. The surface of the shank is kept unfinished as in the case of black bolts. These bolts are tightened to a proof load using calibrated wrenches. Hence they grip the members tightly. In addition nuts are prevented by using clamping devices. If the joint is subjected to shearing load it is primarily resisted by frictional force between the members and washers. The shank of the bolts is not subjected to any shearing. This result into no slippage in the joint. Hence such bolts can be used to connect members subjected to dynamic loads also. Commonly available nominal diameter of HSFG bolts are 16, 20, 24, 30 and 36 mm.

For commonly used HSFG bolts (Grade 8.8),

Ultimate stress f_{ub} = 800 N/mm^2, and yield stress f_{yb} = 0.8 x 800 = 640 N/mm^2.

Shear capacity (P - 76, Cl 10.4)

1) No slip condition: (Friction bolts)

Shear capacity :

$$V_{dsf} = \frac{V_{nsf}}{\gamma_{mf}} = \frac{\mu_f n_e k_h F_o}{\gamma_{mf}}$$

μ_f = Friction co - efficient (Slip factor) = 0.55 ,
if specific condition of surface treatment is given ref Table 20 of Page – 77.
n_e = Number of effective interfaces offering frictional resistance to slip

$n_e = 1$ for single shear

$n_e = 2$ for double shear (Butt joint with 2 cover plates)

$k_h = 1$

K_h = 1 for standard clearance hole,
K_h = 0.85 for oversized or short slotted,
K_h = 0.7 for long slotted hole,

Proof Load $F_o = A_{nb} \times f_o$

Proof Stress $f_o = 0.7 f_{ub}$

$$A_{nb} = \text{Area of thread} = 0.78 \times \frac{\pi \times d^2}{4}$$

$$\gamma_{mf} = 1.1 \text{ (Service load)}$$

2) Slip condition: (Bearing bolts)

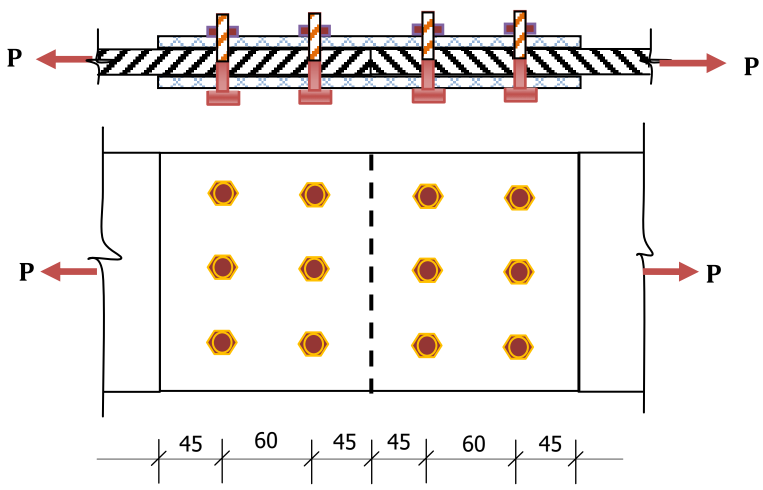
Shear capacity

$$V_{dsf} = \frac{V_{nsf}}{\gamma_{mf}} = \frac{\mu_f n_e k_h F_o}{\gamma_{mf}}$$

$\gamma_{mf} = 1.25$ (Ultimate load)

Problem:

Determine the shear capacity of bolts used in connecting two plates as shown in fig.
if (i) Slip resistance is designated at service load
(ii) Slip resistance is designated at ultimate load



Given:

i. HSFG bolts of grade 8.8 are used.
ii. Fasteners are in clearance holes.
iii. Coefficient of friction = 0.3.

Solution:

For HSFG bolts of Grade 8.8
Ultimate stress f_{ub} = 800 N/mm^2,
Coefficient of friction $\mu_f = 0.3$.

Shear capacity :

$$V_{dsf} = \frac{V_{nsf}}{\gamma_{mf}} = \frac{\mu_f n_e k_h F_o}{\gamma_{mf}}$$

n_e = Number of effective interfaces offering frictional resistance to slip

$n_e = 2$ for double shear (Butt joint with 2 cover plates)

$k_h = 1$

Proof Load $F_o = A_{nb} \times f_o$

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

Proof Stress $f_o = 0.7\,f_{ub} = 0.7 \times 800 = 560\ N/mm^2$

Proof Load $F_o = A_{nb} \times f_o = 245.04 \times 560 = 137.22 \times 10^3\ N$

$\gamma_{mf} = 1.1$ (Service load)

$\gamma_{mf} = 1.25$ (Ultimate load)

1) Design capacity of joint for Slip resistance designated at service load

$$V_{dsf} = N \times \frac{V_{nsf}}{\gamma_{mf}} = N \times \frac{\mu_f n_e k_h F_o}{\gamma_{mf}} = 6 \times \frac{0.3 \times 2 \times 1 \times 137.22 \times 10^3}{1.1 \times 1000} = 449.09\ kN$$

2) Design capacity of joint, if Slip resistance is designated at ultimate load

$$V_{dsf} = N \times \frac{V_{nsf}}{\gamma_{mf}} = N \times \frac{\mu_f n_e k_h F_o}{\gamma_{mf}} = 6 \times \frac{0.3 \times 2 \times 1 \times 137.22 \times 10^3}{1.25 \times 1000} = 395.20\ kN$$

Advantages of HSFG Bolts:

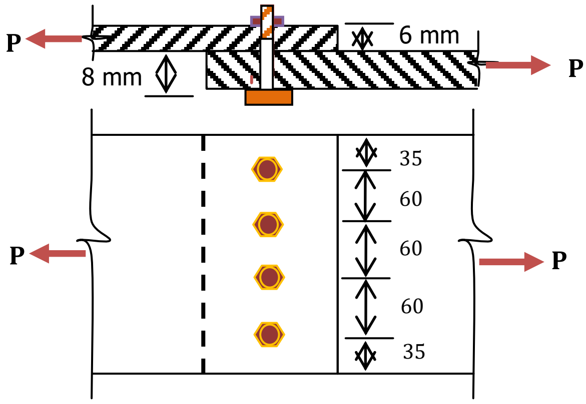
1. Joints are rigid, i.e., No slip takes place in the joint.
2. As load transfer is mainly by friction, the bolts are not subjected to shearing and bearing stresses.
3. High static strength due to high frictional resistance.
4. Smaller number of bolts results in smaller size of gusset plate.

Disadvantages of HSFG Bolts:

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

Ex 1:®

Two plates of 6 mm and 8 mm thick are connected by single bolted lap joint with 4–20 mm diameter bolts at 60 mm gauge width. Calculate the efficiency of the joint. Take f_u of plate as 410 MPa and assume 4.6 grade bolts. Take end distance = 35 mm.



Solution:

Dia of bolt d = 20 mm

Dia of hole = d_o = 20 + 2 = 22 mm

Gauge distance (g) = 60 mm, e = 35 mm

f_u of plate = 410 MPa

For 4.6 grade of bolt

f_u of bolt = 400 MPa

b= 2 x 35 + 3 x 60 = 250 mm

t = 6mm (Min thickness of 6mm and 8mm)

1) Strength of bolt in Single shear(P-75, Cl: 10.3.3)

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

No of bolts N = 4.

Assuming Shank is interfering the shear plane

$n_n = 0$ $n_s = 1$, $\gamma_{mb} = 1.25$

$$A_{sb} = \frac{\pi}{4}(d)^2 = \frac{\pi}{4}(20)^2 = 314.16\text{mm}^2,$$

$$V_{dsb} = 4 \times \frac{400}{\sqrt{3}} \times \left(\frac{0 + 1 \times 314.16}{1.25 \times 1000}\right) = 232.16\text{KN}$$

2)Strength of bolt in Bearing $V_{dpb} = N \times \dfrac{2.5 \times k_b \times d \times t \times f_u}{\gamma_{mb}}$

k_b is the least of the following:

1) $\dfrac{e}{3d_0} = \dfrac{35}{3 \times 22} = 0.53$

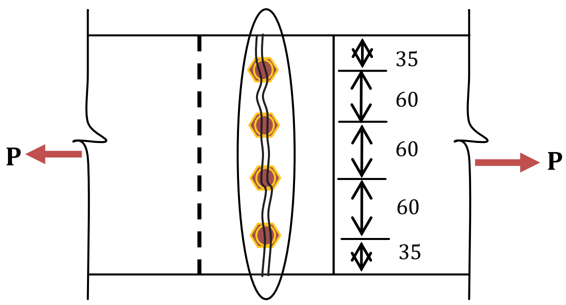
2) $\dfrac{p}{3d_0} - 0.25 = \dfrac{60}{3 \times 22} - 0.25 = 0.66$

3) $\dfrac{f_{ub}}{f_u} = \dfrac{400}{410} = 0.98$ 4) 1

$$V_{dpb} = 4 \times \frac{2.5 \times 0.53 \times 20 \times 6 \times 400}{1.25 \times 1000} = 203.53 \text{ kN}$$

3) Strength of plate in tearing

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



n = No of bolts along tearing section = 4

$$T_{dn} = \frac{0.9(b - nd_0) \times t \times f_u}{\gamma_{m1}}$$

$$= \frac{0.9 \times (250 - 4 \times 22) \times 6 \times 410}{1.25 \times 1000} = 286.93 \text{ KN}$$

The strength of bolted joint of plate is the least of V_{dsb}, V_{dpb} and T_{dn}.

$\therefore$ *The strength of bolted joint =* ***203.52*** **KN**

$$\text{Strength of solid plate} = \frac{0.9 \times b \times t \times f_u}{\gamma_{mb}}$$

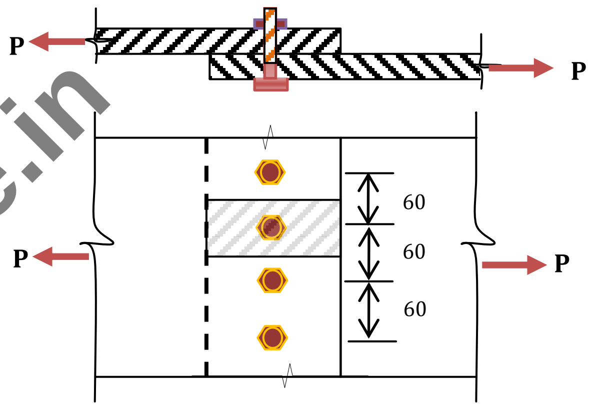
$$= \frac{0.9 \times 250 \times 6 \times 410}{1.25 \times 1000} = 442.8 \text{ KN}$$

$$\text{Efficiency of bolted joint } (\eta): \frac{\text{Strength of bolted joint}}{\text{Strength of solid plate}} \times 100$$

$$\eta = \frac{203.53}{442.8} \times 100 = 45.96\%$$

Ex – 2®:

Plates of 6 mm thick are connected by single bolted lap joint with 20 mm diameter bolts at 60 mm gauge width. Calculate the efficiency of the joint. Take f_u of plate as 410 MPa and assume 4.6 grade bolts.



Solution: Dia of bolt d = 20 mm

Dia of hole = d_o = 20 + 2 = 22 mm

Gauge distance (g) = 60 mm

f_u of plate = 410 MPa

For 4.6 grade of bolt

f_u of bolt = 400 MPa

t = 6mm

1) Strength of joint in Single shear of bolts per gauge width (**P-75, Cl: 10.3.3)**

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

No of bolts n = 1.

Assuming thread is interfering the shear plane

$n_n = 1 \qquad n_s = 0 \quad , \gamma_{mb} = 1.25$

$$A_{nb} = 0.78 \times \frac{\pi}{4}(d)^2 = 0.78 \times \frac{\pi}{4}(20)^2 = 245.62 \text{mm}^2,$$

$$V_{dsb1\text{-}1} = 1 \times \frac{400}{\sqrt{3}} \times \left(\frac{1 \times 245.62 + 0}{1.25 \times 1000}\right) = 45.27 \text{ KN}$$

2) Strength of bolt in Bearing per gauge width

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

k_b is the least of the following:

1) $\frac{e}{3d_0} \Rightarrow$ NA 2) $\frac{p}{3d_0} - 0.25 \Rightarrow$ NA 3) $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.98$ 4) 1

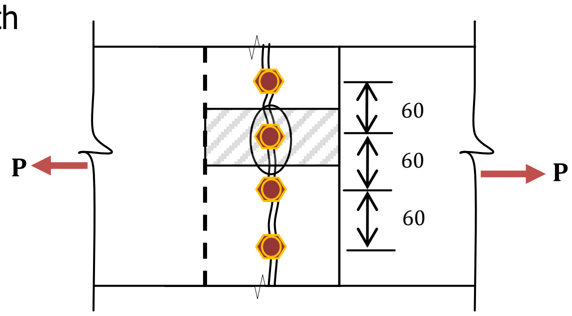
$$V_{dpb1-1} = 1 \times \frac{2.5 \times 0.98 \times 20 \times 6 \times 400}{1.25 \times 1000} = 94.08 \text{ KN}$$

3) Strength of plate in tearing per gauge width

$$T_{dn1-1} = \frac{0.9(g - nd_0) \times t \times f_u}{\gamma_{m1}}$$

$$= \frac{0.9 \times (60 - 1 \times 22) \times 6 \times 410}{1.25 \times 1000}$$

$$T_{dn1-1} = 67.31 \text{ KN}$$



The strength of bolted joint per gauge width of plate is the least of V_{dsb1-1}, V_{dpb1-1} and T_{dn1-1}.

∴ *The strength of bolted joint per gauge width* = ***45.27*** **KN**

$$\text{Strength of solid plate per gauge width} = \frac{0.9 \times g \times t \times f_u}{\gamma_{mb}}$$

$$= \frac{0.9 \times 60 \times 6 \times 410}{1.25 \times 1000} = 106.27 \text{ KN}$$

$$\text{Efficiency of bolted joint per gauge } (\eta): \frac{\text{Strength of bolted joint per gauge}}{\text{Strength of solid plate per gauge}} \times 100$$

$$\eta = \frac{45.27}{106.27} \times 100 = 42.60\%$$

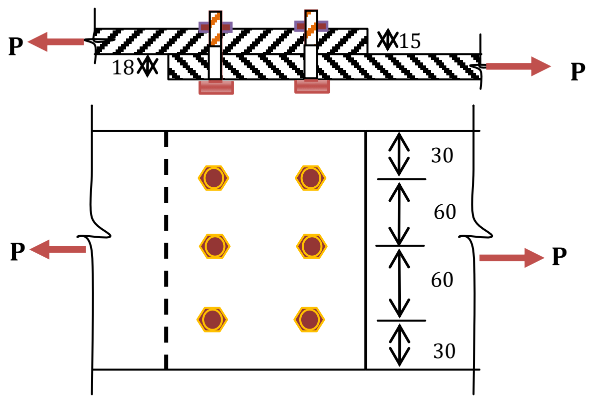
Example 3.®: Find the maximum force which can be transmitted through the lap joint shown in the fig. Find also the efficiency of the joint. Take f_u of plate as 410 MPa and assume 5.6 grade bolts.

Solution:

t = 15mm (Min of 15 mm and 18 mm)

Dia of bolt using Unwin's formula

$$d = 6.04\sqrt{t} = 6.04 \times \sqrt{15} = 23.39 \text{ mm}$$



Usually unwin's formula gives higher value, so diameter of bolt can be assumed lesser than calculated.

Say d = 20 m

Table 19, P-73

Dia of hole = d_o = 20 + 2 = 22 mm

Gauge (g) = 60 mm, e = 30 mm

f_u of plate = 410 MPa

For 4.6 grade of bolt

f_u of bolt = 500 MPa

b= 2 x 30 + 2 x 60 = 180 mm

1) Strength of bolt in Single shear

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

No of bolts N = 6.

Assuming Shank is interfering the shear plane

$$n_n = 0 \qquad n_s = 1 \quad , \gamma_{mb} = 1.25$$

$$A_{sb} = \frac{\pi}{4}(d)^2 = \frac{\pi}{4}(20)^2 = 314.16\ mm^2,$$

$$V_{dsb} = 6 \times \frac{500}{\sqrt{3}} \times \left(\frac{0 + 1 \times 314.16}{1.25 \times 1000}\right) = 435.31\ KN$$

2) Strength of bolt in Bearing $V_{dpb} = N \times \dfrac{2.5 \times k_b \times d \times t \times f_u}{\gamma_{mb}}$

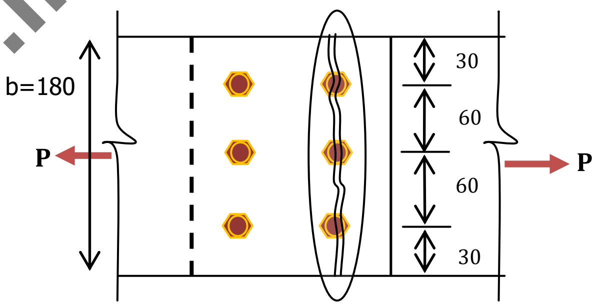
k_b is the least of the following:

1) $\dfrac{e}{3d_0} = \dfrac{30}{3 \times 22} = 0.454$ 2) $\dfrac{p}{3d_0} - 0.25 = \dfrac{60}{3 \times 22} - 0.25 = 0.66$

P = 2.5 x 20 = 50 mm Say 60mm

3) $\dfrac{f_{ub}}{f_u} = \dfrac{500}{410} = 1.22$ 4) 1

$$V_{dpb} = 6 \times \frac{2.5 \times 0.454 \times 20 \times 15 \times 500}{1.25 \times 1000} = 817.2\ KN$$



3. Strength of plate in tearing

$$T_{dn} = \frac{0.9(b - nd_0) \times t \times f_u}{\gamma_{m1}}$$

$$= \frac{0.9 \times (180 - 3 \times 22) \times 15 \times 410}{1.25 \times 1000} = 504.80\ KN$$

The strength of bolted joint of plate is the least of V_{dsb}, V_{dpb} and T_{dn}.

$\therefore$ *The strength of bolted joint = 384.25*KN

$$\text{Strength of solid plate} = \frac{0.9 \times b \times t \times f_u}{\gamma_{mb}}$$

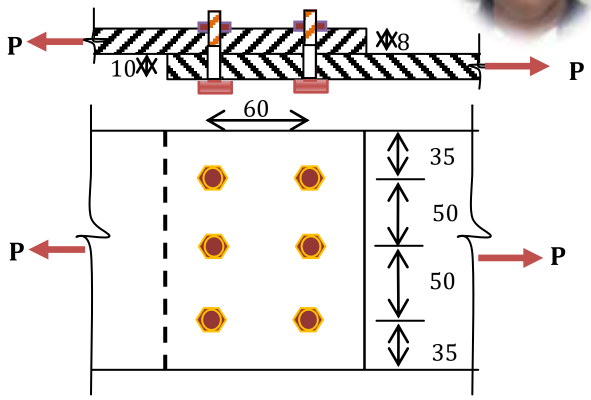
$$= \frac{0.9 \times 180 \times 15 \times 410}{1.25 \times 1000} = 797.04\ KN$$

$$\text{Efficiency of bolted joint } (\eta) : \frac{\text{Strength of bolted joint}}{\text{Strength of solid plate}} \times 100$$

$$\eta = \frac{435.31}{797.04} \times 100 = 54.61\ \%$$

Example 4.®: Find the maximum force which can be transmitted through the lap joint connection of 2 plates of 8 mm and 10 mm thick with 8 bolts at a pitch of 60 mm, gauge distance of 50 mm and edge distance of 35 mm. Also find the efficiency of the joint using bolts of property class 8.8.

Solution:



t = 8 mm (Min of 8 mm and 10 mm)

Dia of bolt using Unwin's formula

$$d = 6.04\sqrt{t} = 6.04 \times \sqrt{8} = 17.08 \text{ mm}$$

Say d = 16mn

Table 19, P-73

Dia of hole = d_o = 16 + 2 = 18 mm

Pitch = 60 mm, e = 35 mm

$f_u$of plate = 410 MPa

For 8.8 grade of bolt (HSFG bolt)

f_u of bolt = 800 MPa

b= 2 x 35 + 2 x 50 = 170 mm

1) <u>Strength of bolt in shear:</u> P-76, Cl: 10.4

$$V_{dsf} = N \times \frac{V_{nsf}}{\gamma_{mf}} = N \times \frac{\mu_f n_e k_h F_o}{\gamma_{mf}}$$

No of bolts N = 6, μ_f = 0.55, n_e = 1, k_h = 1

Proof Load $F_o = A_{nb} \times f_o$

$$A_{nb} = \text{Area of thread} = 0.78 \times \frac{\pi \times d^2}{4} = 0.78 \times \frac{\pi \times 16^2}{4} = 156.83 \text{ mm}^2$$

$$\text{Proof Stress } f_o = 0.7 f_{ub} = 0.7 \times 800 = 560 \text{ N/mm}^2$$

$$\text{Proof Load } F_o = A_{nb} \times f_o = 156.83 \times 560 = 87.82 \times 10^3 \text{ N}$$

$$V_{dsf} = 6 \times \frac{0.55 \times 1 \times 1 \times 87.82 \times 10^3}{1.1 \times 1000} = 263.46 \text{ kN}$$

2) <u>Strength of bolt in Bearing</u>

2) $$V_{dpb} = N \times \frac{2.5 \times k_b \times d \times t \times f_u}{\gamma_{mb}}$$

$k_b$is the least of the following:

1) $\frac{e}{3d_0} = \frac{35}{3 \times 18} = 0.65$ 2) $\frac{p}{3d_0} - 0.25 = \frac{60}{3 \times 18} - 0.25 = 0.86$

3) $\frac{f_{ub}}{f_u} = \frac{800}{410} = 1.95$ 4) 1

$$V_{dpb} = 6 \times \frac{2.5 \times 0.65 \times 16 \times 8 \times 800}{1.25 \times 1000} = 798.72 \text{ kN}$$

3) <u>Strength of plate in tearing:</u>

$$T_{dn} = \frac{0.9(b - nd_0) \times t \times f_u}{\gamma_{m1}}$$

$$= \frac{0.9 \times (170 - 3 \times 18) \times 8 \times 410}{1.25 \times 1000} = 273.95 \text{ KN}$$

The strength of bolted joint of plate is the least of V_{dsb}, V_{dpb} and T_{dn}.

∴ *The strength of bolted joint = 263.46 kN*

$$\text{Strength of solid plate} = \frac{0.9 \times b \times t \times f_u}{\gamma_{mb}}$$

$$= \frac{0.9 \times 170 \times 8 \times 410}{1.25 \times 1000} = 401.47 \text{ kN}$$

$$\text{Efficiency of bolted joint } (\eta) : \frac{\text{Strength of bolted joint}}{\text{Strength of solid plate}} \times 100$$

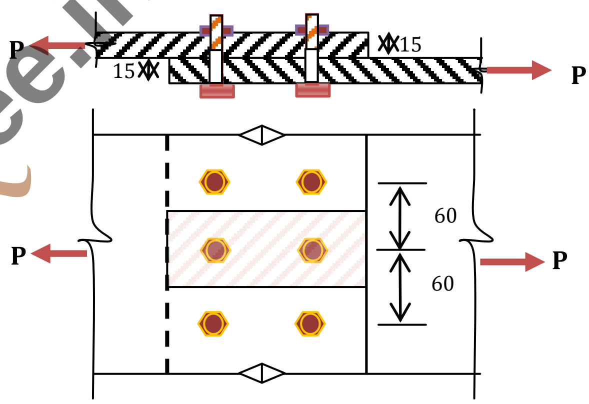
$$\eta = \frac{263.46}{401.47} \times 100 = 65.62\ \%$$

Example 5 ®: Find the maximum force per gauge width which can be transmitted through the lap joint shown in the fig. Find also the efficiency of the joint. Take f_u of plate as 410 MPa and assume 4.6 grade bolts.

Solution:

Dia of bolt using Unwin's formula

$$d = 6.04\sqrt{t} = 6.04 \times \sqrt{15} = 23.39\text{mm}$$



Say d = 20 m

Table 19, P-73

Dia of hole = d_o = 20 + 2 = 22 mm

Pitch = 60 mm, e = 30 mm

f_u of plate = 410 MPa

For 4.6 grade of bolt

f_u of bolt = 400 MPa

1) <u>Strength of bolt in Single shear</u>

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

No of bolts (n) = 2.

Assuming Shank is interfering the shear plane

$n_n = 0 \qquad n_s = 1 \quad , \gamma_{mb} = 1.25$

$$A_{sb} = \frac{\pi}{4}(d)^2 = \frac{\pi}{4}(20)^2 = 314.16 \text{ mm}^2,$$

$$V_{dsb} = 2 \times \frac{400}{\sqrt{3}} \times \left(\frac{0 + 1 \times 314.16}{1.25 \times 1000}\right) = 116.08\text{KN}$$

2) <u>Strength of bolt in Bearing</u>

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

k_b is the least of the following:

1) $\dfrac{e}{3d_0} = \dfrac{35}{3 \times 22} = 0.53$

e = 1.5 x do = 1.5 x 22 = 33 mm say 35 mm

2) $\dfrac{p}{3d_0} - 0.25 = \dfrac{60}{3 \times 22} - 0.25 = 0.66$

p = 2.5 x 20 = 50 mm say 60 mm

3) $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.98$ 4) 1

$$V_{dpb} = 2 \times \frac{2.5 \times 0.53 \times 20 \times 15 \times 400}{1.25 \times 1000} = 254.4 \text{ kN}$$

3) <u>Strength of plate in tearing:</u>

$$T_{dn} = \frac{0.9(p - d_0) \times t \times f_u}{\gamma_{m1}} = \frac{0.9 \times (60 - 22) \times 15 \times 410}{1.25 \times 1000} = 168.26 \text{ kN}$$

The strength of bolted joint of plate is the least of V_{dsb}, V_{dpb} and T_{dn}.

$\therefore$ *The strength of bolted joint = 116.08* KN

$$\text{Strength of solid plate per pitch length} = \frac{0.9 \times p \times t \times f_u}{\gamma_{mb}}$$

$$= \frac{0.9 \times 60 \times 15 \times 410}{1.25 \times 1000} = 265.68 \text{ kN}$$

$$\text{Efficiency of bolted joint } (\eta) : \frac{\text{Strength of bolted joint}}{\text{Strength of solid plate}} \times 100$$

$$\eta = \frac{116.08}{265.68} \times 100 = 43.70\%$$

Example 6: ®Determine the strength and efficiency of the lap joint shown in fig. Bolts are of 20 mm dia and of 4.6 grade. The two plates to be joined are 10 mm and 12 mm thick.

Solution:

Say d = 20 m

Table 19, P-73

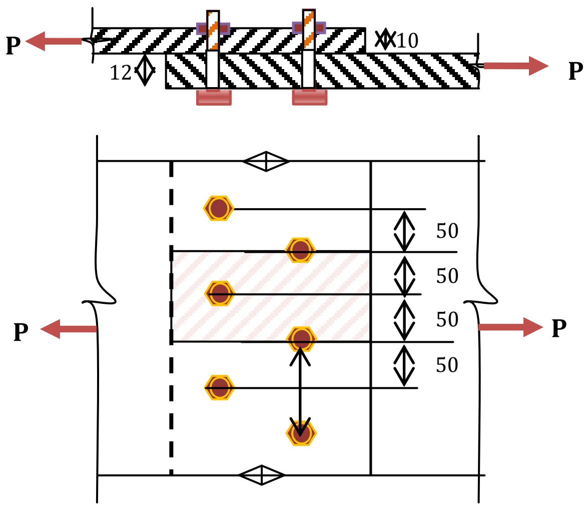
Dia of hole = d_o = 20 + 2 = 22 mm

g = 100 mm,

f_u of plate = 410 MPa

For 4.6 grade of bolt

f_u of bolt = 400 MPa



1) <u>Strength of bolt in Single shear per gauge width</u>

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

No of bolts (n) = 2.

Assuming Shank is interfering the shear plane

$n_n = 0$ $n_s = 1$, $\gamma_{mb} = 1.25$

$$A_{sb} = \frac{\pi}{4}(d)^2 = \frac{\pi}{4}(20)^2 = 314.16 \text{ mm}^2,$$

$$V_{dsb} = 2 \times \frac{400}{\sqrt{3}} \times \frac{0 + 1 \times 314.16}{1.25 \times 1000} = 116.08KN$$

2) <u>Strength of bolt in Bearing</u>

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

k_b is the least of the following:

1) $\frac{e}{3d_0} = NA$ 2) $\frac{p}{3d_0} - 0.25 = NA$ 3) $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.98$ 4) 1

$$V_{dpb} = 2 \times \frac{2.5 \times 0.98 \times 20 \times 10 \times 400}{1.25 \times 1000} = 313.60 \text{ kN}$$

3.**Strength of plate in tearing:**

$$T_{dn} = \frac{0.9(g - nd_0) \times t \times f_u}{\gamma_{m1}} = \frac{0.9 \times (100 - 1 \times 22) \times 10 \times 410}{1.25 \times 1000} = 230.26 \text{ KN}$$

The strength of bolted joint of plate is the least of V_{dsb}, V_{dpb} and T_{dn}.

$\therefore$ *The strength of bolted joint = 116.08* KN

$$\text{Strength of solid plate per pitch length} = \frac{0.9 \times p \times t \times f_u}{\gamma_{mb}}$$

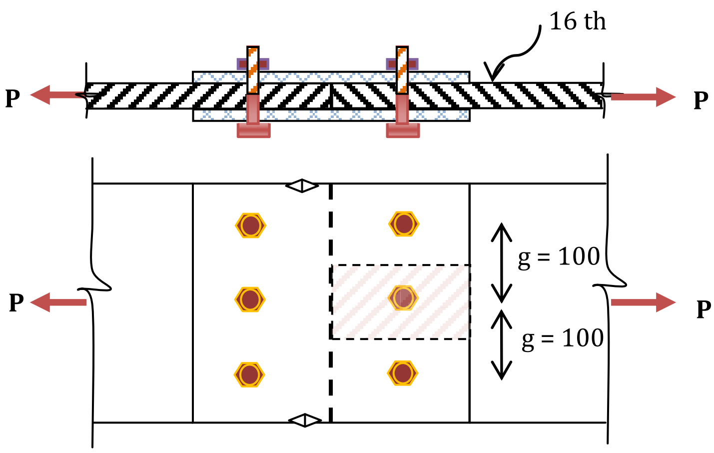
$$= \frac{0.9 \times 100 \times 10 \times 410}{1.25 \times 1000} = 295.2 \text{ kN}$$

$$\text{Efficiency of bolted joint } (\eta): \frac{\text{Strength of bolted joint}}{\text{Strength of solid plate}} \times 100$$

$$\eta = \frac{116.08}{295.26} \times 100 = 39.32\%$$

Example 7.®: A single bolted double cover butt joint of plates 16 mm thick is made with 22 mm dia bolts at a gauge of 100 mm. find the safe load per gauge length of the joint. Find also the efficiency of the joint.

Solution:



Dia of bolt d = 22mm

Table 19, P-73

Dia of hole = d_0 = 22 + 2 = 24 mm

Gauge length (g) = 100 mm

f_u of plate = 410 MPa

Assuming 4.6 grade of bolt

f_u of bolt = 400 MPa

f_y = 0.6 x 400 = 240 MPa

Thickness of double cover plate

$$\not< \frac{5}{8} \times t = \frac{5}{8} \times 16 = 10mm$$

Say t_c = 12 mm

1) Strength of bolt in double shear per gauge width

$$V_{dsb} = n\left[\left(\frac{f_u}{\sqrt{3}}\right)\times\left(\frac{n_n A_{nb} + n_s A_{sb}}{\gamma_{mb}}\right)\right]$$

Assuming both shank and thread interfere the shear plane.

$$n_n = 1, \quad n_s = 1, \quad \gamma_{mb} = 1.25$$

$$A_{sb} = \frac{\pi}{4} d^2 = \frac{\pi}{4}\times 22^2 = 380.13\ \text{mm}^2$$

$$A_{nb} = 0.78\times\frac{\pi}{4} d^2 = 0.78\times 380.13 = 296.50\ \text{mm}^2$$

$$V_{dsb} = 1\times\left[\frac{400}{\sqrt{3}}\times\left(\frac{1\times 380.13 + 1\times 296.50}{1.25\times 1000}\right)\right] = 125.01\ \text{KN}$$

2) Strength of bolt in Bearing

$$V_{dpb} = n\times\left[\frac{2.5\times k_b\times d\times t^*\times f_u}{\gamma_{mb}}\right]$$

t* = Min thickness of a) Thickness of main plate = 16 mm

b) Sum of the thickness of cover plate = 12+12=24 mm

Therefore t* *= 16 mm*

k_b is the least of the following:

1) $\frac{e}{3d_0} = NA$ 2) $\frac{p}{3d_0} - 0.25 = NA$ 3) $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.98$ 4) 1

$$V_{dpb} = 1\times\left[\frac{2.5\times 0.98\times 22\times 16\times 400}{1.25\times 1000}\right] = 276\ \text{KN}$$

3) Strength of plate in tearing per gauge width

$$T_{dn} = \frac{0.9(g - d_0)\times t\times f_u}{\gamma_{m1}} = \frac{0.9\times(100 - 24)\times 16\times 410}{1.25\times 1000} = 358.96\ \text{KN}$$

The strength of bolted joint per gauge width of plate is the least of V_{dsb}, V_{dpb} and T_{dn}.

∴ *The strength of bolted joint per gauge width =* ***125.01*** **KN**

$$\text{Strength of solid plate per gauge length} = \frac{0.9\times g\times t\times f_u}{\gamma_{mb}}$$

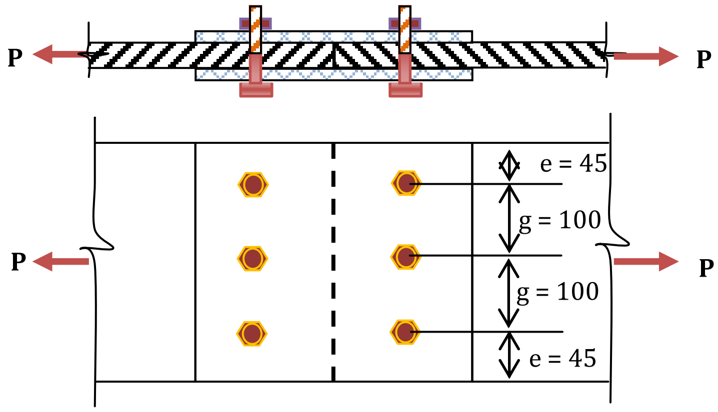
$$= \frac{0.9\times 100\times 16\times 410}{1.25\times 1000} = 472.32\ \text{KN}$$

$$\text{Efficiency of bolted joint } (\eta): \frac{\text{Strength of bolted joint per gauge length}}{\text{Strength of solid plate per gauge length}}\times 100$$

$$\eta = \frac{125.01}{472.32}\times 100 = 26.47\%$$

Example 8:®: A single bolted double cover butt joint of plates 16 mm thick is made with 3-22 mm dia bolts at a gauge of 100 mm. Find the safe load of the joint. Also, determine the efficiency of the joint.

Solution:



Dia of bolt d = 22mm

Table 19, P-73

Dia of hole = d_o = 22 + 2 = 24 mm

Gauge length = 100 mm

f_u of plate = 410 MPa

Assuming 4.6 grade of bolt

f_u of bolt = 400 MPa

Thickness of double cover plate

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

Say t_c = 12 mm

1) **Strength of bolted joint in double shear:**

$$V_{dsb} = N \times \left[\left(\frac{f_u}{\sqrt{3}}\right)\times\left(\frac{n_n A_{nb} + n_s A_{sb}}{\gamma_{mb}}\right)\right]$$

Assuming both shank and thread interfere the shear plane.

$$n_n = 1, \quad n_s = 1, \quad \gamma_{mb} = 1.25$$

$$A_{sb} = \frac{\pi}{4}d^2 = \frac{\pi}{4}\times 22^2 = 380.13\ mm^2$$

$$A_{nb} = 0.78\times\frac{\pi}{4}d^2 = 0.78\times 380.13 = 296.50\ mm^2$$

$$V_{dsb} = 3\times\left[\frac{400}{\sqrt{3}}\times\left(\frac{1\times 380.13 + 1\times 296.50}{1.25\times 1000}\right)\right] = 375.03KN$$

2) **Strength of bolted joint in Bearing:**

$$V_{dpb} = N\times\left[\frac{2.5\times k_b\times d\times t^*\times f_u}{\gamma_{mb}}\right]$$

t* = Min thickness of a) Thickness of main plate = 16 mm

b) Sum of the thickness of cover plate = 12+12=24 mm

Therefore t = 16 mm

k_b is the least of the following:

1) $\frac{e}{3d_0} = \frac{40}{3\times 24} = 0.56$ Edge distance e = 1.5 x 24 = 36 mm say 40 mm

2) $\frac{p}{3d_0} - 0.25 = \frac{100}{3\times 24} - 0.25 = 1.14$ 3) $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.98$ 4) 1

$$V_{dpb} = 3\times\left[\frac{2.5\times 0.56\times 22\times 16\times 400}{1.25\times 1000}\right] = 473.10\ KN$$

3) Strength of plate in tearing for full width of plate:

b = 2 x 40 + 2 x 100 = 280 mm

$$T_{dn} = \frac{0.9(b - nd_0) \times t \times f_u}{\gamma_{m1}}$$

$$T_{dn} = \frac{0.9 \times (280 - 3 \times 24) \times 16 \times 410}{1.25 \times 1000} = 982.43 \text{ KN}$$

The strength of bolted joint is the least of V_{dsb}, V_{dpb} and T_{dn}.

$\therefore$ *The strength of bolted joint =**375.03*** **KN**

$$\text{Strength of solid plate for full width} = \frac{0.9 \times b \times t \times f_u}{\gamma_{mb}}$$

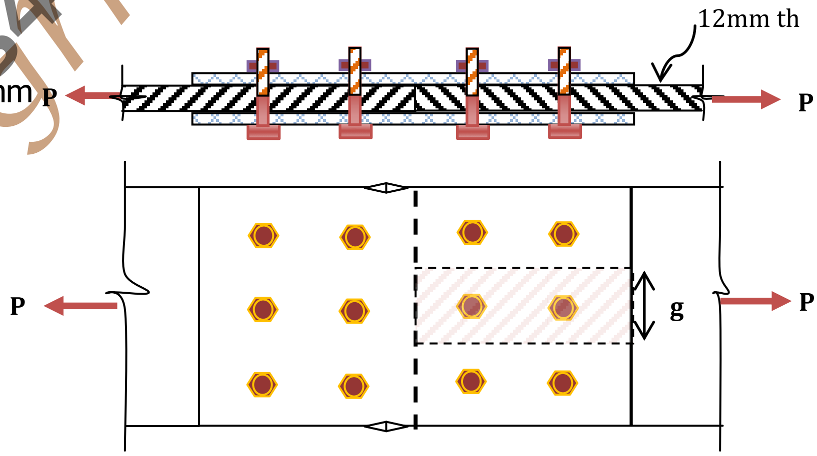
$$= \frac{0.9 \times 280 \times 16 \times 410}{1.25 \times 1000} = 1322.50 \text{ KN}$$

$$\text{Efficiency of bolted joint } (\eta): \frac{\text{Strength of bolted joint}}{\text{Strength of solid plate}} \times 100$$

$$\eta = \frac{375.03}{1322.50} \times 100 = 28.36\%$$

Example 9®: A double bolted double cover butt joint is used to connect plates of 12 mm thick. Determine the efficiency of the joint.

Solution:



Thickness of cover plate

$\not{t} \frac{5}{8} \times t = \frac{5}{8} \times 12 = 7.5\text{mm}$ Say 8 mm

t* = Min thickness of

a) Thickness of main plate = 12 mm

b) Sum of the thickness of cover plate = 16 mm

Dia of bolt using unwin's formula

$$d = 6.04\sqrt{t^*} = 6.04\sqrt{12} = 20.92 \text{ mm}$$

Say dia of bolt = 20 mm.

Table 19, P-73, Dia of hole = d_o = 20 + 2 = 22 mm

Assuming 4.6 grade of bolt, f_u of bolt = 400 MPa

f_u of plate = 410 MPa

1) Strength of bolt in double shear per gauge width

$$V_{dsb} = n\left[\left(\frac{f_u}{\sqrt{3}}\right) \times \left(\frac{n_n A_{nb} + n_s A_{sb}}{\gamma_{mb}}\right)\right]$$

Assuming both shank and thread interfere the shear plane.

$n_n = 1, \quad n_s = 1, \quad \gamma_{mb} = 1.25$

$$A_{sb} = \frac{\pi}{4} d^2 = \frac{\pi}{4} \times 20^2 = 314.16 \text{ mm}^2$$

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

$$V_{dsb} = 2 \times \left[\frac{400}{\sqrt{3}} \times \left(\frac{1 \times 245.04 + 1 \times 314.16}{1.25 \times 1000} \right) \right] = 206.63 \text{ KN}$$

2) **Strength of bolt in Bearing**

$$V_{dpb} = n \times \left[\frac{2.5 \times k_b \times d \times t^* \times f_u}{\gamma_{mb}} \right]$$

1) $\frac{e}{3d_0} = \frac{35}{3 \times 22} = 0.53$ Edge distance e = 1.5 x 22 = 33 mm say 35 mm

Assuming pitch $\not<$ 2.5 dia of bolt = 2.5 x 20 = 50 mm, Say 60 mm

2) $\frac{p}{3d_0} - 0.25 = \frac{60}{3 \times 22} - 0.25 = 0.66$ 3) $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.98$ 4) 1

$$V_{dpb} = 2 \times \left[\frac{2.5 \times 0.53 \times 20 \times 12 \times 400}{1.25 \times 1000} \right] = 203.52 \text{ KN}$$

The strength of plate in tearing per gauge width of plate is the least of V_{dsb} and V_{dpb}

3) **Strength of plate in tearing per gauge width**

$$T_{dn} = \frac{0.9 A_n \times f_u}{\gamma_{m1}} = 203.52 \times 10^3 \text{ N}$$

$$T_{dn} = \frac{0.9(g - d_0) \times t \times f_u}{\gamma_{m1}} = \frac{0.9 \times (g - 22) \times 12 \times 410}{1.25} = 203.52 \times 10^3 \text{ N}$$

$$g = \frac{203.52 \times 10^3 \times 1.25}{0.9 \times 12 \times 410} + 22 = 79.45 \text{mm}$$

Pitch $\not<$ 2.5 dia of bolt = 2.5 x 20 = 50 mm

Let the pitch or gauge width = 80 mm.

Strength of plate in tearing per gauge width

$$T_{dn} = \frac{0.9(g - d_0) \times t \times f_u}{\gamma_{m1}} = \frac{0.9 \times (80 - 22) \times 12 \times 410}{1.25 \times 1000} = 205.60 \text{ KN}$$

The strength of bolted joint per gauge width of plate is the least of V_{dsb}, V_{dpb} and T_{dn}.

$\therefore$ *The strength of bolted joint per gauge width =* ***203.52*** **KN**

$$\text{Strength of solid plate per gauge width} = \frac{0.9 \times g \times t \times f_u}{\gamma_{mb}}$$

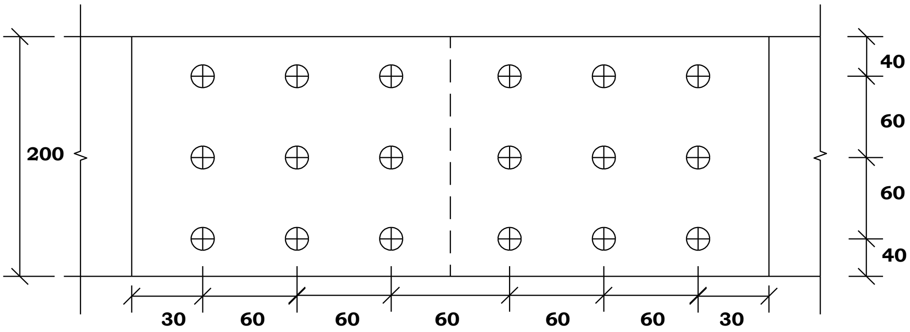
$$= \frac{0.9 \times 80 \times 12 \times 410}{1.25 \times 1000} = 283.40 \text{ KN}$$

$$\text{Efficiency of bolted joint } (\eta) : \frac{\text{Strength of bolted joint per gauge width}}{\text{Strength of solid plate per gauge width}} \times 100$$

$$\eta = \frac{203.52}{283.40} \times 100 = 71.80 \text{ \%}$$

Example 10.®: A double cover butt joint is used to connect two flats 200 ISF 10 with 8 mm cover plates. The two plates are connected by 9 bolts in chain bolting at a pitch of 60 mm, arranged in three rows with three bolts in each row as shown in fig. Determine the strength and efficiency of the joint. The dia of bolt used is 20 mm. Assume grade of bolt as 5.6.

Solution:



Size of flat used = 200 ISF 10, width of flat = 200 mm,

thickness of flat = 10 mm.

Dia of bolt d = 20 mm

Table 19, P-73

Dia of hole = d_o = 20 + 2 = 22 mm

Pitch = 60 mm

Assuming 5.6 grade of bolt, f_u of bolt = 500 MPa, f_u of plate = 410 MPa

Total no of bolts N = 9

1) **Strength of bolted joint in double shear**

$$V_{dsb} = N\left[\left(\frac{f_u}{\sqrt{3}}\right)\times\left(\frac{n_n A_{nb} + n_s A_{sb}}{\gamma_{mb}}\right)\right]$$

Assuming both shank and thread interfere the shear plane.

$$n_n = 1, \quad n_s = 1, \quad \gamma_{mb} = 1.25$$

$$A_{sb} = \frac{\pi}{4}d^2 = \frac{\pi}{4}\times 20^2 = 314.16\ mm^2$$

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

$$V_{dsb} = 9\times\left[\frac{500}{\sqrt{3}}\times\left(\frac{1\times 245.20 + 1\times 314.16}{1.25\times 1000}\right)\right] = 1162.61\ KN$$

2) **Strength of bolt in Bearing**

$$V_{dpb} = N\times\left[\frac{2.5\times k_b\times d\times t^*\times f_u}{\gamma_{mb}}\right]$$

t* = Min thickness of a) Thickness of main plate = 10 mm

b) Sum of the thickness of cover plate = 8 + 8 = 16 mm

Therefore t = 10 mm

k_b is the least of the following:

1) $\frac{e}{3d_0} = \frac{40}{3\times 22} = 0.61$ 2) $\frac{p}{3d_0} - 0.25 = \frac{60}{3\times 22} - 0.25 = 0.66$

3) $\frac{f_{ub}}{f_u} = \frac{500}{410} = 1.22$ 4) 1

$$V_{dpb} = 9 \times \left[\frac{2.5 \times 0.61 \times 20 \times 10 \times 500}{1.25 \times 1000}\right] = 1098 \text{ KN}$$

3) Strength of plate in tearing

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

$$T_{dn} = \frac{0.9(b - nd_0) \times t \times f_u}{\gamma_{m1}} = \frac{0.9 \times (200 - 3 \times 22) \times 10 \times 410}{1.25 \times 1000} = 395.60 \text{ KN}$$

The strength of bolted joint is the least of V_{dsb}, V_{dpb} and T_{dn}.

