DSS-MOD-II

Dr. G.C. BEHERA

TYPES OF WELDING

- Different process of weld connections are there,
- 1. Forge welding
- 2. Thermit welding
- 3. Gas welding
- 4. Resistance welding
- 5. Electric-arc welding..

ADVANTAGES OF WELDING

1. Weld connection does not need any hole in the plate. So there should not be any reduction of area that means the structure members are will be more effective to take the load.
2. In weld connection filler plate, gusset plate, connecting angles etc. are not used that means the total weight of the joint will be less in such cases.
3. Now weld joint should be economical as less materials are required.
4. Efficiency of weld joint is more than that of the riveted or bolted joint, because when bolt connection or riveted connections are used we create hole and because of creation of hole the net effective area of the plate is going to be reduced and this area under tension will be less and therefore, the capacity or strength of the bolt joint will be less compared to the weld joint.
5. The weld joint look better than the bulky riveted and bolted joint. If a particular shape of the joint is to be made , we can make by the weld joint, but in case of bolted joint or riveted joint it will look bulky, because of extra bolt and bolt or rivet will be added.

ADVANTAGES OF WELDING

6. In case of weld joints, speed of fabrication will be much faster than the riveted joint.
7. Complete rigid joint can be achieved through weld process.
8. Alternation and addition of the existing structure is easy as compared to riveted joint, because rivet joint is permanent in nature, it is difficult to add or alter the existing structure.
9. No noise is produced during the welding process as in case of riveting process.
10. Welding process requires less work space in comparison to rivet.
11. Any shape of joint is possible.

DISADVANTAGES OF WELDING

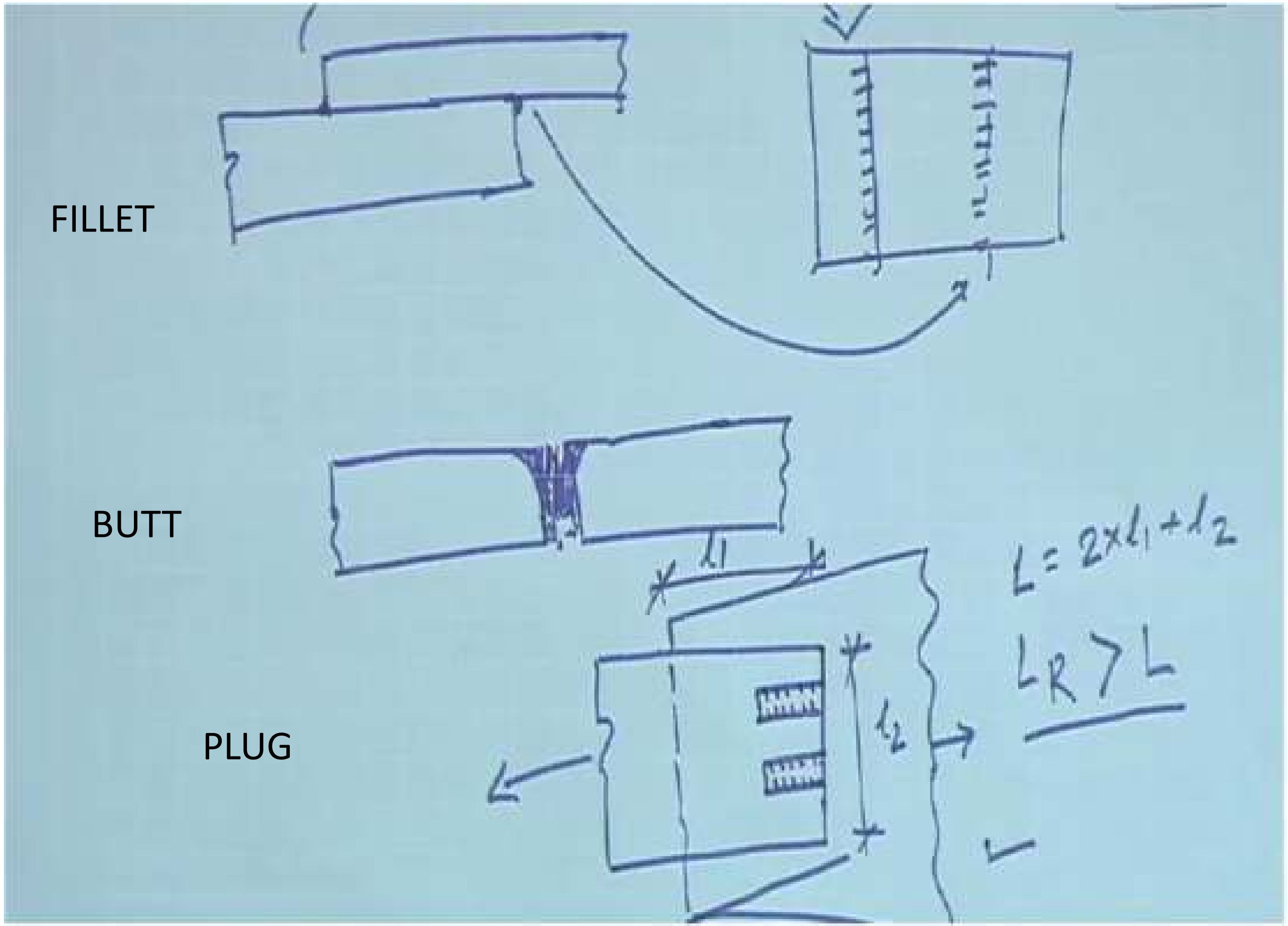
1. Weld joints are basically brittle in nature and therefore, it is means its fatigue strength is comparatively less.
2. Due to uneven heating and cooling of the member during the welding, the members may distort resulting additional stresses.
3. Skilled labor and electricity for welding is required.
4. Then there is no provision for expansion and contraction in welded connection and therefore, there is a possibility of cracks.
5. Iinspection of weld work is more difficult and costlier than riveting.
6. The defects like internal air pockets, slag inclusion, incomplete penetration are difficult to detect.

TYPES OF WELD

There are three types of weld we have; FILLET WELD,BUTT WELD, PLUG WELD

- Fillet weld is used when two members are lapped together. in case of two members joined in a different plane then fillet weld can be used.
- Butt welds we can use when the two members are joined in same plane, suppose this is a member and this is another member, we will be joining in same plane. So in this case we can provide butt weld and we can fill with weld material and it may have complete penetration, it may have incomplete penetration, different type of butt welds are there.

Another type of weld is plug weld, the plug weld is required when two embers are connected together having a limited length of the joint, say suppose one members is as shown and another member is connected with. Limited length means here if we see that length is this much say, this is l_1 and we will get this is l_2, so total length will be available length will be here$2\times l_1+l_2$ and suppose the force along the joint is so high that the required length LR is much more than the available length. So in this case we have to adjust the total length in between, so what we can do we can make a slot here and we can provide way. So in this way we can increase the length of the weld joint by the insertion of slot. So this is how one can make adjustment of the additional length with the insertion of slot.


FILLET
BUTT
PLUG
l1
l2
L = 2×l1 + l2
LR > L

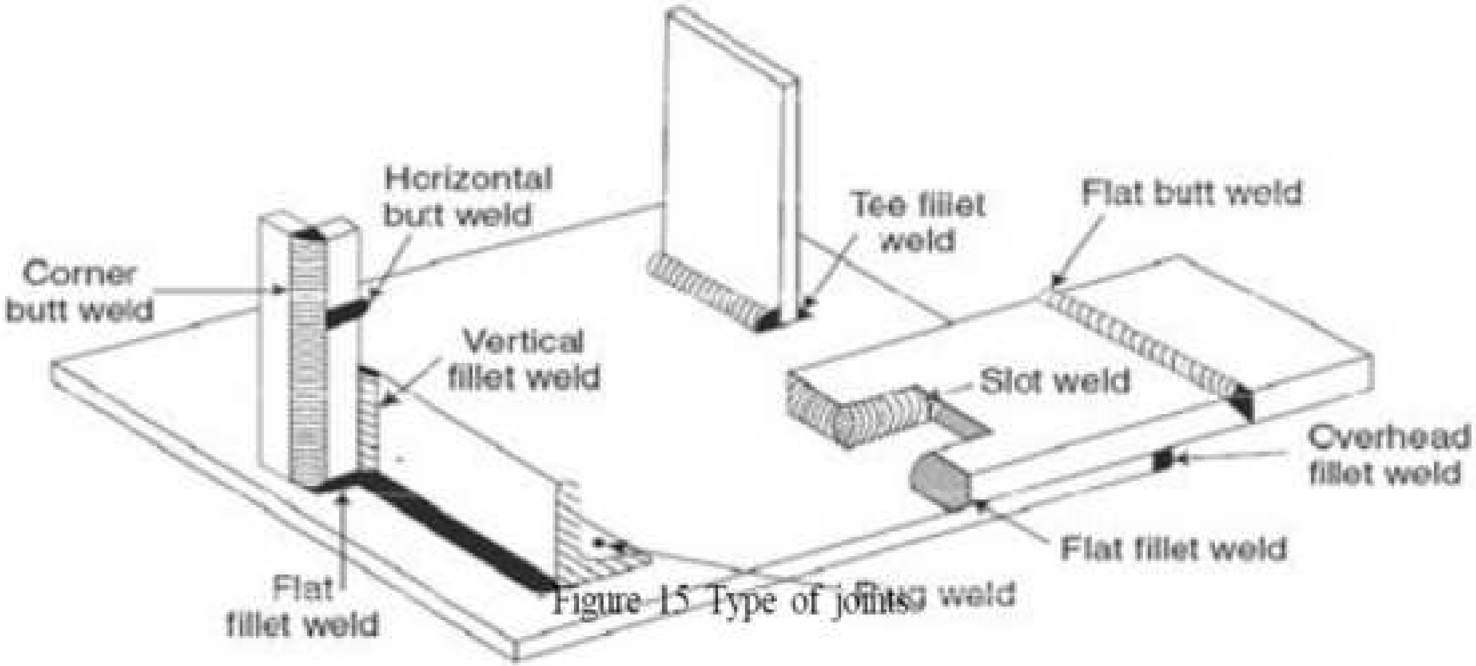


Figure 15 Type of joints

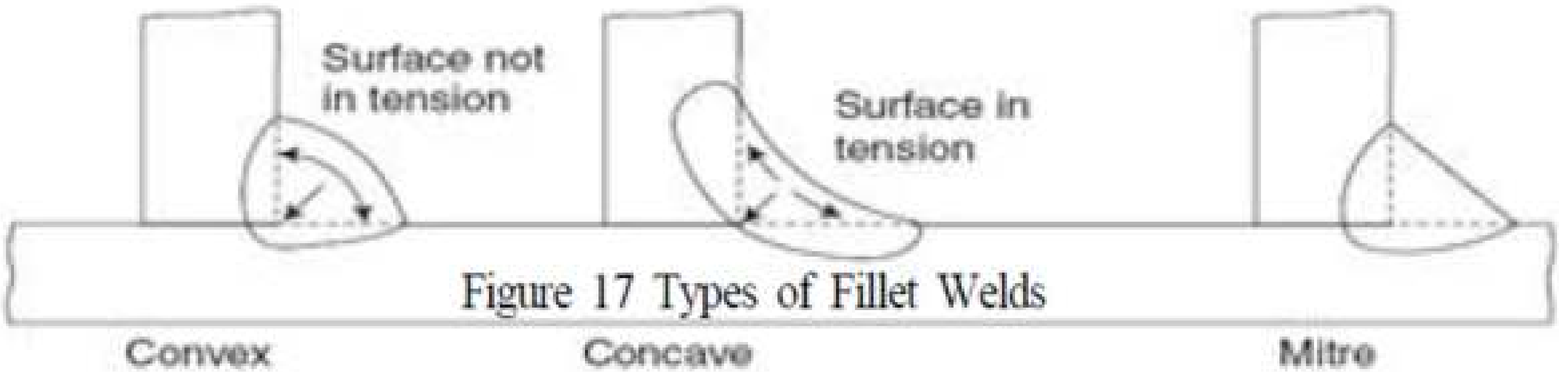


Figure 17 Types of Fillet Welds

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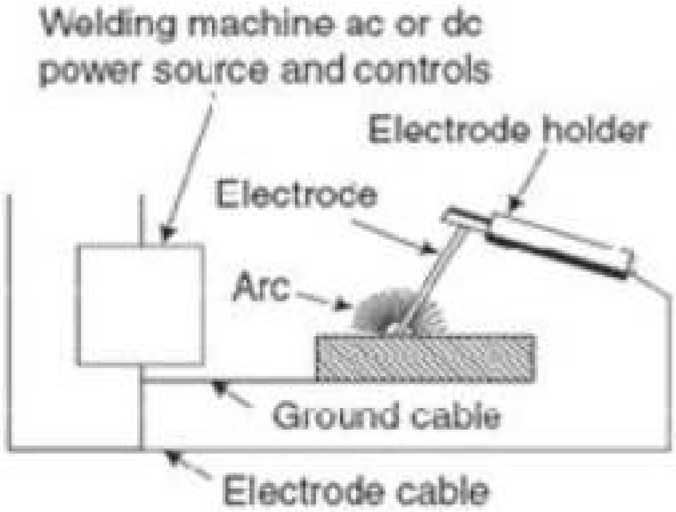
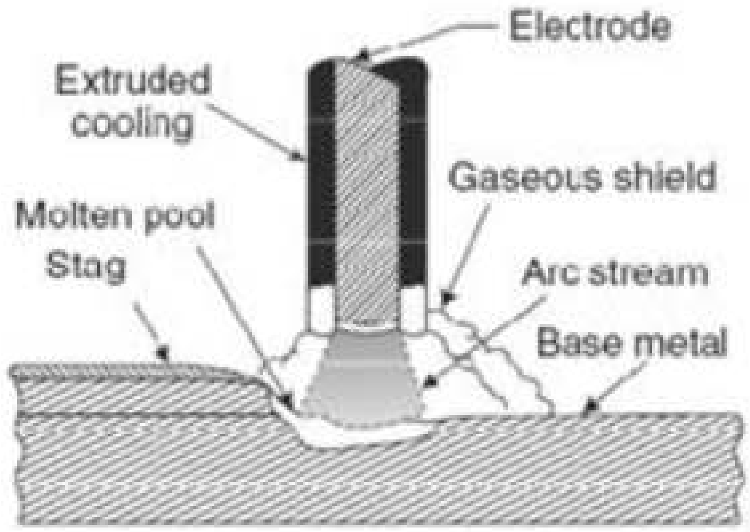


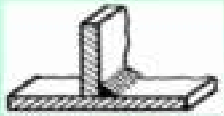
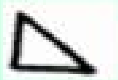
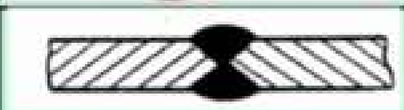
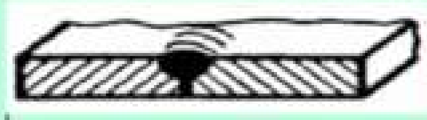
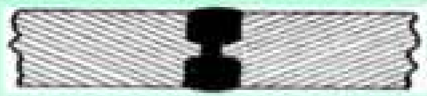
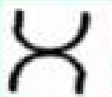
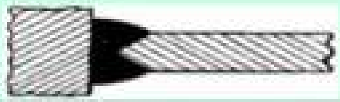
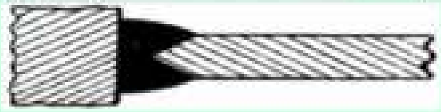
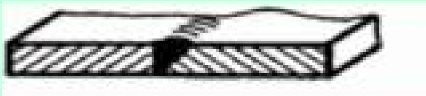
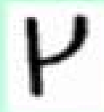
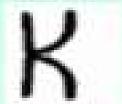


Welding consists of joining two steel sections by means of metallurgical bond between them by the application of pressure and/or fusion. The most commonly used welding process is fusion process.

The bond is produced, in fusion process, by melting the surfaces to be joined and then allowing them to solidify in to a single joint. The most commonly used welding process is the arc welding process .

In this process intense heat required (around 3600^0C) to melt the steel sections is produced by an electric arc. The tremendous heat at the tip of the electrode melts the base metal and the filler metal to form a pool of molten metal called crater which solidifies on cooling produce the joint required.

Basic types of welds and their symbols

Form of weld	Section	Symbol
Fillet		
Square butt		||
Single-V-butt		V
Double-V-butt		X
Single-U-butt		
Double-U-butt		
Single-bevel-butt		
Double -bevel-butt		K
Single-J-butt		
Double-J-butt		

Shape of weld	Symbol
Flat	—
Convex	⌒
Concave	‿

WELDING DEFECTS

- Defects in welding are inevitable in spite of using the good welding techniques, standard electrodes and preparation of joints. Some of the commonly observed welding defects (Figure) are detailed below.

Incomplete fusion

This happens when the base metal fails to completely fuse along with the weld metal. This can be caused by the rapid welding or by the presence of foreign material at the weld surface.

Incomplete penetration

This type of failure occurs due to the failure of the weld metal to penetrate the complete depth of the joint. This is often observed in single V and bevel joints. This can also happen while using electrodes of larger size than is required.

Porosity

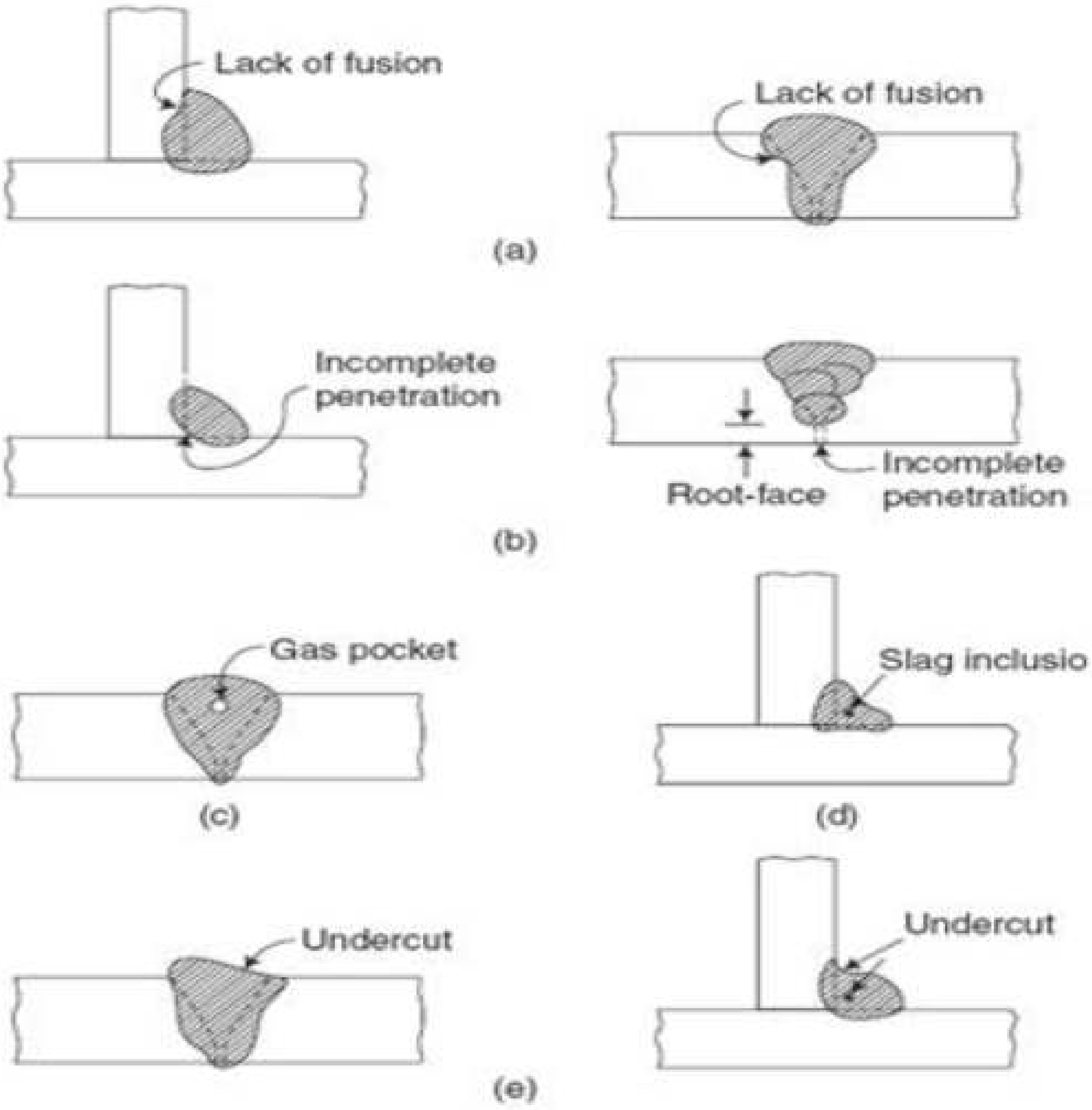
Porosity occurs due to the voids or gas pockets entrapped in the welds while cooling. This results in stress concentration and reduced ductility of the joint. This is mainly due to careless use of backup plates, presence of moisture in the electrodes, presence of hydrogen in the electrodes and excessive current.

Slag inclusion

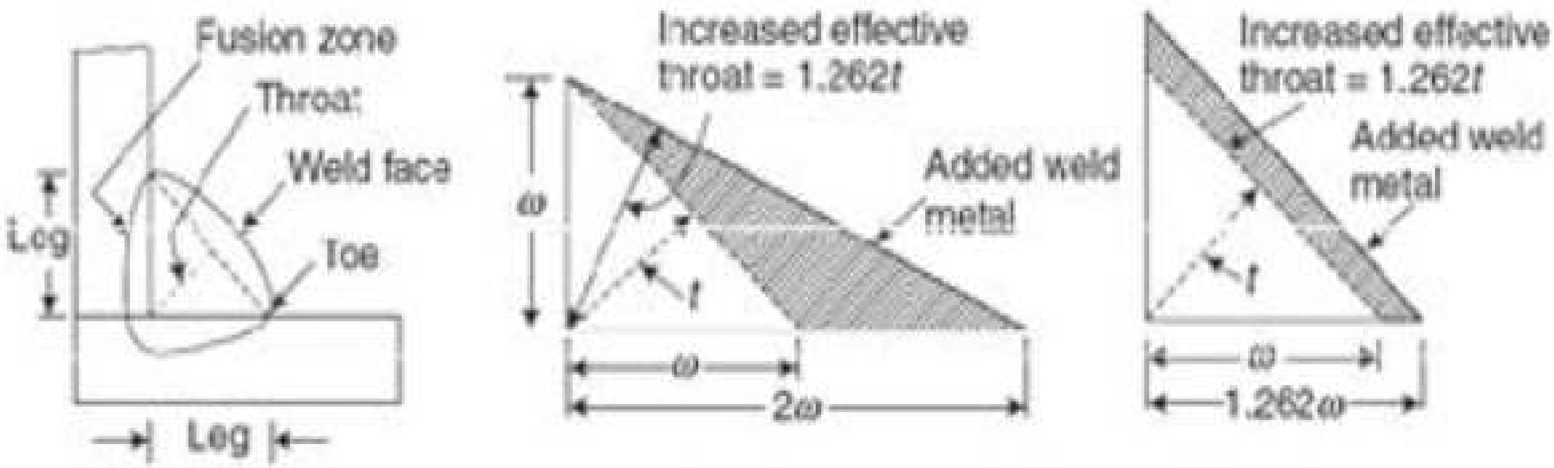
Slag inclusions are metallic oxides and other solid compounds which are sometimes found as elongated or globular inclusions. The slag, being lighter than the molten material of weld, usually rise to the surface and can be removed after cooling of the weld. However if the weld is cooled rapidly possibilities are there to trap these in the weld.

Undercutting

Undercutting is the local decrease in the thickness of parent metal at the weld toe. An undercut results in reduced section and acts like a notch. This can happen due to excessive current and/or long arc.


Lack of fusion
Lack of fusion
(a)
Incomplete penetration
Root-face
Incomplete penetration
(b)
Gas pocket
(c)
Slag inclusio
(d)
Undercut
Undercut
(e)

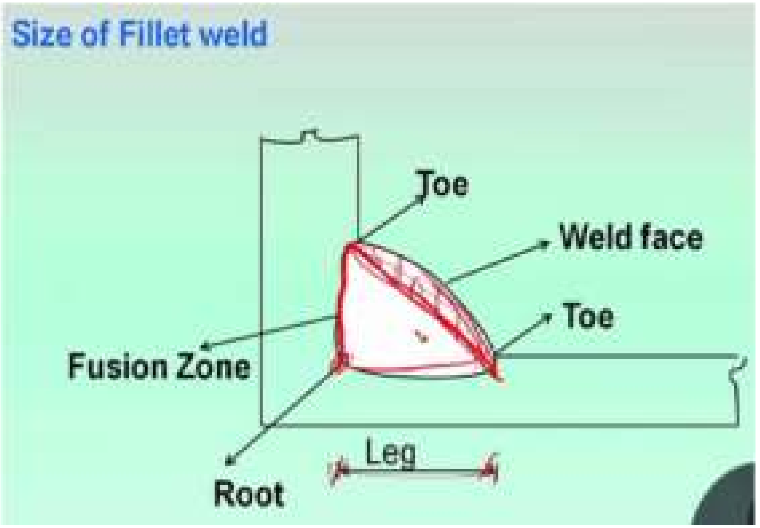
FILLET WELD



(a)

(b) Throat increased by 26%
Area increased by 100%

(c) Throat increased by 26%
Area increased by 59%



TERMS USED IN FILLET WELD

1. Size of the fillet weld
2. Throat of the fillet weld.
3. Effective length of the fillet weld
4. End return
5. Overlap
6. Side effect
7. Intermittent fillet weld
8. Single fillet weld
9. Permissible stress nad strength of fillet weld

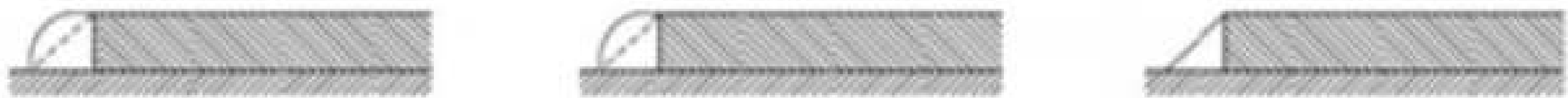
TERMS IN FILLET WELD

1.size of fillet weld.

from root to toe the length is called leg and this will be the size of the weld. This is a weld face and we will see that some extra deposit are there means if we make a straight line from this toe to this, we will see this is the extra deposit which is call reinforcement, right

Cl.10.5.2.1 of IS800-2007 specifies the size of normal fillets shall be taken as the minimum weld leg size. For deep penetration welds, where the depth of penetration beyond the root run is a minimum of 2.4 mm, the size of the fillet should be taken as the minimum leg size plus 2.4 mm. Figure 23 Cl.10.5.2.3 of IS800-2007 restricts minimum size of fillet welds to be 3 mm. The minimum size of the first run or of a single run fillet weld shall be as given in Table 21 of IS800-2007, to avoid the risk of cracking in the absence of preheating. Table 21 if IS800-2007 is reproduced for ready reference as Table below.

- As per Cl.10.5.8.3 of IS800-2007, where the size specified for a fillet weld is such that the parent metal will not project beyond the weld, no melting of the outer cover or covers shall be allowed to occur to such an extent as to reduce



(a) Desirable (b) Acceptable because of full throat thickness (c) Not acceptable because of reduced throat thickness

Table Minimum size of first run or of a single run fillet weld

Sl. No.	Thickness of thicker part (mm)		Minimum size (mm)
	Over	Up to and including	
1	-	10	3
2	10	20	5
3	20	32	6
4	32	50	8 of 1st run, 10 for minimum weld size

The leg length of the weld, is the distance from the root of the weld to the toe of the weld measured along the fusion face. minimum size of the weld we can define on the basis that if the thickness of thinner part.

When the minimum size of the fillet weld is greater than the thickness of the thinner particularly, the minimum size of the weld should be taken as the thickness of the thinner part. Minimum size cannot become more than the thickness of the thinner part.

If the thicker part is more than 50 mm thick special precaution like preheating etc. will be taken care and as per clause 10.5.2.1. For deep penetration weld where the depth of penetration beyond the root run is minimum of 2.4 mm, the size of the filet weld is minimum leg size plus 2.4 mm and this is about the minimum size of the fillet weld.

Now the maximum size is also defined in the code that is the thickness of the thinner part minus 1.5mm. Similarly, in case of angle the maximum size of the fillet weld be three fourth of the nominal thickness of the angle.

- Cl.10.5.8.5 of IS800-2007 specifies for end fillet weld, normal to the direction of force shall be of unequal size with a throat thickness not less than 0.5t, where t is the thickness of the part, as shown in Fig. 19 of IS800-2007.