$\therefore$ *The strength of bolted joint =**395.60*** **KN**

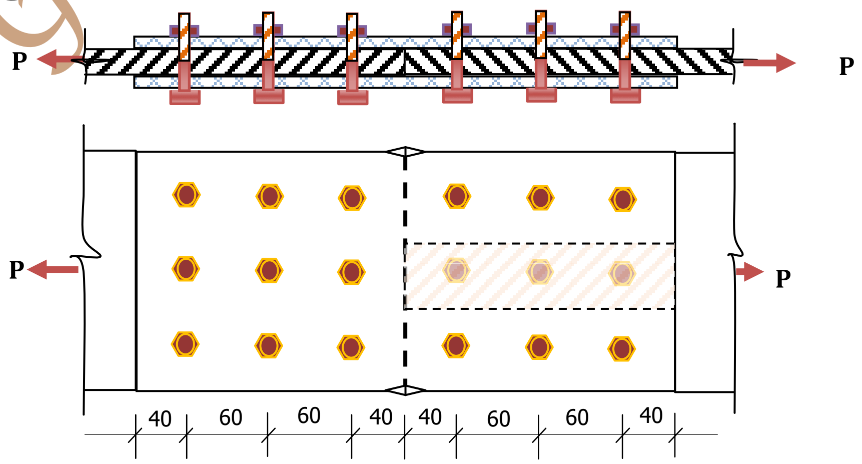
$$\text{Strength of solid plate for full width of plate} = \frac{0.9 \times b \times t \times f_u}{\gamma_{mb}}$$

$$= \frac{0.9 \times 200 \times 10 \times 410}{1.25 \times 1000} = 590.4 \text{ KN}$$

$$\text{Efficiency of bolted joint } (\eta): \frac{\text{Strength of bolted joint}}{\text{Strength of solid plate}} \times 100$$

$$\eta = \frac{395.60}{590.4} \times 100 = 67\%$$

Example 11:® A double cover butt joint is used to connect two flats of 10 mm with 8 mm cover plates. The two plates are connected by chain bolting at a pitch of 60 mm, arranged in three rows with 20 mm dia bolts. Determine the strength and efficiency of the joint. Assume grade of bolt as 4.6.



Solution:

Thickness of flat = 10 mm.

Dia of bolt d = 20 mm

Table 19, P-73, Dia of hole = d_0

= 20 + 2 = 22 mm

Pitch = 60 mm

Assuming 4.6 grade of bolt

f_u of bolt = 400 MPa

f_u of plate = 410 MPa

Total no of bolts per pitch width

(n) = 3

1) Strength of bolt in double shear:

$$V_{dsb} = n\left[\left(\frac{f_u}{\sqrt{3}}\right) \times \left(\frac{n_n A_{nb} + n_s A_{sb}}{\gamma_{mb}}\right)\right]$$

Assuming both shank and thread interfere the shear plane.

$$n_n = 1, \quad n_s = 1, \quad \gamma_{mb} = 1.25 \quad A_{sb} = \frac{\pi}{4} d^2 = \frac{\pi}{4} \times 20^2 = 314.16 \text{ mm}^2$$

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

$$V_{dsb} = 3 \times \left[\frac{400}{\sqrt{3}} \times \left(\frac{1 \times 245.20 + 1 \times 314.16}{1.25 \times 1000} \right) \right] = 310.03 \text{ KN}$$

2) **Strength of bolt in Bearing**

$$V_{dpb} = n \times \left[\frac{2.5 \times k_b \times d \times t^* \times f_u}{\gamma_{mb}} \right]$$

t* = Min thickness of a) Thickness of main plate = 10 mm

b) Sum of the thickness of cover plate = 8 + 8 =16 mm

Therefore t = 10 mm

k_b is the least of the following:

1) $\frac{e}{3d_0} = \frac{40}{3 \times 22} = 0.61$ 2) $\frac{p}{3d_0} - 0.25 = \frac{60}{3 \times 22} - 0.25 = 0.66$ 3) $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.98$

4) 1

$$V_{dpb} = 3 \times \left[\frac{2.5 \times 0.61 \times 20 \times 10 \times 400}{1.25 \times 1000} \right] = 292.8 \text{ KN}$$

3) Strength of plate in tearing per pitch

$$T_{dn} = \frac{0.9(p - nd_0) \times t \times f_u}{\gamma_{m1}} = \frac{0.9 \times (60 - 1 \times 22) \times 10 \times 410}{1.25 \times 1000} = 112.18 \text{ KN}$$

The strength of bolted joint per gauge width of plate is the least of V_{dsb}, V_{dpb} and T_{dn}.

$\therefore$ *The strength of bolted joint per gauge width = 112.18*KN

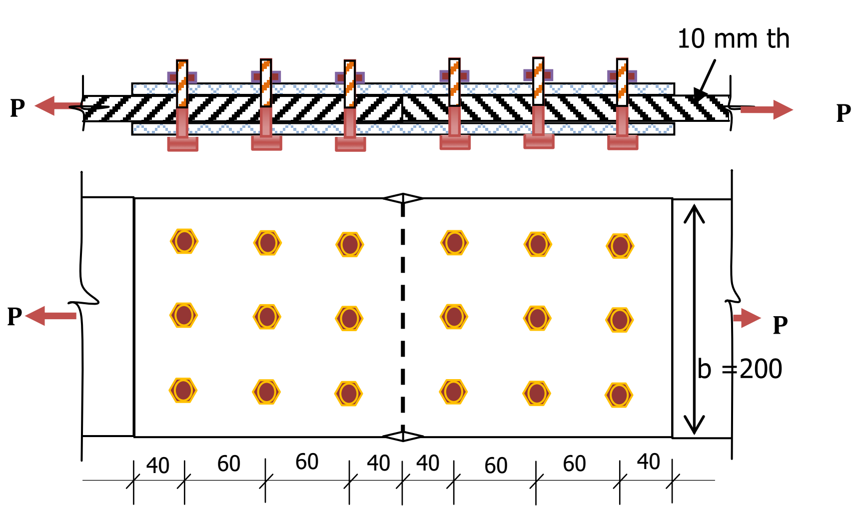
$$\text{Strength of solid plate per pitch length} = \frac{0.9 \times p \times t \times f_u}{\gamma_{mb}}$$

$$= \frac{0.9 \times 60 \times 10 \times 410}{1.25 \times 1000} = 177.12 \text{ KN}$$

$$\text{Efficiency of bolted joint } (\eta) = \frac{\text{Strength of bolted joint per pitch length}}{\text{Strength of solid plate per pitch length}} \times 100$$

$$\eta = \frac{112.18}{177.12} \times 100 = 63.34\%$$

Example 12:® A double cover butt joint is used to connect two flats 200 ISF 10. The two plates are connected by 9 bolts by chain bolting system at a pitch of 60 mm and edge distance of 40 mm arranged in three rows with three bolts in each row. Determine the strength and efficiency of the joint using HSFG bolts of 8.8 grade. The surfaces are not treated.



Solution:

Thickness of cover plate $\not< \frac{5}{8} \times t = \frac{5}{8} \times 10 = 6.25\ \text{mm}$ Say 8 mm

t* = Min thickness of

a) Thickness of main plate = 10 mm

b) Sum of the thickness of cover plate = 8 + 8 = 16 mm

Dia of bolt using unwin's formula

$$d = 6.04\sqrt{t} = 6.04\sqrt{10} = 19.01\ \text{mm}$$

Say Dia of bolt d = 18 mm

Table 19, P-73, Dia of hole = d_o = 18 + 2 = 20 mm

Pitch = 60 mm, HSFG bolts of 8.8 grade, f_u of bolt = 800 MPa

f_u of plate = 410 MPa

1) <u>Strength of bolt in double shear: P-76, Cl: 10.4</u>

$$V_{dsf} = N \times \frac{V_{nsf}}{\gamma_{mf}} = N \times \frac{\mu_f n_e k_h F_o}{\gamma_{mf}}$$

No of bolts N = 9, μ_f = 0.20, P- 77, table 20, n_e = 2 (Double shear), k_h = 1

Proof Load $F_o = A_{nb} \times f_o$

$$A_{nb} = \text{Area of thread} = 0.78 \times \frac{\pi \times d^2}{4} = 0.78 \times \frac{\pi \times 18^2}{4} = 198.49\ \text{mm}^2$$

$$\text{Proof Stress } f_o = 0.7\, f_{ub} = 0.7 \times 800 = 560\ \text{N/mm}^2$$

$$\text{Proof Load } F_o = A_{nb} \times f_o = 198.49 \times 560 = 111.15 \times 10^3\ \text{N}$$

$$V_{dsf} = 9 \times \frac{0.20 \times 2 \times 1 \times 111.15 \times 10^3}{1.1 \times 1000} = 363.76\ \text{kN}$$

2) **<u>Strength of bolt in Bearing</u>**

$$V_{dpb} = N \times \left[\frac{2.5 \times k_b \times d \times t^* \times f_u}{\gamma_{mb}}\right]$$

k_b is the least of the following:

1) $\frac{e}{3d_0} = \frac{40}{3 \times 20} = 0.67$ 2) $\frac{p}{3d_0} - 0.25 = \frac{60}{3 \times 20} - 0.25 = 0.75$

3) $\frac{f_{ub}}{f_u} = \frac{800}{410} = 1.95$ 4) 1

$$V_{dpb} = 9 \times \left[\frac{2.5 \times 0.67 \times 18 \times 10 \times 800}{1.25 \times 1000}\right] = 1736.64\ \text{KN}$$

3) <u>Strength of plate in tearing</u>

$$T_{dn} = \frac{0.9(b - nd_0) \times t \times f_u}{\gamma_{m1}} = \frac{0.9 \times (200 - 3 \times 20) \times 10 \times 410}{1.25 \times 1000} = 413.28\ \text{KN}$$

The strength of bolted joint : Is the least of V_{dsb}, V_{dpb} and T_{dn}.

$\therefore$ *The strength of bolted joint =363.76* KN

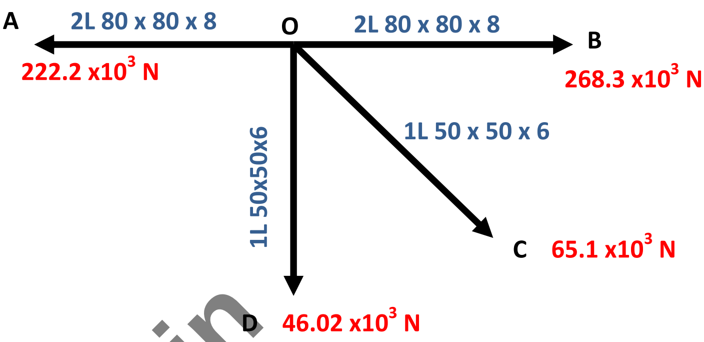
$$\text{Strength of solid plate for full length} = \frac{0.9 \times b \times t \times f_u}{\gamma_{mb}}$$

$$= \frac{0.9 \times 200 \times 10 \times 410}{1.25 \times 1000} = 590.40 \text{ KN}$$

$$\text{Efficiency of bolted joint } (\eta) = \frac{\text{Strength of bolted joint}}{\text{Strength of solid plate}} \times 100$$

$$\eta = \frac{363.76}{590.40} \times 100 = 61.61\ \%$$

Example 13:® Four members OA, OB, OC and OD are to be connected at a joint "O" to a 10 mm thick gusset plate. Details of the members and the loads carried by them are shown in fig. If the connection to the gusset plate be made with 18 mm dia bolts, find the number of bolts required to connect each member to the gusset plate. Sketch the joint details.



Solution:

Dia of bolt = 18 mm

Thickness of gusset plate = 10 mm

Members OA and OB are designed as Discontinuous Members

a) Member OA : 2- ISA 80 x 80 x 8 mm

Force in the member = 222.2 x 10^3N = 222.2 KN

1) Strength of one bolt double shear

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

$n_n = 1,\ \ n_s = 1$ $\because$ Shank and tread interfere the shear plane.

$$\gamma_{mb} = 1.25$$

$$A_{sb} = \frac{\pi}{4} d^2 = \frac{\pi}{4} 18^2 = 254.50 \text{ mm}^2$$

$$A_{nb} = 0.78 \times \frac{\pi}{4} d^2 = 0.78 \times \frac{\pi}{4} 18^2 = 198.50 \text{mm}^2$$

$$V_{dsb} = \left[\frac{400}{\sqrt{3}} \times \left(\frac{1 \times 198.50 + 1 \times 254.50}{1.25 \times 1000}\right)\right] = 83.64 \text{ KN}$$

2) Strength of one bolt in Bearing

$$V_{dpb} = \left[\frac{2.5 \times k_b \times d \times t^* \times f_u}{\gamma_{mb}}\right]$$

t* = Min thickness of a) Thickness of gusset plate = 10 mm

b) Sum of the thickness of angles = 8 + 8 =16mm

Therefore t = 10 mm

k_b is the least of the following:

1) $\frac{e}{3d_0} = \frac{40}{3 \times 20} = 0.66$ Edge distance e = 1.5 x 20 = 30 mm say 40 mm

Assuming Pitch $\not<$ 2.5 dia of bolt = 2.5 x 18 = 45 mm, say 60 mm

2) $\frac{p}{3d_0} - 0.25 = \frac{60}{3 \times 20} - 0.25 = 0.75$ 3) $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.98$ 4) 1

$$V_{dpb} = \left[\frac{2.5 \times 0.66 \times 18 \times 10 \times 400}{1.25 \times 1000}\right] = 95.04 \text{ KN}$$

Bolt value (BV) = Least of V_{dsb}, V_{dpb}.

Bolt value (BV) = 83.64 kN.

$$\text{No of bolts} = \frac{\text{Force}}{\text{Bolt value}} = \frac{222.2}{83.64} = 2.66 \quad \text{Say 03 No's}$$

b) Member OB : 2- ISA 80 x 80 x 8 mm

Force in the member = 268.3 x 10^3 N = 268.3 KN

V_{dsb} = 83.64 KN,

V_{dpb} = 95.04 KN

Bolt value (BV) = 83.64 kN.

$$\text{No of Bolts} = \frac{\text{Force}}{\text{Bolt Value}} = \frac{268.3}{83.64} = 3.21 \quad \text{Say 4}$$

c) Member OC: 1- ISA 50 x 50 x 6 mm

Force in the member = 65.1 x 10^3 N = 65.1 KN

1) Strength of one bolt in Single shear

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

Assuming Shank is interfering the shear plane.

$n_s = 1$, $\gamma_{mb} = 1.25$

$$A_{sb} = \frac{\pi}{4} d^2 = \frac{\pi}{4} 18^2 = 254.50 \text{ mm}^2$$

$$V_{dsb} = \left[\frac{400}{\sqrt{3}} \times \left(\frac{1 \times 254.50}{1.25 \times 1000}\right)\right] = 47.01 \text{ KN}$$

2) Strength ofone bolt in Bearing

$$V_{dpb} = \left[\frac{2.5 \times k_b \times d \times t^* \times f_u}{\gamma_{mb}}\right]$$

t* = Min thickness of a) Thickness of gusset plate = 10 mm

b) Thickness of angle = 6mm

Therefore t = 6 mm

k_b is the least of the following:

1) $\frac{e}{3d_0} = \frac{40}{3 \times 20} = 0.66$ Edge distance e = 1.5 x 20 = 30 mm say 40 mm

Assuming Pitch $\not<$ 2.5 dia of bolt = 2.5 x 18 = 45 mm, say 60 mm

2) $\frac{p}{3d_0} - 0.25 = \frac{60}{3 \times 20} - 0.25 = 0.75$ 3) $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.98$ 4) 1

$$V_{dpb} = \left[\frac{2.5 \times 0.66 \times 18 \times 6 \times 400}{1.25 \times 1000}\right] = 57.02\ \text{KN}$$

Bolt value (BV) = Least of V_{dsb}, V_{dpb}.

Bolt value (BV) = 47.01 kN.

No of bolts = $\frac{\text{Force}}{\text{Bolt value}} = \frac{65.10}{47.01} = 1.38$ Say 02 No's

D) Member OD : 1- ISA 50 x 50 x 6 mm

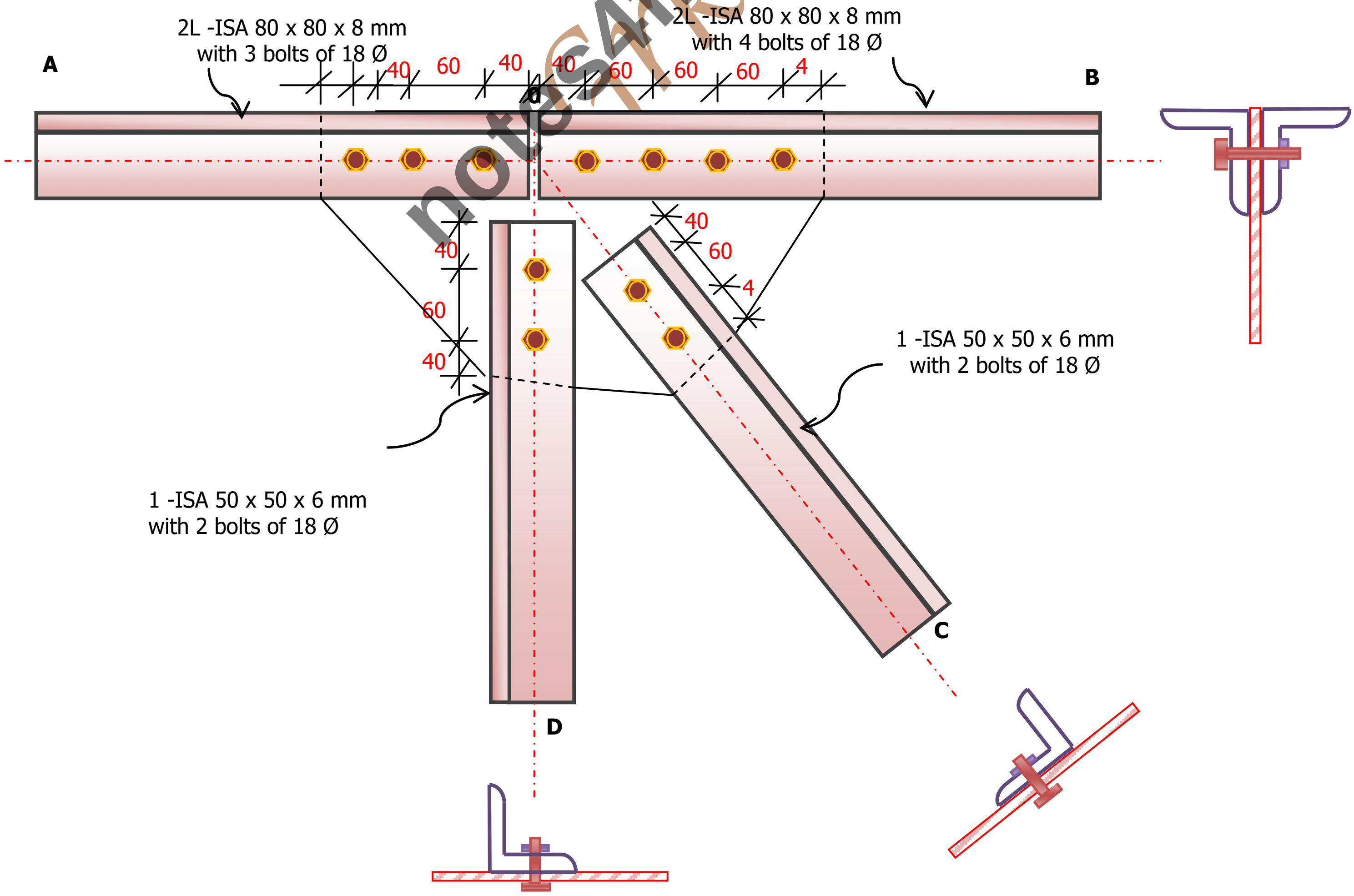
Force in the member = 46.02 x 10^3 N = 46.02 KN

V_{dsb} = 47.01 KN,

V_{dpb} = 57.02 KN

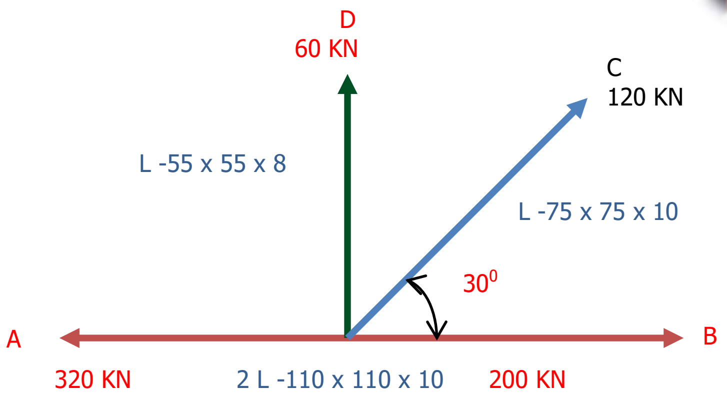
Bolt value (BV) = 47.01 kN.

No of bolts = $\frac{\text{Force}}{\text{Bolt value}} = \frac{46.02}{47.01} = 0.98$ Say 02 No's



Example 14: Figure shows a joint in the lower chord of a roof truss. Design the bolted connection using HSFG bolts of grade 8.8.

Draw the sketch showing the details of joint.



Solution:

Assuming thickness of gusset plate = 10 mm.

Dia of bolt using Unwin's formula $d' = 6.04\sqrt{t} = 6.04 \times \sqrt{8} = 17.10$ mm

Say dia of bolt = 16 mm.

Members OA and OB are designed as a single continuous Member AB

a) Member AB: 2-ISA 110 x 110 x 10 mm

Net Force in the member = 320 – 200 = 120 KN

1) Strength of bolt in double shear: P-76, Cl: 10.4

$$V_{dsf} = \frac{V_{nsf}}{\gamma_{mf}} = \frac{\mu_f n_e k_h F_o}{\gamma_{mf}}$$

$\mu_f = 0.55$, $n_e = 2$ (Double shear), $k_h = 1$

Proof Load $F_o = A_{nb} \times f_o$

$$A_{nb} = \text{Area of thread} = 0.78 \times \frac{\pi \times d^2}{4} = 0.78 \times \frac{\pi \times 16^2}{4} = 156.83\ \text{mm}^2$$

Proof Stress $f_o = 0.7\, f_{ub} = 0.7 \times 800 = 560\ \text{N/mm}^2$

Proof Load $F_o = A_{nb} \times f_o = 156.83 \times 560 = 87.82 \times 10^3$ N

$$V_{dsf} = \frac{0.55 \times 2 \times 1 \times 87.82 \times 10^3}{1.1 \times 1000} = 87.82\ \text{kN}$$

2) Strength of bolt in Bearing

$$V_{dpb} = \left[\frac{2.5 \times k_b \times d \times t^* \times f_u}{\gamma_{mb}}\right]$$

t* = Min thickness of a) Thickness of gusset plate = 10 mm

b) Sum of the thickness of angles = 10 + 10 =20 mm

Therefore t = 10 mm

k_b is the least of the following:

1) $\frac{e}{3d_0} = \frac{30}{3 \times 18} = 0.56$ Edge distance e = 1.5 x 18 = 27 mm say 30 mm

Assuming Pitch $\not<$ 2.5 dia of bolt = 2.5 x 20 = 50 mm, say 60 mm

2) $\frac{p}{3d_0} - 0.25 = \frac{60}{3 \times 18} - 0.25 = 0.86$ 3) $\frac{f_{ub}}{f_u} = \frac{800}{410} = 1.95$ 4) 1

$$V_{dpb} = \left[\frac{2.5 \times 0.56 \times 16 \times 10 \times 800}{1.25 \times 1000}\right] = 143.36 \text{ KN}$$

Bolt value (BV) = Least of V_{dsb}, V_{dpb}.

Bolt value (BV) = 87.82 kN.

$$\text{No of bolts} = \frac{\text{Force}}{\text{Bolt value}} = \frac{120}{87.82} = 1.36 \quad \text{Say 02 No's}$$

b) Design of Members OC and OD:

Strength of the bolt is calculated for 8 mm th angle (conservative side)

1) Strength of one bolt in single shear: P-76, Cl: 10.4

$$V_{dsf} = \frac{V_{nsf}}{\gamma_{mf}} = \frac{\mu_f n_e k_h F_o}{\gamma_{mf}}$$

$\mu_f = 0.55$, $n_e = 1$ (Single shear), $k_h = 1$

$$\text{Proof Load } F_o = A_{nb} \times f_o$$

$$A_{nb} = \text{Area of thread} = 0.78 \times \frac{\pi \times d^2}{4} = 0.78 \times \frac{\pi \times 16^2}{4} = 156.83 \text{ mm}^2$$

$$\text{Proof Stress } f_o = 0.7 f_{ub} = 0.7 \times 800 = 560 \text{ N/mm}^2$$

$$\text{Proof Load } F_o = A_{nb} \times f_o = 156.83 \times 560 = 87.82 \times 10^3 \text{ N}$$

$$V_{dsf} = \frac{0.55 \times 1 \times 1 \times 87.82 \times 10^3}{1.1 \times 1000} = 43.91 \text{ kN}$$

2) Strength of one bolt in Bearing:

$$V_{dpb} = \left[\frac{2.5 \times k_b \times d \times t^* \times f_u}{\gamma_{mb}}\right]$$

t* = Min thickness of a) Thickness of gusset plate = 10 mm

b) Thickness of angle = 8 mm

Therefore t = 8 mm

k_b is the least of the following:

1) $\frac{e}{3d_0} = \frac{30}{3 \times 18} = 0.56$ Edge distance e = 1.5 x 18 = 27 mm say 30 mm

Assuming Pitch $\not<$ 2.5 dia of bolt = 2.5 x 20= 50 mm, say 60 mm

2) $\frac{p}{3d_0} - 0.25 = \frac{60}{3 \times 18} - 0.25 = 0.86$ 3) $\frac{f_{ub}}{f_u} = \frac{800}{410} = 1.95$ 4) 1

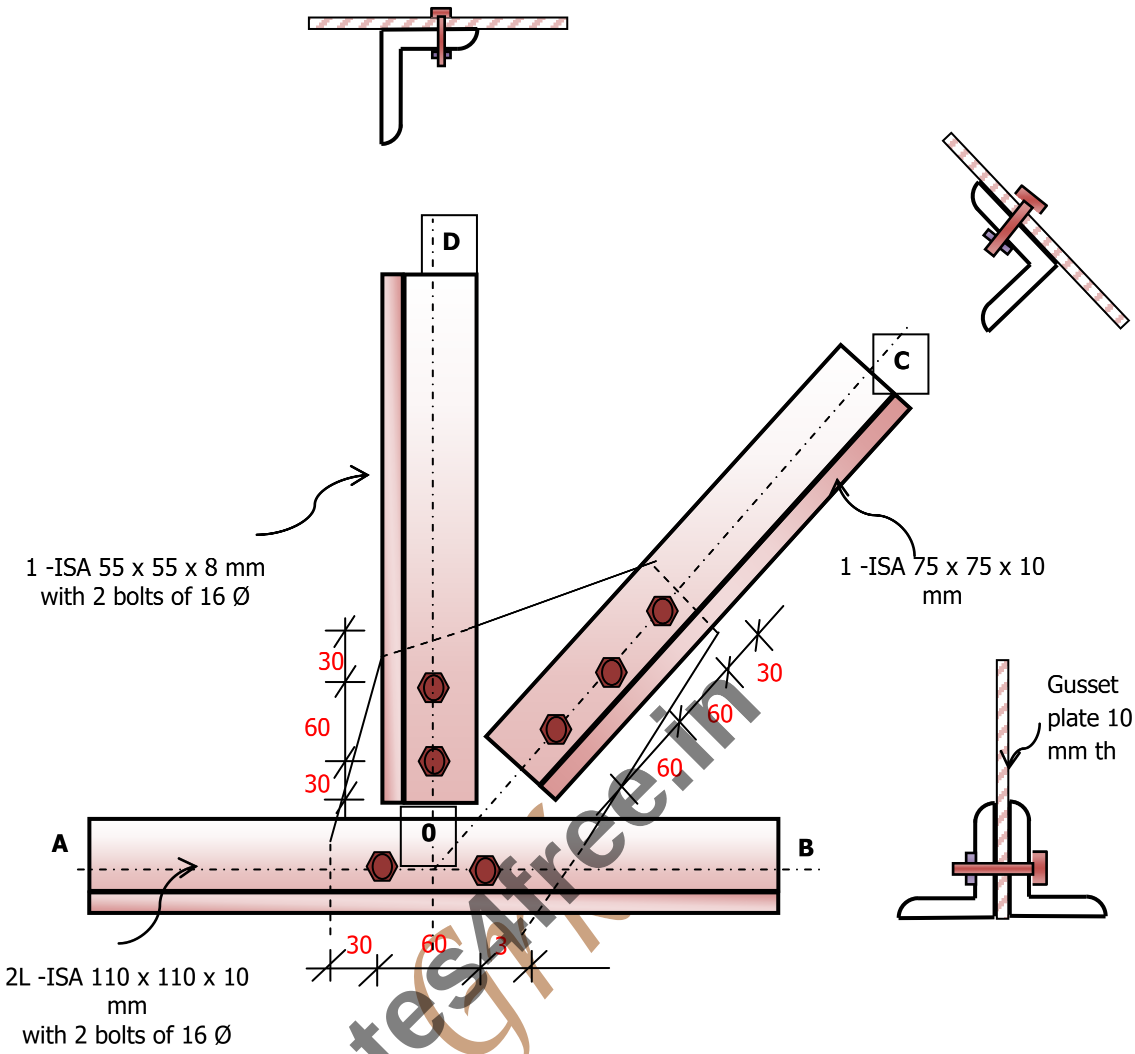
$$V_{dpb} = \left[\frac{2.5 \times 0.56 \times 16 \times 8 \times 800}{1.25 \times 1000}\right] = 114.69 \text{ KN}$$

Bolt value (BV) = Least of V_{dsb}, V_{dpb}.

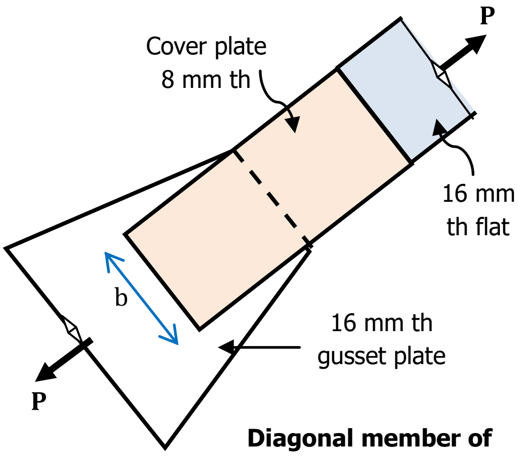
Bolt value (BV) = 43.91kN.

$$\text{No of bolts in member OC} = \frac{\text{Force}}{\text{Bolt value}} = \frac{120}{43.91} = 2.73 \quad \text{Say 03 No's}$$

$$\text{No of bolts in member OD} = \frac{\text{Force}}{\text{Bolt value}} = \frac{60}{43.91} = 1.37 \quad \text{Say 02 No's}$$



Example 15:®A truss of a bridge diagonal consists of 16 mm thick flat and carries a pull of 750 KN connected to a gusset plate by a double cover butt joint. The thickness of each cover plate is 8 mm. Determine the number of bolts required and width of the flat. Use HSFG bolts of 10.8grade. The surfaces are blasted with shot or grit and spray metalized with zinc. Also determine the efficiency of the joint by i) Chain bolting system, ii) Diamond bolting system.



Diagonal member of truss girder

Solution:

Pull P = 750 KN

Dia of bolt using unwin's formula

$$d = 6.04\sqrt{t} = 6.04 \times \sqrt{16} = 24.16mm$$

Say dia of bolt = 22 mm.

Dia of hole d_0 = 22 +2 = 24 mm

1) <u>Strength of one bolt in single shear:</u> <u>P-76, Cl: 10.4</u>

$$V_{dsf} = \frac{V_{nsf}}{\gamma_{mf}} = \frac{\mu_f n_e k_h F_o}{\gamma_{mf}}$$

μ_f = 0.25, Table – 20, P-77, n_e = 2 (Double shear), k_h = 1

Proof Load $F_o = A_{nb} \times f_o$

$$A_{nb} = \text{Area of thread} = 0.78 \times \frac{\pi \times d^2}{4} = 0.78 \times \frac{\pi \times 22^2}{4} = 296.50 \text{ mm}^2$$

$$\text{Proof Stress } f_o = 0.7\, f_{ub} = 0.7 \times 1000 = 700 \text{ N/mm}^2$$

$$\text{Proof Load } F_o = A_{nb} \times f_o = 296.50 \times 700 = 207.55 \times 10^3 \text{ N}$$

$$V_{dsf} = \frac{0.25 \times 2 \times 1 \times 207.55 \times 10^3}{1.1 \times 1000} = 94.34 \text{ kN}$$

2) Strength of one bolt in Bearing

$$V_{dpb} = \left[\frac{2.5 \times k_b \times d \times t^* \times f_u}{\gamma_{mb}}\right]$$

t* = Min thickness of a) Thickness of main plate = 16 mm

b) Sum of the thickness of cover plate = 8 + 8 = 16 mm

Therefore t = 16 mm

k_b is the least of the following:

1) $\frac{e}{3d_0} = \frac{40}{3 \times 24} = 0.56$ Edge distance e = 1.5 x 24 = 36 mm say 40 mm

Assuming Pitch $\not<$ 2.5 dia of bolt = 2.5 x 22 = 55 mm, say 60 mm

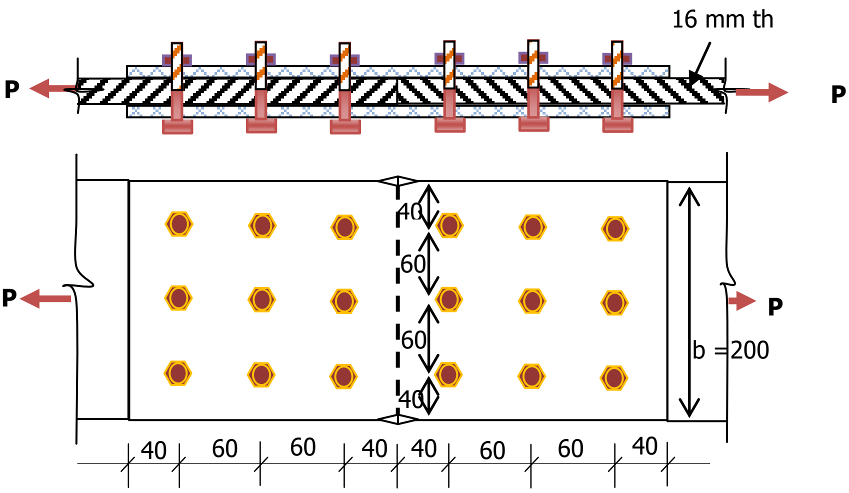
2) $\frac{p}{3d_0} - 0.25 = \frac{60}{3 \times 24} - 0.25 = 0.58$ 3) $\frac{f_{ub}}{f_u} = \frac{1000}{410} = 2.44$ 4) 1

$$V_{dpb} = \left[\frac{2.5 \times 0.56 \times 22 \times 16 \times 1000}{1.25 \times 1000}\right] = 394.24 \text{ KN}$$

Bolt value (BV) = Least of V_{dsf}, V_{dpb}.

Bolt value (BV) = **94.34kN**.

No of bolts = $\frac{\text{Force}}{\text{Bolt value}} = \frac{750}{94.34} = 7.95$ say 9 No's



Case 1: Chain Bolting:

Width of flat = 2 x 40 + 2 x 60 = 200 mm

Strength of bolted joint in double shear = 9 x 94.34 = 849.06kN

Strength of bolted joint in bearing = 9 x 394.34 = 3549.06kN

Strength of plate in tearing:

$$T_{dn} = \frac{0.9A_n \times t \times f_u}{\gamma_{m1}} = \frac{0.9(b - nd_0) \times t \times f_u}{\gamma_{m1}}$$

$$T_{dn} = \frac{0.9 \times (200 - 3 \times 24) \times 16 \times 410}{1.25 \times 1000} = 604.57 \text{ KN}$$

The strength of bolted joint of plate is the least of V_{dsb}, V_{dpb} and T_{dn}.

$\therefore$ *The strength of bolted joint = **604.57*** **KN**

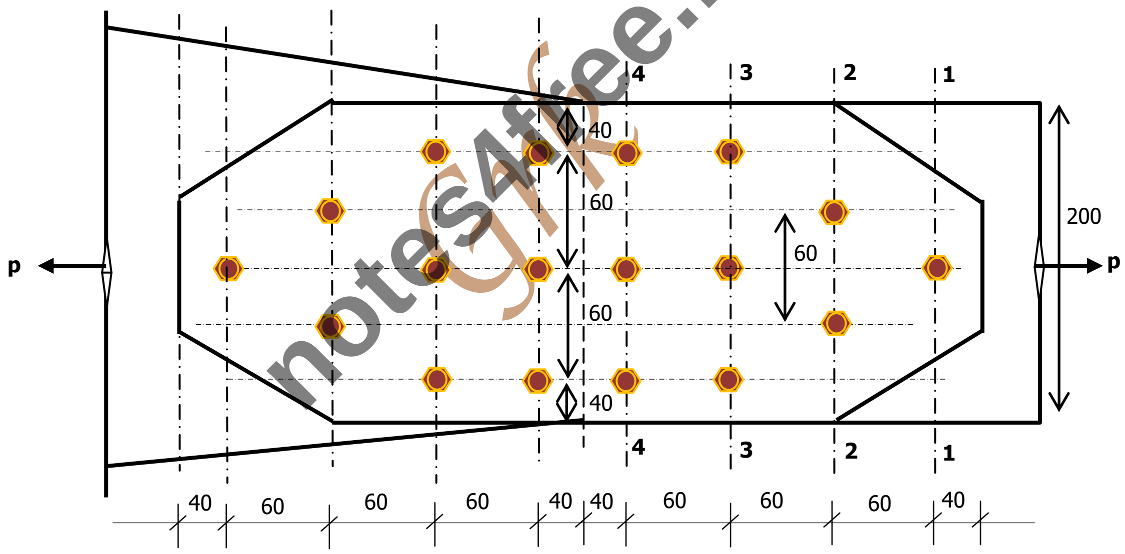
$$\text{Strength of solid plate} = \frac{0.9 \times b \times t \times f_u}{\gamma_{mb}}$$

$$= \frac{0.9 \times 200 \times 16 \times 410}{1.25 \times 1000} = 944.64 \text{ KN}$$

$$\text{Efficiency of bolted joint } (\eta) : \frac{\text{Strength of bolted joint}}{\text{Strength of solid plate}} \times 100$$

$$\eta = \frac{604.57}{944.64} \times 100 = 64\ \%$$

Case 2: DiamondBolting system:



Width of flat = 2 x 40 + 2 x 60 = 200 mm

Strength of bolted joint in double shear = 9 x 94.34 = 849.06 kN

Strength of bolted joint in bearing = 9 x 394.34 = 3549.06 kN

Strength of plate in tearing:

$$T_{dn} = \frac{0.9A_n \times t \times f_u}{\gamma_{m1}} = \frac{0.9(b - nd_0) \times t \times f_u}{\gamma_{m1}}$$

Strength of plate in tearing at section 1-1:

$$\frac{0.9 \times (200 - 1 \times 24) \times 16 \times 410}{1.25 \times 1000} = 831.28 \text{ kN}$$

Strength of plate in tearing at section 2 – 2:

$$T_{dn} = \frac{0.9(b - nd_0) \times t \times f_u}{\gamma_{m1}} + \text{No of bolts before section 2 - 2} \times BV$$

$$T_{dn} = \frac{0.9 \times (200 - 2 \times 24) \times 16 \times 410}{1.25 \times 1000} + 1 \times 94.34 = 812.27 \text{ kN}$$

Strength of plate in tearing at section 3 – 3:

$$T_{dn} = \frac{0.9(b - nd_0) \times t \times f_u}{\gamma_{m1}} + \text{No of bolts before section 3 - 3} \times BV$$

$$T_{dn} = \frac{0.9 \times (200 - 3 \times 24) \times 16 \times 410}{1.25 \times 1000} + 3 \times 94.34 = 887.59 \text{ kN}$$

Strength of plate in tearing at section 4 – 4:

$$T_{dn} = \frac{0.9(b - nd_0) \times t \times f_u}{\gamma_{m1}} + \text{No of bolts before section 4 - 4} \times BV$$

$$T_{dn} = \frac{0.9 \times (200 - 3 \times 24) \times 16 \times 410}{1.25 \times 1000} + 6 \times 94.34 = 1170.61 \text{ kN}$$

The strength of bolted joint of plate is the least of V_{dsb}, V_{dpb} and T_{dn}.

$\therefore$ *The strength of bolted joint = **812.27 KN***

$$\text{Strength of solid plate} = \frac{0.9 \times b \times t \times f_u}{\gamma_{mb}}$$

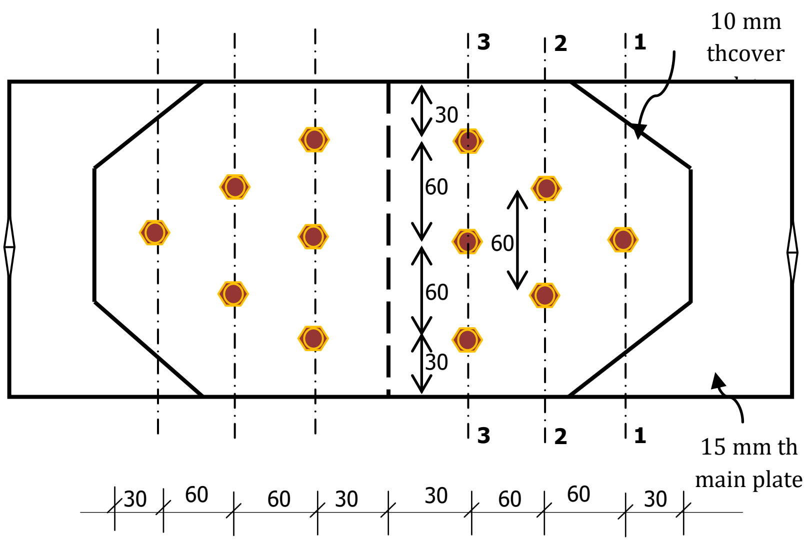
$$= \frac{0.9 \times 200 \times 16 \times 410}{1.25 \times 1000} = 944.64 \text{ KN}$$

$$\text{Efficiency of bolted joint } (\eta): \frac{\text{Strength of bolted joint}}{\text{Strength of solid plate}} \times 100$$

$$\eta = \frac{812.27}{944.64} \times 100 = 86 \ \%$$

Example 16:®

Find the maximum load which can be transferred through the double cover butt joint shown in fig. Find the efficiency of the joint. Use 20 mm dia common bolts.



Solution:

Assuming 4.6 grade bolt

dia of bolt = 20 mm.

Dia of hole d_0 = 20 +2 = 22 mm

1) Strength of one bolt in double shear

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

Assuming both shank and thread interfere the shear plane.

$$n_n = 1, \quad n_s = 1, \quad \gamma_{mb} = 1.25$$

$$A_{sb} = \frac{\pi}{4} d^2 = \frac{\pi}{4} \times 20^2 = 314.16 \text{ mm}^2$$

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

$$V_{dsb} = \left[\frac{400}{\sqrt{3}} \times \left(\frac{1 \times 245.04 + 1 \times 314.16}{1.25 \times 1000}\right)\right] = 103.31 \text{ KN}$$

2) Strength of one bolt in Bearing

$$V_{dpb} = \left[\frac{2.5 \times k_b \times d \times t^* \times f_u}{\gamma_{mb}}\right]$$

t* = Min thickness of a) Thickness of main plate = 15 mm

b) Sum of the thickness of cover plate = 10 + 10 = 20 mm

Therefore t = 15 mm

k_b is the least of the following:

1) $\frac{e}{3d_0} = \frac{30}{3 \times 22} = 0.45$ 2) $\frac{p}{3d_0} - 0.25 = \frac{60}{3 \times 22} - 0.25 = 0.66$ 3) $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.98$

4) 1

$$V_{dpb} = \left[\frac{2.5 \times 0.45 \times 20 \times 15 \times 400}{1.25 \times 1000}\right] = 108 \text{ KN}$$

Bolt value (BV) = Least of V_{dsb}, V_{dpb}.

Bolt value (BV) = **103.31kN**.

No of bolts =6 (given)

Strength of bolted joint in double shear = 6 x 103.31 = 619.86kN

Strength of bolted joint in bearing = 6 x 108 = 648 kN

Width of flat = 2 x 30 + 2 x 60 = 180 mm

Strength of plate in tearing:

$$T_{dn} = \frac{0.9 A_n \times t \times f_u}{\gamma_{m1}} = \frac{0.9(b - nd_0) \times t \times f_u}{\gamma_{m1}}$$

Strength of plate in tearing at section 1-1:

$$\frac{0.9 \times (180 - 1 \times 22) \times 15 \times 410}{1.25 \times 1000} = 699.62 \text{ kN}$$

Strength of plate in tearing at section 2 – 2:

$$T_{dn} = \frac{0.9(b - nd_0) \times t \times f_u}{\gamma_{m1}} + \text{No of bolts before section 2-2} \times \text{BV}$$

$$T_{dn} = \frac{0.9 \times (180 - 2 \times 22) \times 15 \times 410}{1.25 \times 1000} + 1 \times 103.31 = 705.52 \text{ kN}$$

Strength of plate in tearing at section 3 – 3:

$$T_{dn} = \frac{0.9(b - nd_0) \times t \times f_u}{\gamma_{m1}} + \text{No of bolts before section 3 - 3} \times BV$$

$$T_{dn} = \frac{0.9 \times (180 - 3 \times 22) \times 15 \times 410}{1.25 \times 1000} + 3 \times 103.31 = 814.72 \text{ kN}$$

The strength of bolted joint of plate is the least of V_{dsb}, V_{dpb} and T_{dn}.

$\therefore$ *The strength of bolted joint =* ***619.86*** **kN**

$$\text{Strength of solid plate} = \frac{0.9 \times b \times t \times f_u}{\gamma_{mb}}$$

$$= \frac{0.9 \times 180 \times 15 \times 410}{1.25 \times 1000} = 798.04 \text{ kN}$$

$$\text{Efficiency of bolted joint } (\eta): \frac{\text{Strength of bolted joint}}{\text{Strength of solid plate}} \times 100$$

$$\eta = \frac{619.86}{798.04} \times 100 = 77.77\,\%$$

Problem:

Two ISF sections 200 mm x 10 mm each and 1.5m long are to be jointed to make a member length of 3.0m. Design a butt joint with the bolts arranged in a diamond pattern. The flats are supposed to carry a factored tensile force of 450 kN. Steel is of grade Fe 410. 20 mm diameter bolts of grade 5.6 are used to make the connections. Also, determine the efficiency of the joint.

Problem:

Two ISF sections 200 mm x 10 mm each and 1.5m long are to be jointed to make a member length of 3.0m. Design a butt joint with the bolts arranged in a diamond pattern. The flats are supposed to carry a factored tensile force of 450 kN. Adopt HSFG bolts of property class 8.8. Also, determine the efficiency of the joint.

WELDED CONNECTIONS: Introduction, Welding process, Welding electrodes, Advantages of Welding, Types and Properties of Welds, Types of joints, Weld symbols, Weld specifications, Effective areas of welds, Design of welds, Simple joints, Moment resistant connections, Continuous Beam to Column connections, Continuous Beam to Beam connections, Beam Column splices, Tubular connections

6 Hours

Introduction:

- Welded connections are direct and efficient means of transferring forces from one member to the adjacent member. Welded connections are generally made by melting base metal from parts to be joined with weld metal, which upon cooling form the connections. A properly welded joint is stronger than the base metal

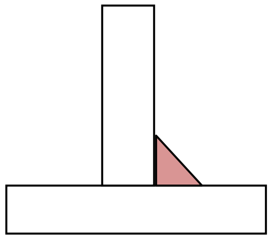
- Welds may be loaded in shear, tension, compression, or a combination of above.
- Capacities for welds are given in the AISC Specification Section J2 (2005)
- The strength of a weld is dependent on multiple factors, including: base metal, filler metal, type of weld, throat and weld size.

Weld types define the configuration of the weld and its underlying design approach

- Fillet welds and groove welds are most common
- Groove welds fall into two categories
  - Full penetration – the entire member cross-section is welded
  - Partial penetration – just part of the member cross-section is welded

Fillet Weld:

- The most commonly used weld is the fillet weld
- Fillet welds are theoretically triangular in cross-section
- Fillet welds join two surfaces at approximately right angles to each other in lap, tee, and corner joints



Fillet

The merits of the fillet welds are:

- No prior edge preparation is necessary.
- Simple, fast and economical to make, and
- Does not require very skilled labour.

The demerits of fillet welds are:

- Not appropriate to transfer forces large in magnitude,
- Poorer performance under fatigue loading, and
- Less attractive in appearance.

GROOVE WELD:

Butt welds, as shown in Fig, are made by butting plate surfaces against one another and filling the gap between contact surfaces with weld metal, in the process fusing the base metal also together. In order to censure full penetration of the weld metal, normally the contact surfaces are cambered to obtain gap for the weld metal to flow easily.