The difference in thickness of the welds shall be negotiated at a uniform slope. Fig. 19 of IS800-2007 is reproduced here. shows the leg length of fillet weld for various cases. Then another term we will use in fillet weld is end return, then overlap, then side fillet,

- intermittent fillet, single fillet weld and permissible stress and strength of fillet weld we have to

THROAT THICKNESS

Throat Thickness:

clause 10.5.3.1 of IS 800, it is told that the throat thickness will not be less than 3 mm and generally not exceeding 0.7t or 1.0t under special circumstances where t is the thickness of thinner plate?

For the angle other than right-angled fillet weld the value of throat thickness is given as:

- *T = KS*
- Where,
- T = Throat thickness of weld
- *K is a constant depends upon the angle between fusion face*
- *S is Thickness of the weld*

$$BA = BC = S$$
$$\therefore AC = \sqrt{2}S$$
$$AB^2 = AD^2 + BD^2$$
$$\Rightarrow BD = \sqrt{AB^2 - AD^2}$$
$$\Rightarrow BD = \sqrt{S^2 - \left(\frac{S}{\sqrt{2}}\right)^2} \Rightarrow BD = \frac{S}{\sqrt{2}}$$
$$\therefore BD = T = 0.707S \approx 0.7S$$

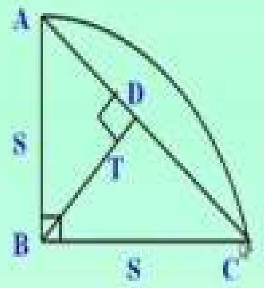


Table Values of K for different angles

Angle between fusion faces in degrees	**60-90**	**91-100**	**101-106**	**107-113**	**114-120**
K	0.7	0.65	0.60	0.55	0.50

EFFECTIVE LENGTH

So effective length can be found out from the area of the weld for which specified size of the effective throat thickness of the weld exist.

total length will be l_e + 2S,

Effective Length of Weld is considered in accordance with Cl.10.5.4 of IS800-2007. As per Cl.10.5.4.1, the effective length of fillet weld shall be taken as only that length which is of the specified size and required throat thickness. In practice the actual length of weld is made of the effective length shown in drawing plus two times the weld size, but not less than four times the size of the weld. So the effective length of the weld is considered as actual length minus twice the weld size.

As per Cl.10.5.1.1, Fillet welds terminating at the ends or sides of parts should be returned continuously around the corners for a distance of not less than twice the size of the weld, unless it is impractical to do so. This is particularly important on the tension end of parts carrying bending loads.

because we assume that the strength will be carried out by the length which is called effective length, but we have to provide little more in two side to make sure the strength is being carried out by this le. So the total length we will make little higher, right and this effective length should not be less than four times the size of the weld.

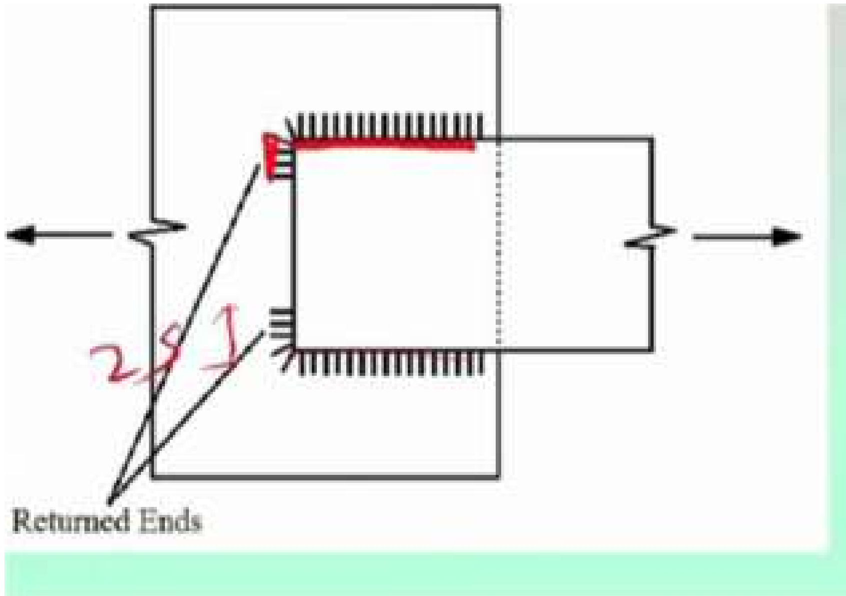
Effective area of fillet welds

The effective area of fillet welds is obtained as the product of effective throat thickness and effective length.

END RETURN

Fillet welds terminating at the ends or sides of parts should be returned continuously around the corners for a distance of not less than twice the size of the weld, unless it is impractical to do so. This is particularly important on the tension end of parts carrying bending loads.

The overlap of lap joints should not be less than four times the thickness of the thinner plate or 40 mm whichever is less



DESIGN STRENGTH OF WELD

- Design Strength of Weld

$$P_{dw} = \frac{f_u L_w t_e}{\sqrt{3}\gamma_{mw}}$$

Where

f_u is the ultimate stress of the weld metal and

L_w is the effective length not the total length and

t_e is the effective throat thickness

γ_{mw} 1.25 for shop weld, 1.5 for site weld.

- **Long joints**

When the length of the welded joint, l_j of a splice or end connection in a compression or tension element is greater than 150 t_t the design capacity of weld, fwd shall be reduced by the factor

PROBLEM

A tie member of a roof truss consists of ISA 100×75×8 of Fe410 grade, is welded to a 10 mm gusset plate. Design the welded connection to transmit a tensile load, T. Assume connection are made in the workshop.

Solution:

Minimum weld size = 3 mm [Table 21, IS 800]

Maximum weld size =3/4 ×8 =6 mm [clause 10.5.8.2, IS 800]

Let us adopt 5 mm thick fillet weld.

Throat thickness, te = 0.7 × 5 = 3.5 mm

For ISA 100×75×8, Gross area, A_g = 1336 mm^2 , C_z = 31mm

Full strength of the angle=fy*Ag/γ_{m0}=250*1336/1.1=303.64 kN

Strength of 5-mm weld, $$P_{dw} = \frac{f_u L_w t_e}{\sqrt{3}\gamma_{mw}}$$

$$303.64 * 1000 = \frac{410 * L_w * 0.7 * 5}{\sqrt{3} * 1.25}$$

$$L_w = 458.11\ mm$$

Total length of weld=458.11 mm

Per unit mm of weld force is

$$P_{dw} = \frac{f_u L_w t_e}{\sqrt{3}\gamma_{mw}}$$

$$P_{dw} = \frac{410 * 1 * 3.5}{\sqrt{3} * 1.25} = 0.6628\ kN$$

P2=0.6628*100=66.28 kN

Taking moment of all forces on bottom of P3,

$P*31=P_2*50+P_1*100$

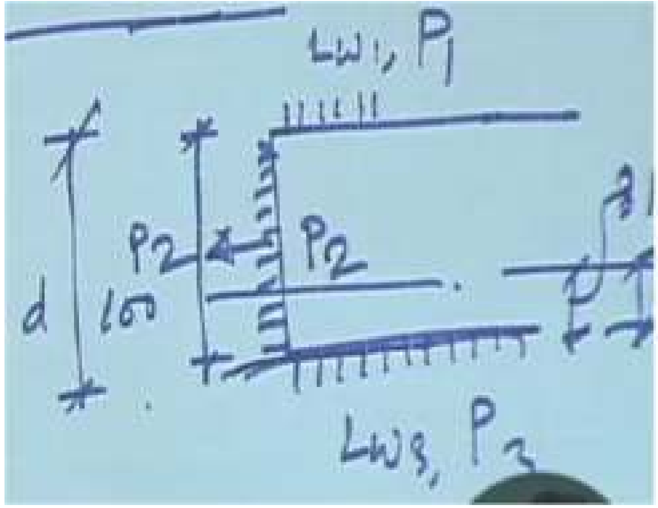
$303.64*100=66.28*50+P_1*100=60.98$ kN say 61 kN

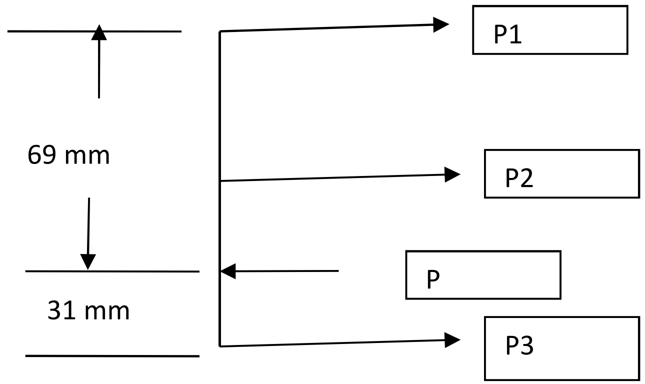
P_3=303.64-66.2-61=176.44 kN

P_1=61 kN, L_{w1}=61/0.6628=92.03 mm-92 mm

P_3=303.64-66.2-61=176.44 kN

L_{w3}=176.44/0.6628=262.02mm

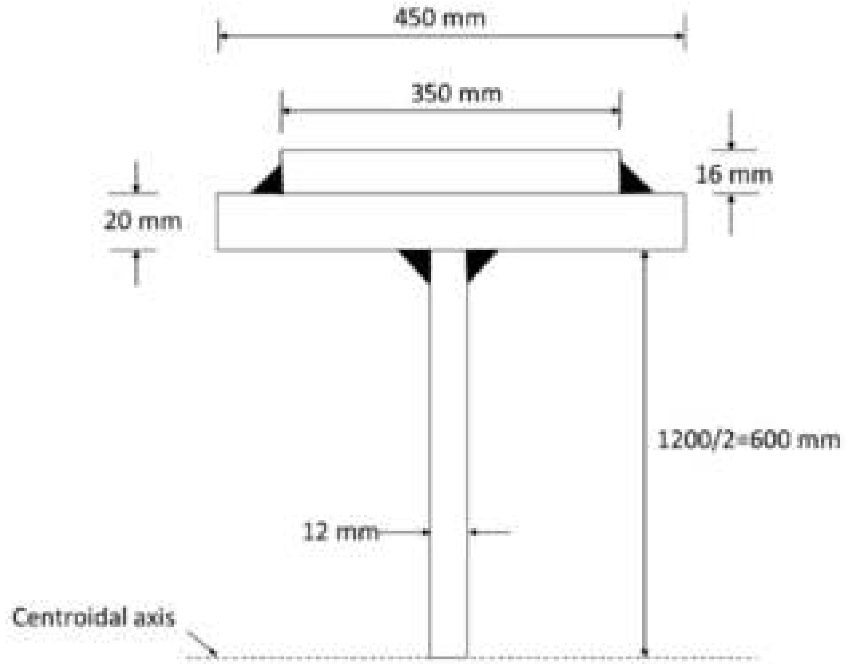




PROBLEM-2

Design a suitable fillet weld to connect web plate to flange plate and flange plate to flange cover plate of a built-up girder as shown in the figure, for the following data. Assume

- shop welding
- Web plate: 1200 mm × 12 mm
- Flange plate: 450 mm × 20 mm
- Flange cover plate: 350 mm × 16 mm
- Maximum Factored shear force: 1600 Kn
- For Fe 410 steel: fu=410MPa
- For shop weld:γ_{mw}=1.25
- Permissible shear stress=f_u/[sqrt(3)*1.25]=410/ /[sqrt(3)*1.25]= 189.37 N/mm^2

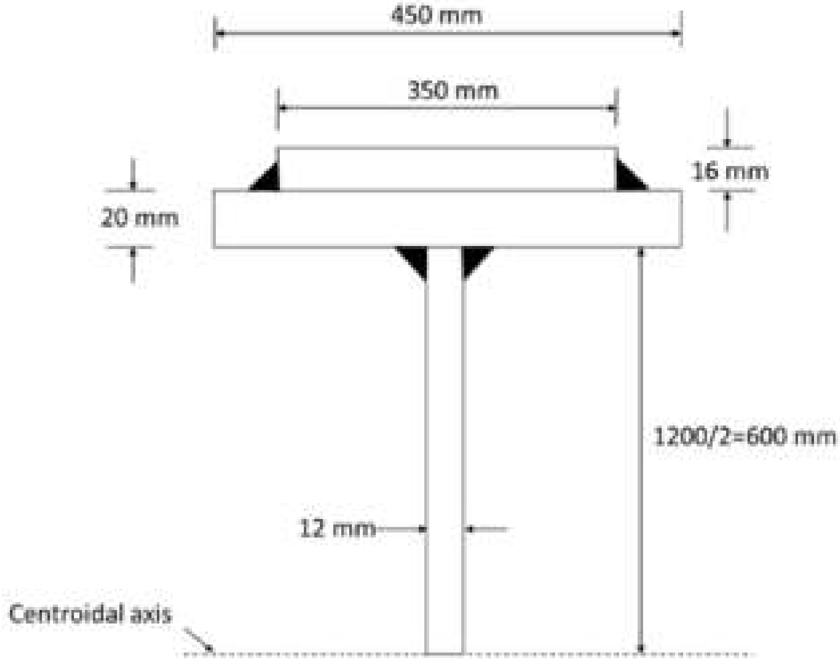


- **Connection of web plate to flange plate:**
- Size of weld: Minimum = 5 mm **[Table 21, IS 800]**
- Maximum = 12 – 1.5 =10.5 mm **[clause 10.5.8.1, IS 800]**
- Let us provide 7 mm size of fillet weld.
- Effective throat thickness of weld
- =0.7*7=4.9mm
- *For both sides=9.8 mm*

$$A\bar{y} = 450 * 20(600 + 10) + 350 * 16 * (600 + 20 + 8)$$
$$= 900.68 * 10^4 mm^3$$
$$I_{zz} = 2\left[\frac{350 * 16^3}{12} + 350 * 16 * 628^2 + \frac{450 * 20^3}{12} + 450 * 20 * 610^2\right] + \frac{12 * 1200^3}{12} = 12.8 * 10^9 mm^4$$

- Shear Stress

$$\frac{VA\bar{y}}{I_{zz}t_e} = \frac{1600 * 10^3 * (900.68 * 10^4)}{12.8 * 10^9 * 9.8} = 114.9\frac{N}{mm^2} < 189.37\frac{N}{mm^2}$$



BUTT WELD

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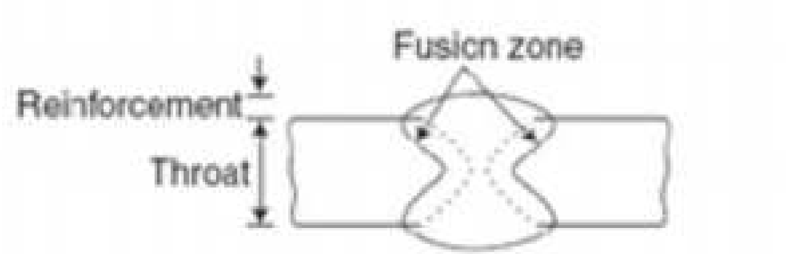
BUTT WELD OR GROOVE WELD

So butt weld is basically a type of weld when two plates are joined at the same plane. when a T joint is desired in that case also we can use this butt weld.