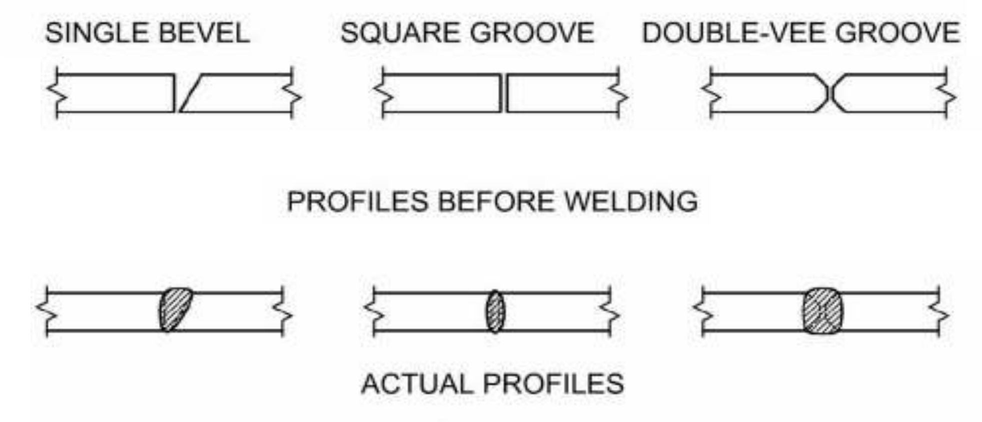
The merits of butt welds are :

- Easily designed and fabricated to be as strong as the member.
- Better fatigue characteristics, compared to fillet welds.
- Better appearance, compared to fillet welds, and
- Easy to detail and the length of the connection is considerably reduced.

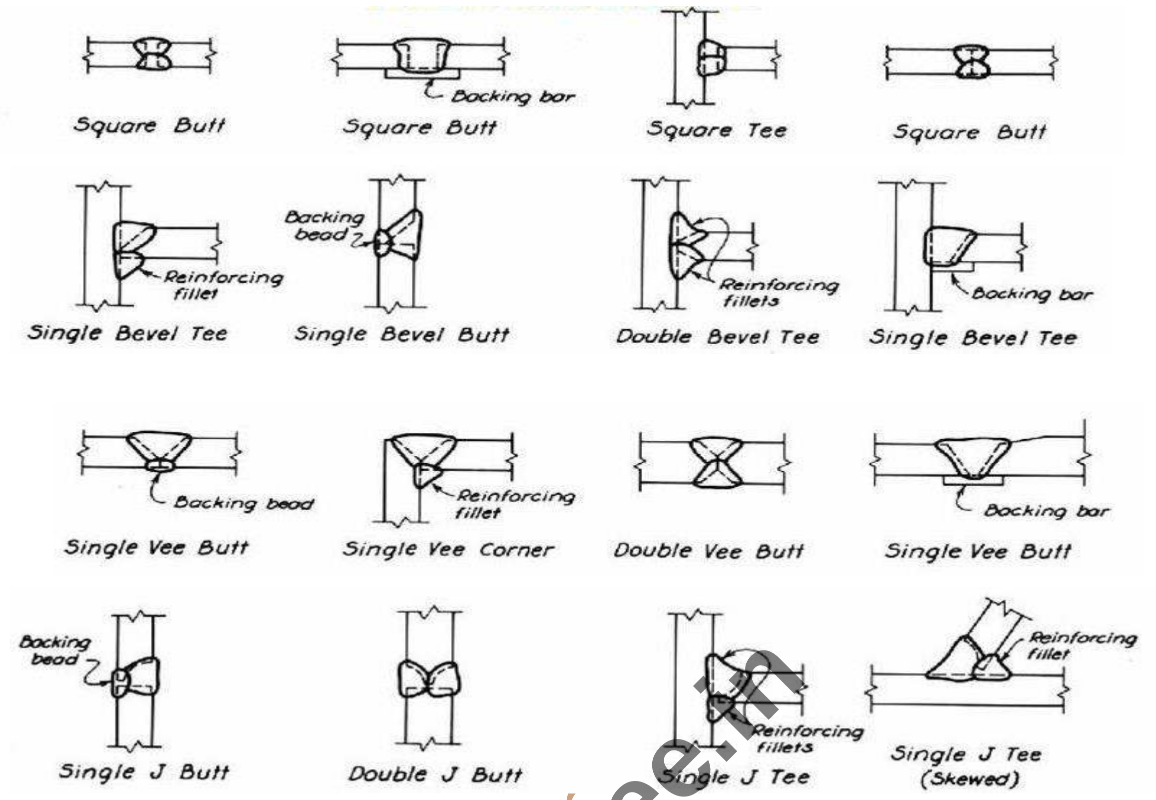
The demerits of the butt welds are:

  - More expensive than fillet welds because of the edge preparation required, and
  - Require more skilled manpower, than that required for filled welds.
- Groove welds are specified when a fillet weld is not appropriate for the job
    - The configuration of the pieces may not permit fillet welding
    - A strength greater than that provided by a fillet weld is required

Groove welds are made in the space or groove between the two pieces being welded.



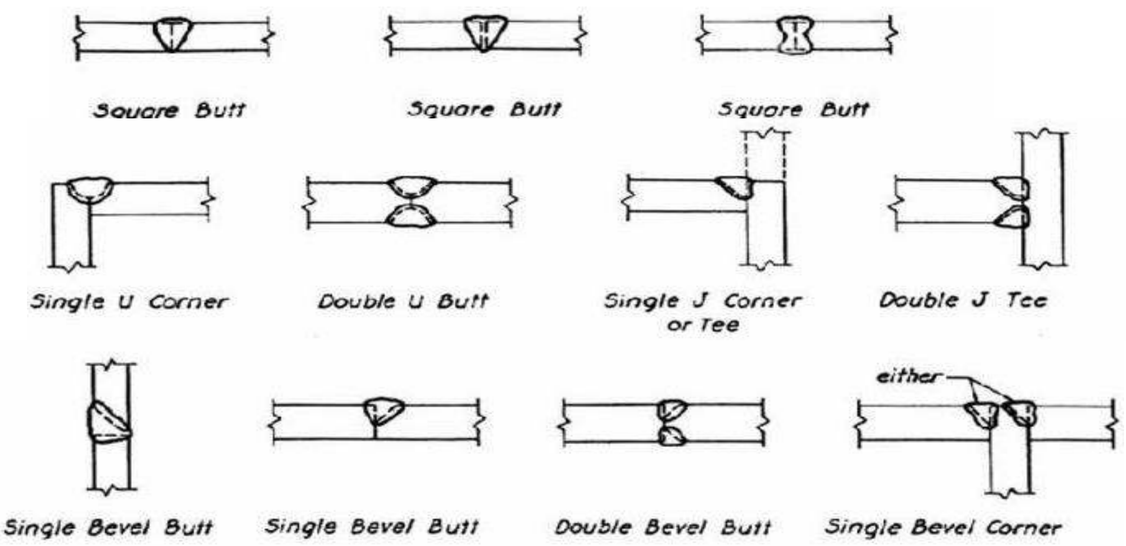
Full Penetration Groove Weld:

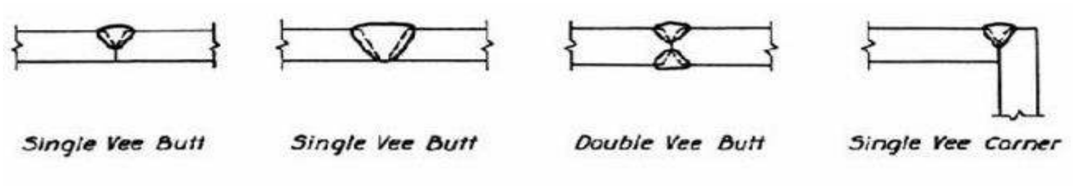


- The bevel or "J" preparation extends over most of or the entire face of the material being joined
- Complete fusion takes place

In some types of full penetration groove welds the material will be beveled from one side of the plate with a separate plate on the opposite side – called backing or a backing bar

Partial Penetration Groove Weld: Partial joint penetration welds are used when it is not necessary for the strength of the joint to develop the full cross section of the members being joined.



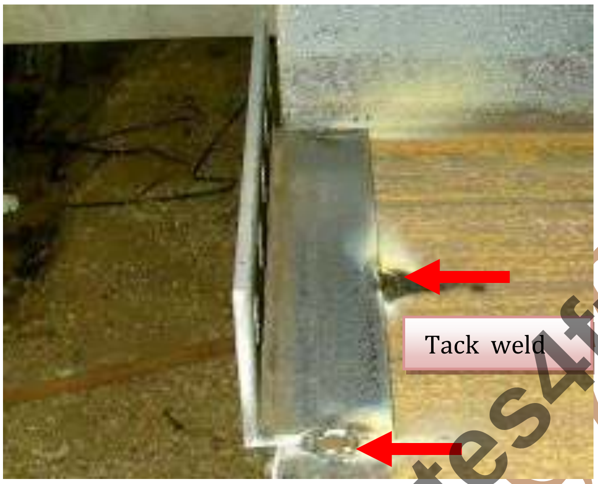


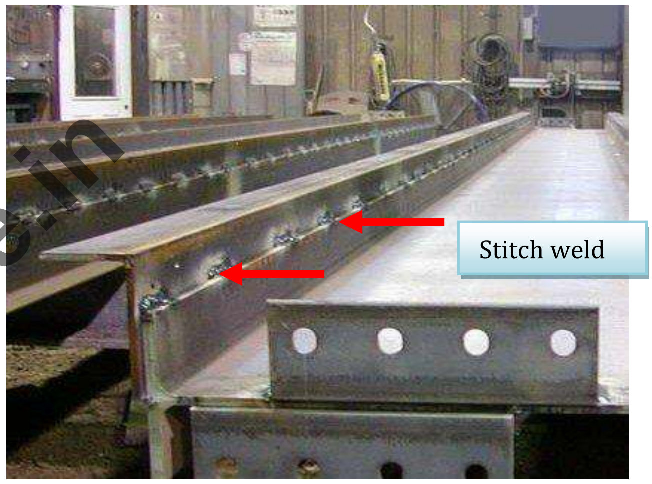
We
lding terminology:

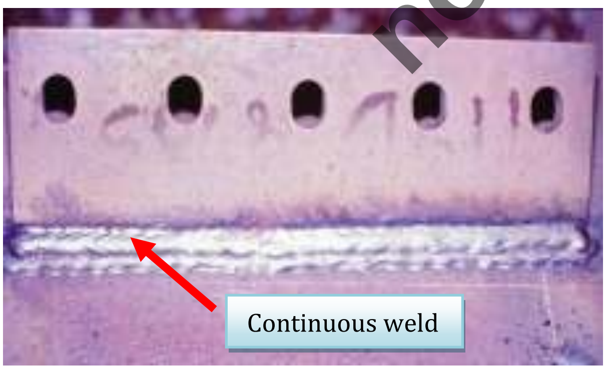
Tack weld: A temporary weld used to hold parts in place while more extensive, final welds are made.

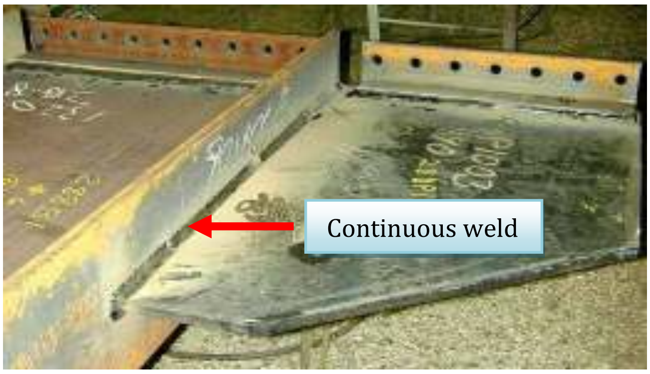
Stitch weld: A series of welds of a specified length that are spaced a specified distances from each other.

Continuous weld: A weld extends continuously from one end of a joint to the other.









ADVANTAGES OF WELDED JOINTS

(i) As no holes are required for welding, the structural members are more effective in taking load.
(ii) The overall weight of structural steel required is reduced by the use of welded joints.
(iii) Welded joints are often economical as less labour and material are required for a joint.
(iv) The welded connections look better than the usually bulky riveted joints.
(v) The speed of fabrication is higher with the welding process.
(vi) Any shape of joint can be made with ease.

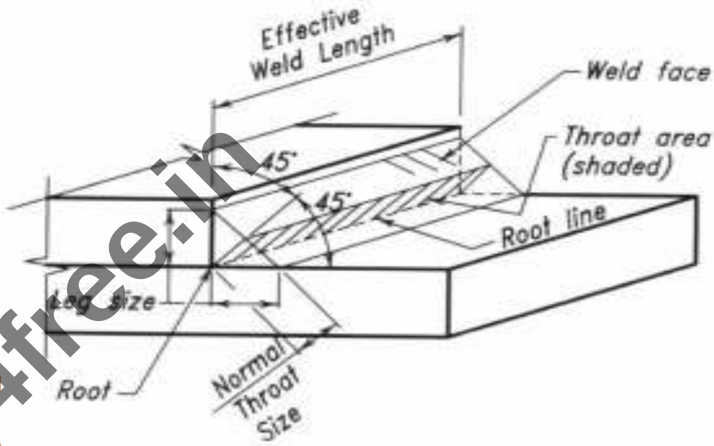
(vii) The welding process requires less working space than the riveting process.
(viii) Complete rigid joints can be provided with the welding process.
(ix) No noise is produced in the welding process as in the riveting process.

DISADVANTAGES OF WELDED JOINTS

(i) Skilled labour and electricity are required for welding.
(ii) Internal stresses and warping are produced due to uneven heating and cooling.
(iii) Welded joints are more brittle and therefore their fatigue strength is less than the members joined.
(iv) Defects like internal air pockets, slag inclusion and incomplete penetration are difficult to detect.

P- 78, Cl 10.5.3.1

Effective throat thickness: The effective throat thickness of a fillet weld is the perpendicular distance from the root to the hypotenuse of the largest isosceles right – angled triangle that can be inscribed within the weld cross – section as shown in fig.



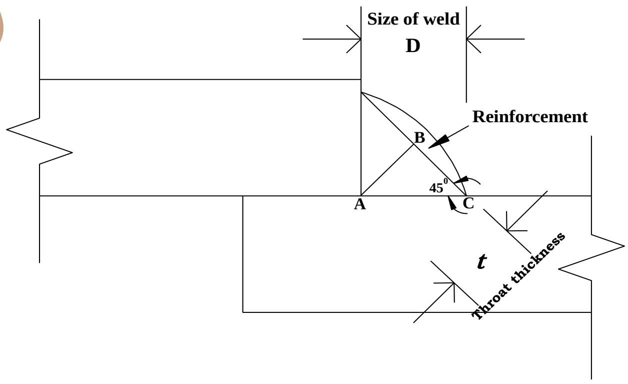
Consider triangle ABC

$$\text{Sin}45^0 = \frac{AB}{AC}$$

$$AB(t) = AC \times \text{Sin}45^0$$

$$t = 0.707 \times D$$

Throat thickness is equal to 0.707 times Size of weld. $\not< 3$ mm



P- 78, Cl 10.5.4.1

Effective length: The effective length of fillet weld shall be taken as only that length specified size and required throat thickness. In practice the actual length of weld is made equal to the effective length shown on the drawing plus twice the weld size, but not less than four times the size of the weld.

P- 79 ,Cl 10.5.7.1

Strength of weld: The strength of the fillet weld is taken equal to the resistance offered by it against shear. It is so, because it is weak in shear as compared to other modes of failure.

Strength of weld = Throat Area x Allowable Shear stress in the weld

Strength of weld = Throat thickness x Length x Allowable Shear stress in the weld

$$\text{Strength of weld} = 0.707 \times D \times L \times f_{wd}$$

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

$$f_{wn} = \frac{f_u}{\sqrt{3}}$$ Where, f_u = Smaller of the ultimate stress of the weld or of the parent metal and

γ_{mw} = Partial safety factor (Table 5, P - 30)

γ_{mw} = 1.25 for Shop weld

γ_{mw} = 1.5 for Field weld

f_u = ultimate stress of weld = 410 N/mm^2

For shop weld

$$f_{wd} = \frac{f_u}{\sqrt{3}\gamma_{mw}} = \frac{410}{\sqrt{3} \times 1.25} = 189.37 \text{N/mm}^2$$

For Field weld

$$f_{wd} = \frac{f_u}{\sqrt{3}\gamma_{mw}} = \frac{410}{\sqrt{3} \times 1.50} = 157.81 \text{N/mm}^2$$

$$\text{Strength of weld} = K \times D \times L \times f_{wd}$$

Cl : 10.5.3.2

For the purpose of stress calculation in fillet welds joining face inclined to each other, the effective throat thickness shall be taken as 'K' times fillet size, where, 'K' is a constant, depending upon the angle between fusion faces, as given in Table 22

Angle Between Fusion Faces	$60^0 - 90^0$	91^0-100^0	101^0-106^0	107^0-113^0	114^0-120^0
Constant 'K'	0.70	0.65	0.60	0.55	0.50

Design of fillet weld. Following specifications as per IS: 816-1956 shall be observed in the design of fillet welds.

- The size of a weld must match the size specified on the drawings
- Some welds may meet the required size after a single pass of the welder
- Larger weld sizes may require multiple passes to meet the size requirement
- Common single pass welds include fillet welds up to and including 5/16 inch and thin plate butt welds with no preparation
- Common multiple pass welds include single bevel full penetration groove welds, single bevel partial penetration groove welds, and fillet welds over 5/16 inch

1. **Size of weld** :

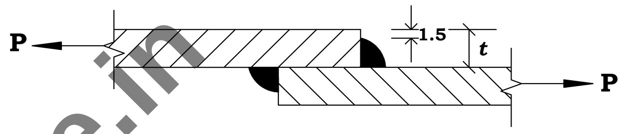
Minimum size:

The minimum size of the single fillet weld for different plate thickness will be as given in Table 21 P-78, Cl : 10.5.2.3

MINIMUM SIZE OF FILLET WELD

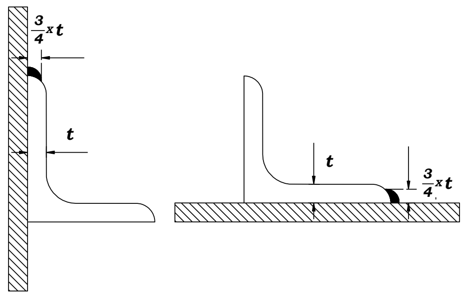
Thickness of thicker part	***Minimum size***
Up to and including 10 mm.	3.0 mm.
Over 10 mm. upto and including 20 mm.	5.0 mm.
Over 20 mm. upto and including 32 mm.	6.0 mm.
Over 32 mm. upto and including 50 mm.	8 mm of first run 10 for minimum size of weld.

Maximum size (cl: 10.5.8.1): Maximum size of fillet weld applied to a square edge should not exceed thickness of the edge minus 1.5 mm.



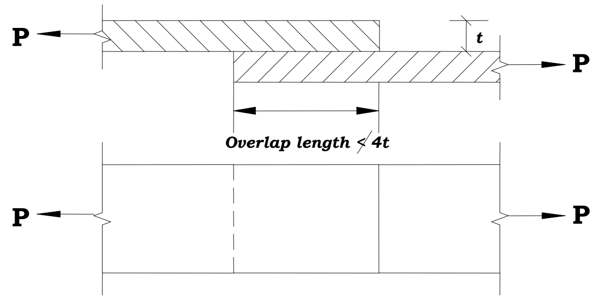
(cl: 10.5.8.2)

Maximum size of fillet weld applied to rounded toe of rolled steel section should not exceed ¾ x the thickness of the rounded toe.



1. **Overlap length:** ***(cl: 10.5.1.2)***:-

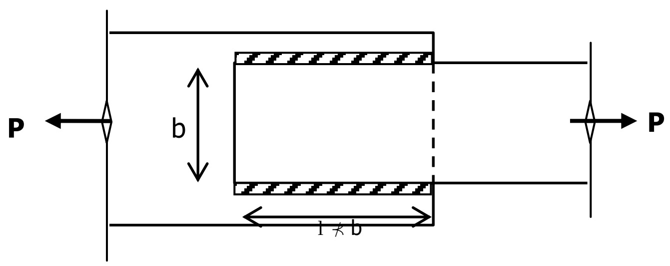
For lap joint as shown in Fig. the overlap should not be less than four times the thickness of the thinner part joined or 40 mm, whichever is more.



i.e., **Overlap length:** It the Max of the following
a) 4t, b) 40 mm.

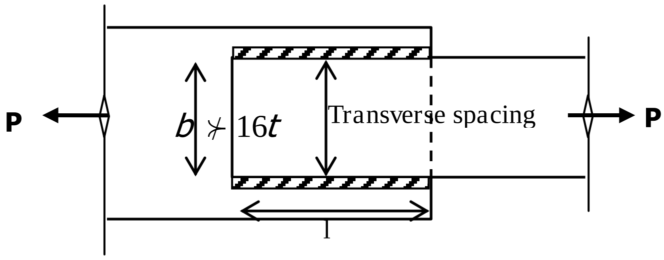
2. **Longitudinal Weld/ Side Weld** ***(cl: 10.5.1.2)*** **:-**

If longitudinal fillet or side fillet welds are used alone in the end connections, the length of each fillet weld should not be less than the perpendicular distance between them as shown in Fig.



Transverse Spacing between longitudinal weld:

The transverse spacing of longitudinal or side fillet welds shall not exceed 16 times the thickness of the thinner part connected, unless an end transverse weld or intermediate plug or slot welds are used to prevent buckling or separation of the parts. This has been shown in Fig.



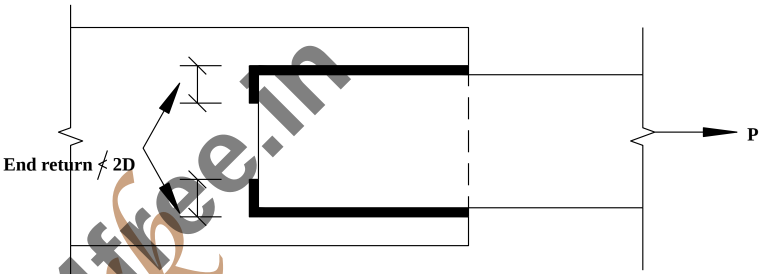
Note:

If b < 16t, only longitudinal weld can be provided

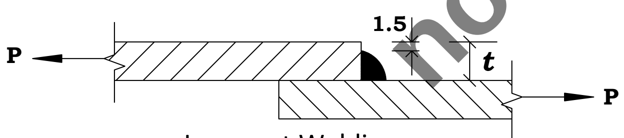
If b > 16t, along with longitudinal weld End weld should also be provided.

End Return weld *(cl: 10.5.1.1)*:-

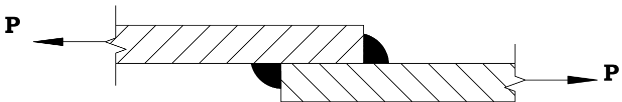
Fillet welds terminating at ends or sides of parts or members should preferably be returned continuously around the corners for a distance not less than twice the weld size as has been shown in Fig.



A Single fillet weld should not be subjected to a bending moment about the longitudinal axis of the fillet as shown in Fig. ***(cl: 10.5.1.3)***.



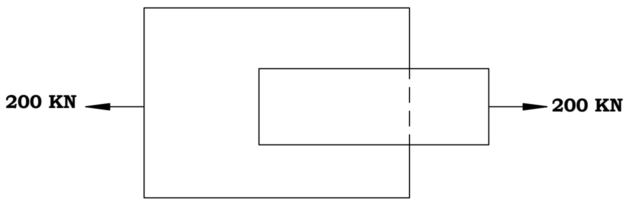
Incorrect Welding



Correct Welding

Problem:®

Design a 6mm size fillet weld for the lap joint shown in the figure. Assume site weld. Fe 410 steel. Assume width of plate = 100 mm.



Solution:

Strength of weld $= 0.707 \times D \times l \times f_{wd}$

$$f_{wd} = \frac{f_u}{\sqrt{3}\gamma_{mw}}$$

Strength of weld $= 0.707 \times 6 \times l \times \dfrac{410}{\sqrt{3} \times 1.5} = 669.43\, l$ - N

In equilibrium,

Strength of weld = Applied force

$$669.43\, l = 200 \text{ kN} = 200 \times 10^3 \text{ N}$$

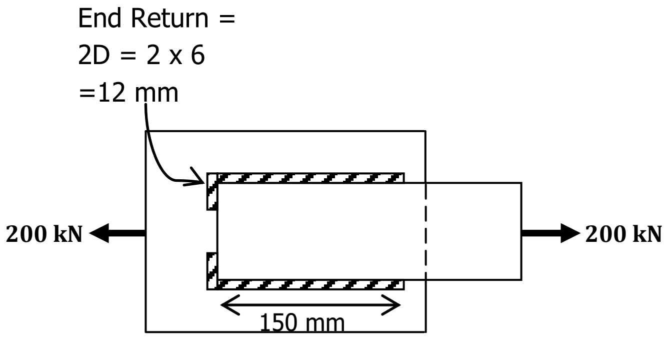
$$l = \frac{200 \times 1000}{669.43} = 298.76 \text{ mm} \qquad \text{Say } 300 \text{ mm}$$

Length of longitudinal weld on each side

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

Length of longitudinal weld required: It is the maximum of the following

1. Overlap length a) 4t b) 40 mm
2. Width of plate (100 mm)
3. Length of longitudinal weld calculated = 150 mm



The overall length of weld provided with end return of (2 x D) = 2 x 150 +2 x 2 x 6 = 324mm

Problem:®

Design a suitable fillet weld to connect a tie bar 60 X 8mm to a 12mm thick gusset plate. Assume shop welds. Use Fe 410 steel.

Solution:

Cl: 6.2, P- 32

Tension capacity of the member/Strength of tie bar $= T_{dg} = \dfrac{A_g f_y}{\gamma_{mo}}$

$$T_{dg} = \frac{60 \times 8 \times 250}{1.1} = 109.10 \times 10^3 \text{ N}$$

Size of weld: P-78, Table 21

Minimum size of weld for 8mm th plate = 3mm

Maximum size of weld = 8 – 1.5 = 6.5 mm. Say 6 mm

Strength of weld $= = 0.707 \times 6 \times l \times \dfrac{410}{\sqrt{3} \times 1.25} = 803.31\, l$ - N

In equilibrium,

Strength of weld = Applied force

$$803.31\, l = 109.10 \times 10^3 \text{ N}$$

$$l = \frac{109.10 \times 10^3}{803.31} = 135.81 \text{ mm} \qquad \text{Say } 140 \text{ mm}$$

Note:

If b < 16t, only longitudinal weld can be provided

If b > 16t, along with longitudinal weld End weld should also be provided.

b= 60 mm, 16 x t = 16 x 8 = 128 mm

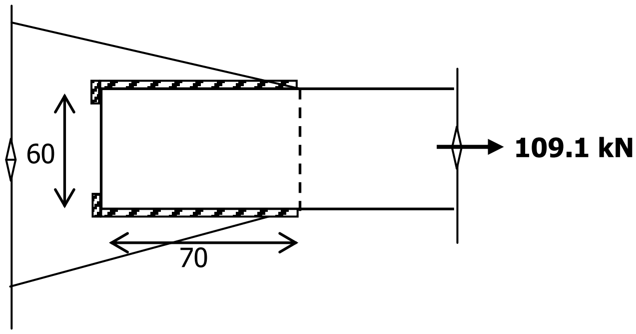
Since b < 16 x t, Only longitudinal weld can be provided.

Length of longitudinal weld on each side $= \dfrac{140}{2} = 70$ mm

Length of longitudinal weld required: It is the maximum of the following

1. Overlap length a) 4 x 8 = 32 mm b) 40 mm
2. Width of plate (60 mm)
3. Length of longitudinal weld calculated = 70 mm

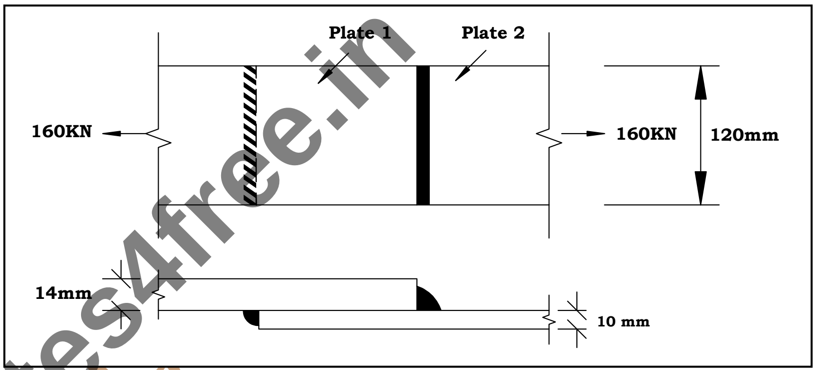
The overall length of weld provided with end return of (2 x D) = 2 x 70 +2 x 2 x 6 = 164 mm



Problem:

A tie bar of 120 mm x 10 mm is to be connected to other of size 120 mm x 14 mm. if the tie bars are to be loaded by a pull of 160 KN, Find out the size of end fillets such that the stresses in both the end fillets are same. Take f_u=410 N/ mm^2, Assume shop welding.

Solution:



Both the plates elongate equally under the load and therefore the stress will be equal in both plates. Force carried by both the plates will be different since the cross sectional areas are different.

Hence the weld size proportional to thickness of plate is provided.

Let D_1 and D_2 be the size of the weld of Plate (1) and Plate (2)

$$\frac{D_1}{D_2} = \frac{14}{10}$$

$$D_1 = 1.4D_2$$

Length of weld in each case = 120 mm

Strength of weld in plate (1) =

$$0.707 \times D_1 \times b \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}} = 0.707 \times 1.4D_2 \times 120 \times \frac{410}{\sqrt{3} \times 1.25}$$

Strength of weld in plate (1) = 22492.72D_2 N

Strength of weld in plate (2) =

$$0.707D_2 \times b \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}} = 0.707 \times D_2 \times 120 \times \frac{410}{\sqrt{3} \times 1.25}$$

Strength of weld in plate (2) = 16066.23D_2 N

Load carried by tie bar = 160 KN

Since there are two sizes of weld,

Total strength of weld $= 22492.72D_2 + 16066.23D_2 = 160 \times 10^3 N$

$$D_2 = \frac{160 \times 10^3}{(22492.72 + 16066.23)} = 4.15\text{mm} \qquad \text{Say } 5\text{ mm} < 10 - 1.5 = 8.5\text{ mm} \qquad \text{Safe}$$

$$D_1 = 1.4 \times D_2 = 1.4 \times 5 = 7\text{mm} \qquad 7\text{ mm} \le 14 - 1.5 = 12.5\text{ mm} \qquad \text{Safe}$$

Overlap length: Max of the following

a) 4 x t = 4 x 8 = 32 mm, b) 40 mm.

Provide 7 mm size weld for plate (1) and 5 mm size weld for plate (2).

Problem:

Find the sizes of the weld for two plates of size 100 mm x 8 mm and 100 mm x 12 mm. The ultimate stress in weld is 410 MPa. Assume field welding.

Solution:

Both the plates elongate equally under the load and therefore the stress will be equal in both plates. Force carried by both the plates will be different since the cross sectional areas are different.

Hence the weld size proportional to thickness of plate is provided.

Let D_1 and D_2 be the size of the weld of Plate (1) and Plate (2)

$$\frac{D_1}{D_2} = \frac{8}{12}$$

$$D_1 = 0.67 D_2$$

Cl: 6.2, P- 32

We know that,

$$\text{Force} = \text{Stress x Area} = T_{dg} = \frac{A_g f_y}{\gamma_{mo}}$$

A_g = Width of plate x Thickness of plate

Strength of plate (1) = Stress x area = $b \times t_1 \times \frac{f_y}{\gamma_{mo}} = \frac{100 \times 8 \times 250}{1.1 \times 1000} = 181.82\text{KN}$

Strength of plate (2) = Stress x area = $b \times t_2 \times \frac{f_y}{\gamma_{mo}} = \frac{100 \times 12 \times 250}{1.1 \times 1000} = 272.73\text{KN}$

Strength of joint = Full strength of thinner plate =181.82 KN

Since there are two sizes of weld,

Total strength of weld

$$= 0.707 D_1 \times b \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}} + 0.707 D_2 \times b \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}} = 181.82 \times 10^3\text{N}$$

$$0.707 \times 0.67 D_2 \times 100 \times \frac{410}{\sqrt{3} \times 1.5} + 0.707 \times D_2 \times 100 \times \frac{410}{\sqrt{3} \times 1.5} = 181.82 \times 10^3$$

$$7475.26 D_2 + 11157.10 D_2 = 181.82 \times 10^3$$

$$D_2 = \frac{181.82 \times 10^3}{(7475.26 + 11157.10)} = 9.76\text{mm} < \text{Max size} = t - 1.5 = 12 - 1.5 = 10.5\text{ mm} \quad \text{Safe}$$

$D_1 = 0.67 \times D_2 = 0.67 \times 9.76 = 6.5\text{mm} \le < \text{Max size} = t - 1.5 = 8 - 1.5 = 6.5 \text{ mm}$ Safe

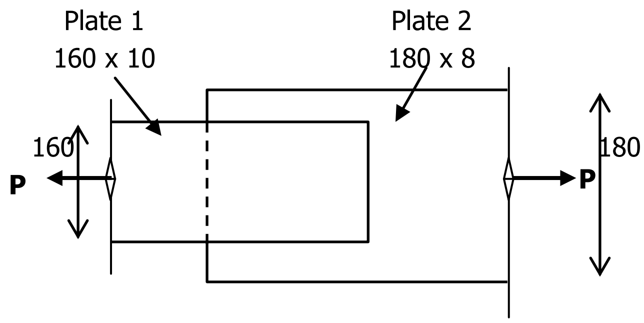
Overlap length: Max of the following

b) 4 x t = 4 x 8 = 32 mm, b) 40 mm.

Provide *6.5 mm size weld for plate (1) and 9.76 mm size weld for plate (2)*

Problem:

Two plates 160 mm x 10 mm and 180 mm x 8 mm are to be connected by a lap joint using 8 mm size weld. Assuming field welding. Assume Fe 410 steel. Design the joint.



Solution:

Cl: 6.2, P- 32

Tension capacity of the member (1)/Strength of plate (1) =

$$T_{dg} = \frac{A_g f_y}{\gamma_{mo}} = \frac{b \times t_1 \times f_y}{\gamma_{mo}} = \frac{160 \times 10 \times 250}{1.1 \times 1000} = 363.64 \text{KN}$$

Tension capacity of the member (2)/Strength of plate (2) =

$$T_{dg} = \frac{A_g f_y}{\gamma_{mo}} = \frac{b \times t_2 \times f_y}{\gamma_{mo}} = \frac{180 \times 8 \times 250}{1.1 \times 1000} = 327.30 \text{KN}$$

Strength of joint = Full strength of thinner plate =327.30 KN

Fillet weld is done for the plate (1) of size 160 mm x 10 mm.

Size of weld = 8 mm (given)

Strength of weld =

$$0.707 \times D \times l \frac{f_u}{\sqrt{3} \times \gamma_{mw}} = 0.707 \times 8 \times \frac{410}{\sqrt{3} \times 1.50} = 892.60 \text{ N}$$

Total length of weld = $l = \dfrac{327.30 \times 1000}{892.60} = 366.65$ mm

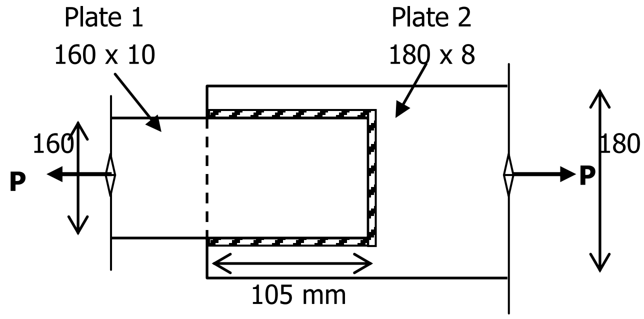
b = 160 mm, 16 x t = 16 x 8 = 128 mm, since b > 16 x t, End fillet weld of length 160 mm has to be provided along with longitudinal side fillet weld.

Length of longitudinal weld on each side = $\dfrac{366.65 - 160}{2} = 103.32\text{mm}$

Say 105 mm.

<u>**Length of longitudinal weld required:**</u> It is the maximum of the following

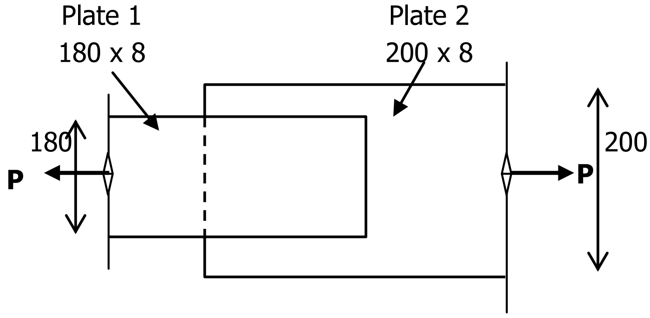
1. Overlap length a) 4 x 8 = 32 mm b) 40 mm
2. Length of longitudinal weld calculated = 105 mm.



Provide 8 mm fillet weld for a longitudinal length of 105 mm and a transverse length of 160 mm length.

Problem:®

Design the fillet welded joint between two plates of size 180 mm x 8 mm and 200 mm x 8 mm to develop the full strength of the smaller plate in tension. Assuming field welding.



Solution: Cl: 6.2, P- 32

Tension capacity of the member

(1)/Strength of 180 x 8 mm plate

$$= T_{dg} = \frac{A_g f_y}{\gamma_{mo}} = \frac{b \times t_1 \times f_y}{\gamma_{mo}} = \frac{180 \times 8 \times 250}{1.1 \times 1000} = 327.27 \text{ KN}$$

Tension capacity of the member (2)/Strength of 200 x 8 mm plate

$$= T_{dg} = \frac{A_g f_y}{\gamma_{mo}} = \frac{b \times t_2 \times f_y}{\gamma_{mo}} = \frac{200 \times 8 \times 250}{1.1 \times 1000} = 363.64 \text{ KN}$$

Strength of joint = Full strength of thinner plate =327.27 KN

Maximum size of weld = 8 – 1.5 = 6.5 mm Say 6 mm.

Strength of weld =

$$0.707 D \times l \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}} = 0.707 \times 6 \times l \times \frac{410}{\sqrt{3} \times 1.50} = 669.43\ l \ \text{- N}$$

$$\text{Total length of weld} = l = \frac{327.27 \times 1000}{669.43} = 488.88 \text{ mm}$$

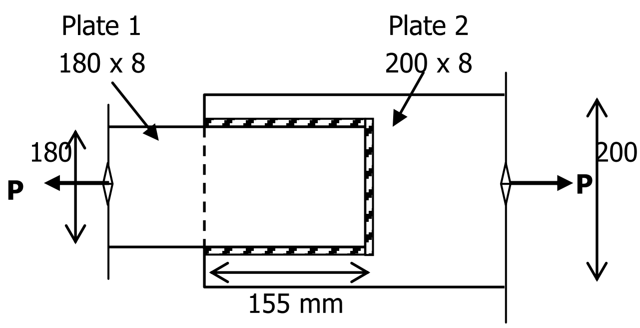
b = 180 mm, 16 x t = 16 x 8 = 128 mm, since b > 16 x t, End fillet weld of length 180 mm has to be provided along with longitudinal side fillet weld.

$$\text{Length of longitudinal weld on each side} = \frac{488.88 - 180}{2} = 154.44 \text{ mm}$$

Say 155 mm.

Length of longitudinal weld required: It is the maximum of the following

1. Overlap length a) 4 x 8 = 32 mm
   b) 40 mm
2. Length of longitudinal weld calculated = 155 mm.



Provide 6 mm fillet weld for a longitudinal length of 155 mm and a transverse length of 180 mm length.

Problem: ®

A tie member of a truss consisting of an angle section ISA 90 x 90 x 6 of Fe 410 grade is welded to an 8 mm gusset plate. Design a weld to transmit a load equal to the full strength of the member. Assume shop welding.

Solution:

Properties of ISA90 x 90 x 6

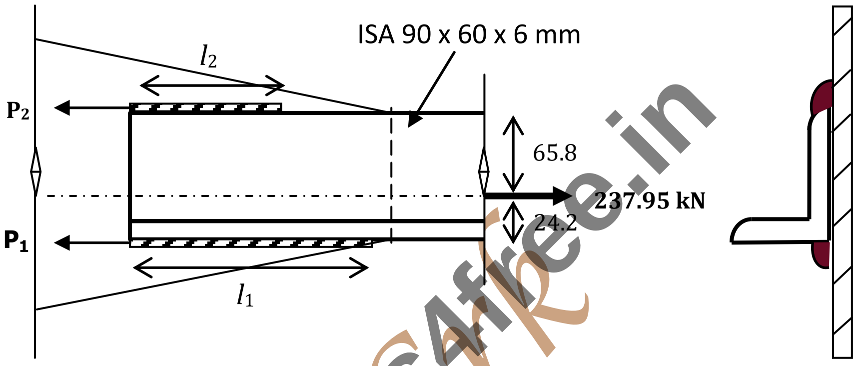
$A = 10.47\ cm^2 = 1047\ mm^2 = A_g$

$C_{zz} = 2.42$ cm $= 24.2$ mm, $e_{zz} = 65.8$ mm

Cl: 6.2, P- 32

Tension capacity of the member = T_{dg}

$$T_{dg} = \frac{A_g f_y}{\gamma_{mo}} \frac{1047 \times 250}{1.1} = 237.95 \times 10^3 N = 237.95\ KN$$



$$\text{Size of weld} \not> \frac{3}{4} \times 6 = 4.5\text{mm} \quad \text{say} \quad 4\text{ mm} > 3\text{ mm}$$

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

Let P_1 and P_2 be the strength of weld at bottom and top edge resp.

$$\text{Strength of weld at bottom}(P_1) = 0.707 \times D \times l_1 \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}}$$

$$= 0.707 \times 4 \times l_1 \times \frac{410}{\sqrt{3} \times 1.25} = 535.54\ l_1 \text{ - N}$$

$$\text{Strength of weld at top}(P_2) = 0.707 \times D \times l_2 \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}}$$

$$= 0.707 \times 4 \times l_2 \times \frac{410}{\sqrt{3} \times 1.25} = 535.54\ l_2 \text{ - N}$$

$$P_1 + P_2 = P$$

Distributing weld in such a way that c.g of the weld coincides with that of the angle section.

Taking moment of force 'P' and bottom weld P_1 about top edge.

$$P_1 \times 90 = P \times 65.8$$

$$535.54 \times l_1 \times 90 = 237.95 \times 65.8$$

$$l_1 = \frac{237.95 \times 65.8}{535.54 \times 90} = 324.85\text{ mm} \qquad \text{Say } 325\text{ mm}$$

$P_1 = 535.54 \times 325 = 174.05 \times 10^3$ N

$P_2 = P - P_1 = 237.95 \times 10^3 - 174.05 \times 10^3 = 63.9 \times 10^3$ N

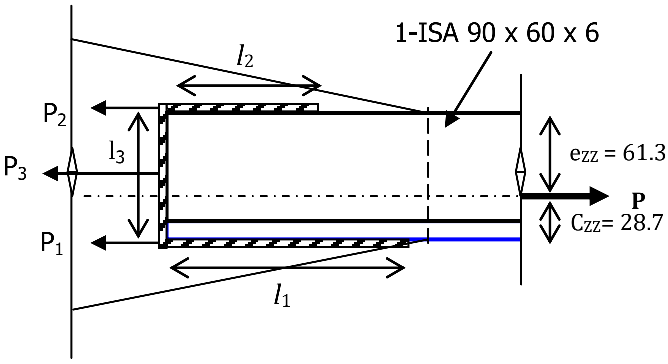
$l_2 = \dfrac{63.9 \times 10^3}{535.54 \times 90} = 119.31$ mm   Say 120 mm

Effective length of l_1 = 325 + 2 x 4 = 333 mm   say 335 mm

Effective length of l_2 = 120 + 2 x 4 = 128 mm   say 130 mm

Problem: ®

Calculate the pull on a single angle 90 x 60 x 6 mm member connected to a gusset plate by fillet weld as shown in the fig. Also determine the length of the weld ($l_1+l_2+l_3$) to carry the load. Assume shop welding.



Solution:

Properties of 1-ISA 90 x 60 x 6

Area = 8.8 cm^2 = 880 mm^2

C_{zz} = 2.42 cm =28.7mm, e_{zz} = 61.3 mm

Cl: 6.2, P- 32

Tension capacity of the member $= T_{dg} = \dfrac{A_g f_y}{\gamma_{mo}} = \dfrac{880 \times 250}{1.1} = 200 \times 10^3\ \mathrm{N} = 200\,\mathrm{KN}$

Size of weld: Min size = 3mm,

Max Size $\ngtr = \dfrac{3}{4} \times 6 = 4.5$ mm say 4 mm

Let P_1 , P_2 and P_3 be the strength of welds at bottom edge,top edge and End resp.

Strength of weld at bottom $(P_1) = 0.707 \times D \times l_1 \times \dfrac{f_u}{\sqrt{3} \times \gamma_{mw}}$

$$= 0.707 \times 4 \times l_1 \times \frac{410}{\sqrt{3} \times 1.25} = 535.54\ l_1\ \text{- N}$$

Strength of weld at top$(P_2) = 535.54\ l_2$ - N

Strength of End weld $(P_3) = 0.707 \times D \times l_3 \times \dfrac{f_u}{\sqrt{3} \times \gamma_{mw}}$

$$= 0.707 \times 4 \times 90 \times \frac{410}{\sqrt{3} \times 1.25} = 48198.6\ \text{- N} = 48.20 \times 10^3\text{N}$$

$$P_1 + P_2 + P_3 = P$$

Distributing weld in such a way that c.g of the welds coincides with that of the angle section.

Taking moment of force 'P', bottom weld P_1 and End weld P_3 about top edge.

$$P_2 \times 90 + P_3\left(\frac{90}{2}\right) = P \times 61.3$$

$$535.54 \times l_1 \times 90 + 48.20 \times 10^3 \times \frac{90}{2} = 200 \times 10^3 \times 61.3$$

$$l_1 = \frac{200 \times 10^3 \times 61.3 - 48.20 \times 10^3 \times 45}{535.54 \times 90} = 209.36 \text{ mm} \qquad \text{Say } 210 \text{ mm}$$

$$P_1 = 535.54 \times 210 = 112.46 \times 10^3 \text{ N}$$

$P_2 = P - P_1 - P_3 = 200 \text{ x } 10^3 - 112.46 \text{ x } 10^3 - 48.20 \text{ x } 10^3 = 39.34 \text{ x } 10^3$ N

$$P_2 = 535.54 \times l_2 = 39.34 \times 10^3$$

$$l_2 = \frac{39.34 \times 10^3}{535.54} = 73.46 \text{ mm} \qquad \text{Say } 80 \text{ mm}$$

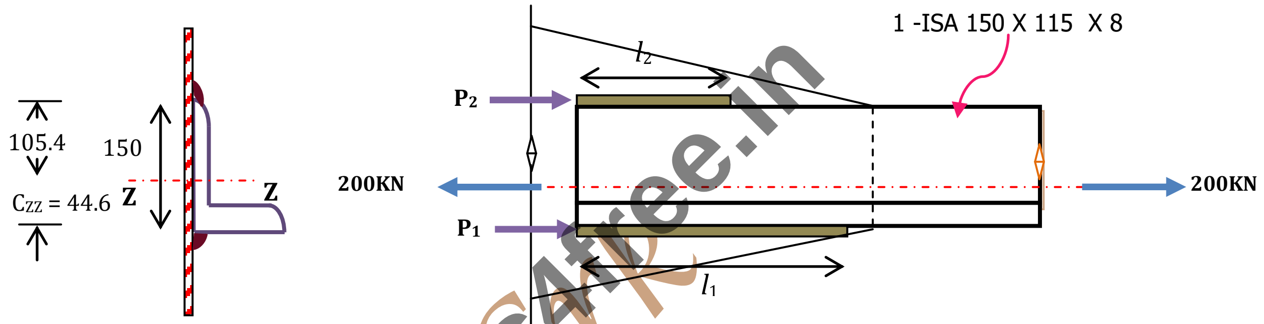
Problem: ®

An ISA 150 x 115 mm x 8 mm angle carrying a tensile load of 200 KN is to be connected to a gusset plate by 6 mm fillet welds at the end with its ***longer leg***. Design the joint assuming field welding.

Solution:

Pull = 200 KN, Size of weld = 6 mm, Taking, f_u = 410 N/mm^2.

C_{zz} = 4.46 cm =44.6 mm, e_{zz} = 105.4 mm,



$$\text{Strength of weld at bottom}(P_1) = 0.707 \times D \times l_1 \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}}$$

$$= 0.707 \times 6 \times l_1 \times \frac{410}{\sqrt{3} \times 1.50} = 670 l_1 \text{ N}$$

$$\text{Strength of weld at top}(P_2) = 670 l_2 \text{ N}$$

$$P_1 + P_2 = P$$

Distributing weld in such a way that c.g of the weld coincides with that of the angle section.

Taking moment of force 'P' and bottom weld P_1 about top edge.

$$P_1 \times 150 = P \times 105.4$$

$$670 \times l_1 \times 150 = 200 \times 10^3 \times 105.4$$

$$l_1 = \frac{200 \times 10^3 \times 105.4}{670 \times 150} = 209.75 \text{ mm} \qquad \text{Say } 210 \text{ mm}$$

$$P_1 = 670 \times 210 = 140.7 \times 10^3 \text{ N}$$

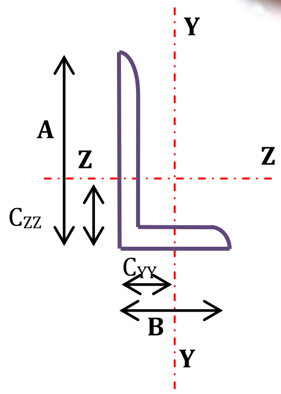
$$P_2 = P - P_1 = 200 \times 10^3 - 140.7 \times 10^3 = 59.3 \times 10^3 \text{ N}$$

$$P_2 = 670\, l_2 = 59.3 \times 10^3 \text{N}$$

$$\therefore l_2 = \frac{59.3 \times 10^3}{670} = 88.5 \text{ mm} \qquad \text{Say } 90 \text{ mm}$$

Problem:®

The 150 x 115 mm x 8 mm angle carrying a tensile load of 200 KN is to be connected to a gusset plate by 6 mm fillet welds at the ends with its ***shorter leg.*** Design the joint assuming field welding.



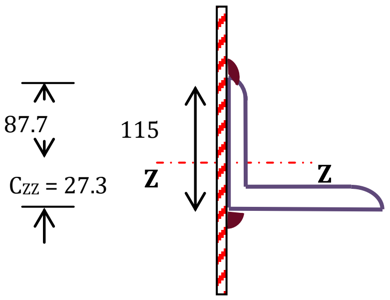
Solution:

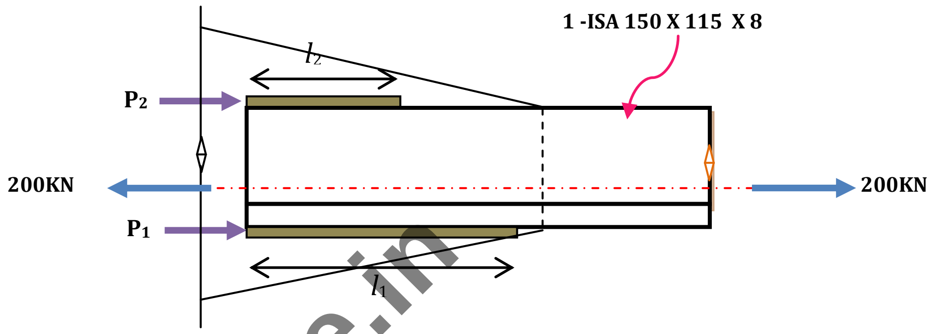
Pull = 200 KN, Size of weld = 6 mm, Taking, f_u = 410 N/mm^2.

C_{zz} = 4.46 cm = 44.6 mm

C_{YY} = 2.73 cm = 27.3 mm

Note: In this case short leg is connected to the gusset plate, therefore, C_{yy} becomes C_{zz} after tilting and connecting to gusset plate.





$$\text{Strength of weld at bottom}(P_1) = 0.707 \times D \times l_1 \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}}$$

$$= 0.707 \times 6 \times l_1 \times \frac{410}{\sqrt{3} \times 1.50} = 670 l_1 \text{ N}$$

$$\text{Strength of weld at top}(P_2) = 670 l_2 \text{ N}$$

$$P_1 + P_2 = P$$

Distributing weld in such a way that c.g of the weld coincides with that of the angle section.

Taking moment of force 'P' and bottom weld P_1 about top edge.

$$P_2 \times 115 = P \times 87.7$$

$$670 \times l_1 \times 115 = 200 \times 10^3 \times 87.7$$

$$l_1 = \frac{200 \times 10^3 \times 87.7}{670 \times 115} = 227.85 \text{ mm} \qquad \text{Say } 230 \text{ mm}$$

$$P_1 = 670 \times 230 = 153.97 \times 10^3 \text{ N}$$

$$P_2 = P - P_1 = 200 \times 10^3 - 153.97 \times 10^3 = 46.03 \times 10^3 \text{N}$$

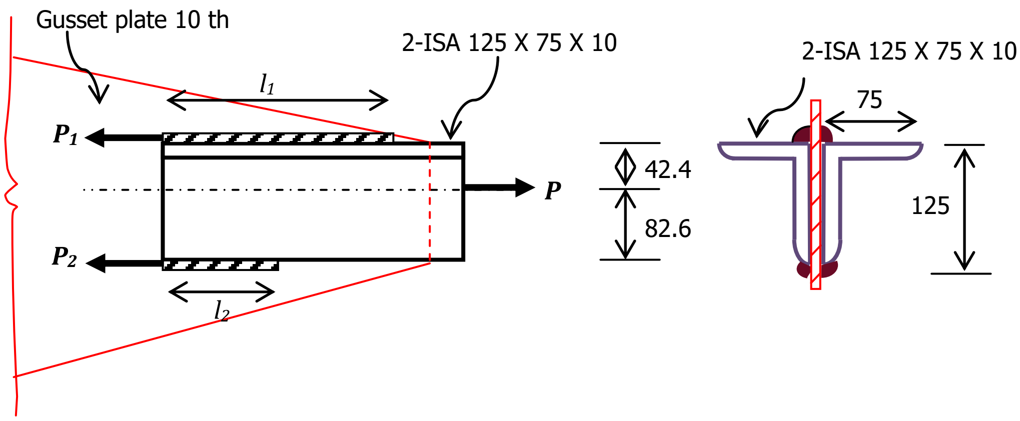
$$P_2 = 670\, l_2 = 46.03 \times 10^3 \text{N}$$

$$\therefore l_2 = \frac{46.03 \times 10^3}{670} = 68.75 \text{ mm} \qquad \text{Say 70 mm}$$

Problem:®

A tie member of a roof truss consists of 2-ISA 125 x 75 x 10 mm is subjected to a pull of 250 KN. The angles are connected ***on either side of a gusset plate*** of 10 mm thick with ***long legs back to back***. Design the weld.

Solution:



Let P_1 and P_2 be the strength of welds at top and bottom edges resp.

Max size of weld = ¾ x 10 = 7.5 mm Say 6 mm

Fillet weld of 6 mm size is provided on both sides of gusset plate.

Taking $f_u = 410\ N/mm^2$ and site welding $\gamma_{mw} = 1.5$.

$$\text{Strength of weld at top } (P_1) = 0.707 \times D \times l_1 \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}}$$

$$= 0.707 \times 6 \times l_1 \times \frac{410}{\sqrt{3} \times 1.50} = 670\ l_1\ N$$

$$\text{Strength of weld at bottom } (P_2) = 0.707 \times D \times l_2 \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}} = 670 l_2\ N$$

Welding calculation is done for single angle section and the same is provided for the two section on either side

$$P_1 + P_2 = P'$$

$$\text{Load carried by single angle section } (P') = \frac{250}{2} = 125\ KN$$

Distributing weld in such a way that c.g of the weld coincides with that of the angle section.

Taking moment of force 'P' and bottom weld P_2 about top edge.

$$P_2 \times 125 = P' \times 42.4$$

$$670 \times l_2 \times 125 = 125 \times 10^3 \times 42.4$$

$$l_2 = \frac{125 \times 10^3 \times 42.4}{670 \times 125} = 63.30\ mm \qquad \text{Say 70 mm}$$

$$P_2 = 670 \times 70 = 46.9 \times 10^3 N$$

$$P_1 = P - P_2 = 125 \times 10^3 - 46.9 \times 10^3 = 78.1 \times 10^3 N$$

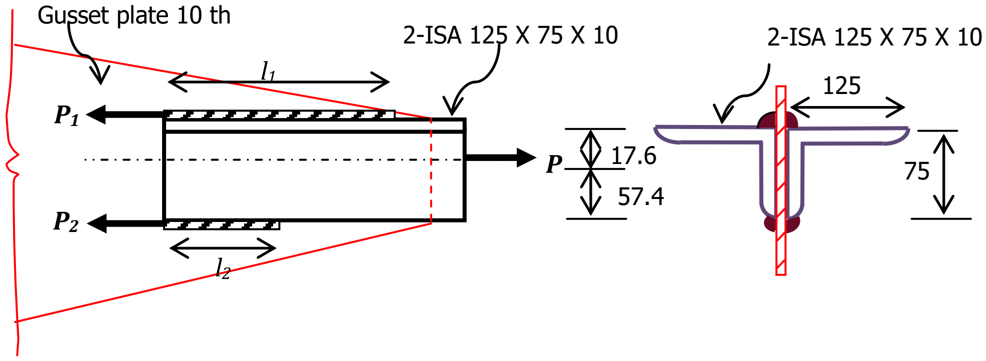
$$P_1 = 670 l_1 = 78.1 \times 10^3 N$$

$$\therefore l_1 = \frac{78.1 \times 10^3}{670} = 116.57\ mm \qquad \text{Say 120 mm}$$

Problem:®

A tie member of a roof truss consists of 2-ISA 125 x 75 x 10 mm is subjected to a pull of 250 KN. The angles are connected ***on either side of a gusset plate*** of 10 mm thick with ***short legs back to back***. Design the weld.

Solution:



Let P_1 and P_2 be the strength of welds at top and bottom edges resp.

Max size of weld = ¾ x 10 = 7.5 mm Say 6 mm

Fillet weld of 6 mm size is provided on both sides of gusset plate.

Taking $f_u = 410\ N/mm^2$ and site welding $\gamma_{mw} = 1.5$.

$$\text{Strength of weld at top } (P_1) = 0.707 \times D \times l_1 \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}}$$

$$= 0.707 \times 6 \times l_1 \times \frac{410}{\sqrt{3} \times 1.50} = 670\ l_1\ N$$

$$\text{Strength of weld at bottom } (P_2) = 0.707 \times D \times l_2 \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}} = 670 l_2\ N$$

Welding calculation is done for single angle section and the same is provided for the two section on either side

$$P_1 + P_2 = P'$$

$$\text{Load carried by single angle section } (P') = \frac{250}{2} = 125\ KN$$

Distributing weld in such a way that c.g of the weld coincides with that of the angle section.

Taking moment of force 'P' and bottom weld P_2 about top edge.

$$P_2 \times 75 = P' \times 17.6$$

$$670 \times l_2 \times 75 = 125 \times 10^3 \times 17.6$$

$$l_2 = \frac{125 \times 10^3 \times 17.6}{670 \times 75} = 43.78\ mm \qquad \text{Say 50 mm}$$

$$P_2 = 670 \times 50 = 33.5 \times 10^3 N$$

$$P_1 = P - P_2 = 125 \times 10^3 - 33.5 \times 10^3 = 91.5 \times 10^3\ N$$

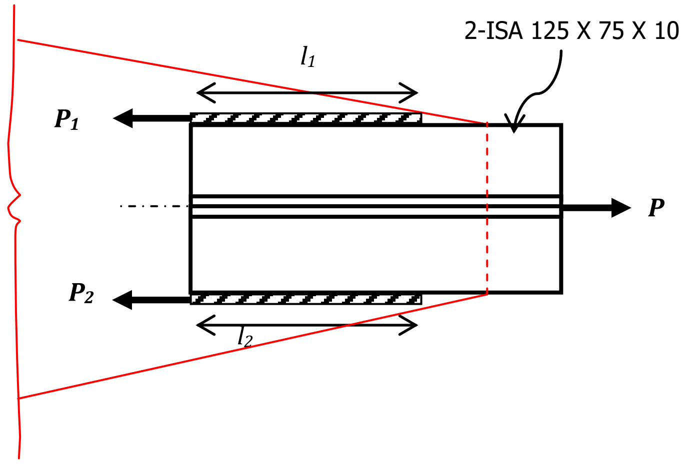
$$P_1 = 670 l_1 = 91.5 \times 10^3 N$$

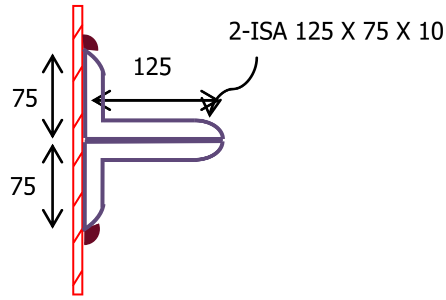
$$\therefore l_1 = \frac{91.5 \times 10^3}{670} = 136.57\ mm \qquad \text{Say 140 mm}$$

Problem:

A tie member of a roof truss consists of 2-ISA 125 x 75 x 10 mm is subjected to a pull of 250 KN. The angles are connected ***on same side of a gusset plate*** of 10 mm thick with ***long legs back to back***. Design the weld.

Solution:





Let P_1 and P_2 be the strength of welds at top and bottom edges resp.

Max size of weld = ¾ x 10 = 7.5 mm Say 6 mm

Taking $f_u = 410\ N/mm^2$ and site welding $\gamma_{mw} = 1.5$.

$$\text{Strength of weld at top } (P_1) = 0.707 \times D \times l_1 \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}}$$

$$= 0.707 \times 6 \times l_1 \times \frac{410}{\sqrt{3} \times 1.50} = 670\ l_1\ N$$

$$\text{Strength of weld at bottom } (P_2) = 0.707 \times D \times l_2 \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}} = 670 l_2\ N$$

$$P_1 + P_2 = P = 250\ kN$$

Distributing weld in such a way that c.g of the weld coincides with that of the angle section.

Taking moment of force 'P' and bottom weld P_2 about top edge.

$$P_2 \times 150 = P \times 75$$

$$670 \times l_2 \times 150 = 250 \times 10^3 \times 75$$

$$l_2 = \frac{250 \times 10^3 \times 75}{670 \times 150} = 186.57\ mm \qquad \text{Say 190 mm}$$

$$P_2 = 670 \times 190 = 127.3 \times 10^3 N$$

$$P_1 = P - P_2 = 250 \times 10^3 - 127.3 \times 10^3 = 122.7 \times 10^3\ N$$

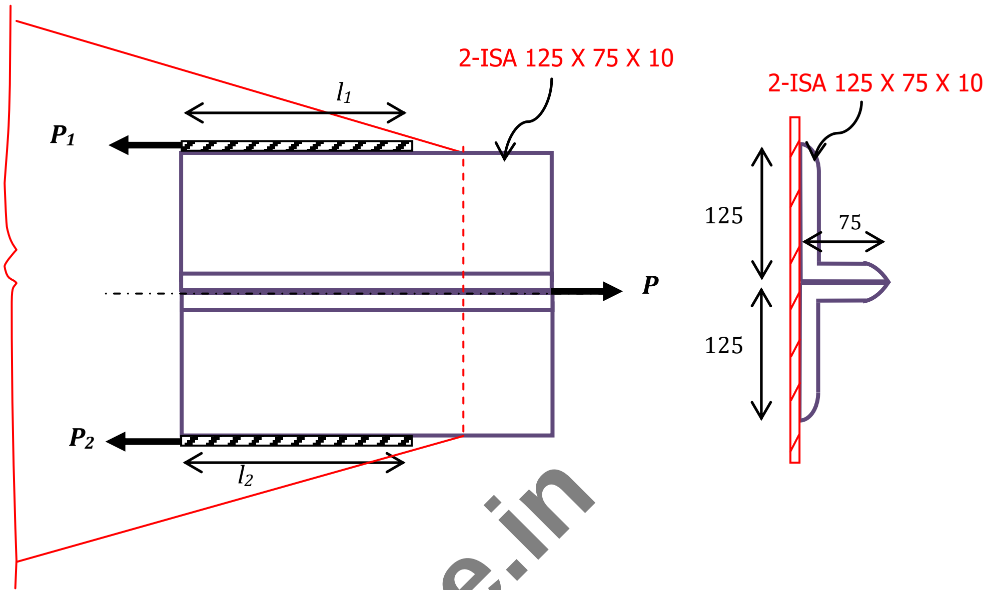
$$P_1 = 670 l_1 = 122.7 \times 10^3 N$$

$$\therefore l_1 = \frac{122.7 \times 10^3}{670} = 183.13\ mm \qquad \text{Say 190 mm}$$

Problem:®

A tie member of a roof truss consists of 2-ISA 125 x 75 x 10 mm. the tie member is subjected to pull of 250 KN. The angles are connected ***on same side of a gusset plate*** of 10 mm thick with ***short legs back to back***. Design the weld.

Solution:



Let P_1 and P_2 be the strength of welds at top and bottom edges resp.

Max size of weld = ¾ x 10 = 7.5 mm Say 6 mm

Taking $f_u = 410\ \text{N/mm}^2$ and site welding $\gamma_{mw} = 1.5$.

$$\text{Strength of weld at top } (P_1) = 0.707 \times D \times l_1 \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}}$$

$$= 0.707 \times 6 \times l_1 \times \frac{410}{\sqrt{3} \times 1.50} = 670\ l_1\ \text{N}$$

$$\text{Strength of weld at bottom } (P_2) = 0.707 \times D \times l_2 \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}} = 670 l_2\ \text{N}$$

$$P_1 + P_2 = P = 250\ \text{kN}$$

Distributing weld in such a way that c.g of the weld coincides with that of the angle section.

Taking moment of force 'P' and bottom weld P_2 about top edge.

$$P_2 \times 250 = P \times 125$$

$$670 \times l_2 \times 250 = 250 \times 10^3 \times 125$$

$$l_2 = \frac{250 \times 10^3 \times 125}{670 \times 250} = 186.57\ \text{mm} \qquad \text{Say 190 mm}$$

$$P_2 = 670 \times 190 = 127.3 \times 10^3\text{N}$$

$$P_1 = P - P_2 = 250 \times 10^3 - 127.3 \times 10^3 = 122.7 \times 10^3\ \text{N}$$

$$P_1 = 670 l_1 = 122.7 \times 10^3\text{N}$$

$$\therefore l_1 = \frac{122.7 \times 10^3}{670} = 183.13\ \text{mm} \qquad \text{Say 190 mm}$$

Problem:

A tie member of a roof truss consists of 2-ISA 90 x 90 x 8 mm. The tie member is subjected to pull of 250 KN. The angles are connected ***on either side of a gusset plate*** of 10 mm thick. Design the weld.

Problem:

A tie member of a roof truss consists of 2-ISA 90 x 90 x 8 mm. The tie member is subjected to pull of 250 KN. The angles are connected ***on same side of a gusset plate*** of 10 mm thick. Design the weld.

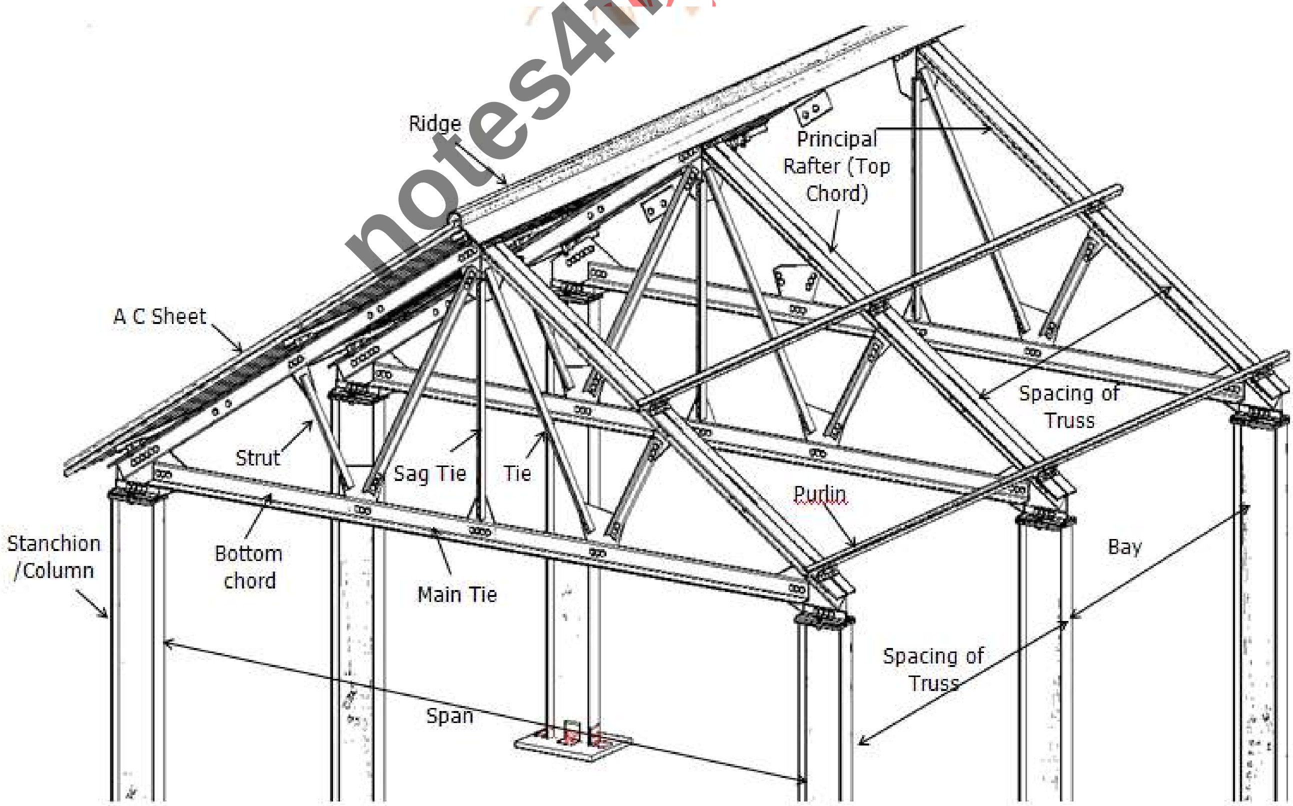
Thursday, January 18, 2018 21:16:04

Design of Compression Members: Introduction, Failure modes, Behaviour of compression members, Sections used for compression members, Effective length of compression members, Design of compression members and built up Compression members, Design of Laced and Battened Systems.

***Introduction* :**

The structural members carrying compressive load in truss are called struts. The vertical members carrying axial loads in a building are called columns or stanchions. The compression member of a crane is called a boom the main compression members of a roof truss are called rafters (Principal rafter and common rafter).

Common hot rolled and built – up steel members used for carrying axial compression, usually fail by flexural buckling. The buckling strength of these members is affected by residual stresses, initial low and accidental eccentricities of load. To account for all these factors, the strength of members subjected to axial compression is defined by class a, b, c, or d as given in Table 7, P - 35.

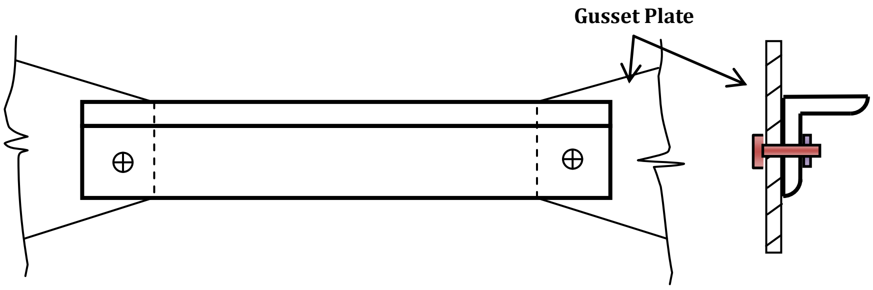


Analysis of single angle struts:

Data given: IS angle section, length of member.

Its required to determine the compressive strength of member

Procedure:



P-34, cl:7.1.2

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

Compressive Strength of member P_d = Compressive Stress (f_{cd}) x Area of the member

f_{cd} can be obtained (P- 34)

$$f_{cd} = \frac{f_y/\gamma_{mo}}{\phi + \left[\phi^2 - \lambda_e^2\right]^{0.5}} = \chi f_y/\gamma_{mo} \le f_y/\gamma_{mo}$$

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

λ = Non - dimensional elective slenderness ratio

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

Where, f_{cc} = Euler Buckling stress $= \dfrac{\pi^2 E}{\left(KL/r\right)^2}$

Where, KL/r = Effective slenderness ratio or ratio of effective length, KL to appropriate radius of gyration 'r'.

α = Imperfection factor given in Table 7 (Page 35).

Based on buckling classification.

Ref P-44, for buckling classification, Table 10, for angles section buckling classification is **"c".**

Table 7: Imperfection factor, α

Buckling Class	A	B	c	D
α	0.21	0.34	0.49	0.76

χ = Stress reduction factor (Table 8) for different buckling class, slenderness ratio and yield stress.

$$\chi = \frac{1}{\left[\phi + \left(\phi^2 - \lambda_e^2\right)^{0.5}\right]}$$

γ_{mo} = Partial safety factor for material strength.

E = Young's modulus of the member = 2 x 10^5 N/mm^2

Effective slenderness ratio: **P-48, Cl 7.5.1.2:**

$$\lambda_e = \sqrt{k_1 + k_2\lambda_{vv}^2 + k_3\lambda_\phi^2}$$

k_1, k_2, k_3 = Constants depending upon the end condition as given in Table 12.

$$\lambda_{vv} = \frac{\left(\dfrac{l}{r_{vv}}\right)}{\varepsilon\sqrt{\dfrac{\pi^2 E}{250}}} \qquad \varepsilon = \text{Yield stress ratio} = \left(\frac{250}{f_y}\right)^{0.5}$$

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

Strength of the member P_d = Compressive Stress (f_{cd}) x Area of the member

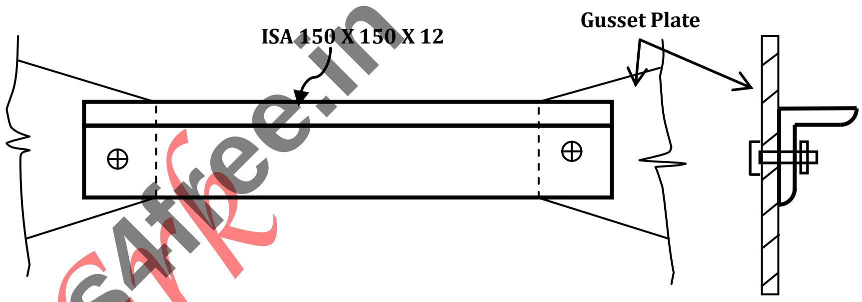
$$\mathbf{P_d = f_{cd} \times A_e}$$

Problem:®

A single angle discontinuous strut ISA 150 x 150 x 12 Th. @ 0.272KN/m with single Bolted connection is 3.5 m long. Calculate flexural buckling strength of section. Assume the fixidity as hinged.

Solution:

Properties of ISA 150 x 150 x 12 @ 0.272 KN / m.



$a = 34.59 cm^2 = 3459 mm^2$

$r_{vv} = 2.93 cm = 29.3 mm$

$r_{vv} = 29.3 m$

$\text{Effective length}(KL) = 3.5 m = 3500 mm$

P-34, cl:7.1.2

Compressive Strength of member P_d = Compressive Stress (f_{cd}) x Area of the member

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

P- 34 f_{cd} can be obtained

$$f_{cd} = \frac{f_y/\gamma_{mo}}{\phi + \left[\phi^2 - \lambda_e^2\right]^{0.5}} \le f_y/\gamma_{mo} \le \frac{250}{1.1} = 227.27 \text{ N/mm}^2$$

Effective slenderness ratio: **P-48, Cl 7.5.1.2:**

$$\lambda_e = \sqrt{k_1 + k_2\lambda_{vv}^2 + k_3\lambda_{\phi}^2}$$

$k_1, k_2, k_3 =$ Constants depending upon the end condition as given in Table 12.

$k_1 = 1.25, \qquad k_2 = 0.5, \qquad k_3 = 60$

$$\lambda_{vv} = \frac{\left(\dfrac{l}{r_{vv}}\right)}{\varepsilon\sqrt{\dfrac{\pi^2 E}{250}}} = \frac{\dfrac{3500}{29.3}}{1 \times \sqrt{\dfrac{\pi^2 \times 2 \times 10^5}{250}}} = 1.34$$

ε = Yield stress ratio $= \left(\frac{250}{f_y}\right)^{0.5} = \left(\frac{250}{250}\right)^{0.5} = 1$

$$\lambda_\phi = \frac{(b_1 + b_2)/2t}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}} = \frac{(150+150)/2\times 12}{1\times\sqrt{\frac{\pi^2\times 2\times 10^5}{250}}} = 0.14$$

$$\lambda_e = \sqrt{1.25 + 0.5\times 1.34^2 + 60\times 0.14^2} = 1.82$$

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

$\alpha =$ Imperfection factor given in Table 7. Based on buckling classification.

Ref P-44, for buckling classification, Table 10, for angles section buckling classification is **"c".**

α = 0.49

$$\phi = 0.5\times\left[1+\alpha(\lambda_e - 0.2)+\lambda_e^2\right] = 0.5\left[1+0.49(1.82-0.2)+1.82^2\right] = 2.55$$

$$f_{cd} = \frac{f_y/\gamma_{mo}}{\phi + \left[\phi^2 - \lambda_e^2\right]^{0.5}} = \frac{250/1.1}{2.55+\left[2.55^2 - 1.82\right]^{0.5}}$$

$$f_{cd} = 52.42\,N/mm^2 \le f_y/\gamma_{mo} = \frac{250}{1.1} = 227.27\,N/mm^2 \qquad \text{Safe}$$

Compressive Strength of member P_d = Compressive Stress (f_{cd}) x Area of the member

$$\mathbf{P_d = f_{cd} \times A}$$

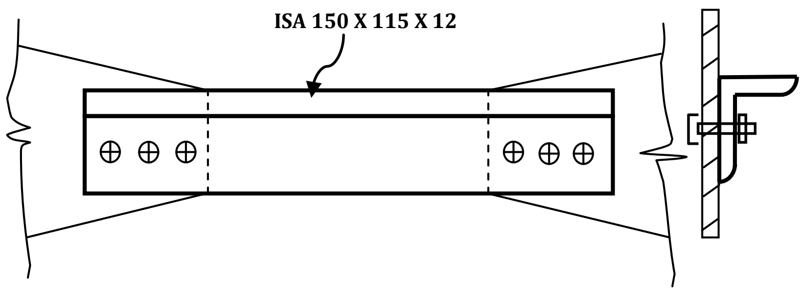
$$P_d = \frac{52.42\times 3459}{1000} = 181.30\ KN$$

Problem:®

A single angle discontinuous strut ISA 150 x 150 x 12 Th. @ 0.272KN/m is 3.5 m long is fixed with more than 2 bolts. Calculate flexural buckling strength of section. Assume the end as fixed.

Solution:

Properties of ISA 150 x 150 x 12 @ 0.272 KN / m.



$a = 34.59cm^2 = 3459mm^2$

$r_{zz} = r_{yy} = 4.61cm = 46.1mm$

$r_{uu} = 5.83cm = 58.3mm$

$r_{vv} = 2.93cm = 29.3mm$

$r_{min} = r_{vv} = 29.3m$

Effective length$(KL) = 3.5m = 3500mm$

Effective slenderness ratio (P-48, Cl 7.5.1.2)

$$\lambda_e = \sqrt{k_1 + k_2\lambda_{vv}^2 + k_3\lambda_\phi^2}$$

$k_1, k_2, k_3 =$ Constants depending upon the end condition as given in Table 12.

$k_1 = 0.2,\qquad k_2 = 0.35,\qquad k_3 = 20$

$$\lambda_{vv} = \frac{\left(\frac{l}{r_{vv}}\right)}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}} = \frac{\frac{3500}{29.3}}{1\times\sqrt{\frac{\pi^2\times 2\times 10^5}{250}}} = 1.34$$

$$\varepsilon = \text{Yield stress ratio} = \left(\frac{250}{f_y}\right)^{0.5} = \left(\frac{250}{250}\right)^{0.5} = 1$$

$$\lambda_\phi = \frac{(b_1+b_2)/2t}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}} = \frac{(150+150)/2\times 12}{1\times\sqrt{\frac{\pi^2\times 2\times 10^5}{250}}} = 0.14$$

$$\lambda_e = \sqrt{0.2+0.35\times 1.34^2+20\times 0.14^2} = 1.1$$

From P- 34 f_{cd} can be obtained

$$f_{cd} = \frac{f_y/\gamma_{mo}}{\phi+\left[\phi^2-\lambda_e^2\right]^{0.5}} \le f_y/\gamma_{mo}$$

Where,

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

α = Imperfection factor given in Table 7. Based on buckling classification.

Ref P-44, for buckling classification, Table 10, for angles section buckling classification is **"c".**

α = 0.49

$$= 0.5\left[1+0.49(1.1-0.2)+1.1^2\right] = 1.33$$

$$f_{cd} = \frac{f_y/\gamma_{mo}}{\phi+\left[\phi^2-\lambda_e^2\right]^{0.5}} = \frac{250/1.1}{1.33+\left[1.33^2-1.1\right]^{0.5}}$$

$$f_{cd} = 109.27\text{N/mm}^2 \le f_y/\gamma_{mo} = \frac{250}{1.1} = 227.27\text{ N/mm}^2 \qquad \text{Safe}$$

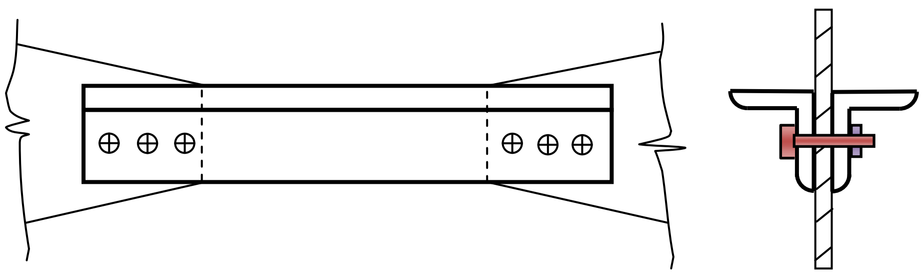
Buckling Strength of the member P_d = Compressive Stress (f_{cd}) x Area of the member

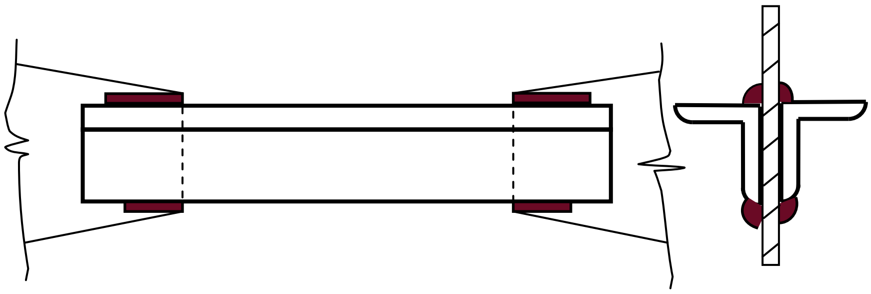
$$P_d = f_{cd} \times A$$

$$P_d = \frac{109.27\times 3459}{1000} = 377.95\text{ KN}$$

<u>Clause 7.5.2.1, P- 48, Double Angle Struts</u>

A) <u>Double angle discontinuous struts back to back connected on both sides of the gusseted by not less than 2 rivets(Bolts) in a line or welding</u>





Data given: Double angle section, length of member.

It's required to determine the compressive strength of member

Procedure:

Effective length :

$$KL = 0.7 \times L \ \text{ to } \ 0.85 \times L$$

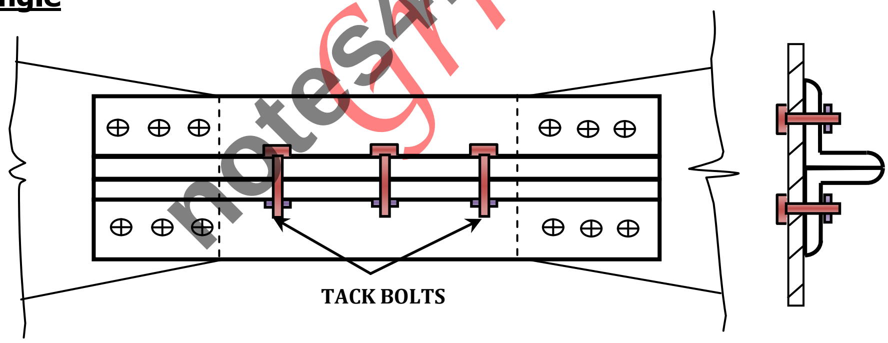
Effective slenderness ratio

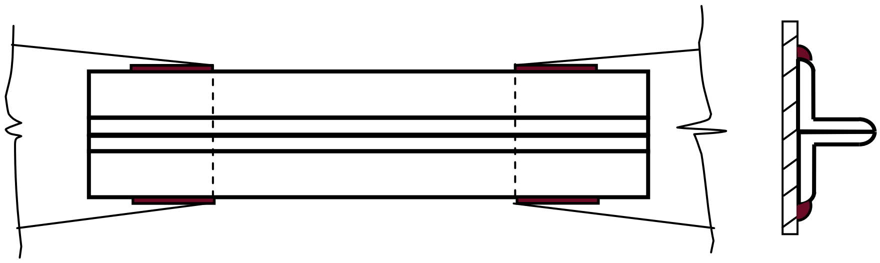
$$\lambda_e = \frac{KL}{r_{min}} \not> 180 \qquad \text{P - 20, Table 3.}$$

Ref Table 9(c) and find f_{cd}

Strength of the member P_d = Compressive Stress (f_{cd}) x Area of the member

B) Clause 7.5.2.2, P-48,Double angle discontinuous struts back to back connected to one side of a gusset by one or more rivets (Bolts) or welding in each angle

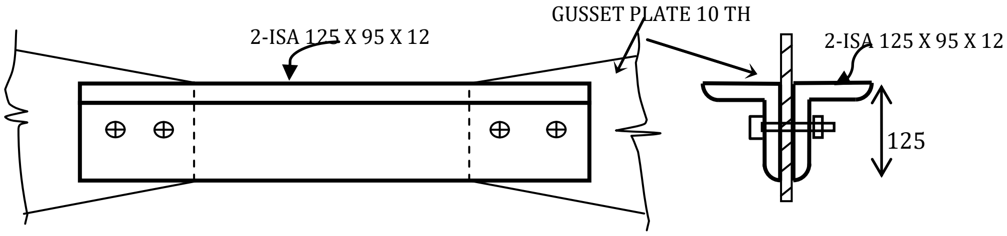




Problem:®

A double angle discontinuous strut ISA 125 x 95 x12 mm, ***long legs back to back*** is connected to both the sides of gusset plate 10 mm thick with 2 bolts. The length of strut b/w c/c of intersection is 4 m. determine the flexural torsional strength of the section.

Solution:



Properties of 2-ISA 125 x 95 x 12

$a = 49.96\ cm^2 = 4996\ mm^2$

$r_{zz} = 3.91\ cm = 39.1\ mm$

$r_{yy} = 4.05\ cm = 40.5\ mm$ (10mm th. gusset plate)

$r_{min} = r_{zz} = 39.1mm$

$KL = 0.7 \times L \ \text{to} \ 0.85 \times L$

Assuming $KL = 0.85 \times L = 0.85 \times 4m = 3400\ mm$

Effective slenderness ratio $= \dfrac{KL}{r_{min}} = \dfrac{3400}{39.1} = 86.96 < 180$ Safe

Ref Table 9(c) , P – 42 for $f_y = 250N/mm^2$

λ	f_{cd}
80	136
86.96	?
90	121
10	15
6.96	? (x)

f_{cd} for $86.96 = 136 - \dfrac{6.96 \times 15}{10} = 125.56\ N/mm^2$

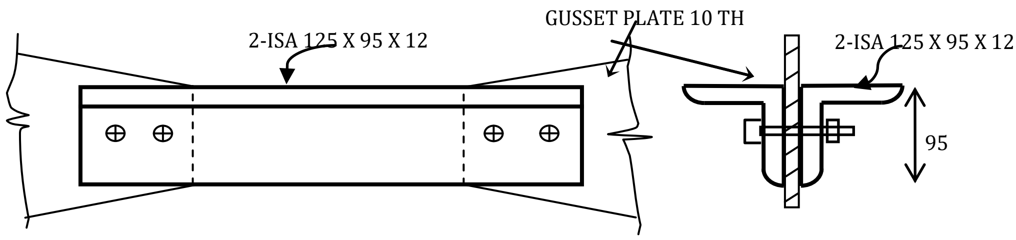
Buckling Strength of the member= Safe stress x area provided

$= f_{cd} \times A = \dfrac{125.56 \times 4996}{1000} = 627.30\ KN.$

Problem:®

A double angle discontinuous strut ISA 125 x 95 x12 mm, ***short legs back to back*** is connected to both the sides of gusset plate 10 mm thick with 2 bolts. The length of strut b/w c/c of intersection is 4 m. Determine the flexural torsional strength of the section.

Solution:



Properties of 2-ISA 125 x 95 x 12

$a = 49.96cm^2 = 4996mm^2$

$r_{yy} = 5.93cm = 59.3mm$ (10mm th. gusset plate)

$r_{zz} = 2.76cm = 27.6mm$

$r_{min} = r_{zz} = 27.6mm$

$KL = 0.7 \times L \ \text{to} \ 0.85 \times L$

Assuming $KL = 0.85 \times L = 0.85 \times 4m = 3400\ mm$

Effective slenderness ratio $= \dfrac{KL}{r_{min}} = \dfrac{3400}{27.6} = 123.20 < 180$ Safe

Ref Table 9(c) , P – 42 for $f_y = 250\ N/mm^2$

$$f_{cd} \text{ for } 123.20 = 83.7 - \frac{3.2 \times 9.4}{10} = 80.69\ N/mm^2$$

λ	f_{cd}
120	83.7
123.20	?
130	74.3
10	9.4
3.20	? (x)

Buckling Strength of the member= Safe stress x area provided

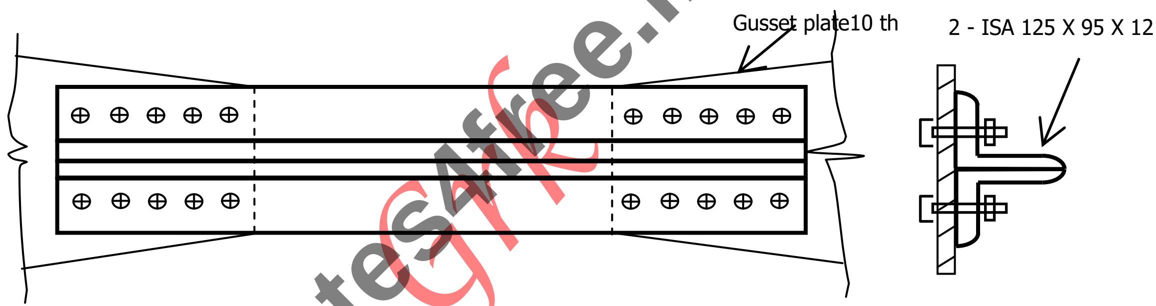
$$P_d = f_{cd} \times A = \frac{80.69 \times 4996}{1000} = 403.13KN.$$

<u>Double angles connected to the same side of gusset plate</u>

<u>Problem:®</u>

A double angle discontinuous strut ISA 125 x 95 x12 mm, ***<u>long legs back to back</u>*** is connected to same side of gusset plate 10 mm thick with 10 bolts on each end. The length of strut b/w c/c of intersection is 4 m. Determine the flexural torsional strength of the section.

<u>Solution:</u>



Properties of 2-ISA 125 x 95 x 12

$a = 49.96cm^2 = 4996mm^2$ $\quad r_{yy} = 3.7cm = 37mm(\text{'0' mm th. gusset plate})$

$r_{ZZ} = 3.91cm = 39.1mm$ $\quad r_{min} = r_{YY} = 37mm$

$$KL = 0.7 \times L \text{ to } 0.85 \times L$$

$$\text{Assuming } KL = 0.85 \times L = 0.85 \times 4m = 3400\ mm$$

$$\text{Effective slenderness ratio} = \frac{KL}{r_{min}} = \frac{3400}{37} = 91.89 < 180 \qquad \text{Safe}$$

Ref Table 9(c) , P – 42 for $f_y = 250\ N/mm^2$

$$f_{cd} \text{ for } 91.89 = 121 - \frac{1.89 \times 14}{10}$$

$$f_{cd} = 118.35\ N/mm^2 \prec \frac{f_y}{\gamma_{mo}} = \frac{250}{1.1} = 227.27\ N/mm^2$$

λ	f_{cd}
90	121
91.89	?
90	107
10	14
1.89	? (x)

Buckling Strength of the member= Safe stress x area provided

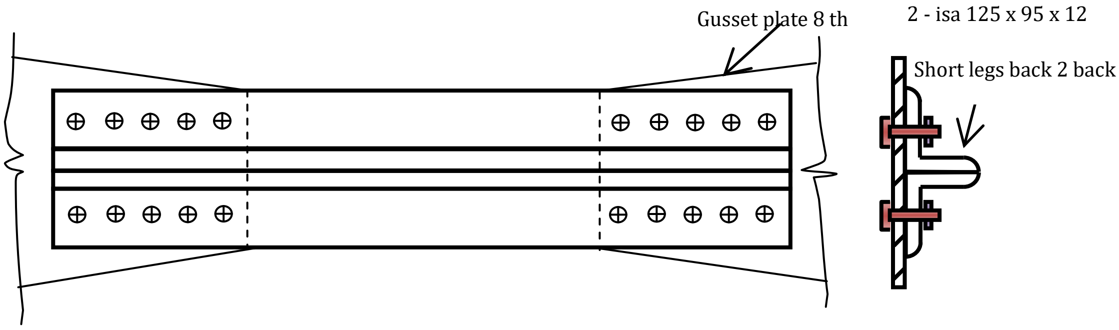
$$= f_{cd} \times A = \frac{118.35 \times 4996}{1000} = 591.30\ KN.$$

Problem:®

A double angle discontinuous strut ISA 125 x 95 x12 mm, ***short legs back to back*** is connected to same side of gusset plate 10 mm thick with 2 bolts or more bolts. The length of strut b/w c/c of intersection is 4 m. Determine the flexural torsional strength of the section.