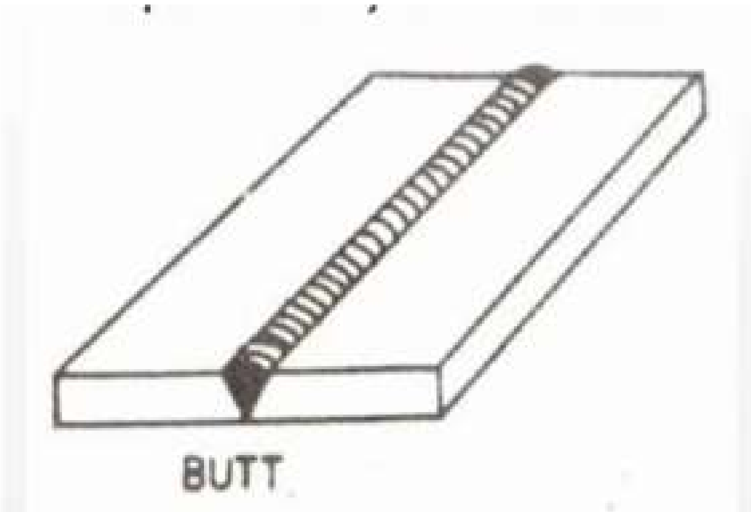
We require the followings

1. Size of the weld
2. Effective length of the weld,
3. Effective area of the butt weld and
4. Reinforcement



Square groove welds are usually employed for sections of thickness up to 8mm. If sections with more than 8mm thickness, U, V, double U or double V butt welds are used.

Size of welded joint is usually specified by throat dimension. This is also called effective throat thickness. Groove welds may be classified in to full penetration groove welds. Complete penetration is difficult to achieve in the case of single U, V, J and bevel welds. However, this can be achieved by using backing strips as shown in Figure.



- # SIZE OF BUTT WELD

As per Cl.10.5.3.3 of IS800-2007, the effective throat thickness of a complete penetration butt weld shall be taken as the thickness of the thinner part joined, and that of an incomplete penetration butt weld shall be taken as the minimum thickness of the weld metal common to the parts joined, excluding reinforcements. However in the case where full penetration groove welds cannot be achieved, an effective throat thickness of 7/8th of the thickness of thinner member is used. But for calculating the strength of the connection, a throat thickness of 5/6th of the thinner member is usually assumed.

Full penetration

Partial Penetration

Now the difference in thickness between two plates should not be more than 25 percent of the thickness or or 3 mm whichever is more. If difference is more than 25% then a tapering is required of 1 in 5.

EFFECTIVE LENGTH

Now the effective length is calculated in a similar way as we have done in case of fillet weld the effective length will be based on the effective area. It is the area of the butt weld for which the specified size that means the effective throat thickness of the weld exists that means the length in which the effective size of the throat thickness are existing that length will be the butt weld length and the minimum length of the butt weld should not be less than 4S where S is the size of the weld.

So minimum length has to be 4 times the size of the weld and the welder must provide an additional length of 2S to get the overall depth. So in drawing we will show the effective length but weld length has to be added 2S for designing.

EFFECTIVE AREA OF GROOVE WELDS

Effective area of weld is obtained as the product of effective length of weld and effective thickness (throat thickness) of weld. As per Cl.10.5.4.2 of IS800-2007, the effective length of butt weld shall be taken as the length of the continuous full size butt weld, but not less than four times the size of the weld.

- **Design strength of Groove welds**

Cl.10.5.7.1.2 of IS800-200 deals with the strength of Butt welds. As detailed in this clause of the code, Butt welds shall be treated as parent metal with a thickness equal to the throat thickness, and the stresses shall not exceed those permitted in the parent metal. Hence the following equations may be used for the design of butt welds.

REINFORCEMENT

Reinforcement is basically an extra deposit over the plate and this is also necessary for effective strength of the joint and this is least 10 percent greater than the thickness of the weld material. So the extra deposit of the metal above the thinner plate between 1 mm to 3 mm is not considered for stress calculation or design. The reinforcement is provided to increase the efficiency of the joint. So reinforcement has not been taken considered for calculation of strength but to make it efficient we need to provide the reinforcement and what will be the permissible stress in the butt weld that also defined, i.e. the stresses of butt weld should be taken equal to the stress of the parent metal in case of shop weld. Therefore, stress of the butt weld will be the stress of the parent metal. Stress of the butt weld we can consider, as a stress of the parent metal but the value will be reduced to 80 percent when it is done in field.

- DESIGN STRENGTH The design strength of butt weld in **tension or compression is governed by yield** $T_{dw} = \frac{f_y L_W t_e}{\gamma_{mw}}$
- L_w = effective length of weld in mm
- f_y = smaller of yield stress of weld and parent material in MPa
- t_e = effective throat thickness in mm
- γ_{mw} = partial safety factor = 1.25 for shop welding and
- = 1.5 for site welding

DESIGN STRENGTH The design strength of butt weld in **SHEAR**

$$v_{dw} = \frac{f_y L_W t_e}{\sqrt{3}\gamma_{mw}}$$

Now stress due to individual force when subjective to different types of compressive or tensile force then the stress we can calculate by the force divided by the effective area.

Effective area = $t_e * L_w$.

When subjected to either compressive or tensile due to axial force or shear force alone, the stress in the weld is given by,

f_a or $q = P/(t_e * L_w)$

f_a = calculated normal stress due to axial force in N/mm^2

q = shear stress in N/mm^2

P = force transmitted (axial force N or shear force Q)

t_e = effective throat thickness of weld in mm

L_w = effective length of weld in mm

When subjected to combined stresses, so in this case we have to find out the equivalent stress.

$$f_e = \sqrt{f_a^2 + 3q^2} \leq \frac{f_u}{\sqrt{3}\gamma_{mw}}$$

- f_a = normal stress, compression or tension, due to axial force or bending moment.
- q = shear stress due to shear force or tension.

If bearing stress combined with Bending and shear, then equivalent stress will be

$$f_e = \sqrt{f_b^2 + f_{br}^2 + f_b f_{br} + 3q^2}$$

- f_e = equivalent stress
- f_b = calculated stress due to bending in N/mm2
- f_{br} = calculated stress due to bearing in N/mm2
- q = shear stress in N/mm2

PROBLEM: Two plates of thickness 12 mm and 10 mm are to be jointed by a groove weld. The joint is subjected to a factored tensile force of 250 kN. Assuming an effective length of 150 mm, check the safety of the joint for

(a) Single-V groove weld joint and,

(b) Double-V groove weld joint.

Assume Fe 410 grade steel plates and that the welds are shop welded.

Solution:

(a) Single-V groove weld:

Throat thickness=te=5/8*t=5/8*10= 6.25 mm

Effective length of weld=L_w=150 mm,

For tensile force: $$\frac{f_y L_w t_e}{\gamma_{mw}} = \frac{150 * 6.25 * 250}{1.25} = 187.5\ kN < 250\ kN$$

- (b) Double-V groove weld:

Throat thickness = thickness of thinner plate = 10 mm

Strength of weld

$$\frac{f_y L_w t_e}{\gamma_{mw}} = \frac{150 * 10 * 250}{1.25} = 300 kN > 250\ kN$$

A joint is subjected to a factored shear force of 300 kN. Assuming single-V groove weld joint find the effective length of the weld if the thickness of thinner plate is 8 mm. Assume Fe 410 grade steel plates and that the welds are shop welded.

Solution:

For single-V groove weld, effective throat thickness =5/8*t=(5/8)*8=8 mm

$$\frac{f_y L_w t_e}{\sqrt{3}\gamma_{mw}} = V_{dw} = \frac{250 * L_w * 5}{\sqrt{3}1.25} = 300 * 1000$$

L_w=520 mm

L=L_w+2s=520+2*8=536 mm

Design of Plug and Slot Weld

If the length of the joint is limited and if we have higher values of tension or compression force then it is difficult to adjust the entire length on this limited length. So, in that case we have to cut some portion of the overlapping portion in terms of slot or plug to make adjust of that additional length.

When the slot is small and completely filled with weld metal then it is called plug weld, but if the periphery of the slot is filled with weld metal then it is called slot weld.

Specifications:

IS816-1969

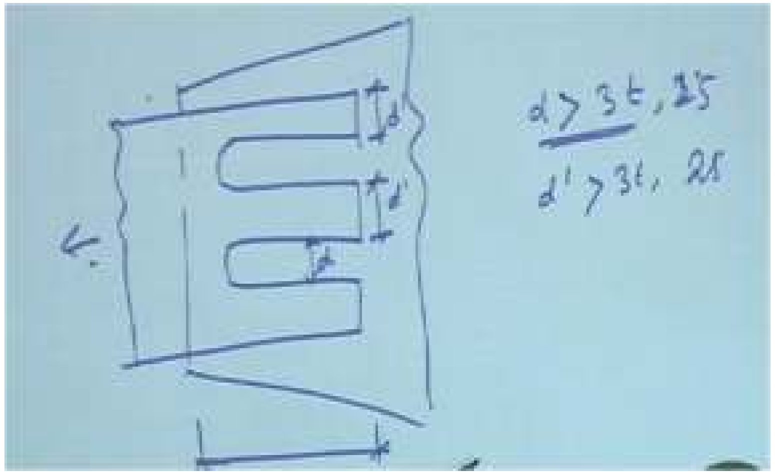
The width or the diameter of the slot should not be less than three times the thickness of the part in which the slot is formed or 25 mm whichever is greater.

The distance between the edge of the part and edge of the slot or plug or between the adjacent slot or plugs should not be less than three times the thickness of thinner member or 25 mm whichever is greater.

- SLOT WELD

Corners at the enclosed ends should be rounded to a radius not less than one and a half times the thickness of the upper plate or 12 mm whichever is greater. So corner at the enclosed end we have to make the rounded corner with a radius not less than one and half times the thickness of the upper plate or 12 mm thickness whichever is greater.

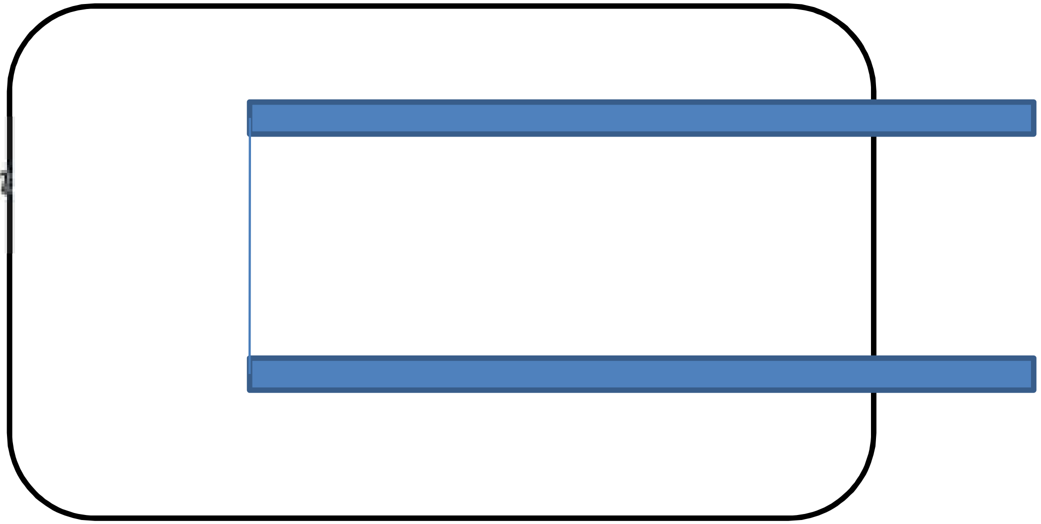
The design stress on a plug or slot weld will be same as that in fillet weld and is specified in clause 10.5.7.1.1 of IS800:2007.



PROBLEM: An ISMC 300 @ 363 N/mm is used to transmit factored force 800 kN. The channel section is connected to a 12mm thick gusset plate. Design a fillet weld connection if the overlap is limited to 300 mm. Use slot welds if required. Assume site welding.

- The properties of ISMC 300 are,
- A= 4630 mm2 , t_f = 13.6 mm , t_w= 7.8 mm
- maximum size of weld= 7.8 - 1.5 = 6.3 mm. (Cl. 10.5.8.1)
- Minimum size of the weld = 3 mm (Table 21 IS 800:2007)
- Adopt a 6-mm size weld.
- Throat thickness = KS = 0.7 x 6 = 4.2 mm.
- Strength of the weld per mm =

$$\frac{f_U L_w t_e}{\sqrt{3}\gamma_{mw}} = \frac{410 * 4.2}{\sqrt{3} * 1.25} = 663\ N/mm$$



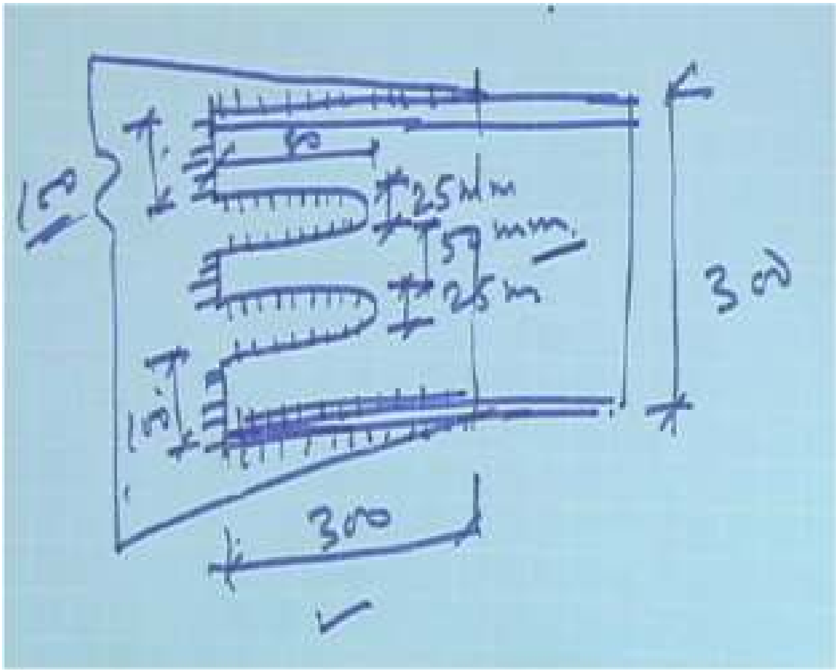
Required length of weld = 800×1000/663 = 1207 mm

The maximum length of weld that can be provided in the channel = 300 × 2 + 300 = 900 mm < 1207 mm

Hence, use two slot welds of width 25 mm (3t = 3 × 7.8 = 23.4 mm or 25 whichever is greater)

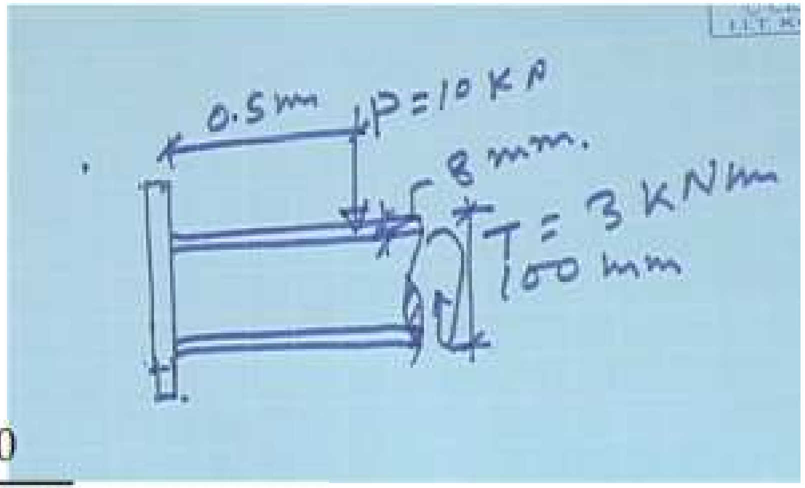
Assume the length of the weld is x mm, then,

- 1207 = 2 × 300 + 300 + 4x
- Or, x = 76.75 mm
- Hence, provide 80 mm × 25 mm slots, two in numbers as shown in the figure



PLUG WELD

A pipe of 100 mm diameter and 8 mm thick is connected to a 16 mm thick plate with fillet weld. It is subjected to a vertical factored load of 10 kN at a distance of 0.5 m from the welded end. It is also subjected to a factored twisting moment of 3 kNm. Find the size of the weld assuming shop welding and steel of grade to be Fe410.



Here, fu = 410 MPa , γ_{mw} = 1.25

Permissible shear stress,N/mm2

Hence, P = 10 kN

M = P. e = 10 x 0.5 = 5 kNm

T = 3 kNm

Polar moment of inertia,

=

$$Permissible\ shear\ stress = \frac{f_u}{\sqrt{3}\gamma_{mw}} = \frac{410}{\sqrt{3} * 1.25}$$

$$= 189.37\ N/mm^2$$

$$Polar\ moment\ of\ Inertia = 2\pi r^3 * t = 2 * \pi * 50^3 * t$$

$$= 785714\ t\ mm^4$$

$$1. Shear\ stress\ due\ to\ direct\ load = \frac{P}{2\pi rt} = \frac{10 * 1000}{2\pi 50t}$$
$$= 31.8181/\ t\ N/mm^2$$

(2) Shear stress due to twisting moment

$$2. Shear\ stress\ due\ to\ Torque = \frac{T * r}{I_p} = \frac{3 * 10^6 * 50}{785714t}$$
$$= 190.90/t$$

(3) Normal stress due to bending= M_z*y/I_{zz}=5*10^6*50/(392857t)=636.36 N/mm^2

$$I_{zz} = I_p/2 = 392857\ t\ mm^4$$

Resultant shear stress due to direct shear and due to torsion=Sqrt of (q_1^2+q_2^2)= SQRT(31.81^2+190.90^2)= 193.54/t,

Equivalent stress due to normal and shear stress,

$$f_e = \sqrt{f_a^2 + 3q^2} = \sqrt{(636.36/t)^2 + 3 * (190.90/t)^2}$$
$$= 719.26/t$$

Thickness must be more than

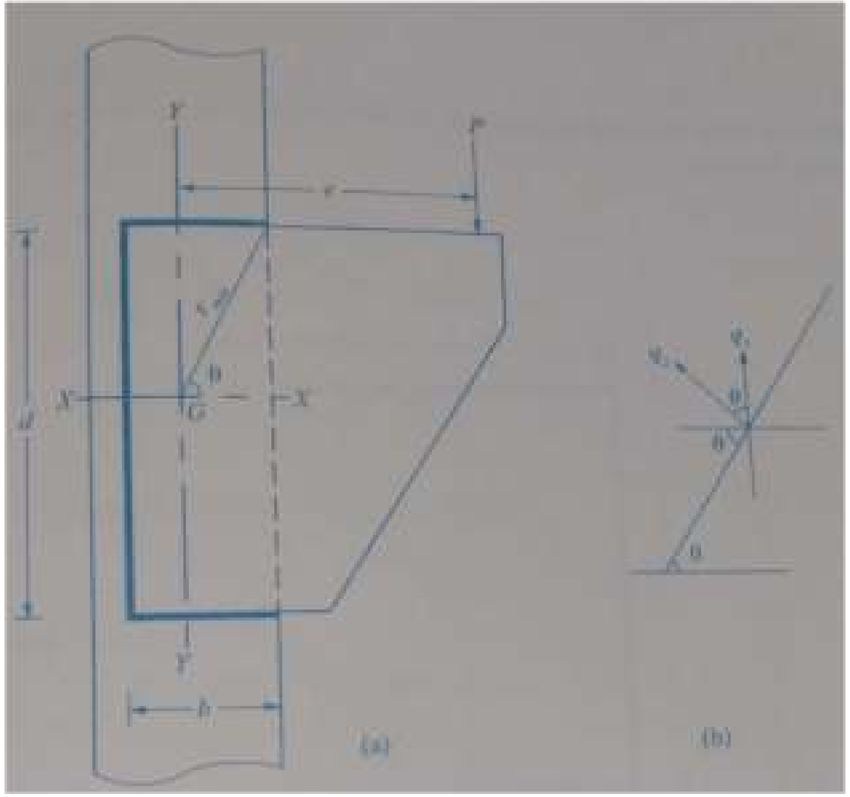
$$\frac{719.26}{t} \leq 189.37, t \geq 3.80\ mm$$

Thickness=k*Size of the weld=0.70*S , S=5.42 mm
Take the size of the weld 6 mm

ECENTRIC CONNECTION-WELD

Dr.G.C. BEHERA

Moment in the of weld



Find out the cg of weld group.

Find out what will be the stresses acting on each portion of the weld.

Similar as bolt connection, maximum stresses will found in outer most.

Shear stress due to direct load=f_s=P/Lw*t= P/[(2b+d)*t]

Now shear stress due to bending can found from the following formulae=f_b =[M=P*e]*r/(I_p)

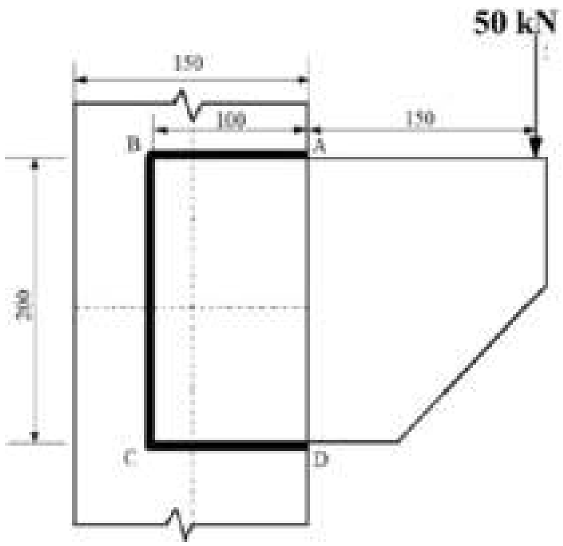
Where M is the moment which is calculated as P×e and r is the radial distance of the welding point from the cg of the weld. Now unless we know the distance of the weld we cannot find out the value of e, e is the distance between load and the cg of the weld group

Ip=I_{xx} + I_{yy}.

Total effective stress= $$f_e = \sqrt{f_s^2 + f_b^2 + 2 * f_s f_b \cos\theta}$$

Where θ is the angle between f_s and f_b

A bracket is subjected to a load of 50 kN and is connected to a stanchion by welding as shown in the figure. Find the size of the weld so that the load can be carried safely.



Solution:

To find the CG of the weld

find out the cg.

Distance of the CG of the welded area from weld line BC,

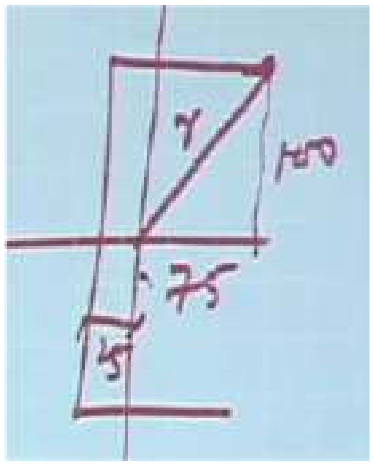
2*100*50*t/[(200+200)*t]=25 mm

Eccentricity of the load = 150 + 100 – 25 = 225 mm

$$I_{xx} = \frac{1}{12} * t * 200^3 + 2 * 100t * 100^2 = 2.67 * 10^6 t\ mm^4$$

$$I_{yy} = 200t * 25^2 + 2 * \frac{1}{12} t100^3 + 2 * 100t * (50 - 25)^2$$
$$= 4.17 * 10^5 t\ mm^4$$

$$I_{zz} = I_{xx} + I_{yy} = 30.87 * 10^5 t\ mm^4$$



A=400t   r=SQRT(100^2+75^2)=125 mm

M=50*(0.15+.075)= 11.25 kNm

Direct shear=f_s=50*1000/(400t)=125/t

Shear stress due to bending f_b=

=(M/I_{zz})*r=11.25*10^6*125/(30.87*10^5)=455.54/t

Cosθ=75/125=0.6

$$f_e = \sqrt{(\frac{125}{t})^2 + \left(\frac{455.54}{t}\right)^2 + 2 * \frac{125}{t} * \frac{455.54}{t} * 0.6}$$

f_e=539.88/t N/mm^2

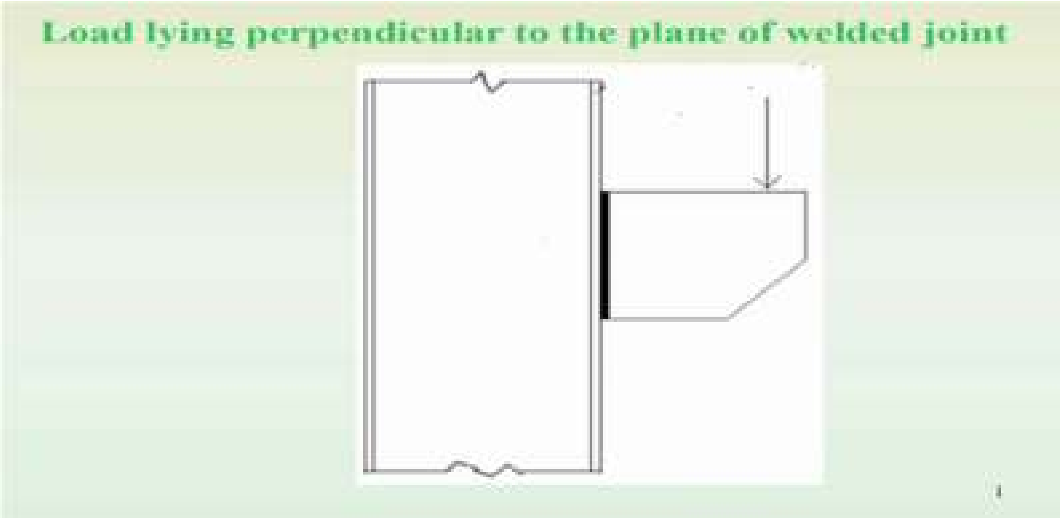
$$Permissible\ shear\ stress = \frac{f_u}{\sqrt{3}\gamma_{mw}} = \frac{410}{\sqrt{3} * 1.25}$$
$$= 189.37\ N/mm^2$$

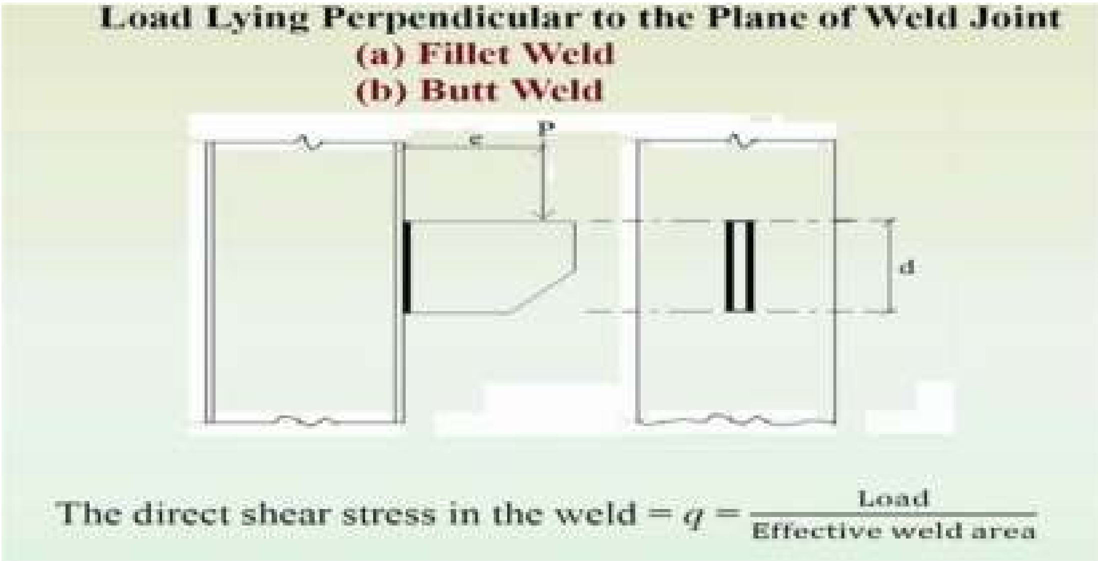
- 539.88/t<189.37
- t>[539.88/189.37 =2.85]
- t=0.7s   s=2.85/0.7=4.07 mm say5 mm is the size of weld
- Minimum size of weld=3 mm maximum=

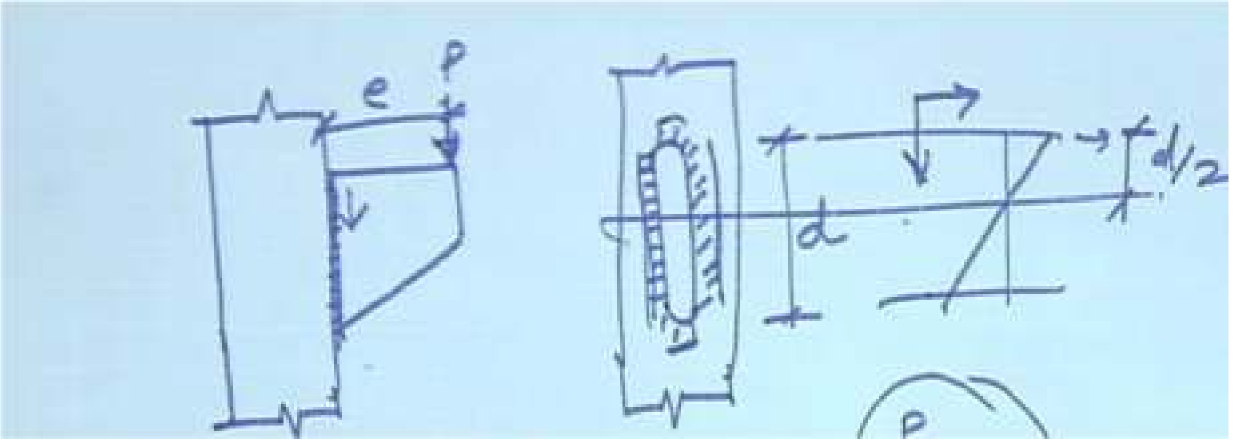
ECCENTRIC CONNECTION (LOAD LYING PERPENDICULAR TO PLANE OF WELDED JOINT)

Dr. G.C.Behera

WELD:where load is lying perpendicular the plane of joint but the connection is welded connections. So in this lecture we will discuss when a welded joint is under eccentric load and load is perpendicular to that welded joint.