Solution:

Short legs back to back



Properties of 2-ISA 125 x 95 x 12

$a = 49.96 cm^2 = 4996\ mm^2$ $\quad r_{yy} = 5.57 cm = 55.7 mm$('0' mm th. gap)

$r_{zz} = 2.76 cm = 27.6\ mm$ $\quad r_{min} = r_{zz} = 27.6\ mm$

$KL = 0.7 \times L$ to $0.85 \times L$

Assuming $KL = 0.85 \times L = 0.85 \times 4m = 3400 mm$

Effective slenderness ratio $= \dfrac{KL}{r_{min}} = \dfrac{3400}{27.6} = 123.20 < 180$ Safe

Ref Table 9(c) , P – 42 for $f_y = 250\ N/mm^2$

f_{cd} for $123.20 = 83.7 - \dfrac{3.2 \times 9.4}{10} = 80.69\ N/mm^2$

λ	f_{cd}
120	83.7
123.20	?
130	74.3
10	9.4
3.20	? (x)

Buckling Strength of the member= Safe stress x area provided

$$P_d = f_{cd} \times A = \frac{80.69 \times 4996}{1000} = 403.13\ KN.$$

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DESIGN PROBLEMS

a) Design Procedure for single angle Struts:

1. Assume Compressive stress between $0.4f_y$ to $0.6f_y$ where, $f_y = 250\ N/mm^2$
2. Calculate Area of section required

$$Area = \frac{Load}{Compressive\quad stress}$$

3. Choose a suitable section from the steel table by assuming 15 % to 25% more than Area required.
4. Calculate Effective slenderness Ratio

$$\lambda_e = \sqrt{k_1 + k_2\lambda_{vv}^2 + k_3\lambda_\phi^2}$$

$k_1, k_2, k_3 =$ Constants depending upon the end condition as given in Table 12.

$$\lambda_{vv} = \frac{\left(\frac{l}{r_{vv}}\right)}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}} \qquad \text{and} \qquad \lambda_{vv} = \frac{(b_1 + b_2)/2t}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}}$$

Where,

l = C/c length of the supporting member,

r_{vv} = radius of gyration about the minor axis,

b_1 , b_2 = Width of the two legs of the angle

t = Thickness of the leg, and

ε = Yield stress ratio = $\left(\frac{250}{f_y}\right)^{0.5}$

Ref P- 42,

$$f_{cd} = \frac{f_y/\gamma_{mo}}{\phi + \left[\phi^2 - \lambda_e^2\right]^{0.5}} \leq f_y/\gamma_{mo}$$

Where,

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

α = Imperfection factor given in Table 7.

α = 0.49

Strength of the member P_d = Compressive Stress (f_{cd}) x Area of the member

$$\mathbf{P_d = f_{cd} \times A > P}$$

End Connection:

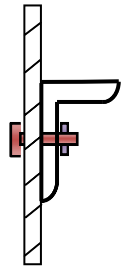
1) Bolted connection:

Bolt Value (BV):

The strength of a bolt in **shearing** and in **bearing** is computed and the **lesser** is called the **Bolt value (BV)** (i.e., Least of V_{nsb} and V_{npb})

1) Strength of one bolt in single shear

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



2) Strength of bolt in bearing

$$V_{dpb} = \left(\frac{2.5k_b \times d \times t \times f_u}{\gamma_{mb}}\right)$$

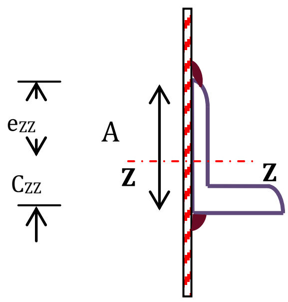
$$\text{No of bolts} = \frac{\text{Force}}{\text{Bolt value}}$$

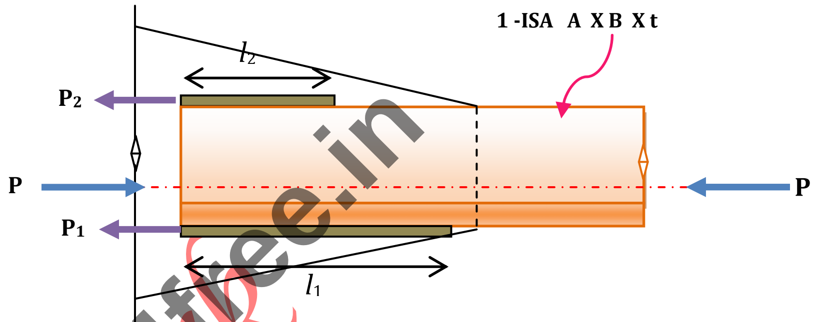
2) Welded connection:

Size of weld:

a) Min size as given table based on thickness of connecting material

b) Max size $\not> \frac{3}{4} \times t$





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

$$\text{Strength of weld at bottom}(P_1) = 0.707 \times D \times l_1 \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}}\ \text{N / mm}$$

$$\text{Strength of weld at top}(P_2) = 0.707 \times D \times l_2 \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}}\ \text{N / mm}$$

$$P_1 + P_2 = P$$

Distributing weld in such a way that c.g. of the weld coincides with that of the angle section

Taking moment about P_2

$$P_1 \times A = P \times e_{xx}$$

$$l_1 = \ ? \quad \text{and} \quad l_2 = \ ?$$

Prob:®

Design a single angle strut for a roof truss carrying a compressive load of 100 KN. The length of strut between c/c intersections is 210 cm. Also design

a) Bolted End Connection, b) Welded End Connection.

Solution:

Load = 100 KN, Factored load = 1.5 x 100 = 150 KN

L= 210 cm = 2100 mm

Assuming 2 or more bolts for connections

Assuming Compressive stress between $0.4f_y$ to $0.6f_y$ where, $f_y = 250\ \text{N/mm}^2$

Permissible stress = 0.4 x f_y = 0.4 x 250 = 100 N/mm^2

Area of section required

$$\text{Area} = \frac{\text{Factored Load}\ (P_u)}{\text{Compressive stress}\ (f_{cd})} = \frac{150 \times 10^3}{100} = 1500\ mm^2$$

Try 1-ISA 100 x 100 x 10 mm @146.2 N/m

Properties of ISA 100 x 100 x 10 mm @ 0.272 KN / m.

$$a = 19.03cm^2 = 1903mm^2$$
$$r_{zz} = r_{yy} = 3.05cm = 30.5mm$$
$$r_{uu} = 3.85cm = 38.5mm$$
$$r_{vv} = 1.94cm = 19.4mm$$
$$r_{min} = r_{vv} = 19.4m$$

$$\text{Effective length}\ (KL) = 210\ cm = 2100\ mm$$

P-48, Cl 7.5.1.2:

Effective slenderness ratio

$$\lambda_e = \sqrt{k_1 + k_2\lambda_{vv}^2 + k_3\lambda_\phi^2}$$

k_1, k_2, k_3 = Constants depending upon the end condition as given in Table 12, P - 48.

$k_1 = 0.2$, $k_2 = 0.35$, $k_3 = 20$

$$\lambda_{vv} = \frac{\left(\frac{l}{r_{vv}}\right)}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}} = \frac{\frac{2100}{19.4}}{1 \times \sqrt{\frac{\pi^2 \times 2 \times 10^5}{250}}} = 1.22$$

$$\varepsilon = \text{Yield stress ratio} = \left(\frac{250}{f_y}\right)^{0.5} = \left(\frac{250}{250}\right)^{0.5} = 1$$

$$\lambda_\phi = \frac{(b_1 + b_2)/2t}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}} = \frac{(100 + 100)/2 \times 10}{1 \times \sqrt{\frac{\pi^2 \times 2 \times 10^5}{250}}} = 0.11$$

$$\lambda_e = \sqrt{0.2 + 0.35 \times 1.22^2 + 20 \times 0.11^2} = 0.98$$

Ref P-34, f_{cd} can be obtained

$$f_{cd} = \frac{f_y/\gamma_{mo}}{\phi + \left[\phi^2 - \lambda_e^2\right]^{0.5}} =\leq f_y/\gamma_{mo}$$

Where,

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

α = Imperfection factor given in Table 7 for class 'c'.

α = 0.49

$$\phi = 0.5\left[1 + 0.49(0.98 - 0.2) + 0.98^2\right] = 1.17$$

$$f_{cd} = \frac{f_y/\gamma_{mo}}{\phi + \left[\phi^2 - \lambda_e^2\right]^{0.5}} = \frac{250/1.1}{1.17 + \left[1.17^2 - 0.98\right]^{0.5}}$$

$$= 125.63 N/mm^2 \leq f_y/\gamma_{mo} = \frac{250}{1.1} = 227.27 N/mm^2$$ Safe

Buckling Strength of the member P_d = Compressive Stress (f_{cd}) x Area of the member

$$\mathbf{P_d = f_{cd} \times A}$$

$$P_d = \frac{125.63 \times 1903}{1000} = 239.10\text{KN} > 150\text{KN}$$

Safe

Provide 1-ISA100 x 100 x 10 mm .

Connection Details:

Assuming 20 mm bolts of grade 4.6

Dia of hole (d_0) = 20+2 = 22 mm

P-75, Cl: 10.3.3

1) For Single shear of bolts

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

Assuming Thread is interfering the shear plane

$$n_n = 1 \qquad n_s = 0\,, \qquad \gamma_{mb} = 1.25$$

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

$$V_{dsb} = \frac{400}{\sqrt{3}} \times \left(\frac{1 \times 245.04}{1.25 \times 1000}\right) = 45.27\ \text{KN}$$

2) Strength of bolt in Bearing $V_{dpb} = \dfrac{2.5 \times k_b \times d \times t \times f_u}{\gamma_{mb}}$

k_b is the least of the following:

1) $\dfrac{e}{3d_0} = \dfrac{40}{3 \times 22} = 0.61$ Edge distance e = 1.5 x 22 =33 mm say 40 mm

2) $\dfrac{p}{3d_0} - 0.25 = \dfrac{60}{3 \times 22} - 0.25 = 0.66$

P = 2.5 x 20 = 50 mm, Say 60mm

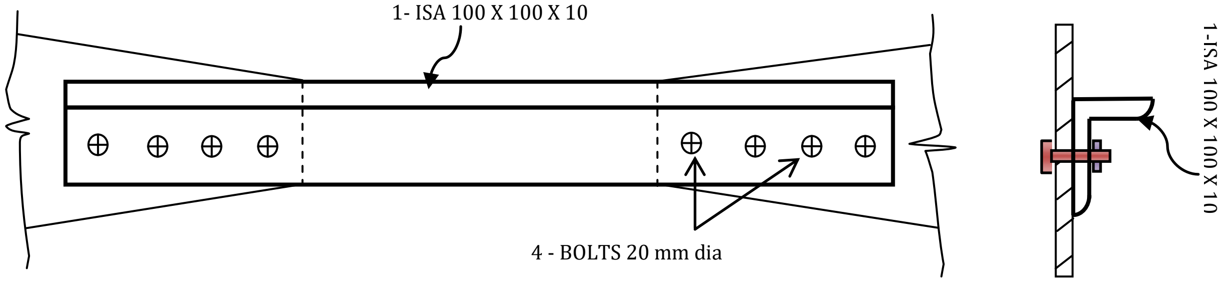
3) $\dfrac{f_{ub}}{f_u} = \dfrac{400}{410} = 0.98$ 4) 1

k_b = 0.51

$$V_{dpb} = \frac{2.5 \times 0.61 \times 20 \times 10 \times 400}{1.25 \times 1000} = 97.6\ \text{KN}$$

Bolt value (BV) = 45.27 KN.

No of bolts = $\dfrac{150}{45.27} = 3.31$ $\boxed{\text{Say 4 No's}}$

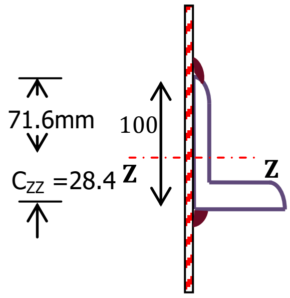


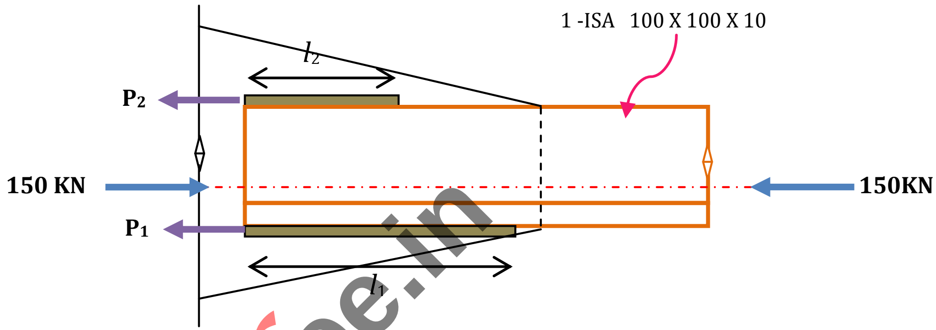
Welded connection:

C_{zz} = 2.84 cm =28.4mm, e_{zz} = 7.16 cm =71.6 mm,

Size of weld:

a) Min size = 3 mm





b) Max size $\not> \frac{3}{4} \times t = \frac{3}{4} \times 10 = 7.5mm$ Say D = 6 mm.

Assuming field weld, $\gamma_{mw} = 1.50$

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

$$\text{Strength of weld at bottom}(P_1) = 0.707 \times D \times l_1 \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}}$$

$$= 0.707 \times 6 \times \frac{410}{\sqrt{3} \times 1.50} = 670\, l_1 \text{N/mm}$$

$$\text{Strength of weld at top}(P_2) = 0.707 \times D \times l_2 \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}} = 670 l_2 \text{ N/mm}$$

$$P_1 + P_2 = P$$

Distributing weld in such a way that c.g. of the weld coincides with that of the angle section. Taking moment about P_2

$$P_2 \times 100 = P \times 71.6$$

$$670 \times l_1 \times 100 = 150 \times 10^3 \times 71.6$$

$$l_1 = \frac{150 \times 10^3 \times 71.6}{670 \times 100} = 160.30 \text{ mm} \qquad \text{Say } 165 \text{ mm}$$

$$P_1 = 670 \times 165 = 110.55 \times 10^3 \text{N}$$

$$P_2 = P - P_1 = 150 \times 10^3 - 110.55 \times 10^3 = 39.45 \times 10^3 \text{N}$$

$$P_2 = 670 l_2 = 39.45 \times 10^3 \text{N}$$

$$\therefore l_2 = \frac{39.45 \times 10^3}{670} = 58.88 \text{ mm} \qquad \text{Say } 65 \text{ mm}$$

FEB 1997 –15 MARKS

3) b) Design a compression member of a roof truss to carry an axial load of 150 KN. Design the member using a single ***unequal angle*** and the corresponding connections to a gusset plate using 20mm dia bolts of 4.6 grade, ***connecting*** the ***longer legs*** to the gusset plate of 8mm thick. Take length of the member = 2.5 m

Solution:

Load = 150 KN, Factored load = 1.5 x 150 = 225 KN

L= 2.5 m = 2500 mm

Assuming Compressive stress between $0.4f_y$ to $0.6f_y$ where, $f_y = 250\ \text{N/mm}^2$

Permissible stress = 0.4 x f_y = 0.4 x 250 = 100 N/mm^2

Area of section required

$$\text{Area} = \frac{\text{Factored Load}\,(P_u)}{\text{Compressive stress}\,(f_{cd})} = \frac{225\times10^3}{100} = 2250\ \text{mm}^2$$

Try 1-ISA 150 x 75 x 12 mm

Properties of ISA 150 x 75 x 12 mm

$a = 25.62\text{cm}^2 = 2562\text{mm}^2$ $\qquad r_{uu} = 4.93\text{cm} = 49.3\text{mm}$

$r_{zz} = 4.79\text{cm} = 39.6\text{mm}$ $\qquad r_{vv} = 1.58\text{cm} = 15.8\text{mm}$

$r_{yy} = 1.97\text{cm} = 19.7\text{mm}$ $\qquad r_{min} = r_{vv} = 15.8\text{mm}$

$$\text{Effective length}(l) = 2.5\text{m} = 2500\text{mm}$$

Ref P-34, f_{cd} can be obtained

$$f_{cd} = \frac{f_y/\gamma_{mo}}{\phi + \left[\phi^2 - \lambda_e^2\right]^{0.5}} = \chi\, f_y/\gamma_{mo} \le f_y/\gamma_{mo}$$

Effective slenderness ratio **(P-48, Cl 7.5.1.2):**

$$\lambda_e = \sqrt{k_1 + k_2\lambda_{vv}^2 + k_3\lambda_\phi^2}$$

Assuming 2 or more bolts for connections and end is fixed

$k_1, k_2, k_3 =$ Constants depending upon the end condition as given in Table 12, P - 48.

$k_1 = 0.2,\qquad k_2 = 0.35,\qquad k_3 = 20$

$$\lambda_{vv} = \frac{\left(\frac{l}{r_{vv}}\right)}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}} = \frac{\frac{2500}{15.8}}{1\times\sqrt{\frac{\pi^2\times2\times10^5}{250}}} = 1.78$$

$$\varepsilon = \text{Yield stress ratio} = \left(\frac{250}{f_y}\right)^{0.5} = \left(\frac{250}{250}\right)^{0.5} = 1$$

$$\lambda_\phi = \frac{(b_1+b_2)/2t}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}} = \frac{(150+75)/2\times10}{1\times\sqrt{\frac{\pi^2\times2\times10^5}{250}}} = 0.105$$

$$\lambda_e = \sqrt{0.2 + 0.35\times1.78^2 + 20\times0.105^2} = 1.236$$

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

α = Imperfection factor given in Table 7 for class 'c'.

α = 0.49

$$= 0.5\left[1+0.49(1.78-0.2)+1.78^2\right] = 1.517$$

$$f_{cd} = \frac{f_y/\gamma_{mo}}{\phi+\left[\phi^2-\lambda_e^2\right]^{0.5}} = \frac{250/1.1}{1.76+\left[1.76^2-1.39^2\right]^{0.5}}$$

$$f_{cd} = 94.83\text{N/mm}^2 \le f_y/\gamma_{mo} = \frac{250}{1.1} = 227.27\text{N/mm}^2 \qquad \text{Safe}$$

Buckling Strength of the member P_d = Compressive Stress (f_{cd}) x Area of the member

$P_d = f_{cd}$ x A

$$P_d = \frac{94.83\times 2562}{1000} = 242.95\text{ KN} > 225\text{ KN} \qquad \text{Safe}$$

Provide 1- ISA 150 x 75 x 12 mm.

Connection Details:

Taking 20 mm bolts of grade 4.6

Dia of hole (d_0) = 20 +2 =22 mm

P-75, Cl: 10.3.3

1) Strength of one bolt in Single shear:

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

Assuming thread is interfering the shear plane

$n_n = 1 \qquad n_s = 0 \;, \gamma_{mb} = 1.25$

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

$$V_{dsb} = \frac{400}{\sqrt{3}}\times\left(\frac{1\times 245.04}{1.25\times 1000}\right) = 45.27\text{ KN}$$

2) Strength of bolt in Bearing $V_{dpb} = \dfrac{2.5\times k_b\times d\times t\times f_u}{\gamma_{mb}}$

k_b is the least of the following:

1) $\dfrac{e}{3d_0} = \dfrac{40}{3\times 22} = 0.61$ Edge distance e = 1.5 x 22 =33 mm say 40 mm

2) $\dfrac{p}{3d_0} - 0.25 = \dfrac{50}{3\times 22} - 0.25 = 0.51$ P = 2.5 x 20 = 50 mm

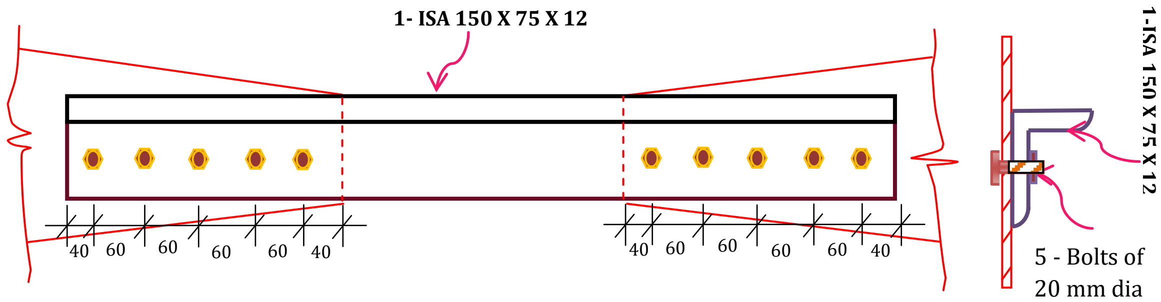
3) $\dfrac{f_{ub}}{f_u} = \dfrac{400}{410} = 0.98$ 4) 1

k_b = 0.51

$$V_{dpb} = \frac{2.5\times 0.51\times 20\times 8\times 400}{1.25\times 1000} = 65.28\text{KN}$$

Bolt value (BV) = 45.27 KN.

No of bolts = $\frac{225}{45.27} = 4.97$ $\boxed{\text{Say 5 No's}}$



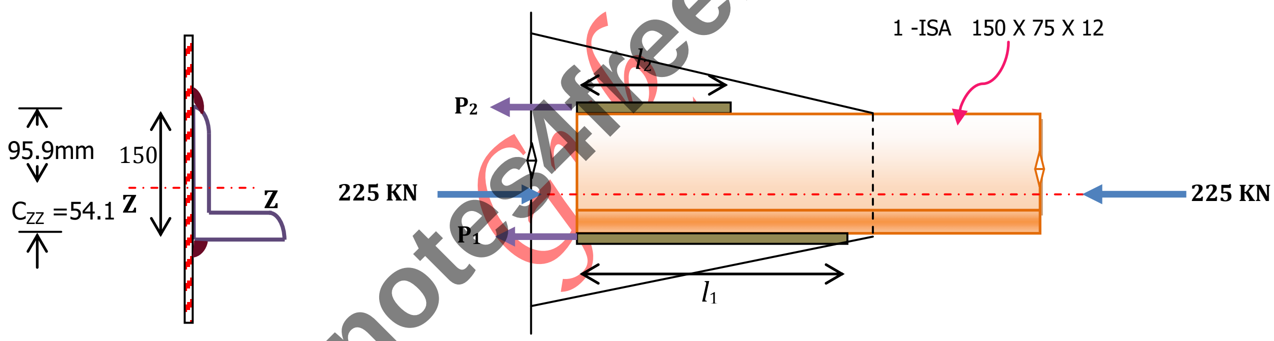
Welded connection:

C_{zz} = 5.41 cm =54.1 mm, e_{zz} = 9.59 cm =95.9 mm,

Size of weld:

a) Min size = 3 mm

b) Max size $\ngtr \frac{3}{4} \times t = \frac{3}{4} \times 12 = 9\text{ mm}$ Say D = 6 mm.



Assuming field weld, $\gamma_{mw} = 1.50$

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

$$\text{Strength of weld at bottom}(P_1) = 0.707 \times D \times l_1 \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}}$$

$$= 0.707 \times 6 \times \frac{410}{\sqrt{3} \times 1.50} = 670 l_1 \text{N / mm}$$

$$\text{Strength of weld at top}(P_2) = 0.707 \times D \times l_2 \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}} = 670 l_2 \text{ N / mm}$$

$$P_1 + P_2 = P$$

Distributing weld in such a way that c.g. of the weld coincides with that of the angle section. Taking moment about P_2

$$P_1 \times 150 = P \times 95.9$$

$$670 \times l_1 \times 150 = 225 \times 10^3 \times 95.9$$

$$l_1 = \frac{225 \times 10^3 \times 95.9}{670 \times 150} = 214.7 \text{ mm} \qquad \text{Say } 220 \text{ mm}$$

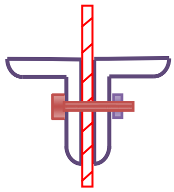
$$P_1 = 670 \times 220 = 147.4 \times 10^3 N$$

$$P_2 = P - P_1 = 225 \times 10^3 - 147.4 \times 10^3 = 77.6 \times 10^3 N$$

$$P_2 = 670 l_2 = 77.6 \times 10^3 N$$

$$\therefore l_2 = \frac{77.6 \times 10^3}{670} = 115.82 \text{ mm} \qquad \text{Say } 120 \text{ mm}$$

b) Design Procedure for double angle Struts:



1. Assume Compressive stress between $0.4f_y$ to $0.6f_y$ where, $f_y = 250 \text{ N/mm}^2$
2. Calculate Area of section required

$$\text{Area} = \frac{\text{Factored Load}(P_u)}{\text{Compressive stress}(f_{cd})}$$

3. Choose a suitable section from the steel table by assuming 15 % to 25% more than Area required.

P-48, Cl 7.5.2.1, Effective length:

$$KL = 0.7 \times L \text{ to } 0.85 \times L$$

Effective slenderness ratio

$$\lambda_e = \frac{KL}{r_{min}} \not> 180$$

Ref P-42, Table 9(c) and find f_{cd}

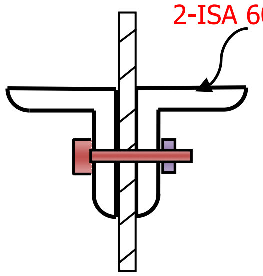
Strength of the member P_d = Compressive Stress (f_{cd}) x Area of the member

$$\mathbf{P_d = f_{cd} \times A > P}$$

1995 Aug - 06 marks

prob:

4(b) Design a compression member using double angles to carry 200 KN load. The length of the member between intersection is 1.5 m. The thickness of gusset plate is 10mm.



Solution:

Load = 200KN, Factored load = 1.5 x 200 = 300 KN

Assuming, stress $f_{ad} = 0.7f_y = 0.7 \times 250 = 175 \text{ N/mm}^2$

$$\text{Area required} = \frac{300 \times 10^3}{175} = 1714.30 \text{ mm}^2 = 17.14 \text{ cm}^2$$

Case 1: Equal angles on either side of gusset plate

Try 2-ISA 60 x 60 x 10

$$a = 22\text{cm}^2 = 2200\text{mm}^2 \quad r_{yy} = 2.95\text{cm} = 29.5\text{mm}(10\text{mm th. gusset plate})$$

$$r_{zz} = 1.78\text{cm} = 17.8\text{mm} \quad r_{min} = r_{zz} = 17.8\text{mm}$$

$$KL = 0.7 \times L \text{ to } 0.85 \times L$$

$$\text{Assuming } KL = 0.8 \times L = 0.8 \times 1500\text{mm} = 1200\text{mm}$$

$$\text{Effective slenderness ratio} = \frac{KL}{r_{min}} = \frac{1200}{17.8} = 67.42 < 180 \qquad \text{Safe}$$

Ref Table 9(c) , P – 42 for $f_y = 250\ N/mm^2$

$$f_{cd} \text{ for } 67.42 = 168 - \frac{7.42 \times 16}{10} = 156.13\ N/mm^2$$

λ	f_{cd}
60	168
67.42	?
70	152
10	16
7.42	?(x)

Buckling Strength of the member= Safe stress x area provided

$$P_d = f_{cd} \times A = \frac{156.13 \times 2200}{1000} = 343.5\ KN > 300\ KN.$$

Safe

Provide 2-ISA 60 x 60 x 10

Connection Details:

1) Bolted Connection:

t* = Min thickness of a) Thickness of gusset plate = 10 mm

b) Sum of the thickness of angles =10+10 =20mm

Dia of bolt using unwin's formula

$$d = 6.04\sqrt{t^*} = 6.04\sqrt{10} = 19.10mm$$

say 18mm

Dia of hole (d_0) = 18 + 2 = 20 mm

1) Strength of bolts in double shear :

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

Assuming shank and thread both interfere the shear plane

$n_n = 1$ $n_s = 1$, $\gamma_{mb} = 1.25$

$$A_{sb} = \frac{\pi}{4} d^2 = \frac{\pi}{4} \times 20^2 = 314.16\ mm^2$$

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

$$V_{dsb} = \frac{400}{\sqrt{3}} \times \left(\frac{1 \times 245.04 + 1 \times 314.16}{1.25 \times 1000}\right) = 103.31\ KN$$

2) **Strength of bolt in Bearing** $V_{dpb} = \dfrac{2.5 \times k_b \times d \times t \times f_u}{\gamma_{mb}}$

k_b is the least of the following:

1) $\dfrac{e}{3d_0} = \dfrac{35}{3 \times 22} = 0.53$ Edge distance e = 1.5 x 20 =30 mm say 35 mm

2) $\dfrac{p}{3d_0} - 0.25 = \dfrac{50}{3 \times 22} - 0.25 = 0.51$

P = 2.5 x 18 = 45 mm, Say 50mm

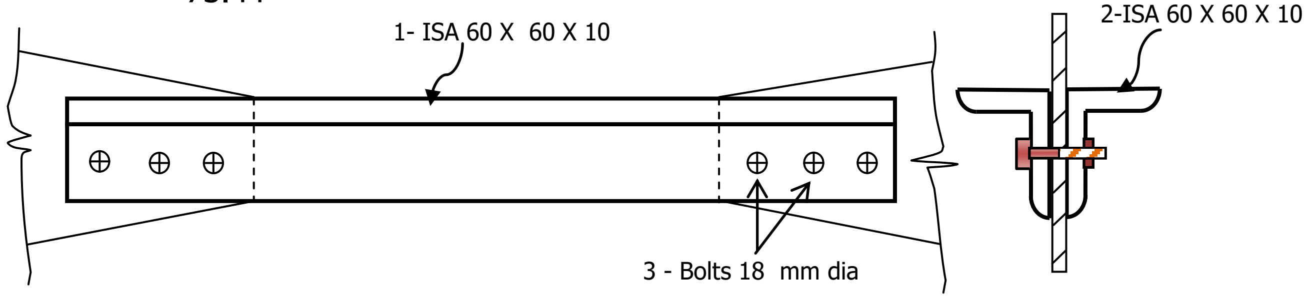
3) $\dfrac{f_{ub}}{f_u} = \dfrac{400}{410} = 0.98$ 4) 1

k_b = 0.51

$$V_{dpb} = \frac{2.5 \times 0.51 \times 18 \times 10 \times 400}{1.25 \times 1000} = 73.44\ KN$$

Bolt value (BV) = 73.44 KN.

No of bolts = $\frac{300}{73.44} = 4.08$ $\boxed{\text{Say 5 No's}}$



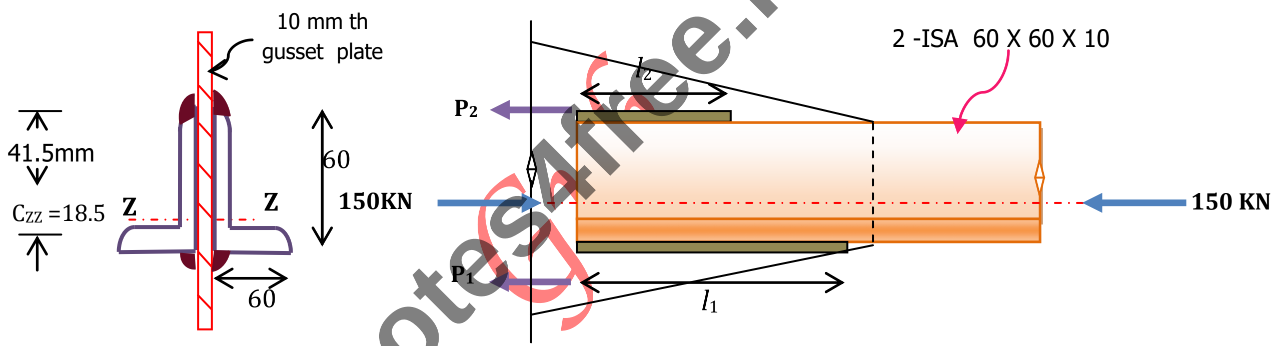
Welded connection:

C_{zz} = 1.85 cm =18.5 mm, e_{zz} = 4.15 cm = 41.5 mm,

Size of weld:

a) Min size = 3 mm

b) Max size $\not> \frac{3}{4} \times t = \frac{3}{4} \times 10 = 7.5\text{ mm}$ Say D = 6 mm.



Assuming field weld, $\gamma_{mw} = 1.50$

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

$$\text{Strength of weld at bottom}(P_1) = 0.707 \times D \times l_1 \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}}$$

$$= 0.707 \times 6 \times \frac{410}{\sqrt{3} \times 1.50} = 670 l_1 \text{N / mm}$$

$$\text{Strength of weld at top}(P_2) = 0.707 \times D \times l_2 \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}} = 670 l_2 \text{ N / mm}$$

Welding is done to both the angles on either side of the gusset plate, let us design one side of the gusset plate for a load of P' = P/2 = 300/2 =150 KN

$$P_1 + P_2 = P'$$

Distributing weld in such a way that c.g. of the weld coincides with that of the angle section

Taking moment about P_2

$$P_1 \times 60 = P' \times 71.6$$

$$670 \times l_1 \times 60 = 150 \times 10^3 \times 41.5$$

$$l_1 = \frac{150 \times 10^3 \times 41.5}{670 \times 60} = 154.85 \text{ mm} \qquad \text{Say } 160 \text{ mm}$$

$$P_1 = 670 \times 160 = 107.20 \times 10^3 \text{N}$$

$$P_2 = P - P_1 = 150 \times 10^3 - 107.20 \times 10^3 = 42.80 \times 10^3 \text{N}$$

$$P_2 = 670 l_2 = 42.80 \times 10^3 \text{N}$$

$$\therefore l_2 = \frac{42.80 \times 10^3}{670} = 63.88 \text{mm} \qquad \text{Say } 70 \text{ mm}$$

Case 2: Equal angles on same side of gusset plate

Try 2-ISA 60 x 60 x 10

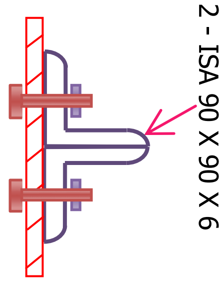
$a = 22 cm^2 = 2200\ mm^2$ $\qquad r_{yy} = 2.95 cm = 29.5 mm$ (10mm th. gusset plate)

$r_{zz} = 1.78 cm = 17.8\ mm$ $\qquad r_{min} = r_{zz} = 17.8\ mm$

$KL = 0.7 \times L$ to $0.85 \times L$

Assuming $KL = 0.8 \times L = 0.8 \times 1500mm = 1200\ mm$

Effective slenderness ratio $= \dfrac{KL}{r_{min}} = \dfrac{1200}{17.8} = 67.42 < 180$ Safe



Ref Table 9(c) , P – 42 for $f_y = 250 N/mm^2$

$$f_{cd} \text{ for } 67.42 = 168 - \frac{7.42 \times 16}{10} = 156.13 N/mm^2$$

λ	f_{cd}
60	168
67.42	?
70	152
10	16
7.42	?(x)

Buckling Strength of the member= Safe stress x area provided

$$P_d = f_{cd} \times A = \frac{156.13 \times 2200}{1000} = 343.5 \text{ KN} > 300 \text{ KN.}$$

Safe

Provide 2-ISA 60 x 60 x 10

Connection Details:

A) Bolted Connection:

t* = Min thickness of a) Thickness of gusset plate = 10 mm

b) Thickness of angle =10 mm

Dia of bolt using unwin's formula

$$d = 6.04\sqrt{t^*} = 6.04\sqrt{10} = 19.10 mm$$

say 18mm

Dia of hole (d_0) = 18 + 2 = 20 mm

1) Strength of bolts in single shear :

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

Assuming shank is interfering the shear plane

$$n_s = 1, \quad \gamma_{mb} = 1.25 \quad A_{sb} = \frac{\pi}{4} d^2 = \frac{\pi}{4} \times 20^2 = 314.16\ mm^2$$

$$V_{dsb} = \frac{400}{\sqrt{3}} \times \left(\frac{1 \times 314.16}{1.25 \times 1000}\right) = 58.04 KN$$

2) Strength of bolt in Bearing $V_{dpb} = \dfrac{2.5 \times k_b \times d \times t \times f_u}{\gamma_{mb}}$

k_b is the least of the following:

1) $\dfrac{e}{3d_0} = \dfrac{35}{3 \times 22} = 0.53$ Edge distance e = 1.5 x 20 =30 mm say 35 mm

2) $\dfrac{p}{3d_0} - 0.25 = \dfrac{50}{3 \times 22} - 0.25 = 0.51$

P = 2.5 x 18 = 45 mm, Say 50mm

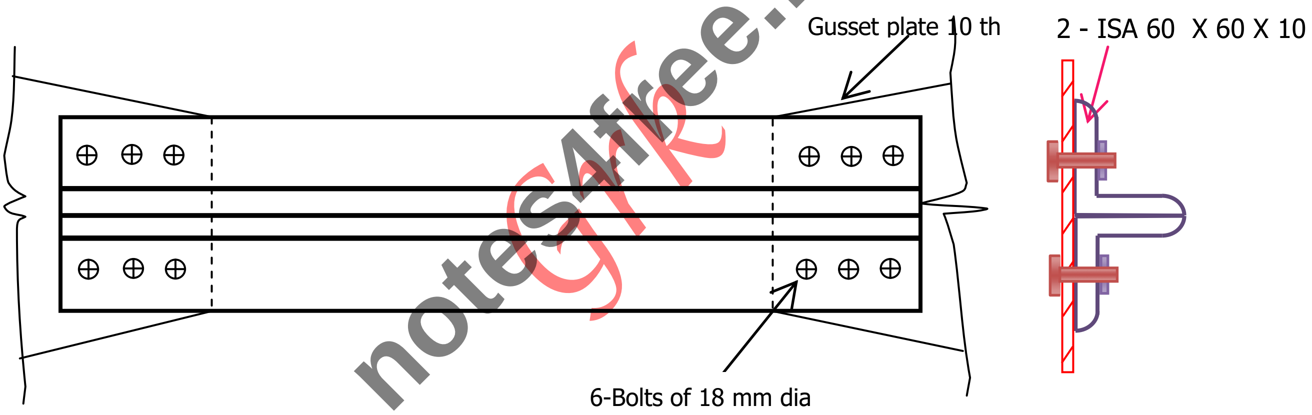
3) $\dfrac{f_{ub}}{f_u} = \dfrac{400}{410} = 0.98$ 4) 1

$k_b = 0.51$

$$V_{dpb} = \frac{2.5 \times 0.51 \times 18 \times 10 \times 400}{1.25 \times 1000} = 73.44 \text{ KN}$$

Bolt value (BV) = 58.04 KN.

No of bolts = $\dfrac{300}{58.04} = 5.17$ $\boxed{\text{Say 6 No's}}$



Welded connection:

Size of weld:

a) Min size = 3 mm

b) Max size $\not> \frac{3}{4} \times t = \frac{3}{4} \times 10 = 7.5\text{mm}$ Say D = 6 mm.

Assuming field weld, $\gamma_{mw} = 1.50$

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

$$\text{Strength of weld at bottom}(P_1) = 0.707 \times D \times l_1 \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}}$$

$$= 0.707 \times 6 \times \frac{410}{\sqrt{3} \times 1.50} = 670 l_1 \text{N / mm}$$

$$\text{Strength of weld at top}(P_2) = 0.707 \times D \times l_2 \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}} = 670 l_2 \text{ N / mm}$$

$$P_1 + P_2 = P$$

Distributing weld in such a way that c.g. of the weld coincides with that of the angle section

Taking moment about P_2

$$P_1 \times 120 = P \times 60$$

$$670 \times l_1 \times 120 = 300 \times 10^3 \times 60 \qquad l_1 = \frac{300 \times 10^3 \times 60}{670 \times 120} = 223.88 \text{ mm Say } 230 \text{ mm}$$

Since the load is acting exactly at the centre of its connection, therefore $l_1 = l_2 = 230$ mm.

Case 3: Unequal angles on either side of gusset plate (Short legs back to back)

Case 4: Unequal angles on either side of gusset plate (Long legs back to back)

Case 5: Unequal angles on same side of gusset plate (Short legs back to back)

Case 6: Unequal angles on same side of gusset plate (Long legs back to back)

Feb-1996 –10 marks

Prob:

4(b) A strut in a roof truss carries an axial load of 200 KN. Design a suitable double angle section for the strut. The effective length of the strut is 2 m and yield stress for the steel is 260 MPa.The thickness of the gusset plate is 20mm.

Prob: Negi

A strut in a roof truss carries an axial compressive load of 180 KN. Design a suitable double angle section for the compression member. The length of strut between center to center of intersection is 2.3 m and yield stress of steel is 250 Mpa.

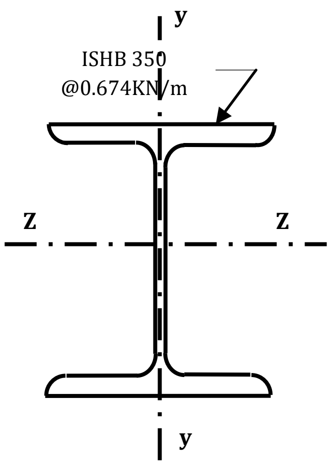
COLUMNS (STANCHION):

ANALYSIS PROBLEMS:

1) Depending on the boundary condition
   L_{eff} is calculated using table 11, P – 45
2) Determine buckling class of cross section from Table 10, Page 44
   $\frac{h}{b_f}$ & t_f values
3) Effective Slenderness ratio $\lambda_{ZZ} = \frac{KL}{r_{ZZ}}$ & $\lambda_{YY} = \frac{KL}{r_{YY}}$
4) Based on Slenderness ratio obtain the f_{cd} value from corresponding table from page No's 40 to 44.
5) Design stress f_{cd} = Min of the $f_{cd)ZZ}$ & $f_{cd)YY}$
6) Safe load = Design stress (f_{ad}) x Area provided

Problem:

A rolled steel beam section ISHB 350 @ 0.674 KN/m is used as stanchion. If the unsupported length of stanchion is 4 m, determine the safe load carrying capacity of stanchion.



Solution:

Properties of ISHB 350 @ 0.674 KN/m

$a = 85.91cm^2 = 85.91 \times 100mm^2$

$h = 350mm, \quad b_f = 250\text{ mm}, \; t_f = 11.6\text{ mm}, t_w = 8.3\text{ mm}.$

$r_{ZZ} = 14.93cm = 149.3mm, \quad r_{yy} = 5.34cm = 53.4mm$

$l_{eff} = 4m = 4000mm$

Determination of buckling class of cross section

Since

$$\frac{h}{b_f} = \frac{350}{250} = 1.4 > 1.2 \quad \text{and} \quad t_f = 11.6 < 40\text{ mm}$$

We should use buckling class 'a' about Z-Z axis and 'b' about y-y axis, Referring to Table 10, P- 44, IS 800 – 2007.

P- 34, Cl 7.1.2.1

Compressive Stress (f_{cd}):

1) About Z-Z axis :

$$f_{cd} = \frac{f_y/\gamma_{mo}}{\phi + \left[\phi^2 - \lambda^2\right]^{0.5}} \le f_y/\gamma_{mo}$$

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

λ = non dimensional effective slenderness ratio

$$\lambda = \sqrt{\frac{f_y}{f_{cc}}} = \sqrt{\frac{f_y\left(\frac{KL}{r}\right)^2}{\pi^2 E}} \qquad \text{Euler buckling stress} = f_{cc} = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}$$

Where,

KL/r = Effective slenderness ratio or ratio of effective length, KL to appropriate radius of gyration 'r'.

α = Imperfection factor given in Table 7, P -35

$$\chi = \frac{1}{\phi + \left[\phi^2 - \lambda^2\right]^{0.5}}$$

Effective length based on end condition

L= 4000 mm

$$\lambda_{ZZ} = \sqrt{\frac{f_y\left(\frac{KL}{r_{zz}}\right)^2}{\pi^2 E}} = \sqrt{\frac{250\left(\frac{4000}{149.3}\right)^2}{\pi^2 \times 2 \times 10^5}} = 0.30$$

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

$$\phi = 0.5\left[1 + 0.21 \times (0.3 - 0.2) + 0.3^2\right] = 0.56$$

$$\chi = \frac{1}{\phi + \left[\phi^2 - \lambda^2\right]^{0.5}}$$

$$\chi = \frac{1}{0.56 + \left[0.56^2 - 0.3^2\right]^{0.5}} = 0.97$$

$$f_{cd} = \frac{0.97 \times 250}{1.1} = 220.04\ \text{N/mm}^2$$

2) About Y-Y axis :

$$f_{cd} = \frac{f_y / \gamma_{mo}}{\phi + \left[\phi^2 - \lambda^2\right]^{0.5}} = \chi \times f_y / \gamma_{mo} \le f_y / \gamma_{mo}$$

Table 7, P -35

α = Imperfection factor =0.34

Effective length based on end condition

L= 4000 mm

$$\lambda_{YY} = \sqrt{\frac{f_y\left(\frac{KL}{r_{YY}}\right)^2}{\pi^2 E}} = \sqrt{\frac{250\left(\frac{4000}{53.4}\right)^2}{\pi^2 \times 2 \times 10^5}} = 0.84$$

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

$$\phi = 0.5\left[1 + 0.34 \times (0.84 - 0.2) + 0.84^2\right] = 0.96$$

$$\chi = \frac{1}{\phi + \left[\phi^2 - \lambda^2\right]^{0.5}}$$

$$\chi = \frac{1}{0.96 + \left[0.96^2 - 0.84^2\right]^{0.5}} = 0.70$$

$$f_{cd} = \frac{0.70 \times 250}{1.1} = 159.52\text{N/mm}^2$$

Compressive stress min of the above two values

$$f_{cd} = 75\ \text{N/mm}^2$$

Load carrying capacity = Safe stress x area provided

$$= \frac{159.52 \times 85.91 \times 10^2}{1000} = 1370.50 \text{ KN.}$$

OR

We should use buckling class 'a' about Z-Z axis and 'b' about y-y axis, Referring to Table 10, P- 44, IS 800 – 2007.

Compressive Stress (f_{cd}): P- 34, Cl 7.1.2.1

About Z-Z axis :

$$\lambda_{zz} = \frac{KL}{r_{zz}} = \frac{4000}{149.3} = 26.80$$

REF TO TABLE 9(a) P-40

λ_{zz}	f_{cd}
20	226
26.80	?
30	220
10	06
6.80	?

$$f_{cd} \text{ for } 26.80 = 226 - \frac{6.8 \times 06}{10} = 221.92 \text{ N/mm}^2$$

Compressive Stress (f_{cd}):

About Y-Y axis :

$$\lambda_{zz} = \frac{KL}{r_{zz}} = \frac{4000}{53.4} = 74.91$$

REF TO TABLE 9(b) P-41

λ_{zz}	f_{cd}
70	166
74.91	?
80	150
10	16
4.91	?

$$f_{cd} \text{ for } 74.91 = 166 - \frac{4.91 \times 16}{10} = 158.14 \text{N/mm}^2$$

f_{cd} is the min of 164.95 N/mm^2, and 158.14 N/mm^2.

i.e., f_{cd} = 158.14N/mm^2.

Load carrying capacity = Safe stress x Area provided

$$= \frac{158.14 \times 8591}{1000} = 1358.60 \text{ KN.}$$

Problem:

Determine the design strength of the rolled steel beam section ISHB 300 @ 0.588 kN/m to be used as stanchion. Effective length of stanchion is 3 m.

Solution:

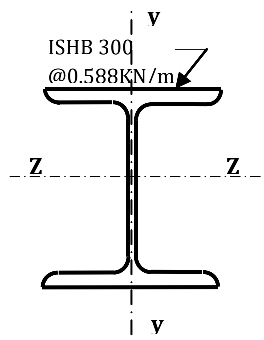
Properties of ISHB 300 @ 0.588 KN/m

$a = 74.85\text{cm}^2 = 7485\text{mm}^2$

$h = 300\text{mm}, \quad b_f = 250 \text{ mm}, \; t_f = 10.6 \text{ mm}.$

$r_{zz} = 12.95\text{cm} = 129.5\text{mm}, \quad r_{yy} = 5.41\text{cm} = 54.1\text{mm}$

$KL = 3000\text{mm}$



Determination of buckling curve classification

Since

$$\frac{h}{b_f} = \frac{300}{250} = 1.2 \quad \text{and} \quad t_f = 10.6 \ \leq 100\text{ mm}$$

We should use buckling class 'b' about Z-Z axis and 'c' about y-y axis, Referring to Table 10, P- 44, IS 800 – 2007.

P- 34, Cl 7.1.2.1

Compressive Stress (f_{cd}):

About Z-Z axis :use buckling class 'b'

$$\lambda_{ZZ} = \frac{KL}{r_{zz}} = \frac{3000}{129.5} = 23.17$$

REF TO TABLE 9(b) P-41

λ_{zz}	f_{cd}
20	225
23.17	?
30	216
10	09
3.17	?

$$f_{cd} \text{ for } 23.17 = 225 - \frac{3.17 \times 09}{10} = 222.15\text{ N/mm}^2$$

Compressive Stress (f_{cd}):

About Y-Y axis : use buckling class 'c'

$$\lambda_{ZZ} = \frac{KL}{r_{zz}} = \frac{3000}{54.1} = 55.45$$

REF TO TABLE 9(c) P-42

λ_{zz}	f_{cd}
50	183
55.45	?
60	168
10	15
5.45	?

$$f_{cd} \text{ for } \ 55.45 = 183 - \frac{5.45 \times 15}{10} = 174.83\text{N/mm}^2$$

f_{cd} is the min of 222.15 N/mm^2, and 174.83 N/mm^2.

i.e., f_{cd} = 174.83 N/mm^2.

Load carrying capacity = Safe stress x Area provided

$$= \frac{174.83 \times 7485}{1000} = 1308.60\text{ KN.}$$

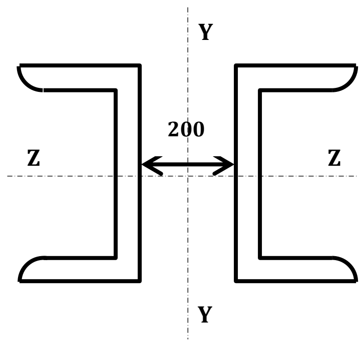
Problem: 1996-Feb (B.U) 20 marks

A steel stanchion is formed of two channels of ISMC 350 placed back to back with a clear spacing of 200 mm. If the effective length of channel is 6m, find safe axial load that the column can carry.

Calculate the extra load the column can carry if 2 plates of 400mm x 10 mm are welded to the channel flanges one on each side.

Solution:

Case-I **Properties of ISMC 350**



$a = 107.32\text{cm}^2 = 107.32 \times 100\text{mm}^2$

$r_{ZZ} = 13.66\text{cm} = 1366\text{mm}, \qquad r_{yy} = 12.76\text{cm} = 1276\text{mm}$

$r_{min} = r_{yy} = 1276\text{mm}$

$l_{eff} = KL = 6\text{m} = 6000\text{mm}$

Effective slenderness ratio

$$\lambda = \frac{KL}{r_{min}} = \frac{6000}{1276} = 47.02$$

Buckling curve classification according to Table 10 – P- 44 is class 'c'

Ref page 42 Table 9 (c) for $f_y = 250 N/mm^2$

$$f_{cd} \text{ for } 47.02 = 198 - \frac{7.02 \times 15}{10} = 187.47 N/mm^2$$

λ	σ_{ac}
40	198
47.02	?
50	183
10	15
7.02	?

flexural buckling strength = Safe stress x Area provided

$$= \frac{187.47 \times 107.08 \times 10^2}{1000} = 2007.50 \text{ KN.}$$

$$\text{Safe load} = \frac{2007.50 \times 10^3}{1.5} = 1338.33 \text{ KN.}$$

<u>Case-II</u> Properties of 2- ISMC 350

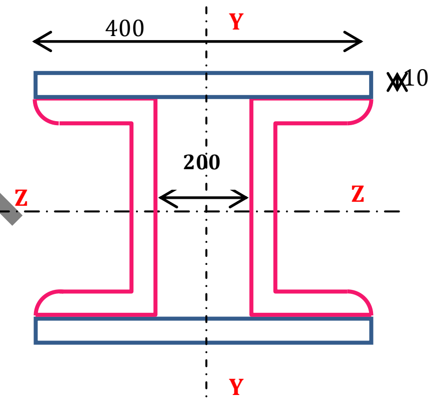
$a = 107.32 cm^2 = 107.32 \times 100 mm^2$

$I_{zz} = 20016 cm^4 = 20016 \times 10^4 mm^4$

$I_{yy} = 17469.4 cm^4 = 17469.4 \times 10^4 mm^4$

$l_{eff} = KL = L = 6m = 6000mm$

$Area(A) = 10732 + 2 \times 400 \times 10 = 18732 mm^2$



<u>I_{ZZ} of the built up section</u>

$$I_{zz} = 20016 \times 10^4 + 2\left[\frac{400 \times 10^3}{12} + 400 \times 10\left(\frac{350}{2} + \frac{10}{2}\right)^2\right]$$

$$= 459.43 \times 10^6 mm^4$$

<u>I_{yy} of the built up section</u>

$$I_{yy} = 17469.4 \times 10^4 + 2\left[\frac{10 \times 400^3}{12}\right]$$

$$= 281.36 \times 10^6 mm^4$$

$$\therefore I_{min} = I_{yy} = 281.36 \times 10^6 mm^4$$

$$r_{min} = \sqrt{\frac{I_{min}}{A}} = \sqrt{\frac{281.36 \times 10^6}{18732}} = 122.55mm$$

Effective slenderness ratio

$$S.R(\lambda) = \frac{6000}{122.55} = 48.95$$

Buckling curve classification according to Table 10 – P- 44 is class 'c'

Ref page 42, Table9 (c) for $f_y = 250 N/mm^2$

$$f_{cd} \text{ for } 48.95 = 198 - \frac{8.95 \times 15}{10} = 184.58 N/mm^2$$

λ	σ_{ac}
40	198
48.95	?
50	183
10	15
8.95	?

flexural buckling strength = = Safe stress x area provided

$$= \frac{184.58 \times 18732}{1000} = 3457.55 KN.$$

Safe load= $\frac{3457.55 \times 10^3}{1.5} = 2305$ KN.

P-187 , DSS BY B.C. PUNMIA:

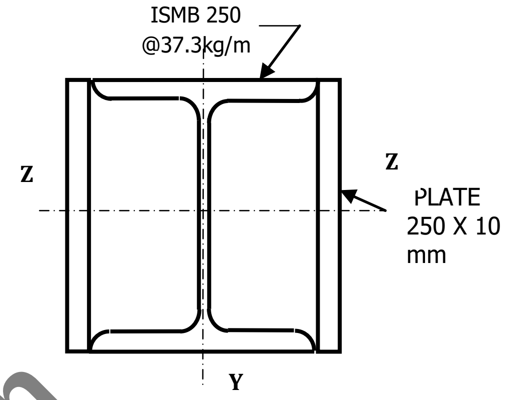
An I- joist ISMB 250 @ 37.3 kg/m has an effective length of 5 m. It is used as a stanchion with two plates 250 x 10 mm welded to its sides, as shown in fig. compute the load carrying capacity. What will be its load carrying capacity if one plate is attached to each flange.

Solution: Properties of ISMB 250 37.3 kg/m.

a = 47.55 cm^2; I_{ZZ} = 5131.6 x 10^4 mm^4;

I_{yy} = 334.5 x 10^4 mm^4.

A = 4755 + 2 x (250 x 10) = 9755 mm^2.



a) Plates attached to sides:

I_{ZZ} of the built up section

$$I_{ZZ} = 5131.6 \times 10^4 + 2\left[\frac{10 \times 250^3}{12}\right] = 77.35 \times 10^6 mm^4$$

I_{yy} of the built up section

$$I_{yy} = 334.5 \times 10^4 + 2\left[\frac{250 \times 10^3}{12} + 250 \times 10\left(\frac{125}{2} + \frac{10}{2}\right)^2\right] = 26.17 \times 10^6 mm^4$$

$$\therefore I_{min} = I_{yy} = 26.17 \times 10^6 \text{ mm}^4$$

$$r_{min} = \sqrt{\frac{I_{min}}{A}} = \sqrt{\frac{26.17 \times 10^6}{9755}} = 51.8 \text{ mm}$$

Effective slenderness ratio

$$S.R(\lambda) = \frac{KL}{r_{min}} = \frac{5000}{51.8} = 96.53$$

λ	σ_{ac}
90	121
96.53	?
100	107
10	14
6.53	?

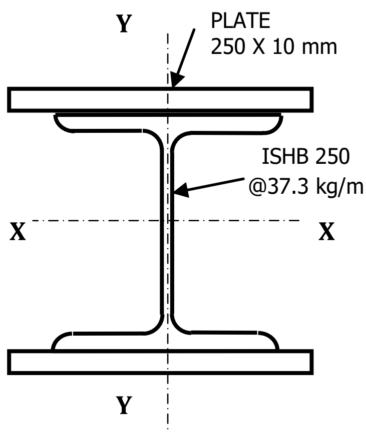
Buckling curve classification according to Table 10 – P- 44 is class 'c'

Ref page 42, Table9 (c) for $f_y = 250 N/mm^2$

$$f_{cd} \text{ for } 96.53 = 121 - \frac{6.53 \times 14}{10} = 111.86 N/mm^2$$

Load carrying capacity = Safe stress x area provided

$$= \frac{111.86 \times 9755}{1000} = 1091.20 \text{ KN.}$$



b) Plates attached to the flange:

I_{ZZ} of the built up section

$$I_{ZZ} = 5131.6 \times 10^4 + 2\left[\frac{250 \times 10^3}{12} + 250 \times 10\left(\frac{250}{2} + \frac{10}{2}\right)^2\right]$$

$$= 135.86 \times 10^6 mm^4$$

I_{yy} of the built up section

$$I_{yy} = 334.5 \times 10^4 + 2\left[\frac{10 \times 250^3}{12}\right]$$

$$= 29.387 \times 10^6 mm^4$$

$$\therefore I_{min} = I_{yy} = 29.387 \times 10^6 mm^4$$

$$r_{min} = \sqrt{\frac{I_{min}}{A}} = \sqrt{\frac{29.387 \times 10^6}{9755}} = 54.89 \text{ mm}$$

Effective slenderness ratio

$$S.R(\lambda) = \frac{5000}{54.89} = 91.1$$

λ	σ_{ac}
90	121
91.1	?
100	107
10	14
1.1	?

Buckling curve classification according to Table 10 – P- 44 is class 'c'

Ref page 42, Table9 (c) for $f_y = 250N/mm^2$

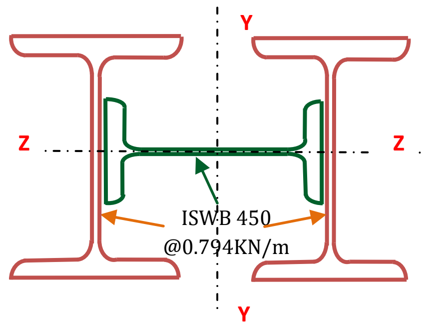
$$f_{cd} \text{ for } 96.53 = 121 - \frac{1.1 \times 14}{10} = 119.46N/mm^2$$

Load carrying capacity = Safe stress x area provided

$$= \frac{119.46 \times 9755}{1000} = 1155.33 \text{ KN.}$$

PROBLEM:

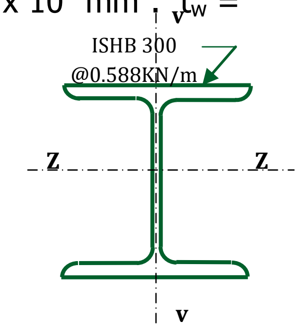
A built – up column consists of three rolled steel beam sections WB 450 @ 0.794 KN/m, connected effectively to act as one column as shown in fig. determine the safe load carrying capacity of built – up section. Unsupported length of column is 4.25m.



Solution:

Properties of 1- ISWB 450 0.794KN/m.

a =101.15 cm^2; I_{ZZ} =35057.6 x 10^4 mm^4; I_{yy} = 1706.7 x 10^4 mm^4, t_w = 9.2 mm.



Total area of built up section

A = 10115 + 2 x (10115) = 30345mm^2.

I_{ZZ} of the built up section:

$$I_{ZZ} = 2 \times 35057.6 \times 10^4 + 1706.7 \times 10^4 = 718.22 \times 10^6 mm^4$$

I_{yy} of the built up section:

$$I_{yy} = 35057.6 \times 10^4 + 2 \times \left[1706.7 \times 10^4 + 10115\left(\frac{450}{2} + \frac{9.2}{2}\right)^2\right] = 1451.15 \times 10^6 mm^4$$

$$\therefore I_{min} = I_{ZZ} = 718.22 \times 10^6 mm^4$$

$$r_{min} = \sqrt{\frac{I_{min}}{A}} = \sqrt{\frac{718.22 \times 10^6}{30345}} = 153.84 \text{ mm}$$

Effective slenderness ratio

$$S.R(\lambda) = \frac{4250}{153.84} = 27.62$$

Buckling curve classification according to Table 10 – P- 44 is class 'c'

Ref page 42, Table9 (c) for $f_y = 250\ \text{N/mm}^2$

$$f_{cd}\ \text{for}\ 27.62 = 224 - \frac{7.62 \times 13}{10} = 214.09\ \text{N/mm}^2$$

λ	f_{cd}
20	224
27.62	?
30	211
10	13
7.62	?

Load carrying capacity = Safe stress x Area provided

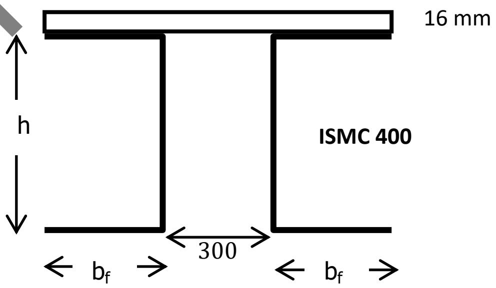
$$= \frac{214.09 \times 30345}{1000} = 6496.56\ \text{kN.}$$

$$\text{Safe load} = \frac{6496.56}{1.5} = 4331\ \text{kN.}$$

Jan / Feb 2006 – 10 marks

Determine the allowable load which the member shown in fig can support, if the member is of 5.5 m effective length. Assume f_y = 250 N/mm^2.

Solution:



Properties of 1- ISMC 400

a =62.93 cm^2; b_f = 100mm, h = 400mm,

I_{ZZ} =15082.8 x 10^4 mm^4; I_{yy} = 504.8 x 10^4 mm^4.

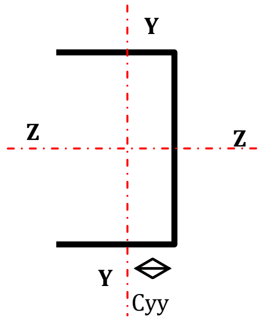
t_w = 9.2 mm. c_{yy} = 2.42 cm = 24.2 mm.

Width of plate at top = b_f + gap + b_f

= 100 + 300 + 100 = 500 mm

A = 2 x 6293 + 500 x 16 = 20586 mm^2.

Centroidal axis distance from Bottom AA reference axis $\bar{Y}$



$$\bar{Y} = \frac{a_1 y_1 + a_2 y_2 + a_3 y_3}{a_1 + a_2 + a_3} =$$

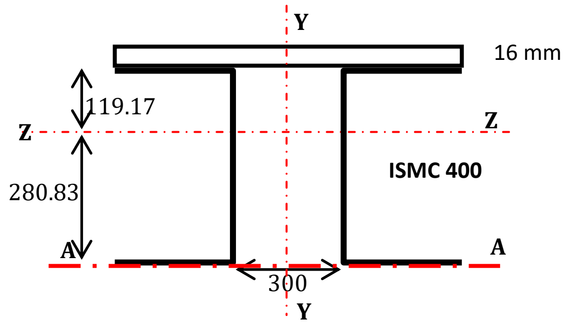
$$\frac{\left[2 \times 6293 \times \left\{\frac{400}{2}\right\}\right] + \left[500 \times 10 \times \left(400 + \frac{16}{2}\right)\right]}{[2 \times 6293] + [500 \times 16]} = 280.83\ \text{mm}$$

I_{ZZ} of the built up section:

$$I_{ZZ} = 2 \times \left[15082.8 \times 10^4 + 6293 \times \left(280.83 - \frac{400}{2}\right)\right] + \left[\frac{500 \times 16^3}{12} + 500 \times 16 \times \left(119.17 + \frac{16}{2}\right)^2\right]$$

$$I_{ZZ} = 512.40 \times 10^6 \text{mm}^4$$

I_{yy} of the built up section:



$$I_{yy} = 2 \times \left[504.8 \times 10^4 + 6293\left(\frac{300}{2} + 24.2\right)^2\right]$$

$$+ \frac{16 \times 500^3}{12} = 558.69 \times 10^6 \text{mm}^4$$

$$\therefore I_{min} = I_{ZZ} = 512.4 \times 10^6 mm^4$$

$$r_{min} = \sqrt{\frac{I_{min}}{A}} = \sqrt{\frac{512.4 \times 10^6}{20586}} = 157.76 \text{ mm}$$

Effective slenderness ratio

$$S.R(\lambda) = \frac{KL}{r_{min}} = \frac{5500}{157.76} = 34.86$$

Buckling curve classification according to Table 10 – P- 44 is class 'c'

Ref page 42, Table9 (c) for $f_y = 250N/mm^2$

λ	f_{cd}
30	211
34.86	?
40	198
10	13
4.86	?

$$f_{cd} \text{ for } 34.86 = 211 - \frac{4.86 \times 13}{10} = 204.68 \text{ N/mm}^2$$

Flexural Strength of member = Safe stress x Area provided

$$= \frac{204.68 \times 20586}{1000} = 4213.58 \text{ kN.}$$

$$\text{Allowable load} = \frac{4213.58}{1.5} = 2809.06 \text{ kN.}$$

OR

Solution:

Properties of 2- ISMC 400

a = 125.86 cm^2= 12586 mm^2;

I_{ZZ} =30165.6 x 10^4 mm^4; I_{yy} = 39202.6x 10^4 mm^4.

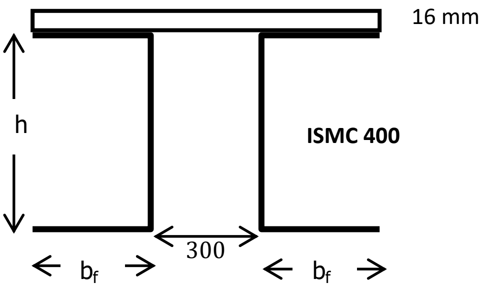
Breadth and depth of single channel section

b_f = 100mm, h = 400mm,

Width of plate at top = b_f + gap + b_f

100 + 300 + 100 = 500 mm

A = 12586 + 500 x 16 = 20586 mm^2.



Centroidal axis distance from Bottom AA reference axis $\bar{Y}$

$$\bar{Y} = \frac{a_1 y_1 + a_2 y_2}{a_1 + a_2} = \frac{\left[12586 \times \left\{\frac{400}{2}\right\}\right] + \left[500 \times 10 \times \left(400 + \frac{16}{2}\right)\right]}{[12586] + [500 \times 16]} = 280.83 \text{ mm}$$

<u>I_{ZZ} of the built up section:</u>

$$I_{ZZ} = \left[30165.6 \text{ x } 10^4 \times 10^4 + 12586 \times \left(280.83 - \frac{400}{2}\right)^2\right] +$$

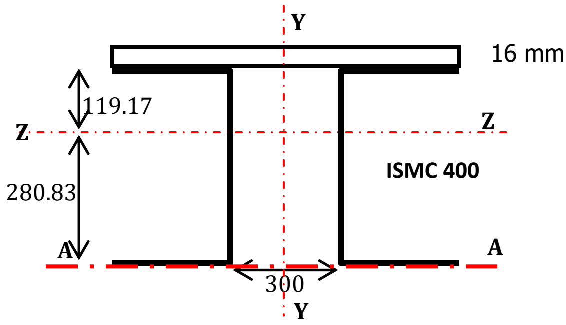
$$\left[\frac{500 \times 16^3}{12} + 500 \times 16 \times \left(119.17 + \frac{16}{2}\right)^2\right] = 512.40 \times 10^6 \text{ mm}^4$$

<u>I_{yy} of the built up section:</u>

$$I_{yy} = \left[39202.6 \times 10^4\right] + \frac{16 \times 500^3}{12} = 558.69 \times 10^6 mm^4$$

$$\therefore I_{min} = I_{ZZ} = 512.4 \times 10^6 mm^4$$

$$r_{min} = \sqrt{\frac{I_{min}}{A}} = \sqrt{\frac{512.4 \times 10^6}{20586}} = 157.76 \text{ mm}$$



Effective slenderness ratio

$$S.R(\lambda) = \frac{KL}{r_{min}} = \frac{5500}{157.76} = 34.86$$

Buckling curve classification according to Table 10 – P- 44 is class 'c'

Ref page 42, Table9 (c) for $f_y = 250 \text{ N/mm}^2$

λ	f_{cd}
30	211
34.86	?
40	198
10	13
4.86	?

$$f_{cd} \text{ for } 34.86 = 211 - \frac{4.86 \times 13}{10} = 204.68 \text{ N/mm}^2$$

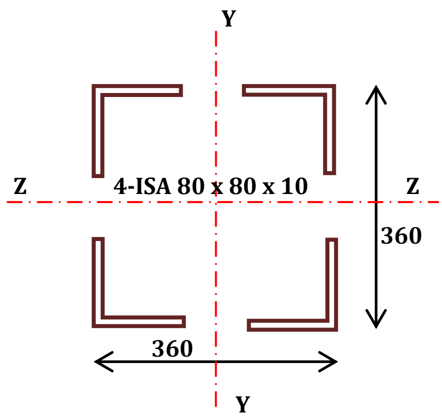
Flexural Strength of member = Safe stress x Area provided

$$= \frac{204.68 \times 20586}{1000} = 4213.58 \text{ kN.}$$

$$\text{Allowable load} = \frac{4213.58}{1.5} = 2809.06 \text{ kN.}$$

Problem:

A column height 5m is hinged at the ends. It is square in cross section (plan) of side 360 mm and consists of 4 angles of ISA 80 x 80 x 10 mm at each corner suitably laced. Find the minimum load on the column.



Solution:

Properties of 1- ISA 80 x 80 x 10 mm

$$a = 15.05 cm^2 = 1505 mm^2, C_{ZZ} = 2.34 cm = 23.4 mm,$$

$$I_{ZZ} = 87.7 cm^4 = 87.7 \times 10^4 \text{ mm}^4 \text{ , } L = 5m$$

End condition: Both ends hinged

$$l_{eff} = KL = L = 5m = 5000 \text{ mm}$$

I_{ZZ}= I_{YY} of the built up section:

$$I_{ZZ} = I_{YY} = 4\left[87.7 \times 10^4 + 1505 \times \left(\frac{360}{2} - 23.4\right)^2\right] = 151.14 \times 10^6 mm^4$$

$$r_{min} = \sqrt{\frac{I_{min}}{A}} = \sqrt{\frac{151.14 \times 10^6}{4 \times 1505}} = 158.45 mm$$

Effective slenderness ratio

$$\lambda = \frac{KL}{r_{min}} = \frac{5000}{158.45} = 31.56$$

Buckling curve classification according to Table 10 – P- 44 is class 'c'

Ref page 42 Table 9 (c) for $f_y = 250 N/mm^2$

$$f_{cd} \text{ for } 31.56 = 211 - \frac{1.56 \times 13}{10} = 208.97 \ N/mm^2$$

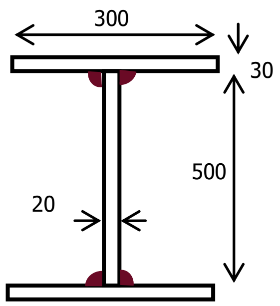
Load carrying capacity = Safe stress x Area provided

$$= \frac{208.97 \times 4 \times 1505}{1000} = 1258 \text{ kN.}$$

λ	σ_{ac}
30	211
31.56	?
40	198
10	13
1.56	?

Prob: P-334, Ex : 7.7, LSD of Steel Structure . S.K. Duggal

For a column section built up of shape as shown in fig, determine the axial load capacity of compression for the data indicated against the fig. f_y = 250 MPa, L = 6 m, t_w = 20 mm, t_f = 30 mm. End condition:- Both ends restrained in direction & position ,



Solution:

A = 2 x 300 x 30 + 500 x 20 = 28000 mm^2.

<u>**I_{ZZ} of the built up section:**</u>

$$I_{ZZ} = 2\left[\frac{300 \times 30^3}{12} + 300 \times 30 \times \left(\frac{500}{2} + \frac{30}{2}\right)^2\right] + \left[\frac{20 \times 500^3}{12}\right]$$

$$= 1473.73 \times 10^6 mm^4$$

<u>**I_{yy} of the built up section:**</u>

$$I_{yy} = 2 \times \frac{30 \times 300^3}{12} + \frac{500 \times 20^3}{12} = 135.33 \times 10^6 \ mm^4$$

$$r_{zz} = \sqrt{\frac{I_{zz}}{A}} = \sqrt{\frac{1473.73 \times 10^6}{28000}} = 229.41 \text{ mm}$$

$$r_{yy} = \sqrt{\frac{I_{yy}}{A}} = \sqrt{\frac{135.33 \times 10^6}{28000}} = 69.52 \text{ mm}$$

Determination of Buckling curve classification according to Table 10 – P-44: $t_f = 30 \text{ mm} < 40 \text{ mm}$

We should use buckling class 'b' about Z-Z axis and 'c' about y-y axis.

Effective slenderness ratio

$$S.R(\lambda) = \frac{KL}{r}$$

End Condition: Both ends restrained in direction & position

L = 6 m, KL = 0.65 x 6000 = 3900 mm

$$S.R(\lambda_{zz}) = \frac{KL}{r_{zz}} = \frac{3900}{229.41} = 17 \qquad S.R(\lambda_{yy}) = \frac{KL}{r_{yy}} = \frac{3900}{69.52} = 56.09$$

Compressive Stress (f_{cd}): About Z-Z axis : use buckling class 'b'

Ref page 41, Table9 (b) for $f_y = 250\ N/mm^2$

λ	f_{cd}
10	227
17	?
20	225
10	02
07	?

$$f_{cd} \text{ for } 17 = 227 - \frac{7 \times 02}{10} = 225.6\ N/mm^2$$

About Y – Y axis : use buckling class 'c'

Ref page 42, Table - 9 (c) for $f_y = 250\ N/mm^2$

λ	f_{cd}
50	183
56.09	?
60	168
10	15
6.09	?

$$f_{cd} \text{ for } 56.09 = 183 - \frac{6.09 \times 15}{10} = 173.87\ N/mm^2$$

Design f_{cd} = 173.87 N/mm^2

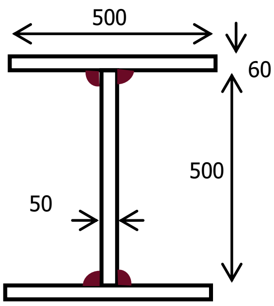
Load carrying capacity of member = Safe stress x Area provided

$$= \frac{173.87 \times 28000}{1000} = 4868.36 \text{ kN.}$$

$$\text{Allowable load} = \frac{4868.36}{1.5} = 3245.57 \text{ kN.}$$

P- 759, Design of steel structures by N. Subramaniam:

A heavy column is required to support a gantry Girder and a special H – Section is to be fabricated. The trial section is shown in fig. check its suitability to support a fabricated load of 11,000 KN, assuming both ends are pinned and a length of 8m. Steel of design strength 250 N/mm^2 is to be used.



Solution:

A = 2 x 500 x 60 + 500 x 50 = 85000 mm^2.

I_{zz} of the built up section:

$$I_{zz} = 2\left[\frac{500 \times 60^3}{12} + 500 \times 60 \times \left(\frac{500}{2} + \frac{60}{2}\right)^2\right] + \left[\frac{50 \times 500^3}{12}\right] = 5.243 \times 10^9 mm^4$$

I_{yy} of the built up section:

$$I_{yy} = 2 \times \frac{60 \times 500^3}{12} + \frac{500 \times 50^3}{12} = 1.255 \times 10^9 mm^4$$

$$r_{zz} = \sqrt{\frac{I_{zz}}{A}} = \sqrt{\frac{5.243 \times 10^9}{85000}} = 248.36\text{ mm} \qquad r_{yy} = \sqrt{\frac{I_{yy}}{A}} = \sqrt{\frac{1.255 \times 10^9}{85000}} = 121.51\text{ mm}$$

Determination of Buckling curve classification according to Table 10 – P- 44:

$$t_f = 60\text{ mm} > 40\text{ mm}$$

We should use buckling class 'c' about Z-Z axis and 'd' about y-y axis.

Effective slenderness ratio

$$S.R(\lambda) = \frac{KL}{r}$$

End Condition: Both ends pinned.

L = 8 m, KL = L = 8000 mm

$$S.R(\lambda_{zz}) = \frac{KL}{r_{zz}} = \frac{8000}{248.36} = 32.21 \qquad S.R(\lambda_{yy}) = \frac{KL}{r_{yy}} = \frac{8000}{121.51} = 65.84$$

Compressive Stress (f_{cd}):

About Z-Z axis : use buckling class 'c'

Ref page 42, Table9 (c) for $f_y = 250\text{N/mm}^2$

λ	f_{cd}
30	211
32.21	?
40	198
10	13
2.21	?

$$f_{cd} \text{ for } 32.21 = 211 - \frac{13 \times 2.21}{10} = 208.13\text{ N/mm}^2$$

About Y – Y axis : use buckling class 'd'

Ref page 43, Table - 9 (d) for $f_y = 250\text{ N/mm}^2$

λ	f_{cd}
60	168
65.84	?
70	152
10	16
5.84	?

$$f_{cd} \text{ for } 65.84 = 168 - \frac{5.84 \times 16}{10} = 158.65\text{ N/mm}^2$$

Design f_{cd} = 158.65 N/mm^2

Load carrying capacity of member = Safe stress x Area provided

$$= \frac{158.65 \times 85000}{1000} = 13487.8\text{ kN.}$$

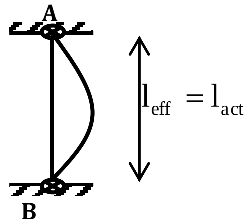
$$\text{Allowable load} = \frac{13487.8}{1.5} = 8992\text{ kN.}$$

Design Problems:

Problem:

Design a rolled steel beam section column to carry an axial load of 1100 KN. The column is 4 m long and adequately restrained in position, but not in direction at both ends.

Solution:



Axial load =1100 KN

Factored load = 1.5 x 1100 = 1650 KN

$l_{act} = 4m.$

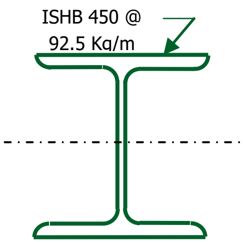
End condition: Adequately restrained in position but not in direction at both ends.

Ref page 45 Table 11 $l_{eff} = l_{act} = 4m$

Assuming permissible stress $(f_{cd}) = 0.6\ f_y = 0.6 \times 250 = 150\ N/mm^2\ (0.4\ f_y \text{ to } 0.6\ f_y)$

Area required = $\dfrac{1650 \times 10^3}{150} = 11000\ mm^2 = 110\ cm^2$

Try ISHB 450 @ 92.5 kg/m.



$a = 117.89 cm^2 = 11789 mm^2, h = 450 mm, b_f = 250 mm, t_f = 13.7 mm$

$r_{xx} = 18.5 cm = 185 mm, r_{yy} = 5.08 cm = 50.8 mm.$

$r_{min} = r_{yy} = 50.8 mm$

Determination of buckling curve classification

$\dfrac{h}{b_f} = \dfrac{450}{250} = 1.8 > 1.2$

$t_f = 13.7 \le 40\ mm$

We should use buckling class 'a' about Z-Z axis and 'b' about y-y axis, Referring to Table 10, P- 44, IS 800 – 2007.

P- 34, Cl 7.1.2.1

Compressive Stress (f_{cd}):

About Z-Z axis :use buckling class 'b'

$\lambda_{zz} = \dfrac{KL}{r_{zz}} = \dfrac{4000}{185} = 21.62$

λ_{zz}	f_{cd}
20	225
21.62	?
30	216
10	09
1.62	?

REF TO TABLE 9(a) P-40

f_{cd} for $21.62 = 225 - \dfrac{1.62 \times 09}{10} = 225.03\ N/mm^2$

Compressive Stress (f_{cd}):

About Y-Y axis :use buckling class 'c'

$\lambda_{yy} = \dfrac{KL}{r_{yy}} = \dfrac{4000}{50.8} = 78.74$

λ_{yy}	f_{cd}
70	166
78.74	?
80	150
10	16
8.74	?

REF TO TABLE 9(b) P-41

f_{cd} for $78.74 = 166 - \dfrac{8.74 \times 16}{10} = 152.02 N/mm^2$

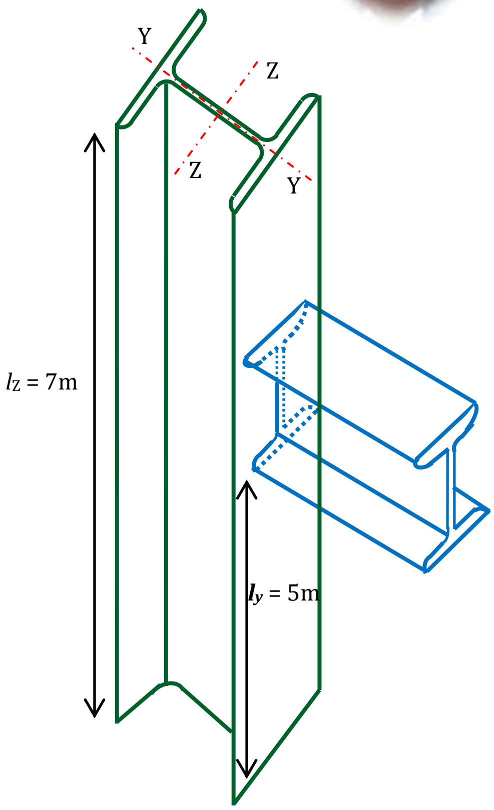
f_{cd} is the min of 222.15 N/mm^2, and 174.83 N/mm^2.

i.e., f_{cd} = 174.83 N/mm^2.

Load carrying capacity = Safe stress x Area provided

$$= \frac{152.02 \times 11789}{1000} = 1792.16KN > 1650KN$$

Safe **Provide ISHB 450 @ 92.5 kg / m.**



P- 332, LSD by steel structures, S.K.Duggal

Design a column to support a factored load of 1050 KN. The column has an effective length of 7 m with respect to Z- axis and 5 m with respect to Y- axis. Use steel of grade Fe 410

Solution:

Factored load = 1050 KN

$l_{ZZ} = 7m, \quad l_{YY} = 5m,$

Assuming permissible stress $(f_{cd}) = 0.6\ f_y \quad (0.4\ f_y \text{ to } 0.6\ f_y)$

$= 0.6 \times 250 = 150\ N/mm^2$

Area required = $\frac{1050 \times 10^3}{150} = 7000mm^2 = 70cm^2$

Try ISHB 350 @ 67.4 kg/m.

$a = 85.91cm^2 = 8591mm^2, h = 350mm, b_f = 250mm, t_f = 11.6mm$

$r_{xx} = 14.95cm = 149.5mm, r_{yy} = 5.34cm = 53.4mm.$

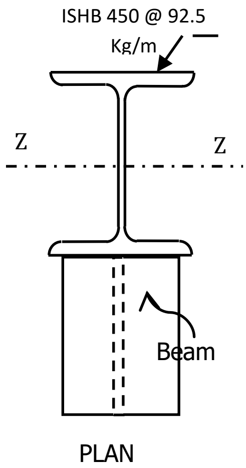
$r_{min} = r_{yy} = 53.4mm$

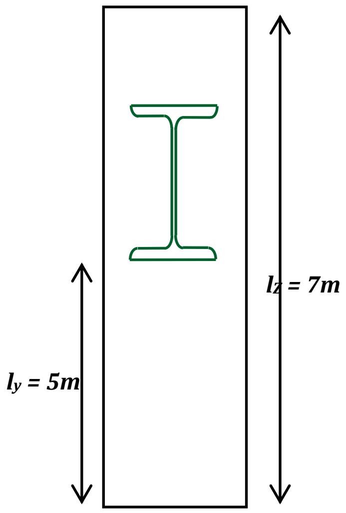
Determination of buckling curve classification

$\frac{h}{b_f} = \frac{350}{250} = 1.4 > 1.2 \qquad t_f = 11.6 \ \leq 100$ mm

We should use buckling class 'a' about Z-Z axis and 'b' about y-y axis, Referring to

Table 10, P- 44, IS 800 – 2007.





P- 34, Cl 7.1.2.1

Compressive Stress (f_{cd}):

About Z-Z axis :use buckling class 'a'

$$\lambda_{ZZ} = \frac{KL}{r_{zz}} = \frac{7000}{149.5} = 46.82$$

REF TO TABLE 9(a) P-40

$$f_{cd} \text{ for } 46.82 = 213 - \frac{6.82 \times 08}{10} = 207.5\ N/mm^2$$

λ_{zz}	f_{cd}
40	213
46.82	?
50	205
10	08
6.82	?

Compressive Stress (f_{cd}):

About Y-Y axis :use buckling class 'b'

λ_{yy}	f_{cd}
90	134
93.63	?
100	118
10	16
3.63	?

$$\lambda_{yy} = \frac{KL}{r_{yy}} = \frac{5000}{53.4} = 93.63$$

REF TO TABLE 9(b) P-41

$$f_{cd} \text{ for } 93.63 = 134 - \frac{16 \times 3.63}{10} = 144.192 \text{ N/mm}^2$$

f_{cd} is the min of 207.5 N/mm^2, and 144.19 N/mm^2.

i.e., f_{cd} = 144.19 N/mm^2.

Load carrying capacity = Safe stress x Area provided

$$= \frac{144.19 \times 8591}{1000} = 1238.75 \text{ kN} > 1050 \text{ kN}$$

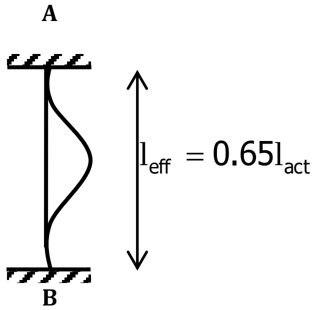
Safe **Provide ISHB 350 @ 67.4 kg / m**

P- 332, LSD by steel structures, S.K.Duggal

Design a column to support a factored load of 800 KN. The column has an effective length of 7 m with respect to Y - axis and 5 m with respect to Z - axis. Use steel of grade Fe 410

Problem:

Design a rolled steel beam section column to carry an axial load of 2500 KN. The column is 5 m long effectively held in position and restrained against rotation at both ends.



Solution:

Axial load = 2500 KN

Factored Load = 1.5 x 2500 = 3750 kN

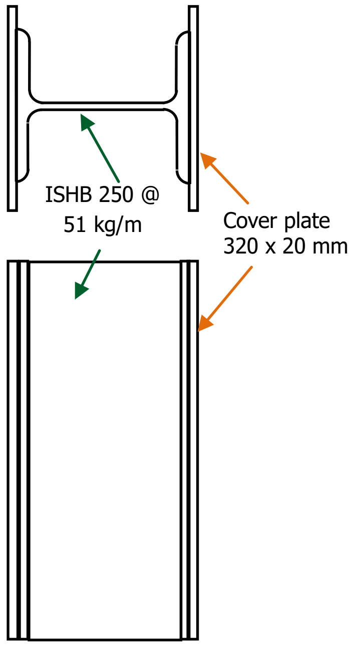
$l_{act} = 5$m.

End condition: Effectively held in position and restrained against rotation at both ends.

$l_{eff} = 0.65 l_{act} = 0.65 \times 5 = 3.25$m (P 45, T - 11)

Assuming permissible stress $(f_{cd}) = 0.8\, f_y = 0.8 \times 250 = 200 \text{ N / mm}^2$

$$\text{Area required} = \frac{3750 \times 10^3}{200} = 18750 \text{ mm}^2 = 187.50 \text{ cm}^2$$



Try ISHB 250 @ 51 kg/m. with additional plates on both flanges of size 320 x 20 mm.

Area (a) = 192.96 cm^2, r_{min} = 8.17 cm = 81.7 mm

$$\text{Slenderness ratio } \lambda_{zz} = \frac{KL}{r_{zz}} = \frac{3250}{81.7} = 39.78 \approx 40$$

Ref page 42 Table 9(c) for $f_y = 250\text{N/mm}^2$

For buckling class 'c' for built up section

f_{cd} for $40 = 198$ N/mm^2

Load carrying capacity = stress x area provided

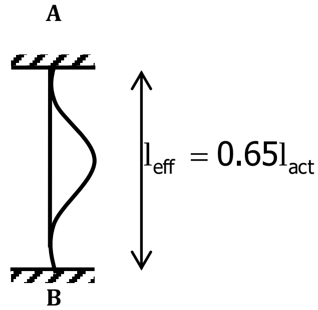
$$= \frac{198 \times 19296}{1000} = 3820.61 \text{ kN} > 3750 \text{ kN}$$

Safe

Provide ISHB 250 @ 51 kg/m. with additional plates on both flanges of size 320 x 25 mm.

Problem:

A column 5 m long is to support a load of 4500 KN. The ends of the columns are effectively held in position and directions. Design the column if rolled steel beams and 18 mm plates are only available.



Solution:

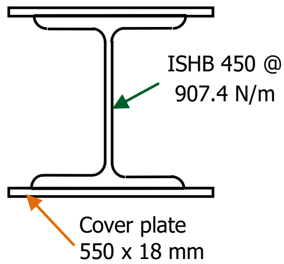
Axial load = 4500 KN, $l_{act} = 5\text{m}$

End condition: Effectively held in position and direction.

Ref page 41 Table 5.2

$$l_{eff} = 0.65 l_{act} = 0.65 \times 5 = 3.25 \text{ m}$$

Assuming, stress $\sigma_{ac} = 150 \text{ N/mm}^2$



$$\text{Area required} = \frac{\text{Load}}{\sigma_{ac}} = \frac{4500 \times 10^3}{150} = 30000 \text{ mm}^2 = 300 \text{ cm}^2$$

Ref steel table, Try ISHB 450 @ 907.4 N/m

$a = 117.89\text{cm}^2 = 11789\text{mm}^2, I_{xx} = 40349.9\text{cm}^4 = 40349.9 \times 10^4 \text{mm}^4$

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

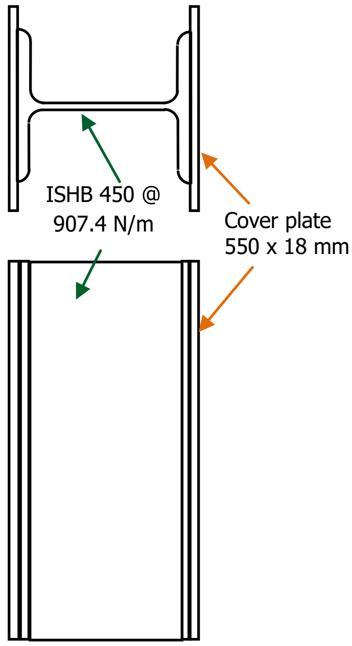
Area required for the two cover plates = $300 - 117.89 = 182.11 \text{ cm}^2$.

Area of one cover plate = $\frac{182.11}{2} = 91.06 \text{ cm}^2 = 9106 \text{ mm}^2$.

Given Th. of cover plate = 18 mm.

Width of cover plate = $\frac{9106}{18} = 505.9 \text{ mm}$, say 550mm

Try a cover plate of size = 550 x 18 mm.



Check for local buckling

$$\frac{\text{Outstanding width}}{\text{thickness}} \not> 16$$

$$\frac{\frac{550-140}{2}}{18} = 11.38 < 16 \qquad \text{Satisfactory}$$

$$\text{Area}(A) = 11789 + 2(550 \times 18) = 31589 \text{ mm}^2$$

Ixx of the built up section

$$I_{xx} = 40349.9 \times 10^4 + 2\left[\frac{550 \times 18^3}{12} + 550 \times 18\left(\frac{450}{2} + \frac{18}{2}\right)^2\right] = 1.49 \times 10^9 \text{mm}^4$$

Iyy of the built up section

$$I_{yy} = 3045 \times 10^4 + 2\left[\frac{18 \times 550^3}{12}\right] = 529 \times 10^6 \text{ mm}^4$$

$$\therefore I_{min} = I_{yy} = 529 \times 10^6 \text{mm}^4$$

$$r_{min} = \sqrt{\frac{I_{min}}{A}} = \sqrt{\frac{529 \times 10^6}{31589}} = 129.4 \text{ mm}$$

$$S.R(\lambda) = \frac{3250}{129.4} = 25.11$$

Ref page 42, Table 9(c) ,for $f_y = 250 \text{ N/mm}^2$

$$f_{cd} \text{ for } 25.11 = 224 - \frac{13 \times 5.11}{10} = 217.36 \text{ N / mm}^2$$

λ	f_{cd}
20	224
25.11	?
30	211
10	13
5.11	?

Load carrying capacity = Safe stress x area provided

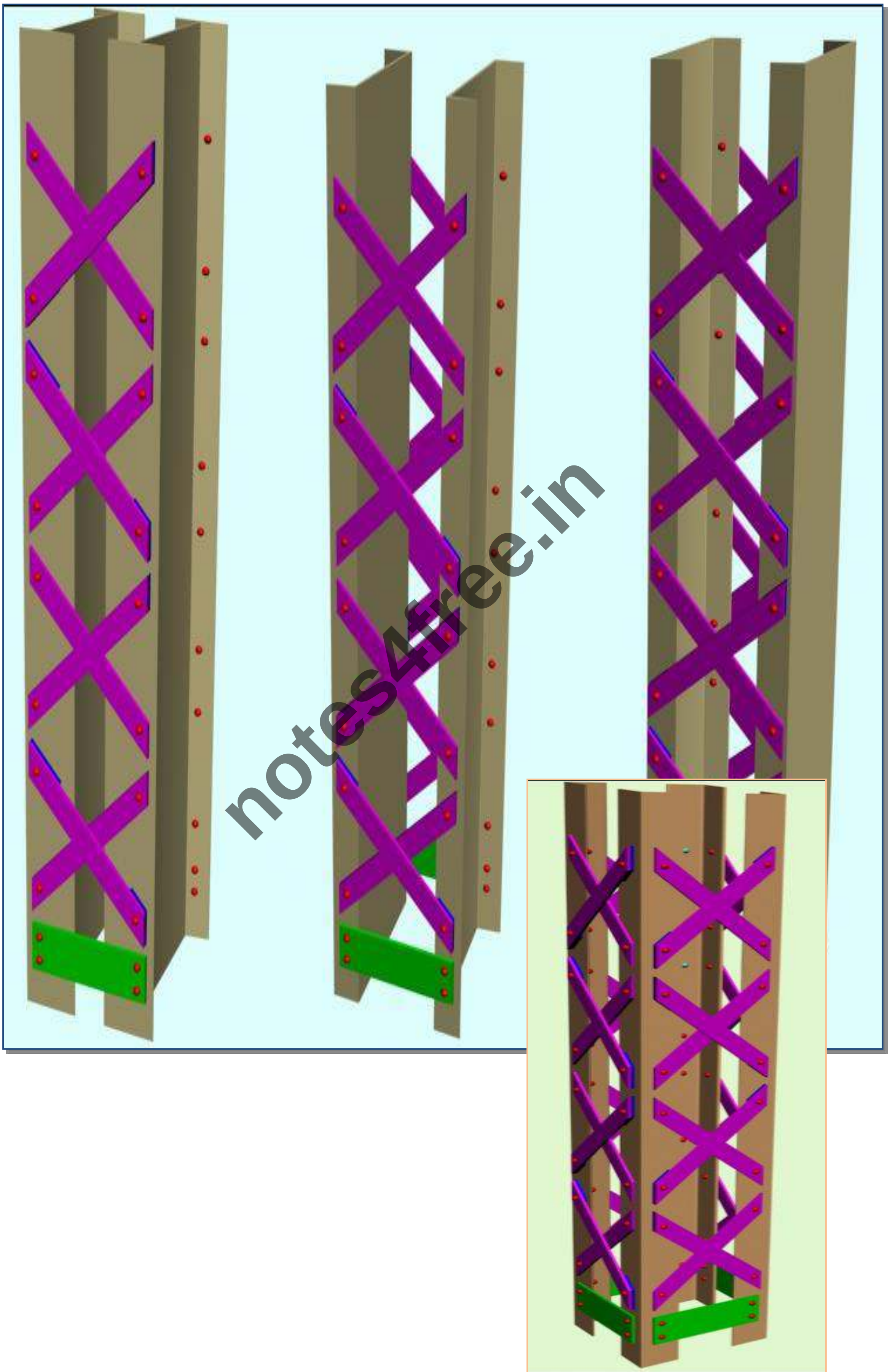
=

$$\frac{217.36 \times 31589}{1000} = 6866 \text{ kN} > 6750 \text{ kN}$$

SAFE

<u>**Provide ISHB 450 @ 907.4N/m with additional cover plates of size 550 x 18mm one on each side.**</u>

Lacing system



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LACING FOR BUILT-UP COMPRESSION MEMBERS (P - 48, 49, 50 Cl : 7.6)

The different components of the built-up section should be placed uniformly at a maximum possible distance from the axis of the column for greater strength of the column. The different components of the built up section are connected together so that they act as single column. Lacing is generally preferred in case of eccentric loads. Battening is normally used for axially loaded columns and where the components are not for apart. Flat bars are generally used for lacing. Angles, channels, and tubular sections are also used for the lacing of very heavy columns. Plates are used for battens.