The direct shear stress in the weld = q =Load/ Effective weld area

(a) Fillet Weld

1. The shear stress in the fillet weld=q=P/(Lw*t_t)=P/(2d*t_t)

Where, P is the load and e is the eccentricity, d is the depth of bracket plate/welding depth, L_w is total effective length of weld, t_t is the throat thickness of the fillet weld.

2. The stress due to bending=P*e/(I/y)

I=2*t_t*d^3/12, y=d/2 M=P*e

$$f_b = \frac{Pe}{2 * \frac{1}{12} t_t d^3} * \frac{d}{2} = \frac{3Pe}{t_t d^2}$$

$$Resultant\ Stress = f_e = \sqrt{q^2 + f_b^2} \leq weld\ strength$$

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

- **Design Steps:**

i. Select a suitable size of weld s and then compute throat thickness t_t and weld strength,

$$Strength\ of\ weld = R_w = \frac{f_u}{\sqrt{3}\gamma_{mw}}$$

ii. strength of weld per unit length=

iii. Calculate the depth of weld using the following expression:

$$d = \sqrt{\frac{3Pe}{t_t R_w}}$$

iv. Increase depth d to certain percentage to accommodate shear stress as well.

v. Calculate direct shear stress, q, which should be less than R_w. $q=P/(2d*t_t)$

vi. Compute stress due to bending, f_b*, which should be less than* R_w

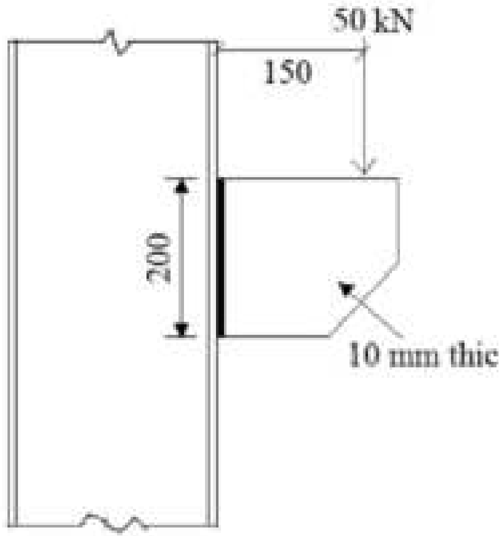
$$f_b = \frac{Pe}{2*\frac{1}{12}t_t d^3} * \frac{d}{2} = \frac{3Pe}{t_t d^2}$$

vii. Find the resultant stress

$$Resultant\ Stress = f_e = \sqrt{q^2 + f_b^2} \leq weld\ strength = \frac{f_u}{\sqrt{3}\gamma_{mw}}$$

If the equivalent stress exceeds the design weld strength R_w then length of weld should increase and above process be repeated till the checks are satisfied.

Design a fillet weld to connect a 10 mm thick bracket to the flange of a column as shown in the figure below.



So in this case the depth is given but size of the weld is unknown.

Let s = size of weld;

Throat thickness, $t_t = 0.707s$

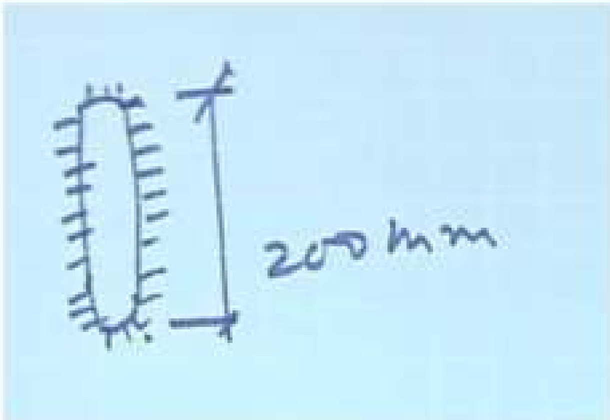
Therefore, Vertical shear stress, $$q = \frac{P}{2dt_t} = \frac{50\times10^3}{2\times200\times0.707s} = \frac{176.8}{s}\ \text{MPa}$$

Horizontal shear stress due to bending,

$$f_b = \frac{6M}{2t_t d^2} = \frac{6\times P\times e}{2t_t d^2}$$

$$= \frac{6\times50\times10^3\times150}{2\times0.707s\times200^2}$$

$$= \frac{795.6}{s}\ \text{MPa}$$



$$Resultant\ Stress = f_e = \sqrt{(\frac{176.8}{s})^2 + (\frac{795.6}{s})^2} = 815/s$$

Allowable stress R_w

$$R_w = 410 / \left(\sqrt{3}\times1.25\right) = 189.37\text{MPa}$$

815/s=189.37 S=4.3 mm

- Strength of weld

adapt a weld size of 5 mm.

GROOVE WELD

In case of groove weld, the weld has been done by grooving in the plate. The depth of weld as d and thickness as t then we can find out the shear stress and the bending stress in a similar fashion.

1. The shear stress in the fillet weld,

$$q = \frac{P}{l_w t_e} = \frac{P}{d \times t_e}$$

2. The stress due to bending,

y=d/2

$$f_b = \frac{M}{I} y \quad I = \frac{1}{12} t_e d^3$$

$$f_b = \frac{Pe}{\frac{1}{12} t_e d^3} \frac{d}{2} = \frac{6Pe}{t_e d^2}$$

$$f_e = \sqrt{3q^2 + f_b^2} \le \frac{f_y}{\gamma_{m0}}$$

DESIGN STEPS

1. Select a suitable size of weld and then compute effective thickness and weld strength. $R_w = f_y/\gamma_{m0}$

2. Calculate the depth of weld using the following expression: $d = \sqrt{\frac{6Pe}{t_e R_w}}$

3. Increase depth d to a certain percentage to accommodate shear stress as well.

4. Calculate direct shear stress q, which should be less than R_w

$$q = P/d*t_e$$

5. Compute stress due to bending, f_b, which should be less than R_w. $f_b = \frac{6Pe}{t_e \times d^2}$

6. Calculate equivalent stress, f_e which should be less than R_w

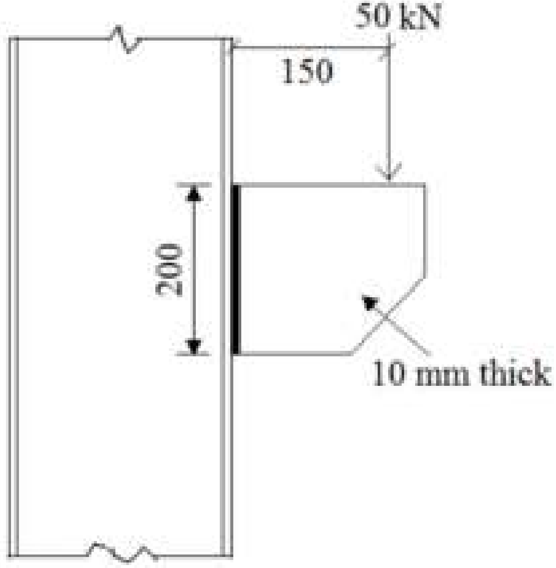
$$f_e = \sqrt{3q^2 + f_b^2}$$

7. If the equivalent stress exceeds the design weld strength R_w, then the length of the weld should be increased and above process be repeated till the checks are satisfied.

,

Design a groove weld to connect a 10 mm thick bracket to the flange of a column as shown in the figure below.

- **Solution:**



Let provide a double J groove weld.
Therefore, effective throat thickness = thickness of bracket plate = t_e = *10 mm*
Therefore, vertical shear stress,

$$q = \frac{P}{dt_e} = \frac{50\times10^3}{200\times10} = 25 \text{ MPa}$$

$$f_b = \frac{6M}{t_e d^2} = \frac{6\times P\times e}{t_e d^2}$$

$$= \frac{6\times50\times10^3\times150}{10\times200^2} = 112.5 \text{ MPa}$$

Resultant stress at extreme fiber,

$$f_e = \sqrt{3q^2 + f_b^{\,2}} = \sqrt{3\times(25)^2 + (112.5)^2} = 120.55$$

Now weld strength,

$$R_w = f_y / \gamma_{m0} = 250 / \ 1.1 \ = \ 227.27\text{MPa}$$

The resultant stress is less than the weld strength that means the joint is safe.

TENSION MEMBERS

Dr.G.C. BEHERA

TENSION MEMBER

A member may be in tension, compression or in bending or combination of above.

When a member is in tension, it is better to use steel section rather than RCC as RC sections weak in tension.

In case of industrial building or bridges members are subjected to tension because of different type of loads including vehicle load for bridges and wind load, earthquake load for industrial structure etc. Therefore, we found that many members undergoes axial tension.

Due to change in direction of wind or earthquake forces, members subjected to compression may be in tension.

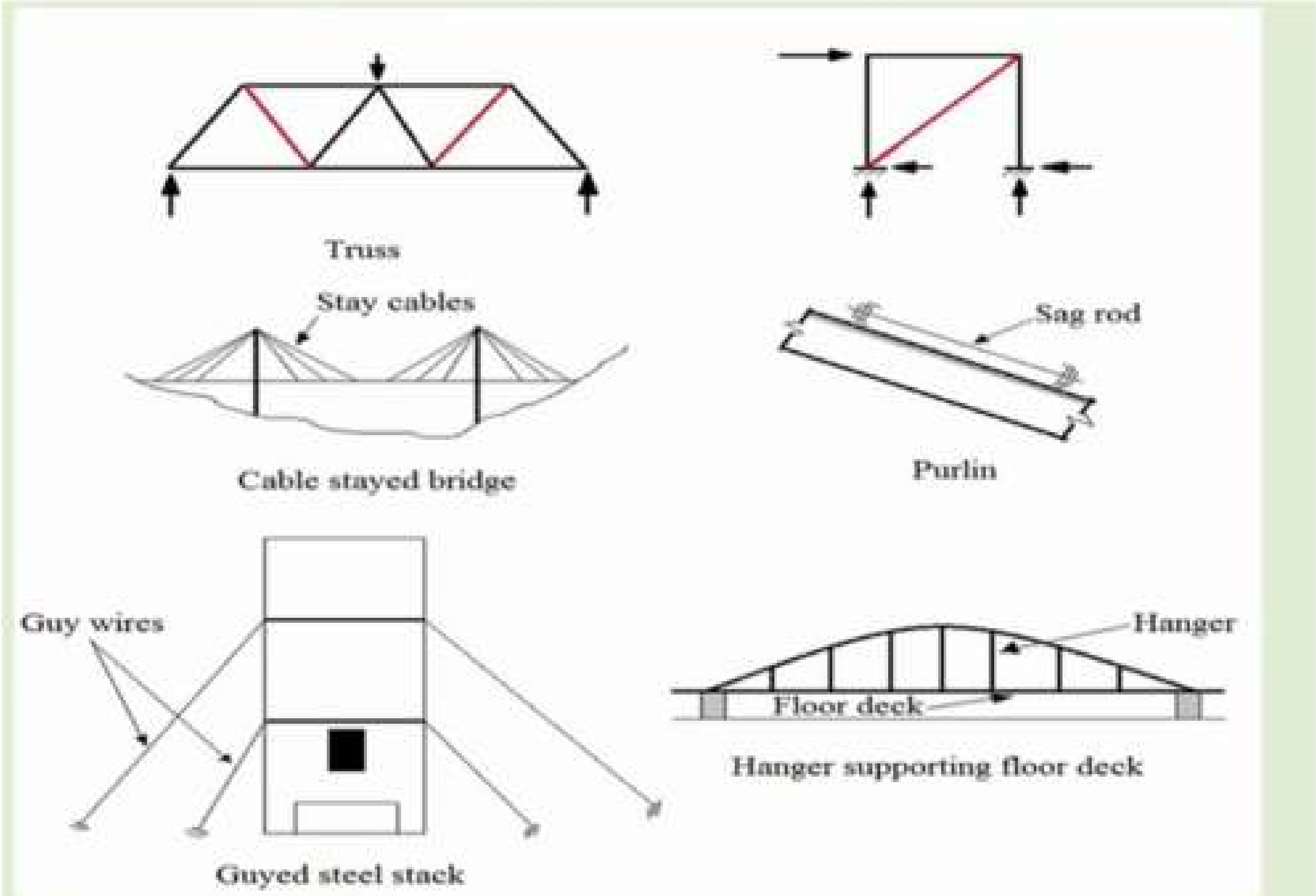
Truss member, cables in suspension bridges, bracings for buildings, these are

often subjected to axial tensile forces which is be designed properly and

cross sectional configuration may be used where circular rods and rolled angle sections are commonly used.

A member carrying direct tension is also known as tie members

TENSION MEMBERS



Types of Tension Members

Wires, strands and cables

A strand consists of individual wires wound helically around a central core. A wire rope consists of a number of strands wound helically around a core. Cables are group of individual strands wound helically around a core.

Bars and rods

Bars and rods are straight member which have considerable cross section. These can be either circular square or rectangular in cross section. Unlike cables, wires and strands, they are used individually as structural members. They are often bolted to the other members by means of threaded ends.