Design procedure

<u>Cl 7.6.4 page 50</u>

1. The angle of inclination of the lacing with the longitudinal axis of the column should be between $40°$ to $70°$.

Cl 7.6.6.3 page 50

2. The slenderness ratio $\frac{l_{eff}}{r_{min}}$ of the lacing bars should not exceed 145. The effective length 'l_e' of the lacing bars should be taken as follows.

Type of lacing	Effective length (le)
Single lacing, bolted at ends lacing bar	Length b/w inner ends of bolts on
Double lacing, bolted at ends at Intersection	0.7 times b/w inner ends of bolts on lacing bars (0.7x L)
Welded lacing	0.7 times distance b/w inner ends of effective lengths of welds at ends (0.7 x L)

If flat bars of width 'b' and thickness 't' are used for lacing, the maximum slenderness

Ratio is given by

$$\text{Max. S.R}(\lambda) = \frac{l_e}{r_{min}} = \frac{l_e}{\sqrt{\frac{I}{A}}} = \frac{l_e}{\sqrt{\frac{bt^3}{12} \times \frac{1}{bt}}} = \frac{l_e\sqrt{12}}{t} \not> 145$$

Cl 5.7.6.1 page 51

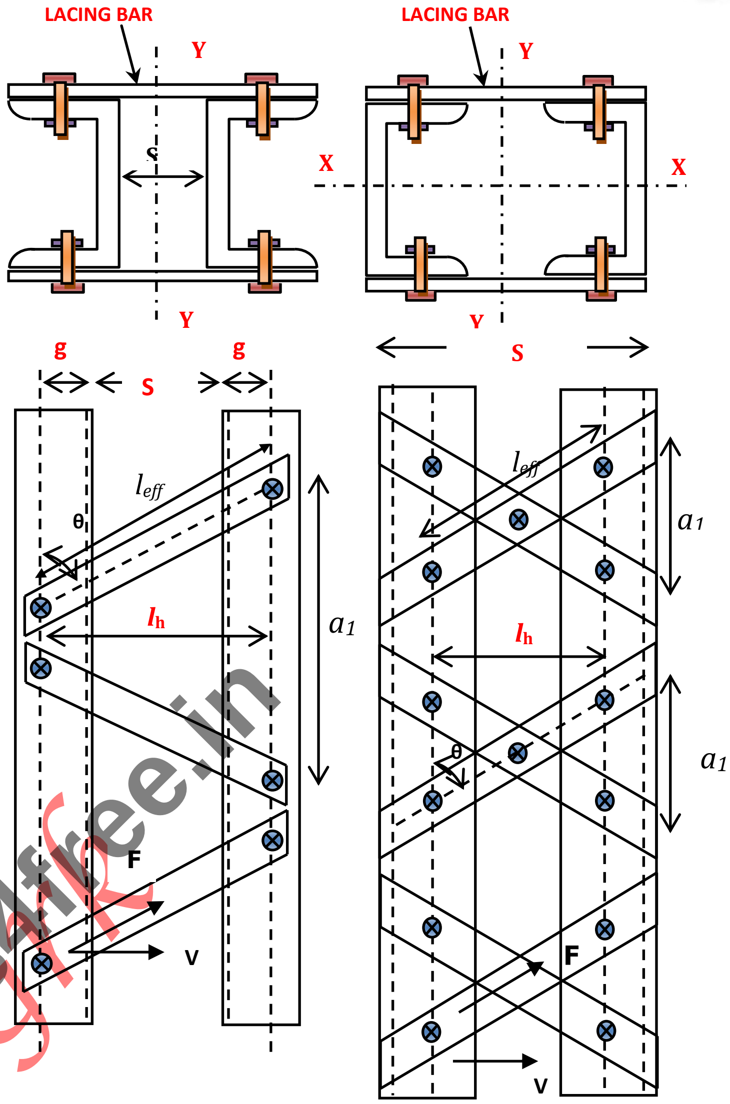
1. For bolted or welded lacing system- ----spacing

$\frac{a_1}{r_1} \not> 50$ or 0.7 times max SR of the compression member as a whole, whichever is less.

Where,

a_1 = Distance b/w the centers of connections of the lattice bars to each components as shown in fig.

r_1= Min radius of gyration of the components of compression members.



Cl 7.6.2 page 50

Width of lacing bars: In bolted/riveted construction, the minimum width of lacing bars shall be three times the nominal diameter of the end bolt.

Cl 7.6.3 page 50

Thickness of lacing bars

$t \nless \frac{l_{eff}}{40}$ for Single lacing

$t \nless \frac{l_{eff}}{60}$ for Double lacing bolted or welded at intersection.

Where,

l = length b/w inner end bolts.

Design of Lacings: Cl : 7.6.6

The lacing of compression members should be designed to resist to transverse shear V = 2.5% of the axial force in member.

This shear is divided equally among all transverse lacing system in parallel planes. The lacing system should be designed to resist additional shear due to bending if the compression member carries bending due to eccentric load, applied end moments, and lateral loading.

For single lacing system on two parallel faces, the force (compressive or tensile) in each bar.

$$\boxed{F = \frac{V}{2\sin\theta}}$$

For double lacing system on two parallel planes, the force (compressive or tensile) in each bar.

$$\boxed{F = \frac{V}{4\sin\theta}}\ \text{vt}$$

If the flat lacing bars of width 'b' and thickness 't' have bolts of diameter 'd' then

$$\text{Compressive stress in bar } \frac{\text{Force}}{\text{Gross area}} = \frac{F}{b \times t} \not> \sigma_{ac}$$

$$\text{Tensile stress in each bar} = \frac{\text{Force}}{\text{Net area}} = \frac{F}{(b - d_o)t} \not> \sigma_{at}$$

$$\sigma_{at} = 0.6 f_y$$

$$\text{Compressive Force} = \text{Compressive Stress} \times \text{Area of lacing bar} \not< F$$

Tensile Force (P - 32)

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}} \not< F$$

OR

$$T_{dn} = \frac{0.9(b - d_o)t \times f_u}{\gamma_{m1}}$$

Connection Details:

$$\text{No. of bolts} = \frac{F}{BV}$$

P-75, Cl: 10.3.3

1) Strength of bolt in Single shear:

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

2) Strength of bolt in Bearing $V_{dpb} = \dfrac{2.5 \times k_b \times d \times t^* \times f_u}{\gamma_{mb}}$

k_b is the least of the following:

1) $\dfrac{e}{3d_0}$ Edge distance e = 1.5 x d_o

2) $\dfrac{p}{3d_0} - 0.25$

P = 2.5 x d

3) $\dfrac{f_{ub}}{f_u}$ 4) 1

t* → Min of 1) Thickness of flange of column section and
2) Thickness of lacing bar

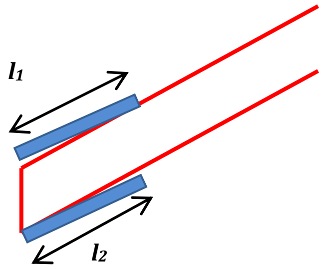
Bolt value (BV) = Min of above two values.

Welded connection;

Lap joint: Overlap $\nless$ 4 times thickness of bar or member, whichever is less.

Butt joint: Full penetration butt weld or fillet weld on each side. Lacing bar should be placed opposite to flange or stiffening member of main member.

Welded connection:



Max Size of weld S = thickness of flat - 1.5

Force in lacing bar = Strength of the weld

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

$$\text{Strength of weld} = 0.707 \times D \times l \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}}$$

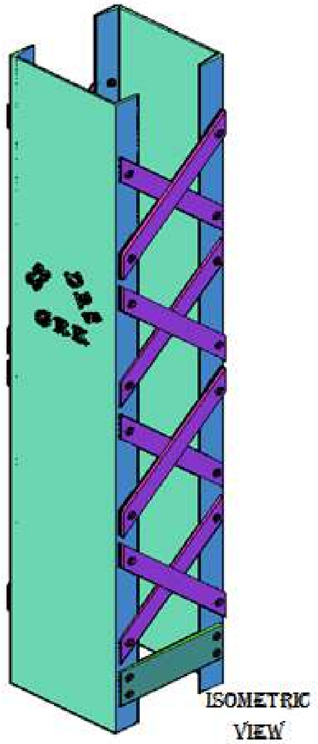
$$\text{Effective length of weld} = \frac{F}{0.707 \times D \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}}}$$

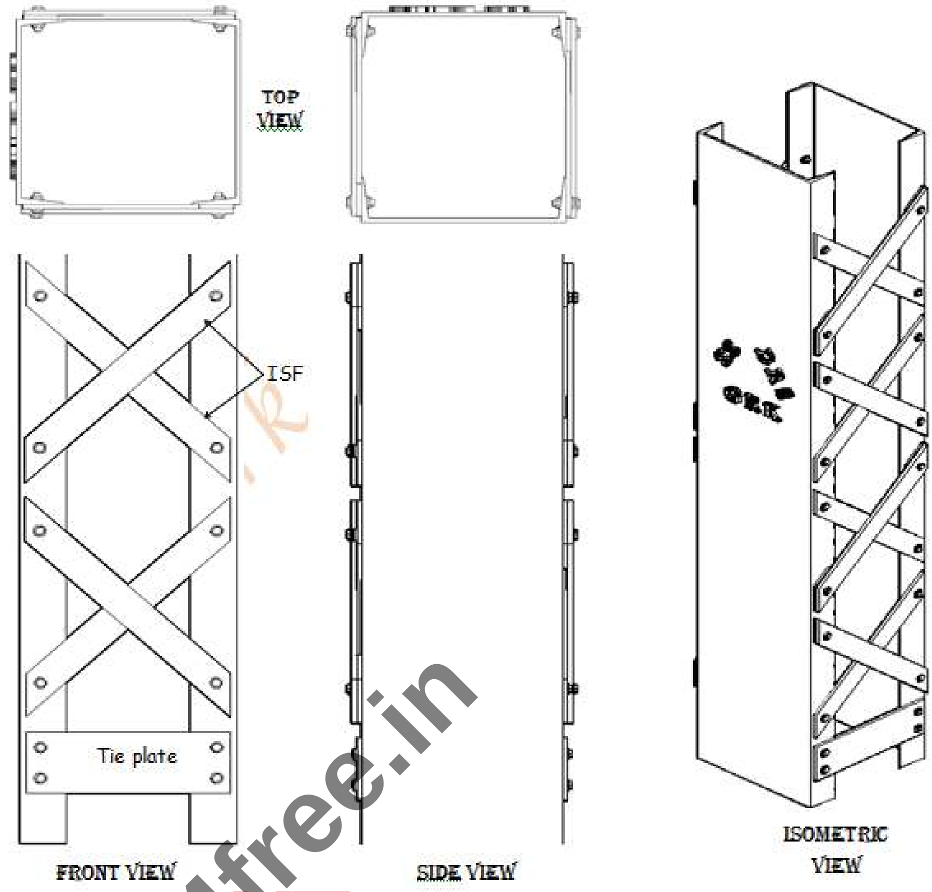
Provide Length of weld on each side of flat

JAN / FEB 2004 – 8 marks

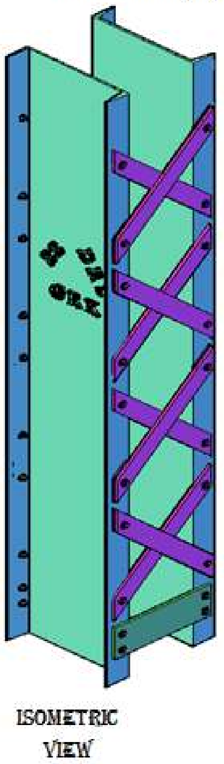
Why are lacing / battens provided in steel columns consisting of more than one section ? Explain with neat sketches the details of different types of lacing and battening system.

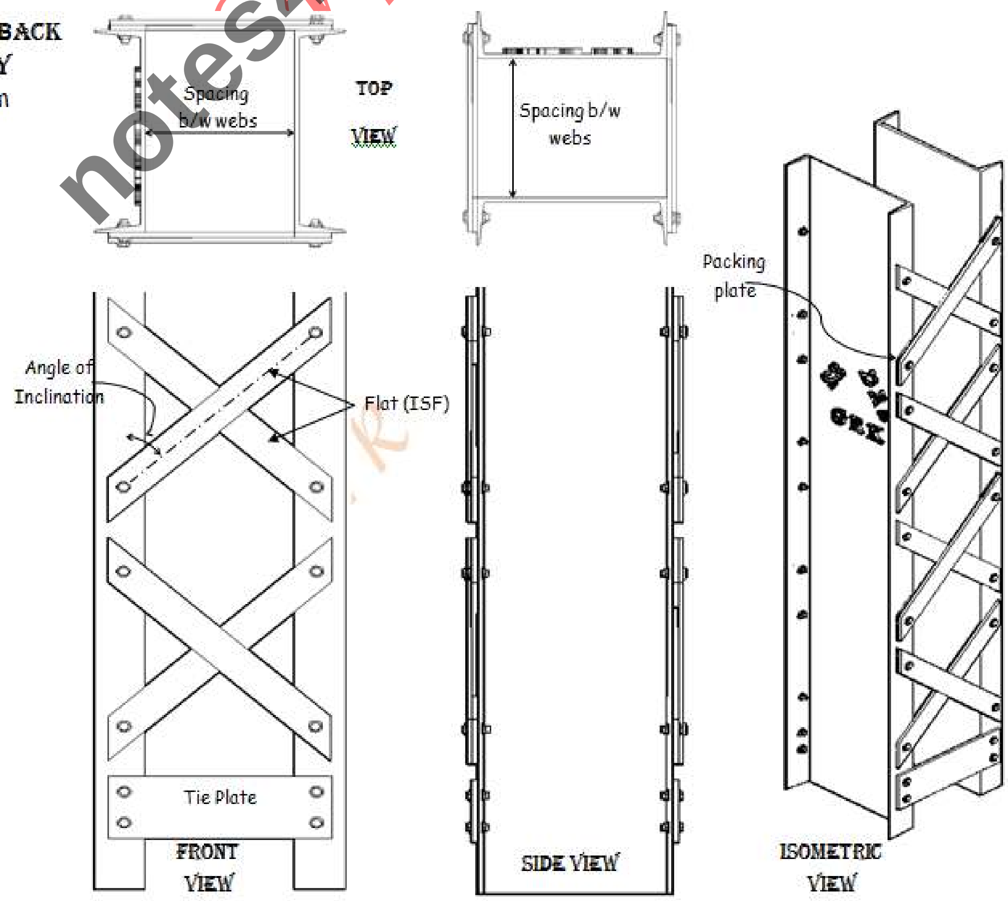
2 = CHANNELS TOE TO TOE
LACED TOGETHER BY
Double lacing system





2 = CHANNELS BACK TO BACK
LACED TOGETHER BY
Double lacing system





Problem:

Design a built- up column with 2 channel sections back to back to carry an axial factored load of 1300 KN. The height of the column is 7 m and effectively held in position at both ends, but not restrained against rotation. Take $f_y = 250\,\text{MPa}$. Design single lacing system with 16 mm dia bolt of 4.6 grade.

Solution:

Design of compression member

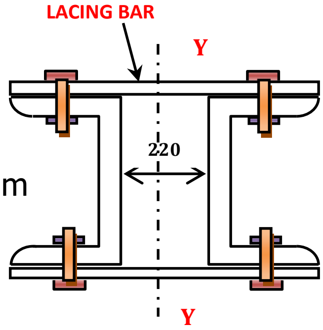
Factored Load= 1300 KN

Assuming permissible stress $= 0.5\,f_y = 0.5 \times 250 = 125\ \text{N/mm}^2 \quad (0.4\,f_y \text{ to } 0.6\,f_y)$

$$\text{Area of 2 channels} = \frac{\text{Load}}{f_{cd}} = \frac{1300 \times 10^3}{125} = 10400\ \text{mm}^2 = 104\ \text{cm}^2$$

Try 2 - ISMC 350 @ 84.2 Kg/m with spacing b/w webs S = 220 mm

[P - 48, Cl: 7.6.1.1
Note :- The spacing is choosen in such a way that
$I_{ZZ} \approx I_{yy}$ & $r_{yy} > r_{ZZ}$]



$a = 107.32\ \text{cm}^2 = 10732\ \text{mm}^2$

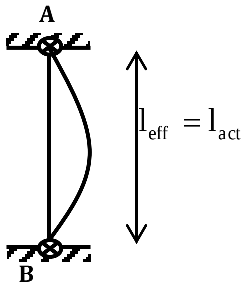
$r_{ZZ} = 13.66\ \text{cm} = 136.6\ \text{mm}, \quad r_{yy} = 13.74\ \text{cm} = 137.4\ \text{mm}$

P-48, Cl: 7.6.1.5

Slenderness ratio of builtup section $(\lambda) = 1.05 \times \dfrac{KL}{r}$

Effective length (Table11, Cl : 7.2.2, P - 45)

End condition : Effectively held in postion at both ends,
but not restained against rotation. (Both ends Hinged)



$KL = L = 7\ \text{m} = 7000\ \text{mm}$

P-44, Table 10, Buckling curve class about any axis 'c'.
P-42, Table 9(c) for f_y = 250 N/mm^2.

Compressive stress about ZZ- axis, ($f_{cd\text{-}zz}$)

$$\lambda_{ZZ} = 1.05 \times \frac{7000}{136.6} = 53.81$$

Compressive stress f_{cd} = 177.30 N/mm^2.

$\lambda\lambda$	f_{cd}
50	183
53.81	?
60	168
10	15
3.81	?

$$f_{cd} = 183 - \frac{3.81 \times 15}{10} = 177.30\ \text{N/mm}^2$$

Load carrying capacity $= f_{cd} \times \text{Area}$

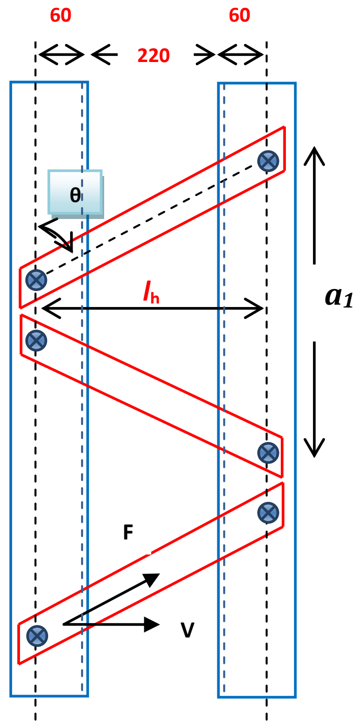
$$= \frac{177.30 \times 10732}{1000} = 1902.80\ \text{KN} > 1300\ \text{KN}$$

Safe

Provide 2 - ISMC 350 @ 84.2 Kg / m.

Design Of Lacing: (single lacing system)

Check for local buckling of column section (P -50, cl 7.6.5)



$\frac{a_1}{r_1} \not> 50$ or 0.7λ.7 builtup section, whichever is less.

0.7 times min of λ_{ZZ} and λ_{YY}

Inclination Of Lacing : (P - 50, Cl 7.6.4)

Assuming Inclination Of Lacing = 45° $(40^0 \prec \theta \prec 70^0)$

The gauge distance 'g' for ISMC 350 is 60 mm.

$r_1 = 2.83$cm

$\therefore$ Horizontal length of lacing $l_h = 60 + 220 + 60 = 340$ mm

Spacing of lacing is c/c distance of adjacent bolts

$= a_1 = 2 \times 340 = 680$ mm

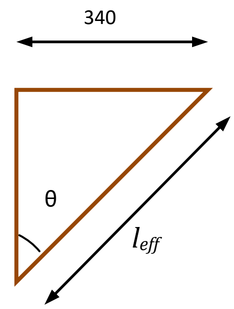
$\frac{a_1}{r_1} = \frac{680}{28.3} = 24.03 < 50$ and $< 0.7 \times 53.81 \Rightarrow 37.45$

The local buckling of the column does not occur,

Hence single lacing system can be adopted.

Dimension of lacing

Width of lacing bar (P- 50, Cl 7.6.2)



Assuming dia of bolt = 16 mm

Width of lacing = 3 x dia of bolt = 3 x 16 = 48 mm Say 50 mm

Thickness of bar (t) (P- 50, Cl 7.6.3)

$t \not< \frac{1}{40}$ of distance of inner bolts

$\sin\theta = \frac{340}{l_{eff}}$

$\frac{340}{l_{eff}} = \frac{340}{\sin 45} = 480.83$mm

$l_{eff} = 480.83$mm For single lacing system

$t = \frac{1}{40} \times l_{eff} = \frac{1}{40} \times 480.83 = 12.04$ mm Say 16 mm

Try a lacing bar of 50 mm width and 16 mm thick

i.e., 50 ISF 16

Note: (P- 50, Cl 7.6.3) Double lacing system $t = \frac{1}{60} \times l_{eff}$

$$r_{min} = \sqrt{\frac{I_{xx}}{A}} = \sqrt{\frac{\frac{l_{eff}\, t^3}{12}}{l_{eff}\, t}}$$

$$r_{min} = \frac{t}{\sqrt{12}}$$

Check for slenderness ratio: (P-50, Cl 7.6.6.3)

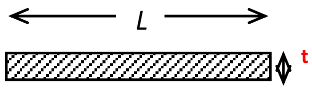
$\lambda = \frac{l_{eff}}{r_{min}} \not> 145$

$\lambda = \frac{l_{eff}}{r_{min}} = \frac{l_{eff} \times \sqrt{12}}{t} = \frac{480.83 \times \sqrt{12}}{16} = 104.10 < 145$ Safe

Note: - (P-50, Cl 7.6.3)

For double lacing system l_{eff} = 0.7 x Length of lacing bar b/w inner bolts.

$$\lambda = \frac{0.7 \times l_{eff}}{r_{min}} \not> 145$$



Check for Compressive Force and Tensile Force:

Force in lacing bar (F) : (P - 48, Cl 5.7.2.1)

Transverse Shear (V) = 2.5 % of axial load

$$= \frac{2.5}{100} \times 1300 = 32.5 \text{ KN}$$

$$\text{Force (F)} = \frac{V}{n \times \sin\theta}$$

n = 2 for single lacing system

n = 4 for double lacing system

$$F = \frac{32.5}{2 \times \sin 45} = 22.98 \text{ KN}$$

OR

$$\text{Force (F)} = \frac{V}{2n} \times \text{cosec}\theta$$

n = 1 for single lacing system

n = 2 for double lacing system

Ref Page 42 , Table 9(c)

λλ	f_{cd}
100	107
104.10	?
110	94.6
10	12.4
4.10	?

$$f_{cd} = 107 - \frac{4.1 \times 12.4}{10} = 101.92 \text{ N/mm}^2$$

Compressive Stress

for $\lambda = 104.10$

Compressive Force = Compressive Stress × Area of lacing bar $\not<$ F

$$\text{Compressive Force} = \frac{101.92 \times 50 \times 16}{1000} = 81.54 > 22.98 \text{KN} \quad \underline{\text{Safe}}$$

Tensile force (P-32):

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}} \not< F$$

$$T_{dn} = \frac{0.9(b - d_o)t \times f_u}{\gamma_{m1}} = \frac{0.9(50 - 18) \times 16 \times 410}{1.25 \times 1000} = 151.14 \text{KN} > 22.98 \text{ KN} \quad \underline{\text{Safe}}$$

Provide 50 ISF 16 as lacing bar

Connection Details:

$$\text{No of bolts} = \frac{F}{BV}$$

Dia of bolt = 16 mm.

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

Strength of one bolt in Single shear: P-75, Cl: 10.3.3

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

Assuming thread is interfering the shear plane

$$n_n = 1 \qquad n_s = 0 \quad , \gamma_{mb} = 1.25 \quad , \quad A_{nb} = 0.78 \times \frac{\pi}{4} d^2 = 0.78 \times \frac{\pi}{4} \times 16^2 = 156.83 \text{mm}^2$$

$$V_{dsb} = \frac{400}{\sqrt{3}} \times \left(\frac{1 \times 156.83}{1.25 \times 1000}\right) = 28.97KN$$

2) Strength of one bolt in Bearing $V_{dpb} = \dfrac{2.5 \times k_b \times d \times t^* \times f_u}{\gamma_{mb}}$

k_b is the least of the following:

1) $\dfrac{e}{3d_0} = \dfrac{35}{3 \times 18} = 0.65$ Edge distance e = 1.5 x 18 = 27 mm say 35 mm

2) $\dfrac{p}{3d_0} - 0.25 = \dfrac{50}{3 \times 18} - 0.25 = 0.68$ P = 2.5 x 16 = 40 mm Say 50 mm

3) $\dfrac{f_{ub}}{f_u} = \dfrac{400}{410} = 0.98$ 4) 1

k_b = 0.65

t → Min of 1) Thickness of flange of channel (13.5) and*

2) thickness of lacing bar (16 mm)

$$V_{dpb} = \frac{2.5 \times 0.65 \times 16 \times 13.5^* \times 400}{1.25 \times 1000} = 112.32KN$$

Bolt value (BV) = 28.97 KN.

No of bolts = $\dfrac{22.98}{28.97} = 0.79$ $\boxed{\text{Say 2 No's (Min) One on each side}}$

Problem:

Design a built- up column with 2 channel sections back to back to carry an axial factored load of 1300 KN. The height of the column is 7 m and effectively held in position at both ends, but not restrained against rotation. Take $f_y = 250\,MPa$. Design single lacing system with field weld.

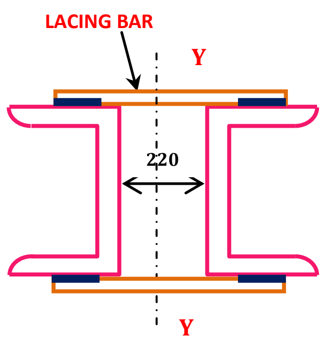
Solution:

Design of compression member

Factored Load= 1300 KN

Assuming permissible stress $= 0.5\,f_y = 0.5 \times 250 = 125\,N/mm^2$ $(0.4\,f_y \text{ to } 0.6\,f_y)$

Area of 2 channels $= \dfrac{Load}{\sigma_{ac}} = \dfrac{1300 \times 10^3}{125} = 10400\,mm^2 = 104\,cm^2$



Try 2 - ISMC 350 @ 84.2 Kg/m with spacing b/w webs S = 220 mm

$$\left[\begin{array}{l} \text{P - 48, Cl : 7.6.1.1} \\ \text{Note : - The spacing is choosen in such a way that} \\ \qquad I_{xx} \approx I_{yy} \quad \& \; r_{yy} > r_{xx} \end{array}\right]$$

$a = 107.32\,cm^2 = 10732\,mm^2$, $r_{ZZ} = 13.66\,cm = 136.6\,mm$,

$r_{yy} = 13.74\,cm = 137.4\,mm$

P-48, Cl: 7.6.1.5

Slenderness ratio of builtup section $(\lambda) = 1.05 \times \frac{KL}{r}$

Effective length (Table11, Cl : 7.2.2, P - 45)

End condition : Effectively held in postion at both ends,

but not restained against rotation. (Both ends Hinged)

$KL = L = 7 \text{ m} = 7000 \text{ mm}$

P-44, Table 10, Buckling curve class about any axis 'c'.

P-42, Table 9(c) for f_y = 250 N/mm^2.

λλ	f_{cd}
50	183
53.81	?
60	168
10	15
3.81	?

$$f_{cd} = 183 - \frac{3.81 \times 15}{10} = 177.30 \text{ N/mm}^2$$

Compressive stress about ZZ- axis, (f_{cd-zz})

$$\lambda_{ZZ} = 1.05 \times \frac{7000}{136.6} = 53.81$$

Compressive stress f_{cd} = 177.30 N/mm^2.

Load carrying capacity $= f_{cd} \times$ Area

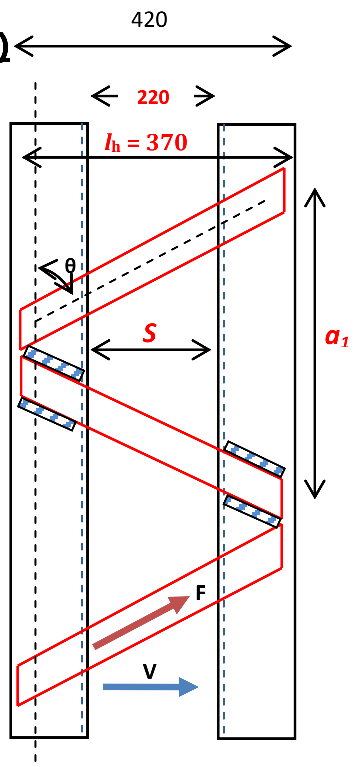
$$= \frac{177.30 \times 10732}{1000} = 1902.80 \text{ KN} > 1300 \text{ KN}$$

Safe

Provide 2 - ISMC 350 @ 84.2 Kg / m.

Design Of Lacing: (single lacing system)

Check for local buckling of column section (P -50, cl 7.6.5)



$$\frac{a_1}{r_1} \not> 50 \text{ or } 0.7\lambda.7 \text{ builtup section, whichever is less.}$$

0.7 times min of λ_{ZZ} and λ_{YY}

Inclination Of Lacing : (P - 50, Cl 7.6.4)

Assuming Inclination Of Lacing = 45° $(40^0 \prec \theta \prec 70^0)$

The min radius of gyration for ISMC 350

$r_1 = 2.83\text{cm} = 28.3 \text{ mm}$

$\therefore$ Horizontal length of lacing $l_h = 100 + 220 + 100 - 50 = 370$ mm

Spacing of lacing is c/c distance of adjacent bars

$= a_1 = 2 \text{ x } 370 = 740 \text{ mm}$

$$\frac{a_1}{r_1} = \frac{740}{28.3} = 26.15 < 50 \text{ and } < 0.7 \text{ x}53.81 \Rightarrow 37.45$$

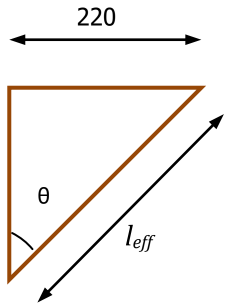
The local buckling of the column does not occur,

Hence single lacing system can be adopted.

Dimension of lacing

Width of lacing bar (P- 50, Cl 7.6.2)

Assuming, Width of lacing = 60 mm



Thickness of bar (t) (P- 50, Cl 7.6.3)

$$t \nless \frac{1}{40} \text{ of distance of inner welds}$$

$$\sin\theta = \frac{220}{l_{eff}}$$

$$l_{eff} = \frac{220}{\sin 45} = 311.13\text{mm}$$

$l_{eff} = 311.13\text{mm}$ For single lacing system

$$t = \frac{1}{40} \times l_{eff} = \frac{1}{40} \times 311.13 = 7.77 \text{ mm} \qquad \text{Say 8 mm}$$

$$r_{min} = \sqrt{\frac{I_{xx}}{A}} = \sqrt{\frac{\frac{lt^3}{12}}{lt}}$$

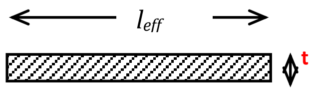
$$r_{min} = \frac{t}{\sqrt{12}}$$

Try a lacing bar of 60 mm width and 8 mm thick

i.e., 60 ISF 8

Note: (P- 50, Cl 7.6.3) Double lacing system $t = \frac{1}{60} \times l_{eff}$

Check for slenderness ratio: (P-50, Cl 7.6.6.3)



$$\lambda = \frac{l_{eff}}{r_{min}} \ngtr 145$$

$$\lambda = \frac{l_{eff}}{r_{min}} = \frac{l_{eff} \times \sqrt{12}}{t} = \frac{311.13 \times \sqrt{12}}{8}$$

$$= 134.72 < 145 \qquad \text{Safe}$$

Note: - (P-50, Cl 7.6.3)

For double lacing system l_{eff} = 0.7 x Length of lacing bar b/w inner bolts.

Check for Compressive Force and Tensile Force:

Force in lacing bar (F) : (P - 48, Cl 5.7.2.1)

Transverse Shear (V) = 2.5 % of axial load

$$= \frac{2.5}{100} \times 1300 = 32.5 \text{ KN}$$

$$\text{Force (F)} = \frac{V}{n \times \sin\theta}$$

n = 2 for single lacing system

n = 4 for double lacing system

$$F = \frac{32.5}{2 \times \sin 45} = 22.98 \text{ KN}$$

OR

$$\text{Force (F)} = \frac{V}{2n} \times \text{cosec}\theta$$

n = 1 for single lacing system, n = 2 for double lacing system

Ref Page 42, Table 9(c)

λ	f_{cd}
130	74.3
134.72	?
140	66.2
10	8.1
4.72	?

$$f_{cd} = 74.3 - \frac{8.1 \times 4.72}{10} = 70.48 \text{ N / mm}^2$$

Compressive Stress
for $\lambda = 134.72$

$$\text{Compressive Force} = \text{Compressive Stress} \times \text{Area of lacing bar} \nless F$$

$$\text{Compressive Force} = \frac{70.48 \times 60 \times 8}{1000} = 33.82\text{KN} > 22.98\text{KN} \qquad \underline{\text{Safe}}$$

$\underline{\text{Tensile Force (P - 32)}}$

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

$$T_{dn} = \frac{0.9 \times b \times t \times f_u}{\gamma_{m1}} = \frac{0.9 \times 60 \times 8 \times 410}{1.25 \times 1000} = 141.70\text{KN} > 22.98 \text{ KN} \qquad \underline{\text{Safe}}$$

Provide 60 ISF 8 as lacing bar

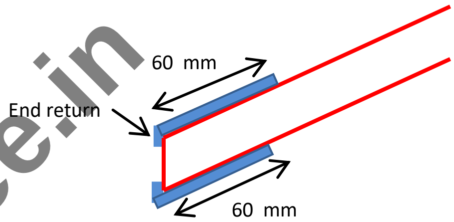
Welded connection:

Max Size of weld S = 8 - 1.5 = 6.5 mm

Say S = 5 mm

Force = Strength of the weld

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



$$\text{Strength of weld} = 0.707 \times D \times l \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}}$$

$$= 0.707 \times 5 \times l \times \frac{410}{\sqrt{3} \times 1.25} = 669.43 l \text{ N - mm}$$

$$\text{Effective length of weld} = \frac{22.98 \times 10^3}{669.43} = 34.33 \text{ mm} \qquad \text{Say 40 mm}$$

Length of weld on each side of flat $= 40/2 = 20$ mm.

Length of longitudinal weld : It is the max of the following

1. Overlap length a) 4t = 4 x 8 = 32mm, b) 40 mm
2. Width of plate = 60 mm

Therefore provide Overlap length of 60 mm.

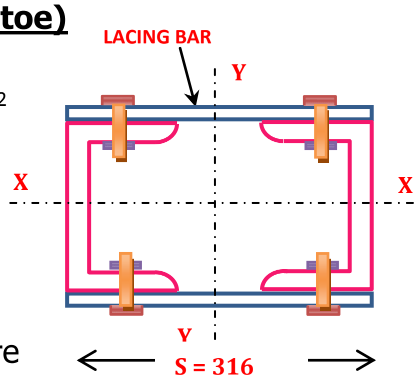
The overall length of weld provided with end return of (2 x D) = 2 x (60 + 2 x 5) = 140 mm

Problem:

Design a built up member to carry an factored load of 1400 KN and effective length in both planes is 6.5m. The column is restrained in position but not in direction at both the ends. Provide double lacing system with bolted connections. Assume steel of grade Fe 410 and bolts of grade 4.6. Design the column with two channels placed toe- to – toe.

Solution:

Design of compression member (Channels toe to toe)



Factored Load= 1400 KN

Assuming permissible stress $= 0.6\,f_y = 0.6 \times 250 = 150\ \text{N}/\text{mm}^2$

$$\text{Area of 2 channels} = \frac{\text{Load}}{\sigma_{ac}} = \frac{1400 \times 10^3}{150}$$

$$= 9333.33\ \text{mm}^2 = 93.33\ \text{cm}^2$$

Try 2 – ISMC 350 @ 42.1 kg /m. Properties of each channels are

$a = 53.66\ \text{cm}^2 = 5366\ \text{mm}^2$ $\quad I_{yy} = 430.6\ \text{cm}^4 = 430 \times 10^4\ \text{mm}^4$,

$I_{ZZ} = 10008\ \text{cm}^4 = 10008 \times 10^4\ \text{mm}^4$, $\quad C_{yy} = 2.44\ \text{cm} = 24.4\ \text{mm}$

Spacing (S): Equate $I_{ZZ} = I_{YY}$ of builtup sections

$$2 \times I_{ZZ} = 2 \times I_{yy}$$

$$2 \times I_{ZZ} = 2 \times \left[I_{yy} + A \times \left(\frac{S}{2} - C_{yy} \right)^2 \right]$$

$$2 \times 10008 \times 10^4 = 2 \times \left[430 \times 10^4 + 5366 \times \left(\frac{S}{2} - 24.4 \right)^2 \right]$$

$$S = 316\text{mm}$$

P-48, Cl: 7.6.1.5

$$\text{Slenderness ratio of builtup section } (\lambda) = 1.05 \times \frac{KL}{r}$$

<u>Effective length (Table11, Cl : 7.2.2, P - 45)</u>

End condition : Effectively held in postion at both ends,

but not restained against rotation. (Both ends Hinged)

$$KL = L = 6.5\ \text{m} = 6500\ \text{mm}$$

$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{2 \times 10008 \times 10^4}{2 \times 5366}} = 136.57$$

44, Table 10, Buckling curve class about any axis 'c'.

P-42, Table 9(c) for f_y = 250 N/mm^2.

Compressive stress about ZZ- axis, ($f_{cd\text{-}zz}$)

$$\lambda_{ZZ} = 1.05 \times \frac{6500}{136.57} = 50$$

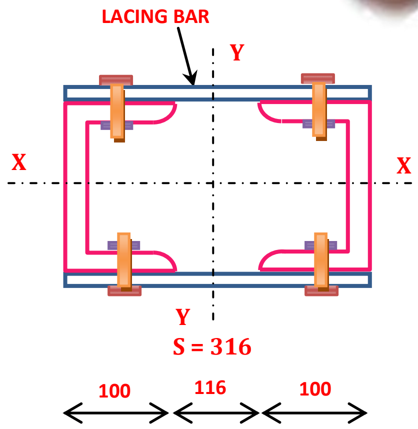
Compressive stress f_{cd} = 183 N/mm^2.

$$\text{Load carrying capacity} = f_{cd} \times \text{Area} = \frac{183 \times 2 \times 5366}{1000} = 1964\ \text{KN} > 1400\ \text{KN}$$

<u>**Safe**</u> **Provide 2 - ISMC 350**

Design Of Lacing: (Double lacing system)

Check for local buckling of column section (P -50, cl 7.6.5)



$\frac{a_1}{r_1} \not> 50$ or $0.7\lambda.7$ builtup section, whichever is less.

0.7 times min of λ_{ZZ} and λ_{YY}

Inclination Of Lacing : (P - 50, Cl 7.6.4)

Assuming Inclination Of Lacing $= 45^\circ \ \left(40^0 \prec \theta \prec 70^0\right)$

The gauge distance 'g' for ISMC 350 is 60 mm.

$r_1 = 2.83\text{cm}$

$\therefore$ Horizontal length of lacing b/w bolts $l_h = 316 - 60 - 60 = 196$ mm

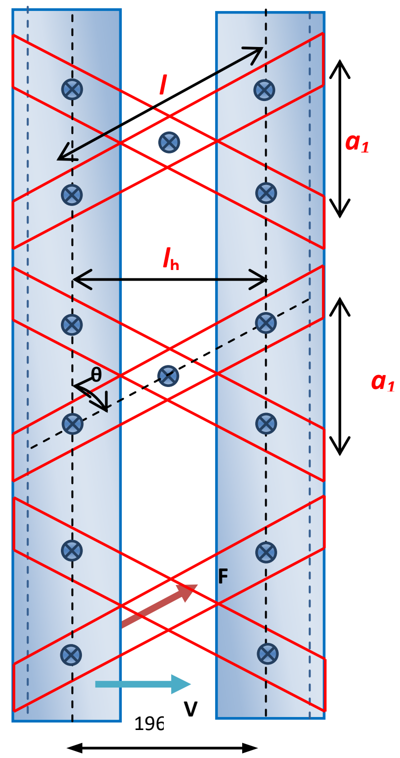
Spacing of lacing is c/c distance of adjacent bolts

$= a_1 = 196$ mm

$$\frac{a_1}{r_1} = \frac{196}{28.3} = 6.93 < 50 \text{ and} < 0.7 \text{ x}50 \Rightarrow 35$$

The local buckling of the column does not occur,

Hence double lacing system can be adopted.



Dimension of lacing

Width of lacing bar (P- 50, Cl 7.6.2)

Assuming dia of bolt = 16 mm

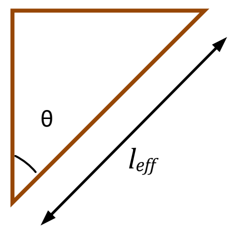
Width of lacing = 3 x dia of bolt = 3 x 16 = 48 mm Say 50 mm

Thickness of bar (t) (P- 50, Cl 7.6.3)

$t \not< \frac{1}{60}$ of distance of inner bolts

$$\sin\theta = \frac{196}{l_{eff}}$$

$$l_{eff} = \frac{196}{\sin 45} = 277.20\text{mm}$$



For double lacing system

$$t = \frac{1}{60} \times l_{eff} = \frac{1}{60} \times 277.20 = 4.62 \text{ mm} \qquad \text{Say } 6 \text{ mm}$$

Try a lacing bar of 50 mm width and 6 mm thick

i.e., 50 ISF 6

Check for slenderness ratio: (P-50, Cl 7.6.6.3)

$$\lambda = \frac{0.7 \times l_{eff}}{r_{min}} \not> 145$$

$$\lambda = \frac{0.7 \times l_{eff}}{r_{min}} = \frac{0.7 \times l_{eff} \times \sqrt{12}}{t}$$

$$= \frac{0.7 \times 277.20 \times \sqrt{12}}{6} = 112.03 < 145 \quad \underline{\text{Safe}}$$

$$r_{min} = \sqrt{\frac{I_{xx}}{A}} = \sqrt{\frac{\frac{lt^3}{12}}{lt}}$$

$$r_{min} = \frac{t}{\sqrt{12}}$$

Check for Compressive Force and Tensile Force:

Force in lacing bar (F) : (P - 48, Cl 5.7.2.1)

Transverse Shear (V) = 2.5 % of axial load

$$= \frac{2.5}{100} \times 1400 = 35 \text{ KN}$$

$$\text{Force (F)} = \frac{V}{n \times \sin\theta}$$

n = 2 for single lacing system

n = 4 for double lacing system

$$F = \frac{35}{4 \times \sin 45} = 12.37 \text{ KN}$$

OR

$$\text{Force (F)} = \frac{V}{2n} \times \operatorname{cosec}\theta$$

n = 1 for single lacing system

n = 2 for double lacing system

Ref Page 42 , Table 9(c)	
λλ	f_{cd}
100	107
112.03	?
110	94.6
10	12.4
2.03	?

$$f_{cd} = 107 - \frac{2.03 \times 12.4}{10} = 104.48 \text{ N/mm}^2$$

Compressive Stress for $\lambda = 112.03$, $f_{cd} = 104.48 \text{ N/mm}^2$

Compressive Force = Compressive Stress × Area of lacing bar $\not<$ F

$$\text{Compressive Force} = \frac{104.48 \times 50 \times 6}{1000} = 31.34 > 12.37\text{KN} \qquad \underline{\text{Safe}}$$

Tensile Force (P - 32)

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}} \not< F$$

$$T_{dn} = \frac{0.9(b - d_o) t \times f_u}{\gamma_{m1}} = \frac{0.9 \times (50 - 18) \times 6 \times 410}{1.25 \times 1000}$$

$= 56.68\text{KN} > 12.37$ KN <u>Safe</u>

Provide 50 ISF 6 as lacing bar

<u>Connection Details:</u>

$$\text{No of bolts} = \frac{F}{BV}$$

Dia of bolt = 16 mm.

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

1) Strength of one bolt in Single shear :

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

Assuming thread is interfering the shear plane

$n_n = 1$ $\quad n_s = 0$, $\gamma_{mb} = 1.25$, $\quad A_{nb} = 0.78 \times \frac{\pi}{4} d^2 = 0.78 \times \frac{\pi}{4} \times 16^2 = 156.83\text{mm}^2$

$$V_{dsb} = \frac{400}{\sqrt{3}} \times \left(\frac{1 \times 156.83}{1.25 \times 1000}\right) = 28.97\text{KN}$$

2) Strength of bolt in Bearing $V_{dpb} = \dfrac{2.5 \times k_b \times d \times t^* \times f_u}{\gamma_{mb}}$

k_b is the least of the following:

1) $\dfrac{e}{3d_0} = \dfrac{35}{3 \times 18} = 0.65$ Edge distance e = 1.5 x 18 =27 mm say 35 mm

2) $\dfrac{p}{3d_0} - 0.25 = \dfrac{50}{3 \times 18} - 0.25 = 0.68$ P = 2.5 x 16 = 40 mm Say 50 mm

3) $\dfrac{f_{ub}}{f_u} = \dfrac{400}{410} = 0.98$ 4) 1

k_b = 0.65

t^* → Min of 1) Thickness of flange of channel (13.5) and
2) thickness of lacing bar (6 mm)

$$V_{dpb} = \frac{2.5 \times 0.65 \times 16 \times 6^* \times 400}{1.25 \times 1000} = 49.92 KN$$

Bolt value (BV) = 28.97 KN.

No of bolts = $\dfrac{12.37}{28.97} = 0.43$ $\boxed{\text{Say 2 No's (Min) One on each side}}$

Problem:

Design a built up member to carry an factored load of 1400 KN and effective length in both planes is 6.5m. The column is restrained in position but not in direction at both the ends. Provide double lacing system with fillet field weld connections. Assume steel of grade Fe 410. Design the column with two channels placed toe – to – toe.

Solution:

Design of compression member (Channels toe to toe)

Factored Load= 1400 KN

Assuming permissible stress $= 0.6\, f_y = 0.6 \times 250 = 150\ N/mm^2$

Area of 2 channels $= \dfrac{Load}{\sigma_{ac}} = \dfrac{1400 \times 10^3}{150} = 9333.33\ mm^2 = 93.33\ cm^2$

Try 2-ISMC 350 @ 42.1Kg/m Properties of each channels are

$a = 53.66\ cm^2 = 5366\ mm^2$ $I_{yy} = 430.6\ cm^4 = 430 \times 10^4\ mm^4$,

$I_{ZZ} = 10008\ cm^4 = 10008 \times 10^4\ mm^4$, $C_{yy} = 2.44\ cm = 24.4\ mm$

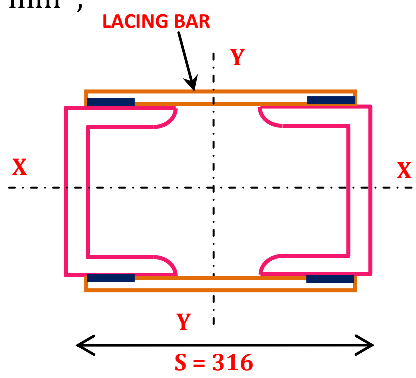
Spacing (S): Equate $I_{ZZ} = I_{YY}$ of builtup sections

$$2 \times I_{ZZ} = 2 \times I_{yy}$$

$$2 \times I_{ZZ} = 2 \times \left[I_{yy} + A \times \left(\frac{S}{2} - C_{yy} \right)^2 \right]$$

$$2 \times 10008 \times 10^4 = 2 \times \left[430 \times 10^4 + 5366 \times \left(\frac{S}{2} - 24.4 \right)^2 \right]$$

$S = 316mm$



P-48, Cl: 7.6.1.5

Slenderness ratio of builtup section $(\lambda) = 1.05 \times \frac{KL}{r}$

Effective length (Table11, Cl : 7.2.2, P - 45)

End condition : Effectively held in postion at both ends,

but not restained against rotation. (Both ends Hinged)

$$KL = L = 6.5 \text{ m} = 6500 \text{ mm}$$

$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{2 \times 10008 \times 10^4}{2 \times 5366}} = 136.57$$

44, Table 10, Buckling curve class about any axis 'c'.