Plates and flat bars.

They are very commonly used. Plates are members where one dimension (thickness) is very small in comparison with the other dimensions. Flat bars are usually rectangular in cross section and the cross sectional dimensions are comparable where as the length is very large in comparison with the cross sectional dimension.

Structural sections

Standard structural steel sections like angles are also used as tension members. These are available in standard dimensions and length.

Built up sections

Built-up sections are also used very frequently in construction. These are formed by using a combination of more than one standard sections and/or plates.

various factors affecting the tensile strength.

Therefore, if we have a plate and if we make a connection with a hole then the net area of the section is going to be reduced. For this reason we will be calculating the net area or the for calculation of the tensile strength we have to reduce the bolt hole area because this bolt hole cannot take tension that is why the strength will be decreased in the presence of hole.

Then another factor is geometry factor. The ratio of gauge length (g) to diameter (d) ,represents geometry factor. A lower ratio of gauge length to its diameter gauge results in contentment of contraction at the net section and hence it is more efficient.

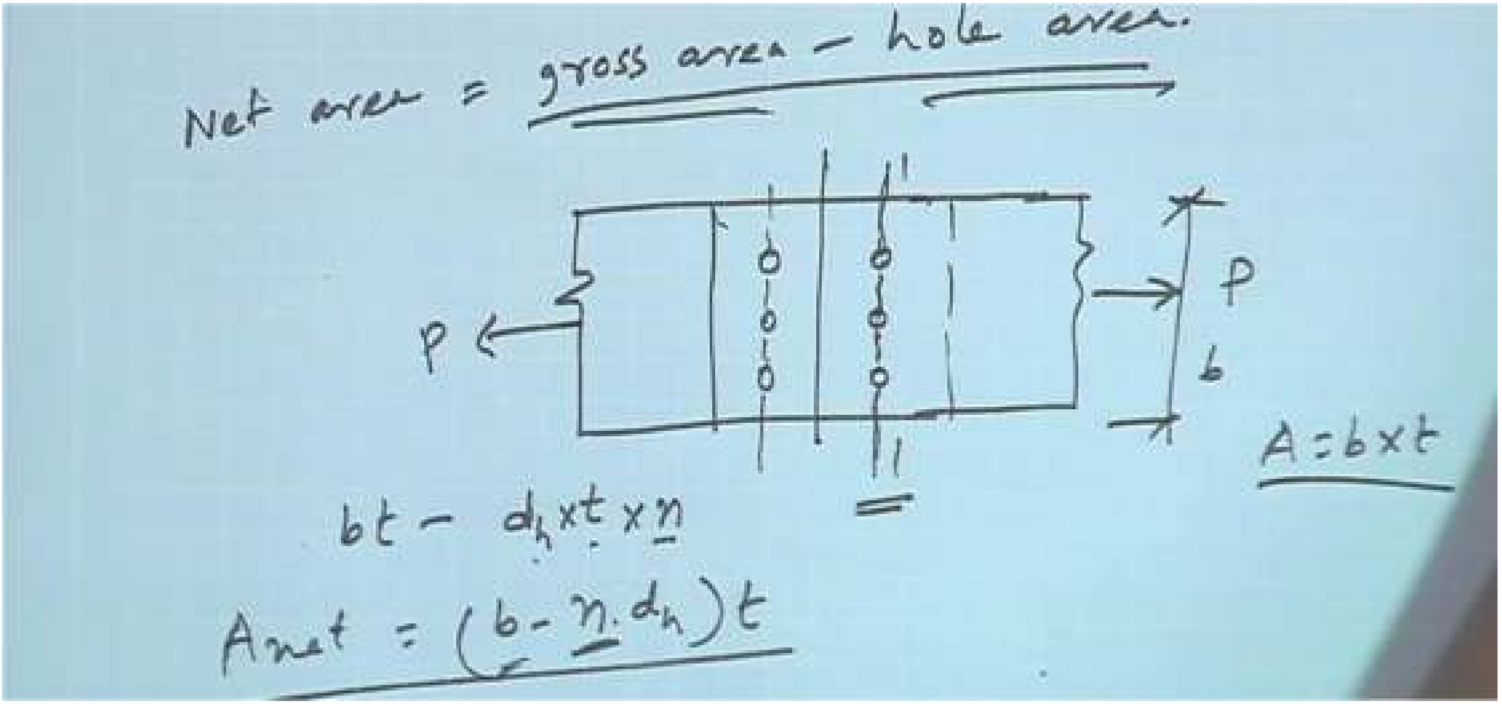
Then another is ductility factor if the members become more ductile then it increases its strength because of even distribution of stress. Then residual strength, where fatigue is involved, we have to count the residual stresses also, how much it is present so accordingly it has to be taken care.

If spacing of fasteners are closer than relative to the diameter then block shear will lead into failure so that has to be also taken care.

FACTORS AFFECTING THE STRENGTH

Shear lag effect: this is very important in case of tension member design. Sometimes the whole member are not connected to the gusset plate or to the system. So when the members are subjected to tension so all the portions of the element or the member are not directly under tension. Therefore, tension force are not distributed throughout its cross section properly, so there is scope of shear lag effect. Because of shear lag effect the strength of the member gets reduced. If an angle is connected to a gusset plate. Now when the tension force is applied so this portion will be under tension directly as it is directly connected but in other portion, it will not occur there will be some lagging. So because of that lagging shear lag effect will come into picture and because of shear lag effect the strength of the member will be reduced.

NET AREA



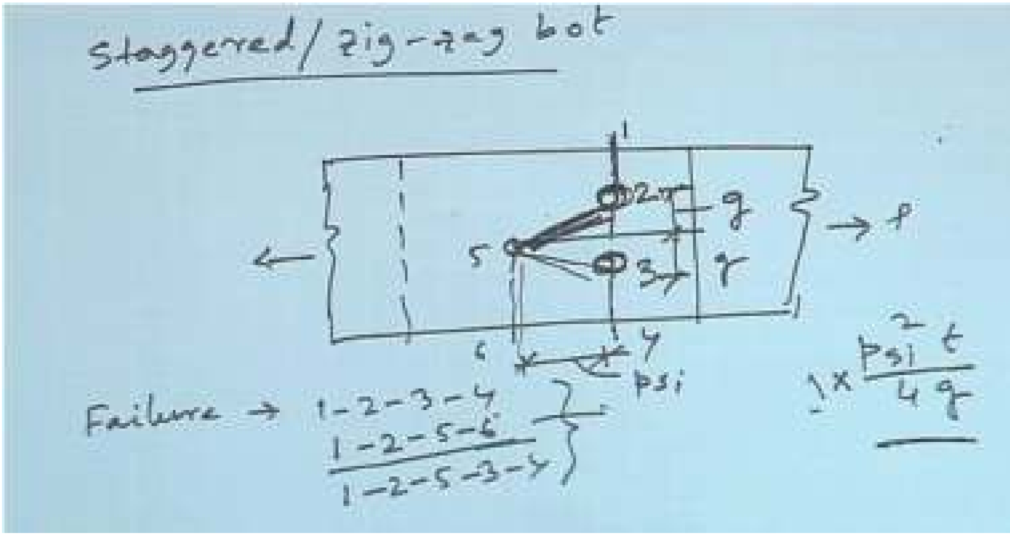
A_{net}=(b−n d_h)t

t = Thickness of the plate
d_h =diameter of hole
b = width of plate
n→ No. of bolts in one line

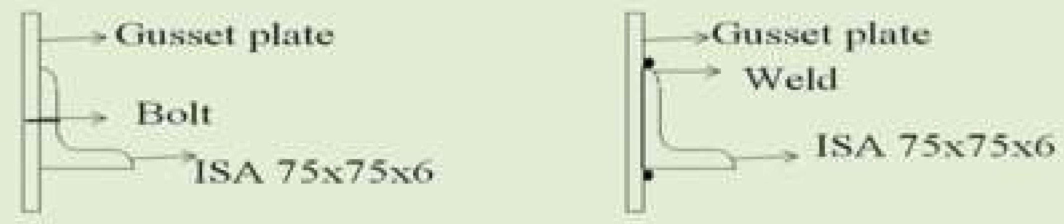
However, in case of staggered bolt we have different formula.

$$A_{net} = \left[b - nd + \left(\frac{p_{s1}^2}{4g_1} + \frac{p_{s2}^2}{4g_2} \right) \right] t$$

- *b , t = width and thickness of the plate respectively.*
- d_h== diameter of the bolt hole (2 mm in addition to the diameter of the hole, in case the directly punched holes).
- *g = gauge length between the bolt holes.*
- *Ps=* staggered-pitch length between line of bolt holes.
- *n = number of bolt holes in the critical section.*



Calculate the net area of an angle ISA 75×75×6 which is connected to the gusset plate through single leg as shown in following figure. Bolts used are M20 grade 4.6.



(a) Bolt Connection (b) Weld Connection

(a) For bolt connection

Diameter of bolt hole = 20+ 2 = 22 mm

Net Area of connected leg, A_{nc}= (75-6/2-22) × 6 = 300 mm^2

Gross area of outstanding leg, A_{go}= (75-6/2) × 6 = 432 mm^2

Net area =A_n= A_{nc}+A_{go}= 300+432=732 mm^2

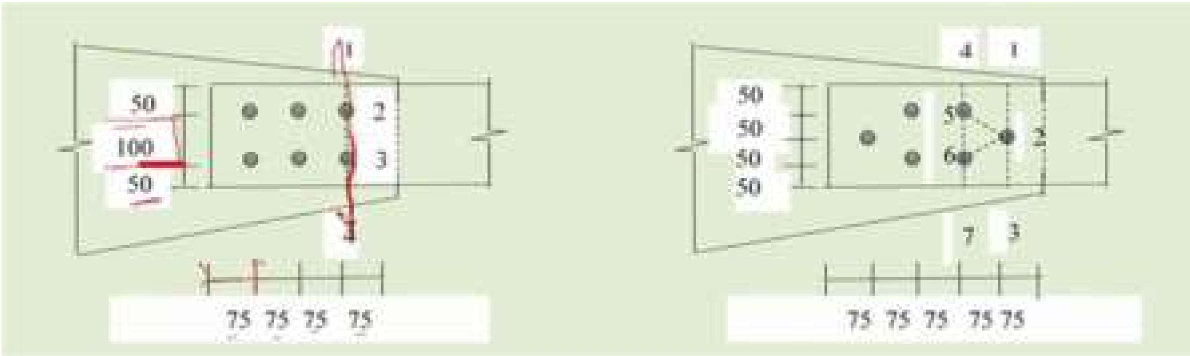
(b) For weld connection

Net Area of connected leg, A_{nc}= (75-6/2) × 6 = 432 mm2

Gross area of outstanding leg, A_{go}= (75-6/2) × 6 = 432 mm^2

Net area, A_n= A_{nc}+ A_{go}= 432 + 432 = 864 mm^2

A flat size of 200 × 8 mm of grade Fe 410 is used as tension member in a roof truss. It is connected to a 12 mm gusset plate by M16 bolt of grade 4.6 using two alternate methods of bolting as shown in following figures. Calculate the net area of the members.



Solution:

Diameter of bolt hole = 16 + 2 = 18 mm

(a) Chain bolting

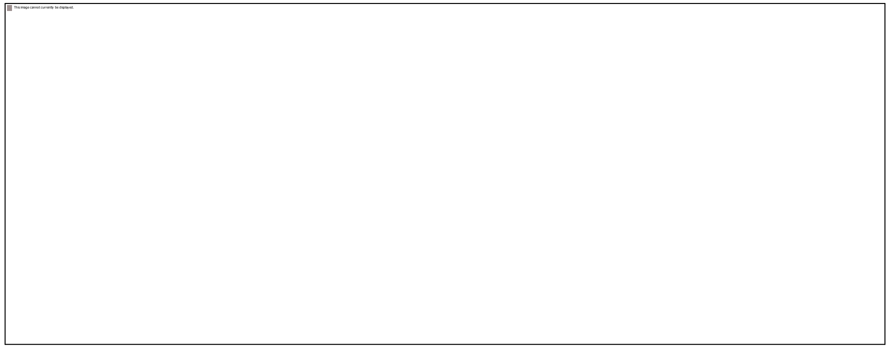
The critical sectional area of the plate will be along 1-2-3-4.

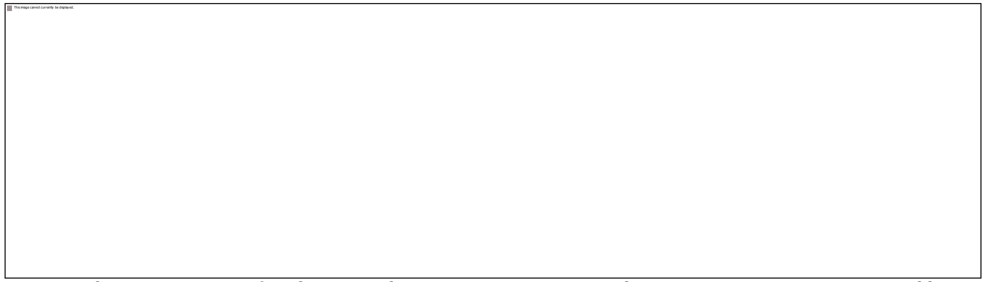
So the net area, $A_n = (200 - 2 \times 18) \times 8 = 1312\ mm^2$

(b) Zig-zag bolting

In this case, the critical section may fail along 1-2-3, 4-5-2-3, 4-5-2-6-7 or 4-5-6-7. Hence, the net area for all possible sections needs to be calculated and the minimum value will be considered as net area.

The net area along 1-2-3, $A_n = (b - n\,d_h)t = (200 - 18) \times 8 = 1456\ mm^2$





It may be noted that the section along 4-5-6-7 will not be critical as the strength of the bolt 1 will be added to this section.

Thus the net sectional area = min of (1456 mm2, 1537 mm2 and 1618 mm2) = 1456 mm^2

Therefore, the most critical sectional area will be along 1-2-3.

STRENGTH OF TENSION MEMBER

Tension member may fail due to yielding of the gross area, because of yielding.

The member may fail due to rupture of the critical section.

Another scope of failure will be the block shear failure as a hole it may fail due to shear which is called block shear failure.

Design strength calculation of tension member in clause 6 of IS: 800-2007. So details you can find out from clause 6 that the design tension T should satisfy the requirement of this Td . Where Td is the design strength of the member under axial tension and Td will be the least of these three, one is the yielding of gross section (Tdg), then rapture of critical section (Tdn) and then block shear failure (Tdb).