P-42, Table 9(c) for f_y = 250 N/mm^2.

Compressive stress about ZZ- axis, ($f_{cd\text{-}zz}$)

$$\lambda_{ZZ} = 1.05 \times \frac{6500}{136.57} = 50$$

Compressive stress f_{cd} = 183 N/mm^2.

$$\text{Load carrying capacity} = f_{cd} \times \text{Area} = \frac{183 \times 2 \times 5366}{1000}$$

$= 1964$ KN > 1400 KN Safe

Provide 2 - ISMC 350

Design Of Lacing: (Double lacing system)

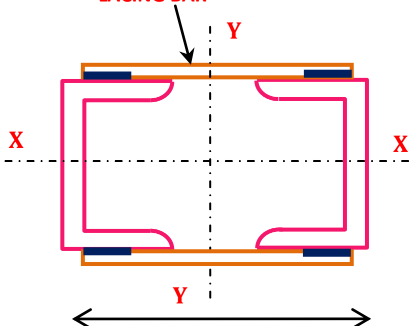
Check for local buckling of column section (P -50, cl 7.6.5)

$\frac{a_1}{r_1} \not> 50$ or 0.7λ.7 builtup section, whichever is less.

0.7 times min of λ_{ZZ} and λ_{YY}

Inclination Of Lacing : (P - 50, Cl 7.6.4)

Assuming Inclination Of Lacing $= 45° \left(40^0 \prec \theta \prec 70^0\right)$



The radius of gyration for single ISMC 350

$$r_1 = 2.83\text{cm} = 28.3\text{mm}$$

$\therefore$ Horizontal length of lacing $l_h = 316 - 50 = 266$ mm

Spacing of lacing $= a_1 =$ 266 mm

$$\frac{a_1}{r_1} = \frac{266}{28.3} = 9.40 < 50 \text{ and} < 0.7 \text{ x}50 \Rightarrow 35$$

The local buckling of the column does not occur,

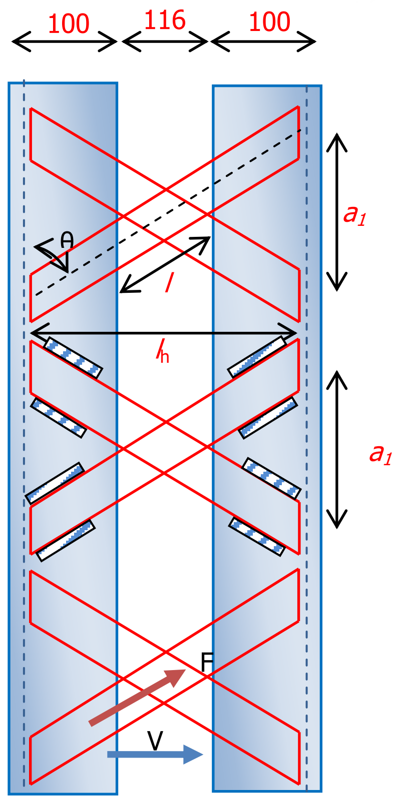
Hence double lacing system can be adopted.

Dimension of lacing

Width of lacing bar (P- 50, Cl 7.6.2)

Assuming,

Width of lacing = 60 mm



Thickness of bar (t) (P- 50, Cl 7.6.3)

$t \nless \frac{1}{60}$ of distance of inner welds

Horizontal length b/w inner welds = 316 - 100 - 100 = 116mm

$$\sin\theta = \frac{116}{l_{eff}}$$

$$l_{eff} = \frac{116}{\sin 45} = 164.05\text{mm}$$

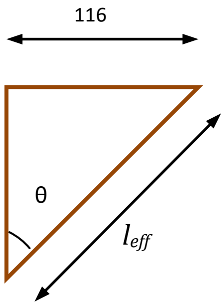
For double lacing system

$$t = \frac{1}{60} \times l_{eff} = \frac{1}{60} \times 164.05 = 2.73 \text{ mm} \qquad \text{Say 6 mm}$$

Try a lacing bar of 60 mm width and 6 mm thick

i.e., 60 ISF 6

Check for slenderness ratio: (P-50, Cl 7.6.6.3)



$$\lambda = \frac{0.7 \times l_{eff}}{r_{min}} \ngtr 145$$

$$\lambda = \frac{0.7 \times l_{eff}}{r_{min}} = \frac{0.7 \times l_{eff} \times \sqrt{12}}{t}$$

$$= \frac{0.7 \times 164.05 \times \sqrt{12}}{6} = 66.30 < 145 \qquad \underline{\text{Safe}}$$

Check for Compressive Force and Tensile Force:

Force in lacing bar (F) : (P - 48, Cl 5.7.2.1)

Transverse Shear (V) = 2.5 % of axial load

$$= \frac{2.5}{100} \times 1400 = 35 \text{ KN}$$

$$\text{Force (F)} = \frac{V}{n \times \sin\theta}$$

n = 2 for single lacing system

n = 4 for double lacing system

$$F = \frac{35}{4 \times \sin 45} = 12.37 \text{ KN}$$

OR

$$\text{Force (F)} = \frac{V}{2n} \times \text{cosec}\theta$$

n = 1 for single lacing system

n = 2 for double lacing system

Compressive Stress
for $\lambda = 66.30$

Compressive Force $=$ Compressive Stress $\times$ Area of lacing bar $\not<$ F

$$\text{Compressive Force} = \frac{157.92 \times 60 \times 6}{1000} = 56.85 > 12.37\,\text{KN}$$

Safe

Ref Page 42, Table 9(c)

λ	f_{cd}
60	168
66.30	?
70	152
10	16
6.30	?

$$f_{cd} = 168 - \frac{6.30 \times 16}{10} = 157.92\ \text{N/mm}^2$$

Tensile Force (P-32)

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}} \not< F$$

$$T_{dn} = \frac{0.9 \times b \times t \times f_u}{\gamma_{m1}} = \frac{0.9 \times 60 \times 6 \times 410}{1.25 \times 1000}$$

$T_{dn} = 106.27\ \text{kN} > 12.37\ \text{kN}$ Safe

Provide 60 ISF 6 as lacing bar

Welded connection:

Max Size of weld S = 6 - 1.5 = 4.5 mm

Say S = 4 mm

Force = Strength of the weld

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

$$\text{Strength of weld} = 0.707 \times D \times l \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}}$$

$$= 0.707 \times 4 \times l \times \frac{410}{\sqrt{3} \times 1.25} = 535.54\ l\ \text{N-mm}$$

$$\text{Effective length of weld} = \frac{12.37 \times 10^3}{535.54} = 23\ \text{mm} \qquad \text{Say } 30\ \text{mm}$$

Length of weld on each side of flat $= 30/2 = 15$ mm.

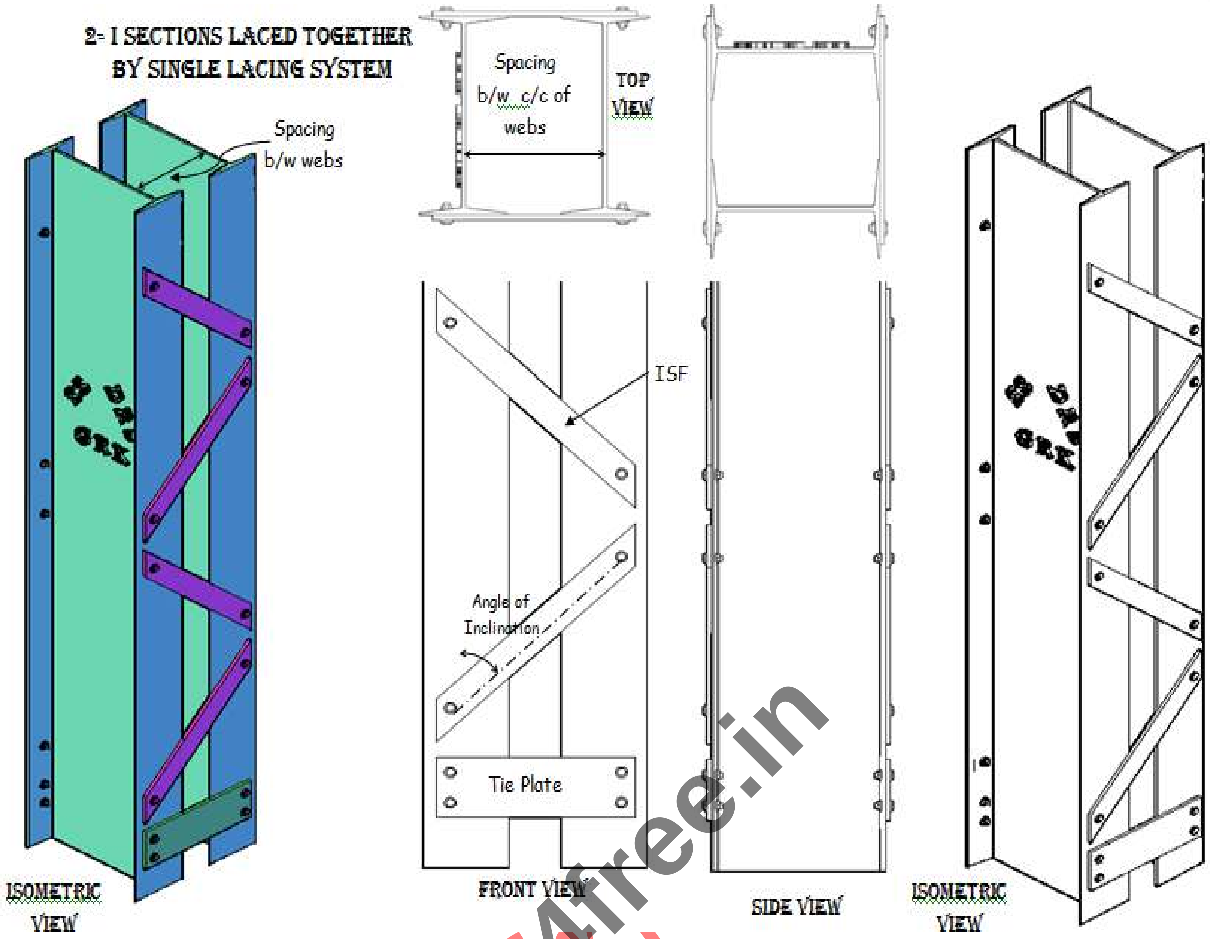
Length of longitudinal weld : It is the max of the following

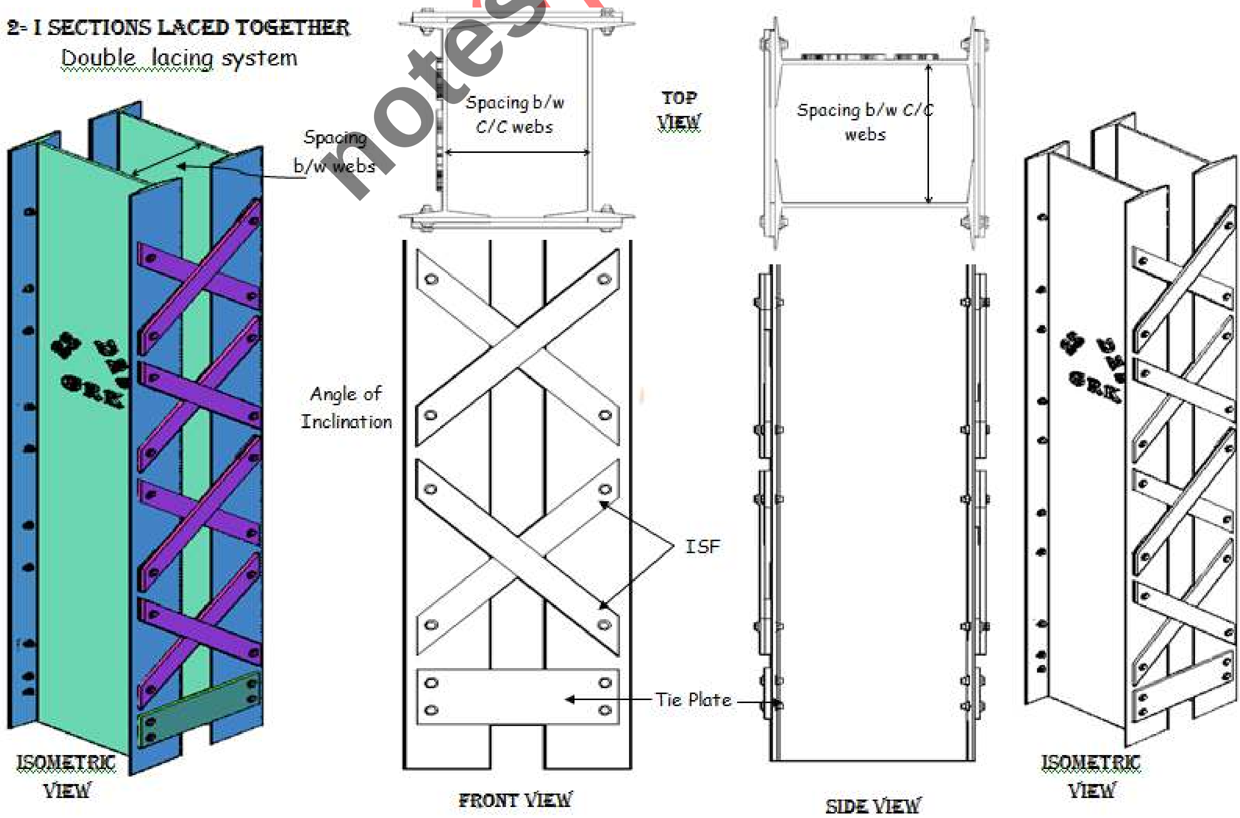
1. Overlap length a) 4t = 4 x 6 = 24mm, b) 40 mm
2. Width of plate = 60 mm

Therefore provide Overlap length of 60 mm.

The overall length of weld provided with end return of (2 x D) = 2 x (60 + 2 x 5) = 140 mm

Problems on two – I sections laced together


2- I SECTIONS LACED TOGETHER
BY SINGLE LACING SYSTEM
Spacing
b/w webs
Spacing
b/w c/c of
webs
TOP
VIEW
ISF
Angle of
Inclination
Tie Plate
ISOMETRIC
VIEW
FRONT VIEW
SIDE VIEW
ISOMETRIC
VIEW


2- I SECTIONS LACED TOGETHER
Double lacing system
Spacing
b/w webs
Spacing b/w
C/C webs
TOP
VIEW
Spacing b/w C/C
webs
Angle of
Inclination
ISF
Tie Plate
ISOMETRIC
VIEW
FRONT VIEW
SIDE VIEW
ISOMETRIC
VIEW

Problem:

The axial load on a steel column is 2000 KN. The column of length 5 m is effectively held in position at both ends and restrained in direction at one end. Design a suitable built up I-section for the column adopting single lacing and sketch the elevation and plan of the column. Permissible stresses confirm to the specification of IS 800 – 2007.

Solution:

Design of compression member

Axial Load= 2000 KN

Factored load = 1.5 x 2000 = 3000 KN

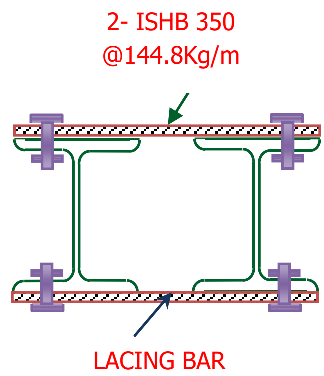
$\text{Assuming permissible stress}(f_{cd}) = 180\ N/mm^2$

$$\text{Area of } 2\text{ I-sections} = \frac{\text{Load}}{\sigma_{ac}} = \frac{3000 \times 10^3}{180}$$

$= 16666.66\ mm^2 = 166.67\ cm^2$

Try 2-ISHB 350 @ 144.8Kg/m with spacing b/w webs $S = 275$ mm

$$\left[\begin{array}{l} \text{P-48, Cl: 7.6.1.1} \\ \text{Note:- The spacing is choosen in such a way that} \\ \quad I_{xx} \approx I_{yy} \quad \& \ r_{yy} > r_{xx} \end{array}\right]$$



$a = 184.42\ cm^2 = 18442\ mm^2$

$r_{zz} = 14.65\ cm = 146.5\ mm,$

$r_{yy} = 14.71\ cm = 147.1\ mm$

P-48, Cl: 7.6.1.5

$$\text{Slenderness ratio of builtup section } (\lambda) = 1.05 \times \frac{KL}{r}$$

Effective length (Table11, Cl: 7.2.2, P-45)

End condition : Effectively held in postion at both ends,

but not restained against rotation. (One end fixed and one end Hinged)

$KL = 0.8 \times L = 0.8 \times 5 = 4\ m = 4000\ mm$

P-44, Table 10, Buckling curve class about any axis 'c'.

P-42, Table 9(c) for f_y = 250 N/mm^2.

Compressive stress about ZZ- axis, ($f_{cd\text{-}zz}$)

$$\lambda_{zz} = 1.05 \times \frac{4000}{146.5} = 28.67$$

Compressive stress f_{cd} = 215.73 N/mm^2.

$\lambda\lambda$	f_{cd}
20	224
28.67	?
30	211
10	13
8.67	?

$$f_{cd} = 224 - \frac{8.67 \times 13}{10} = 215.73\ N/mm^2$$

Load carrying capacity $= f_{cd} \times$ Area

$$= \frac{215.73 \times 18442}{1000} = 3978.50 \text{ KN} > 3000 \text{ KN}$$

<u>Safe</u>

Provide 2 - ISHB 350 @ 144.8 Kg / m.

<u>Design Of Lacing: (single lacing system)</u>

<u>Check for local buckling of column section (P -50, cl 7.6.5.1)</u>

$\frac{a_1}{r_1} \not> 50$ or 0.7λ.7 builtup section, whichever is less.

<u>Inclination Of Lacing : (P - 50, Cl 7.6.4)</u>

Assuming Inclination Of Lacing $= 45° \ (40^0 \prec \theta \prec 70^0)$

The gauge distance 'g' for ISHB 350 is 140 mm.

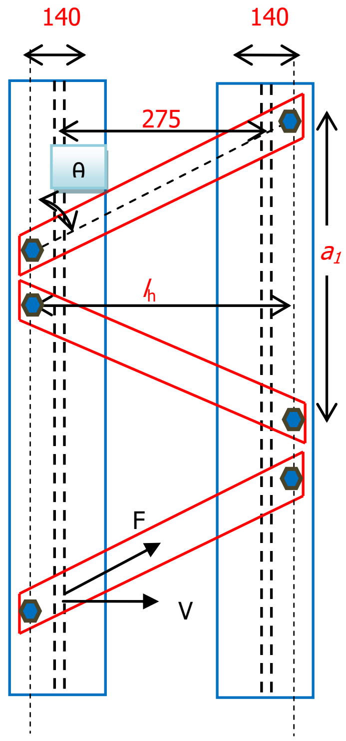
$r_1 = 5.22\text{cm} = 52.2$ mm

$\therefore$ Horizontal length of lacing $= 140/2 + 275 + 140/2$

$l_h = 415$ mm

Spacing of lacing is c/c distance of adjacent bolts

$= a_1 = 2 \text{ x } 415 = 830$ mm



$$\frac{a_1}{r_1} = \frac{830}{52.2} = 15.9 < 50 \text{ and } < 0.7 \text{ x } 28.67 \Rightarrow 20.07$$

The local buckling of the column does not occur,

Hence single lacing system can be adopted.

<u>Dimension of lacing</u>

<u>Width of lacing bar (P- 50, Cl 7.6.2)</u>

Assuming dia of bolt = 20 mm

Width of lacing = 3 x dia of bolt = 3 x 20 = 60 mm

<u>Thickness of bar (t)</u> <u>(P- 50, Cl 7.6.3)</u>

$t \not< \frac{1}{40}$ of distance of inner bolts

$$\sin\theta = \frac{415}{l_{eff}}$$

$$l_{eff} = \frac{415}{\sin 45} = 586.9\text{mm}$$

$l_{eff} = 586.9$mm For single lacing system

$$t = \frac{1}{40} \times l_{eff} = \frac{1}{40} \times 586.9 = 14.67 \text{ mm} \qquad \text{Say 16 mm}$$

Try a lacing bar of 60 mm width and 16 mm thick

i.e., 60 ISF 16

<u>Note:</u> <u>(P- 50, Cl 7.6.3)</u> Double lacing system $t = \frac{1}{60} \times l_{eff}$

Check for slenderness ratio: (P-50, Cl 7.6.6.3)

$$\lambda = \frac{l_{eff}}{r_{min}} \ngtr 145$$

$$\lambda = \frac{l_{eff}}{r_{min}} = \frac{l_{eff} \times \sqrt{12}}{t} = \frac{586.9 \times \sqrt{12}}{16} = 127.1 < 145$$

Safe

$$r_{min} = \sqrt{\frac{I_{xx}}{A}} = \sqrt{\frac{\frac{lt^3}{12}}{lt}}$$

$$r_{min} = \frac{t}{\sqrt{12}}$$

Check for Compressive Force and Tensile Force:

Force in lacing bar (F) : (P - 50, Cl 7.6.6.1)

Transverse Shear $(V_t) = 2.5$ % of axial load

$$= \frac{2.5}{100} \times 3000 = 75 \text{ KN}$$

$$\text{Force } (F) = \frac{V_t}{n \times \sin\theta}$$

n = 2 for single lacing system

n = 4 for double lacing system

$$F = \frac{75}{2 \times \sin 45} = 53.03 \text{KN}$$

Compressive Stress

for $\lambda = 127.1$

Ref Page 42, Table 9(c)

λ	f_{cd}
120	83.7
127.1	?
130	74.3
10	9.4
7.1	?

$$f_{cd} = 83.7 - \frac{9.4 \times 7.1}{10} = 77.03 \text{ N/mm}^2$$

Compressive Force = Compressive Stress × Area of lacing bar $\nless$ F

$$\text{Compressive Force} = \frac{77.03 \times 60 \times 16}{1000} = 73.95 > 53.03 \text{KN}$$

Safe

Tensile Force (P - 32)

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

$$T_{dn} = \frac{0.9(b - d_o)t \times f_u}{\gamma_{m1}} = \frac{0.9(60 - 22) \times 16 \times 410}{1.25 \times 1000} = 179.48 \text{KN} > 53.03 \text{ KN}$$ Safe

Provide 60 ISF 16 as lacing bar

Connection Details:

$$\text{No of bolts} = \frac{F}{BV}$$

Dia of bolt = 20 mm.

Dia of hole (d_0) = 20 + 2 = 22 mm

P-75, Cl: 10.3.3

1) Strength of one bolt in Single shear:

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

Assuming shank is interfering the shear plane

$$n_n = 0 \qquad n_s = 1 \quad , \gamma_{mb} = 1.25, \quad A_{sb} = \frac{\pi}{4} d^2 = \frac{\pi}{4} \times 20^2 = 314.16 \text{ mm}^2$$

$$V_{dsb} = \frac{400}{\sqrt{3}} \times \left(\frac{1 \times 314.16}{1.25 \times 1000} \right) = 58.04 \text{KN}$$

2) Strength of bolt in Bearing $V_{dpb} = \dfrac{2.5 \times k_b \times d \times t^* \times f_u}{\gamma_{mb}}$

k_b is the least of the following:

1) $\dfrac{e}{3d_0} = \dfrac{40}{3 \times 22} = 0.57$ Edge distance e = 1.5 x 22 =33 mm say 40 mm

2) $\dfrac{p}{3d_0} - 0.25 = \dfrac{50}{3 \times 22} - 0.25 = 0.51$

P = 2.5 x 20 = 50 mm

3) $\dfrac{f_{ub}}{f_u} = \dfrac{400}{410} = 0.98$ 4) 1

k_b = 0.51

t → Min of 1) Thickness of flange of I-section (11.6) and*

2) Thickness of lacing bar (16 mm)

$$V_{dpb} = \frac{2.5 \times 0.51 \times 20 \times 11.6^* \times 400}{1.25 \times 1000} = 94.66 \text{ kN}$$

Bolt value (BV) = 58.04 KN.

No of bolts = $\dfrac{53.03}{58.04} = 0.91$ $\boxed{\text{Say 2 No's (Min) One on each side}}$

Problem:

The axial load on a steel column is 2000 KN. The column of length 5 m is effectively held in position at both ends and restrained in direction at one end. Design a suitable built up I-section for the column adopting single lacing system with site welded connection and sketch the elevation and plan of the column. Permissible stresses confirm to the specification of IS 800 – 2007.

Solution:

Design of compression member

Axial Load= 2000 KN

Factored load = 1.5 x 2000 = 3000 KN

Assuming permissible stress$(f_{cd}) = 180 \text{ N/mm}^2$

$$\text{Area of } 2\,\text{I-sections} = \frac{\text{Load}}{\sigma_{ac}} = \frac{3000 \times 10^3}{180} = 16666.66 \text{ mm}^2 = 166.67 \text{ cm}^2$$

Try 2 - ISHB 350 @ 144.8Kg/m with spacing b/w webs S = 275 mm

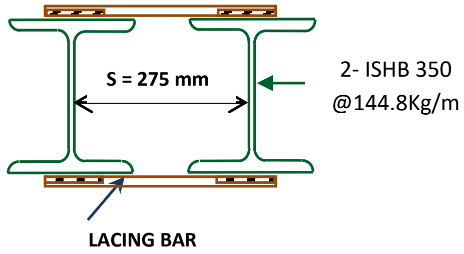
[P - 48, Cl: 7.6.1.1]

Note : - The spacing is choosen in such a way that

$I_{xx} \approx I_{yy}$ & $r_{yy} > r_{xx}$

$a = 184.42\ cm^2 = 18442\ mm^2$, $r_{zz} = 14.65\ cm = 146.5\ mm$,

$r_{yy} = 14.71\ cm = 147.1\ mm$



P-48, Cl: 7.6.1.5

Slenderness ratio of builtup section $(\lambda) = 1.05 \times \frac{KL}{r}$

Effective length (Table11, Cl : 7.2.2, P - 45)

End condition : Effectively held in postion at both ends,

but not restained against rotation. (One end fixed and one end Hinged)

$KL = 0.8 \times L = 0.8 \times 5 = 4\ m = 4000\ mm$

P-44, Table 10, Buckling curve class about any axis 'c'.

P-42, Table 9(c) for f_y = 250 N/mm^2.

Compressive stress about ZZ- axis, (f_{cd-zz})

$$\lambda_{zz} = 1.05 \times \frac{4000}{146.5} = 28.67$$

Compressive stress f_{cd} = 215.73 N/mm^2.

Load carrying capacity $= f_{cd} \times$ Area

$$= \frac{215.73 \times 18442}{1000} = 3978.50\ KN > 3000\ KN$$

Safe

Provide 2 - ISHB 350 @ 144.8 Kg / m.

λ	f_{cd}
20	224
28.67	?
30	211
10	13
8.67	?

$$f_{cd} = 224 - \frac{8.67 \times 13}{10} = 215.73\ N/mm^2$$

Design Of Lacing: (single lacing system)

Check for local buckling of column section (P -50, cl 7.6.5.1)

$\frac{a_1}{r_1} \not> 50$ or $0.7\lambda.7$ builtup section, whichever is less.

Inclination Of Lacing : (P - 50, Cl 7.6.4)

Assuming Inclination Of Lacing = 45° $(40^0 \prec \theta \prec 70^0)$

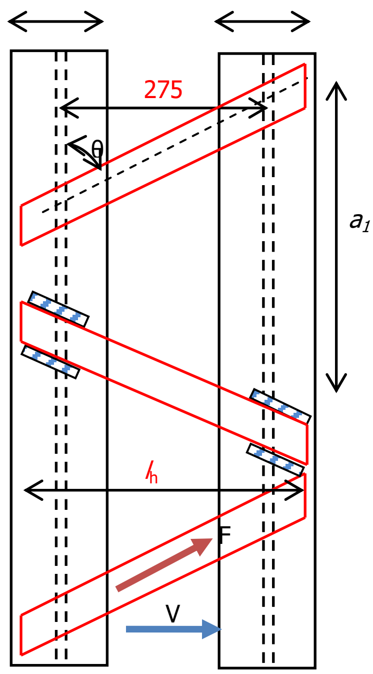
$r_1 = 5.22cm = 52.2mm$

The horizontal distance (l_h):

$l_h = 250/2 + 275 + 250/2 - 250/2 - 50 = 475\ mm$

Spacing of lacing is c/c distance of adjacent bolts

$= a_1 = 2 \times 475 = 950$ mm



$$\frac{a_1}{r_1} = \frac{950}{52.2} = 18.20 \; < 50 \text{ and} < 0.7 \text{ x } 28.67 \Rightarrow 20.10$$

The local buckling of the column does not occur,
Hence single lacing system can be adopted.

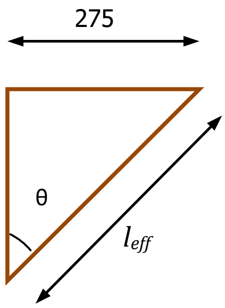
Dimension of lacing

Width of lacing bar (P- 50, Cl 7.6.2)

Assuming ,

Width of lacing = 60 mm

Thickness of bar (t) (P- 50, Cl 7.6.3)



$t \not< \frac{1}{40}$ of distance of inner welds

$$\text{horizontal distance b/w inner welds} = 275 - 2\left(\frac{b_f}{2}\right) = 275 - 250 = 25$$

$$\sin\theta = \frac{25}{l_{eff}}$$

$$l_{eff} = \frac{25}{\sin 45} = 35.36\text{mm}$$

$$l_{eff} = 35.36\text{mm}$$

For single lacing system

$$t = \frac{1}{40} \times l_{eff} = \frac{1}{40} \times 35.36 = 0.88 \text{ mm} \quad \text{Say } 6 \text{ mm}$$

Try a lacing bar of 60 mm width and 6 mm thick

i.e., 60 ISF 6

Check for slenderness ratio: (P-50, Cl 7.6.6.3)

$$\lambda = \frac{l_{eff}}{r_{min}} \not> 145$$

$$\lambda = \frac{l_{eff}}{r_{min}} = \frac{l_{eff} \times \sqrt{12}}{t} = \frac{35.36 \times \sqrt{12}}{6} = 20.42 < 145$$

Safe

$$r_{min} = \sqrt{\frac{I_{xx}}{A}} = \sqrt{\frac{\frac{lt^3}{12}}{lt}}$$

$$r_{min} = \frac{t}{\sqrt{12}}$$

Check for Compressive Force and Tensile Force:

Force in lacing bar (F) : (P - 50, Cl 7.6.6.1)

Transverse Shear $(V_t) = 2.5$ % of axial load

$$= \frac{2.5}{100} \times 3000 = 75 \text{ KN}$$

$$\text{Force (F)} = \frac{V_t}{n \times \sin\theta}$$

$n = 2$ for single lacing system

$n = 4$ for double lacing system

Ref Page 42, Table 9(c)

λ	f_{cd}
20	224
20.42	?
40	211
10	13
0.42	?

$$f_{cd} = 224 - \frac{0.42 \times 13}{10} = 223.45 \text{ N / mm}^2$$

$$F = \frac{75}{2 \times \sin 45} = 53.03KN$$

Compressive Stress

for $\lambda = 20.42$

$$\text{Compressive Force} = \text{Compressive Stress} \times \text{Area of lacing bar} \not< F$$

$$\text{Compressive Force} = \frac{223.45 \times 60 \times 6}{1000} = 80.44 > 53.03 \text{ kN} \qquad \underline{\text{Safe}}$$

$\underline{\text{Tensile Force } (P - 32)}$

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}} \not< F$$

$$T_{dn} = \frac{0.9 \times b \times t \times f_u}{\gamma_{m1}} = \frac{0.9 \times 60 \times 6 \times 410}{1.25 \times 1000} = 106.27KN > 53.03\text{ KN} \qquad \underline{\text{Safe}}$$

Provide 60 ISF 6 as lacing bar

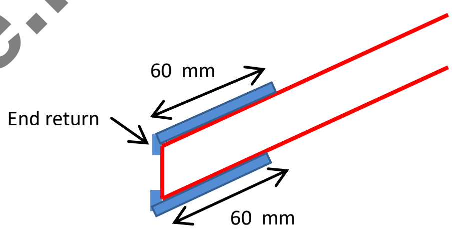
Welded connection:

Max Size of weld S = 6 - 1.5 = 4.5 mm

Say S = 4 mm

Force = Strength of the weld

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



$$\text{Strength of weld} = 0.707 \times D \times l \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}}$$

$$= 0.707 \times 4 \times l \times \frac{410}{\sqrt{3} \times 1.25} = 535.54\, l \text{ N - mm}$$

$$\text{Effective length of weld} = \frac{53.03 \times 10^3}{535.54} = 99.02 \text{ mm} \qquad \text{Say } 100 \text{ mm}$$

Length of weld on each side of flat $= 100/2 = 50$ mm.

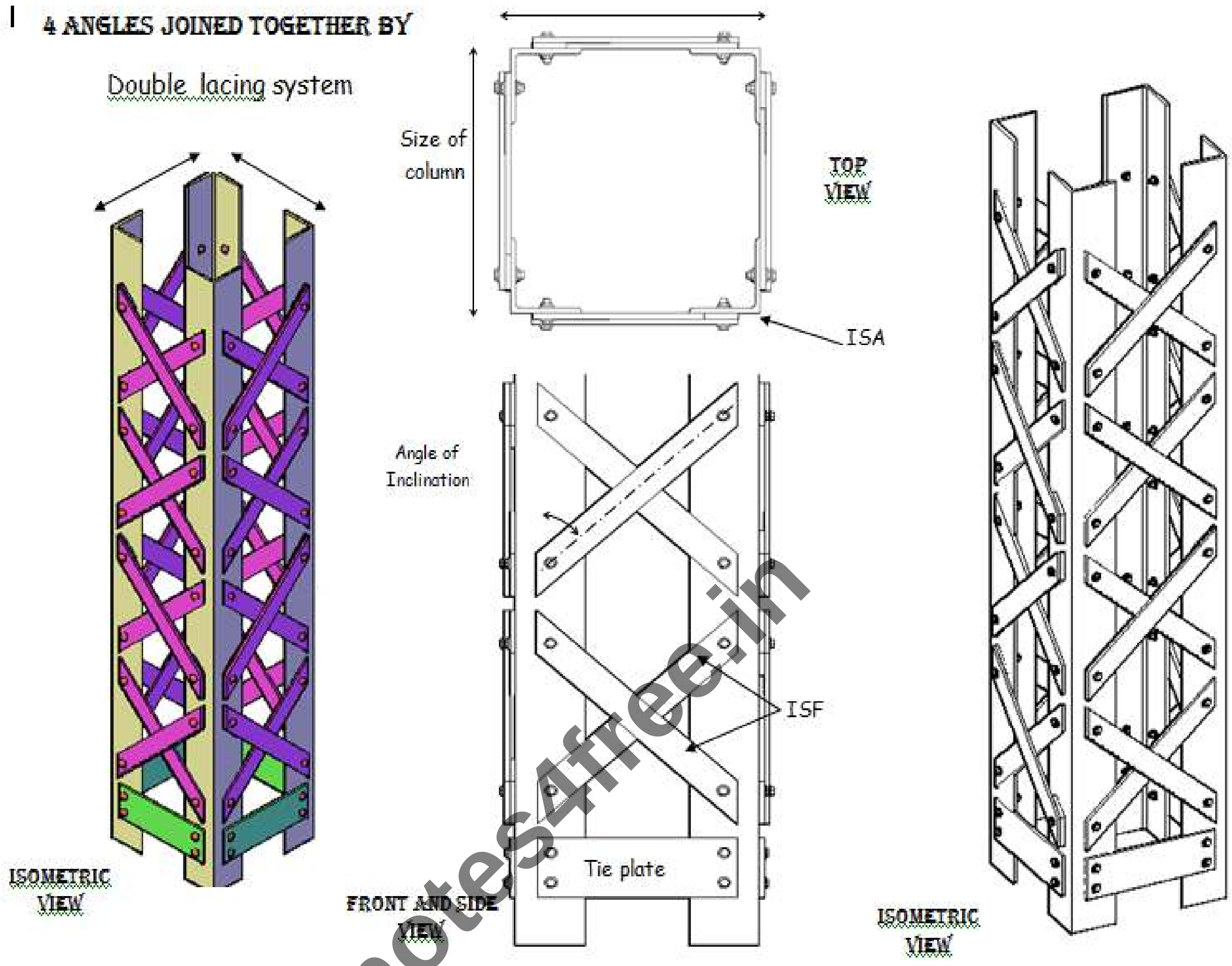
Length of longitudinal weld: It is the max of the following

1. Overlap length a) 4t = 4 x 6 = 24mm, b) 40 mm
2. Width of plate = 60 mm

Therefore provide Overlap length of 60 mm.

The overall length of weld provided with end return of (2 x D) = 2 x (60 + 2 x 5) = 140 mm

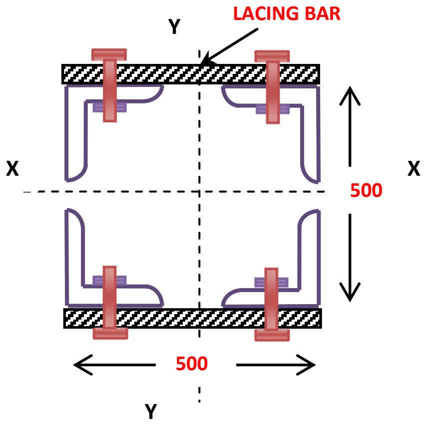
Problem on double lacing system joined together by 4 angles



Problem: 1996 Aug

A mild steel built-up column is to be designed to carry an axial load of 1800 KN. The height of the column is 7 m. The column is considered to be held effectively in position at both the ends and restrained in direction at one end. The column is to be designed using 4-angle section suitably laced together.

Solution:



Design of compression member

Design of compression member

Axial Load= 1800 KN

Factored load = 1.5 x 1800 = 3000 KN

$$\text{Assuming permissible stress}(f_{cd}) = 180\ N/mm^2$$

$$\text{Area od 4-angles} = \frac{\text{Load}}{f_{cd}} = \frac{2700\times10^3}{180} = 15000mm^2 = 150cm^2$$

$$\text{Area of each angle} = \frac{150}{4} = 37.5cm^2$$

Try 4 - ISA $130\times130\times158$ mm @ 28.9 Kg / m

having the following properties

$$a = 36.81cm^2 = 3681mm^2$$

$$I_{ZZ} = I_{yy} = 574.6cm^4 = 574.6\times10^4\ mm^4,$$

$$C_{xx} = C_{yy} = 3.78\ cm = 37.8\ mm$$

P-48, Cl: 7.6.1.5

$$\text{Slenderness ratio of builtup section}(\lambda) = 1.05\times\frac{KL}{r}$$

<u>Effective length (Table11, Cl: 7.2.2, P-45)</u>

End condition : Effectively held in postion at both ends and restained in direction at one end.

$$l_{eff} = 0.8\times l_{act} = 0.8\times7\ m = 5.6\ m = 5600\ mm$$

Let the size of the built-up column with 4 angles be 400 mm x 400 mm

$$I_{ZZ\text{ of built up column}} = 4\left[574.6\times10^4 + 36.81\left(\frac{400}{2} - 37.8\right)^2\right]$$

$$= 410.36\times10^6\ mm^4$$

$$\text{Radius of gyration } r_{zz} = r_{yy} = \sqrt{\frac{410.36\times10^6}{4\times3681}} = 166.94\ mm$$

$$\text{Slenderness ratio } \lambda = 1.05\times\frac{5600}{166.94} = 35.22$$

P-44, Table 10, Buckling curve class about any axis 'c'.

P-42, Table 9(c) for f_y = 250 N/mm^2.

Compressive stress about ZZ- axis, (f_{cd-zz})

Compressive stress f_{cd} = 204.21 N/mm^2.

$\lambda\lambda$	f_{cd}
30	211
35.22	?
40	198
10	13
5.22	?

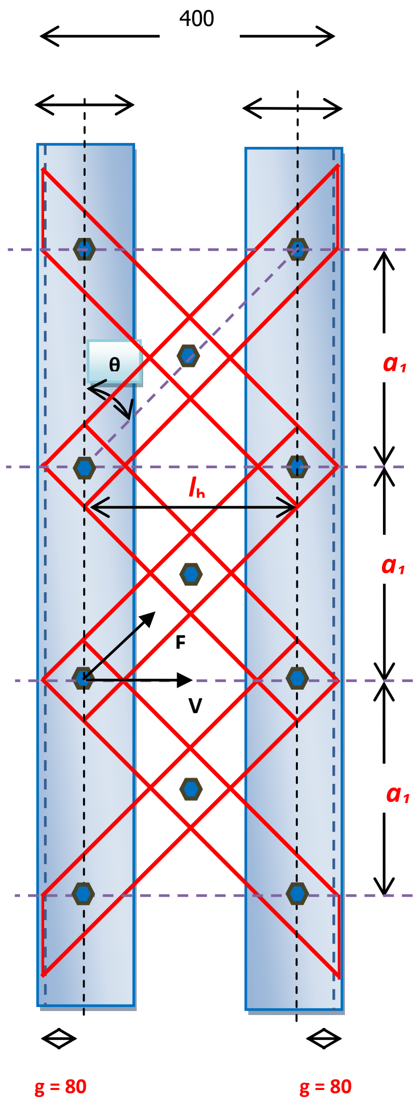
$$f_{cd} = 211 - \frac{5.22 \times 13}{10} = 204.21 \text{ N/mm}^2$$

$$\text{Load carrying capacity} = f_{cd} \times \text{Area}$$

$$= \frac{204.21 \times 4 \times 3681}{1000} = 3007 \text{ KN} > 2700 \text{ KN}$$

Safe

Provide 4 - ISA 130 x 130 x 15 mm @ 28.9 Kg / m.



Design Of Lacing:

Assuming Double lacing system:

Check for local buckling of column section (P -50, cl 7.6.5)

$\frac{a_1}{r_1} \not> 50$ or 0.7λ.7 builtup section, whichever is less.

0.7 times min of λ_{ZZ} and λ_{YY}

Inclination Of Lacing : (P - 50, Cl 7.6.4)

Assuming Inclination Of Lacing = 45° $(40^0 \prec \theta \prec 70^0)$

The gauge distance 'g' for ISA 130 x 130 x 15 is 80 mm.

$\therefore$ Horizontal length of lacing l_h = 400 - 80 - 80 = 240 mm

Spacing of lacing is c/c diatance of adjacent bolts

$= a_1 = 240$ mm

r'_{min} for single channel ISA 130 x130 x 15 = 2.53 cm = 25.3 mm

$$\frac{a_1}{r_1} = \frac{240}{25.3} = 9.49 < 50 \text{ and } < 0.7 \text{ x } 35.22 \Rightarrow 24.65$$

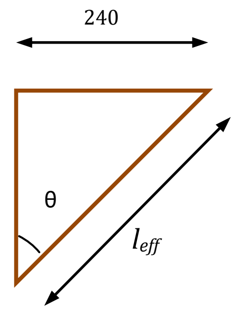
The local buckling of the column does not occur, hence Double lacing system can be adopted.

Dimension of lacing

Width of lacing bar (P- 50, Cl 7.6.2)

Assuming dia of bolt = 16 mm

Width of lacing = 3 x dia of bolt = 3 x 16 = 48 mm Say 0 mm



Thickness of bar (t) (P- 50, Cl 7.6.3)

$t \not< \frac{1}{60}$ of distance of inner bolts

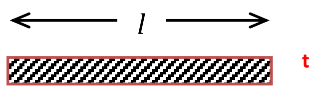
$$\sin\theta = \frac{240}{l_{eff}}$$

$$l_{eff} = \frac{240}{\sin 45} = 339.41\,mm$$

$$t = \frac{1}{60} \times l_{eff} = \frac{1}{60} \times 339.41 = 5.66\ mm \qquad \text{Say 8 mm}$$

Try a lacing bar of 60 mm width and 8 mm thick

i.e., 60 ISF 8



Check for slenderness ratio: (P-50, Cl 7.6.6.3)

$$\lambda = \frac{0.7 \times l_{eff}}{r_{min}} \ngtr 145$$

$$\lambda = \frac{0.7 \times l_{eff}}{r_{min}} = \frac{0.7 \times l_{eff} \times \sqrt{12}}{t}$$

$$= \frac{0.7 \times 339.41 \times \sqrt{12}}{8} = 102.88 < 145 \qquad \underline{\text{Safe}}$$

$$r_{min} = \sqrt{\frac{I_{xx}}{A}} = \sqrt{\frac{\frac{lt^3}{12}}{lt}}$$

$$r_{min} = \frac{t}{\sqrt{12}}$$

Check for Compressive Force and Tensile Force:

Force in lacing bar (F) : (P - 50, Cl 7.6.6.1)

Transverse Shear (V_t) = 2.5 % of axial load

$$= \frac{2.5}{100} \times 2700 = 67.5\ KN$$

$$\text{Force (F)} = \frac{V_t}{n \times \sin\theta}$$

n = 2 for single lacing system

n = 4 for double lacing system

$$F = \frac{67.5}{4 \times \sin 45} = 23.86\,KN$$

Ref Page 42 , Table 9(c)

$\lambda\lambda$	f_{cd}
100	107
102.88	?
110	94.6
10	12.4
2.88	?

$$f_{cd} = 107 - \frac{2.88 \times 12.4}{10} = 103.43\ N/mm^2$$

Compressive Force = Compressive Stress × Area of lacing bar $\nless$ F

$$\text{Compressive Force} = \frac{103.43 \times 60 \times 8}{1000} = 49.65 > 23.86\,KN$$

Safe

Tensile Force (P - 32)

$$T_{dn} = \frac{0.9A_n f_u}{\gamma_{m1}} \not< F$$

$$T_{dn} = \frac{0.9(b - d_o)t \times f_u}{\gamma_{m1}} = \frac{0.9 \times (60 - 18) \times 8 \times 410}{1.25 \times 1000} = 99.20KN > 23.86\ KN \quad \underline{Safe}$$

Provide 60 ISF 8 as lacing bar

<u>Connection Details:</u> No of bolts $= \frac{F}{BV}$

Dia of bolt = 16 mm.

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

P-75, Cl: 10.3.3

1) Strength of one bolt in Double shear :

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

Assuming both thread and shank is interfering the shear plane

$n_n = 1$, $n_s = 1$, $\gamma_{mb} = 1.25$

$$A_{nb} = 0.78 \times \frac{\pi}{4} d^2 = 0.78 \times \frac{\pi}{4} \times 16^2 = 156.83mm^2$$

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

$$V_{dsb} = \frac{400}{\sqrt{3}} \times \left(\frac{1 \times 156.83 + 1 \times 201.06}{1.25 \times 1000}\right) = 66.12KN$$

2) Strength of bolt in Bearing $V_{dpb} = \frac{2.5 \times k_b \times d \times t^* \times f_u}{\gamma_{mb}}$

k_b is the least of the following:

1) $\frac{e}{3d_0} = \frac{35}{3 \times 18} = 0.65$ Edge distance e = 1.5 x 18 =27 mm say 35 mm

2) $\frac{p}{3d_0} - 0.25 = \frac{50}{3 \times 18} - 0.25 = 0.68$ P = 2.5 x 16 = 40 mm Say 50 mm

3) $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.98$ 4) 1

k_b = 0.65

t^* → Min of 1) Thickness of angle (15) and

2) Thickness of lacing bar (8mm)

$$V_{dpb} = \frac{2.5 \times 0.65 \times 16 \times 8^* \times 400}{1.25 \times 1000} = 66.56\ KN$$

Bolt value (BV) = 66.12 KN.

No of bolts = $\frac{23.86}{66.12} = 0.35$ $\boxed{\text{Say 2 No's (Min) One on each side}}$

Problem:

The c/s of a 6 m long, pin ended column consists of 4- ISA 100 x 100 x 10 mm suitably connected with lacing bars. The angles face inwards and the outside dimensions of the c/s are 350 x 350 mm.

1. Determine the safe axial compressive load for the column
2. Also design the lacing bars and their connection of angles.