$T_{dg} = A_g * _{fy} / \gamma_{m0}$

Where,

f_y is the yield stress of material in MPa,

Ag is the gross area of cross-section

γ_{m0} is the partial safety factor of failure in tension by yielding (Table 5, IS 800: 2007).

clause 6.3.1, you will get design strength in tension of a plate,

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

Where, f_u *is the ultimate stress of material in MPa,*

A_n *is the net effective area of cross-section*

γ_{m1} *is the partial safety factor of failure in tension at ultimate stress (Table 5, IS 800: 2007)*

Single angle section, if it is connected with some gusset plate, or some other plates, or some other members then shear lag effect will be going to be occur. So, we have to calculate the T_{dn} value taking care of the shear lag effect.

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

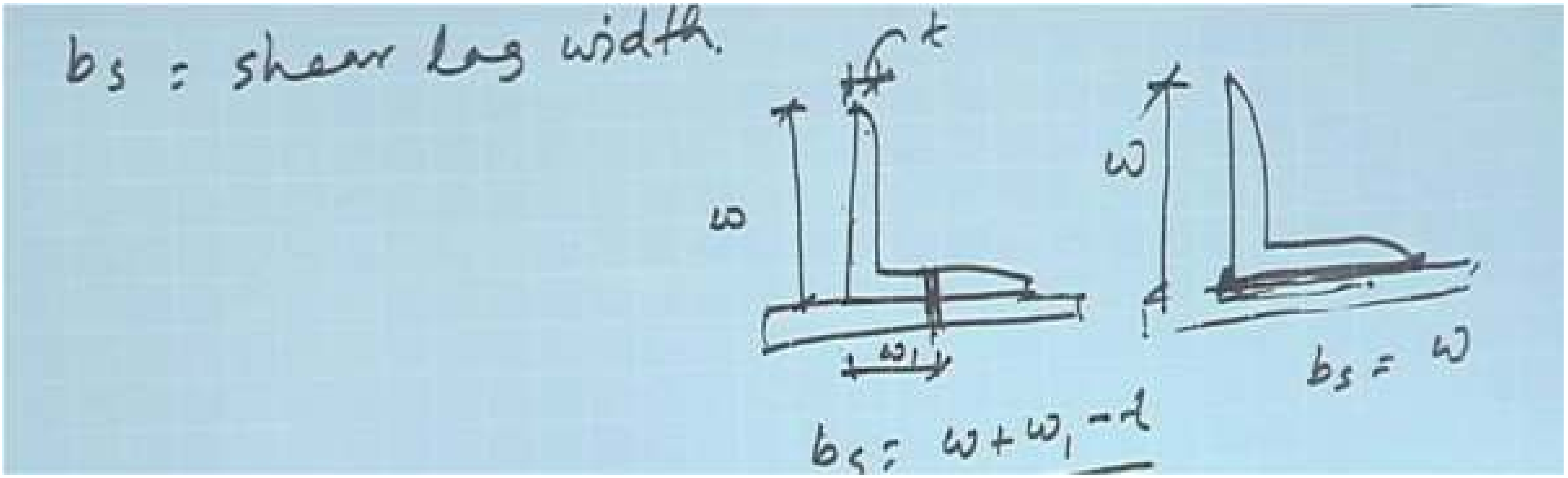
$$\beta = 1.4 - 0.076\,(w/t)\,(f_y/f_u)\,(b_s/L_c) \qquad \le f_u\gamma_{m0}/f_y\gamma_{m1}$$

$$\ge 0.7$$

Here, w = *outstanding leg width,*

b_s = *shear lag width, as shown in figure below.*

L_C = length of the end connection, that is the distance between the outermost bolts in the end joint measured along the load direction or length of the weld along the load direction.



for preliminary sizing we can calculate T_{dn} from this formula

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

Here, α = 0.6 for one or two bolts, 0.7 for three bolts and 0.8 for four or more bolts along the length in the end connection or equivalent weld length.

A_n = net area of the total cross-section;

A_{nc} = net area of the connected leg;

A_{go} = gross area of the outstanding leg; and

t = thickness of the leg.

STRENGTH CALCULATION

For other sections like double angles, channels, I-sections and other rolled steel sections connected by one or more elements to end gusset is also governed by shear lag effect. The design tensile strength of such sections as governed by tearing of net section may also be calculated using equation in clause 6.3.3, where β is the is calculated based on the shear lag distance b_s , and b_s is taken from the furthest edge of the outstanding leg to the nearest bolt or weld line in the connected leg of the cross section. So, for rapture strength calculation other than the single angle section we can use this clause that is clause 6.3.3.

Then the design strength due to block shear can be calculated from clause 6.4 of the IS code,

Bolted Connections Lower of the two

$$T_{db} = \frac{0.9A_{vn}f_u}{\sqrt{3}\gamma_{m1}} + \frac{A_{tg}f_y}{\gamma_{m0}} \; for\ tension\ yield\ and\ shear\ fracture$$

$$T_{db} = \frac{A_{vg}f_y}{\sqrt{3}\gamma_{m0}} + \frac{0.9A_{tn}f_u}{\gamma_{m1}} \; for\ tension\ fracture\ \ and\ shear\ yield$$

A_{vg} and A_{vn} = minimum gross and net area in shear along bolt line parallel to external force, respectively.

A_{tg} and A_{tn}= minimum gross and net area in tension from the bolt hole to the toe of the angle,

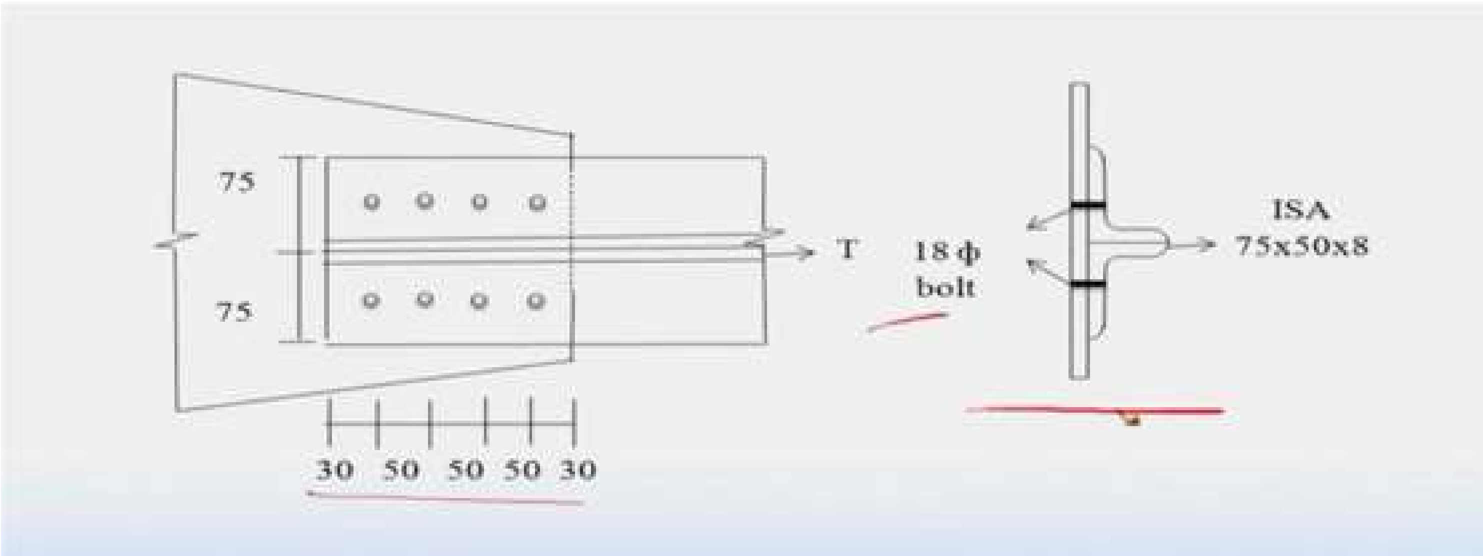
Maximum effective slenderness ratio (Table 3, IS 800: 2007)

Member	Maximum effective slenderness ratio
A tension member in which a reversal of direct stress occurs due to loads other than wind or seismic forces.	180
A member subjected to compressive forces resulting only from a combination of wind/earthquake actions, provided the deformation of such a member does not adversely affect the stresses in any part of the structure.	250
A member normally acting as a tie in a roof truss or a bracing member which is not considered effective when subjected to reversal of stress resulting from the action of wind or earthquake forces	350
Members always in tension (other than pre-tensioned members)	400

Two ISA 75×50×8 are connected to a gusset plate on its same side of thickness 10mm by four M18 grade 4.6 bolts. Find the design tensile strength of the angle if (1) gusset is connected to the longer leg

(2) gusset is connected to the shorter leg.

- **Solution:**
- **(1) Gusset connected to the longer leg**
- 2 ISA 75 × 50 × 8 connected back to back with its longer length.



- **Solution:**

(1) Gusset connected to the longer leg

2 ISA 75 × 50 × 8 connected back to back with its longer length. Thus, the gross area will be *$Ag = 2 \times 938 = 1876\ mm^2$*

- ***Strength due to yielding of gross section:***
- $T_{dg} = f_y \times A_g / \gamma_{m0} = 250 \times 1876/1.1 = 426.36 \times 10^3$ N = 426.36 kN
- Dia. of bolt = 18 mm
- Dia. of hole = 18+2 = 20
- Let us have pitch 50 mm, e=35 mm
- ***Strength governed by rupture of net section:***
- $A_{nc} = (75 - 8/2 - 20) \times 8 = 408\ mm^2$
- $A_{go} = (50 - 8/2) \times 8 = 368\ mm^2$
- $An = 408 + 368 = 776\ mm^2$

$$\beta = 1.4 - 0.076\ (w/t)\ (f_y/f_u)\ (b_s/L_c) \qquad \leq f_u \gamma_{m0} / f_y \gamma_{m1}$$

$$\geq 0.7$$

$$1.4 - 0.076 \times \frac{50+40-8}{3 \times 50} \times \frac{50}{8} \times \frac{250}{410}$$

$$\text{Again,} \quad \beta \leq \frac{f_u \gamma_{m0}}{f_y \gamma_{m1}} \quad \text{and} \quad \geq 0.7$$

$$\frac{f_u \gamma_{m0}}{f_y \gamma_{m1}} = \frac{410 \times 1.1}{250 \times 1.25} = 1.443$$

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

$$\frac{0.9 \times 410 \times 408}{1.25} + \frac{1.242 \times 250 \times 368}{1.1} = 224.31 \times 10^3 \text{ N}$$

The strength due to rupture for two angles = 2 × 224.31 = 448.62 kN

$$\text{Alternatively,} \quad T_{dn} = 2 \times \frac{\alpha \times A_n \times f_u}{\gamma_{m1}} = 2 \times \frac{0.8 \times 776 \times 410}{1.25} = 407\text{kN}$$

- ***Strength governed by block shear:***

$A_{vg} = 8 \times (3 \times 50 + 30) = 1440\ mm^2$

$A_{vn} = 8 \times (3 \times 50 + 30 - 3.5 \times 20) = 880\ mm^2$

$A_{tg} = 8 \times 35 = 280\ mm^2$

$A_{tn} = 8 \times (35 - 0.5 \times 20) = 200\ mm^2$

$$T_{db1} = \frac{0.9\,A_{vn} f_u}{\sqrt{3}\gamma_{m1}} + \frac{f_y A_{tg}}{\gamma_{m0}} = \frac{0.9 \times 410 \times 880}{\sqrt{3} \times 1.25} + \frac{250 \times 280}{1.1} = 213.62 \times 10^3\ \text{N} = 213.62\ \text{kN}$$

Thus, T_{db1} for both the angle will be:

2×213.62 kN = 427.24 kN.

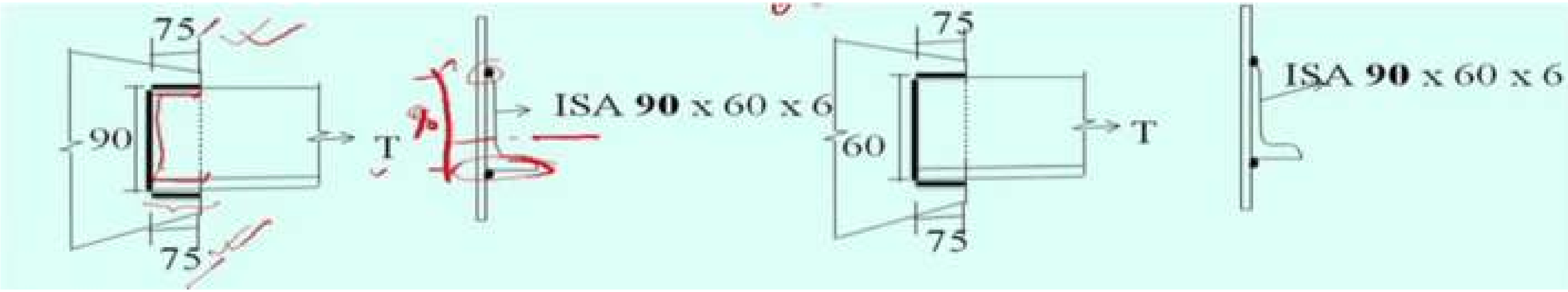
$$T_{db2} = \frac{A_{vg} f_y}{\sqrt{3}\gamma_{m0}} + \frac{0.9 f_u A_{tn}}{\gamma_{m1}} = \frac{1440 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 410 \times 200}{1.25}$$

$$= 248 \times 10^3\ \text{N} = 248\ \text{kN}$$

So, T_{db2} for both the angle will be: 2×248 kN = 496 kN.

Thus, the block shear, *Tdb = 427.24 kN ,*
Design tensile strength of angle will be least of T_{dg} , T_{dn} *and* T_{db} *(426.36 kN, 448.62 kN and* 427.24 kN) = **426.36 kN.**

An ISA 90×60×6 angle of Fe 410 grade steel is connected to a 10 mm thick gusset plate by weld as shown in the figure below. Calculate the design tensile strength of the angle if gusset is connected to the (a) longer leg (b) shorter leg.



Solution:

(a) Gusset is connected to longer leg

Gross area, *Ag for ISA 90 × 60 × 6 = 865 mm²*. *[From IS hand book: SP: 6(1)-1964]*

The net area of connected leg, $A_{nc} = (90 - 6/2) \times 6 = 522\ mm^2$

Gross area of outstanding leg, $A_{go} = (60 - 6/2) \times 6 = 342\ mm^2$

(i) Tensile strength governed by yielding of gross section:

$$T_{dg} = \frac{f_y \times A_g}{\gamma_{m0}} = \frac{250 \times 865}{1.1} \times 10^{-3} = 196.6\ \text{kN}$$

- *(ii) Tensile strength governed by rupture of net section:*

Therefore, the length of outstanding leg will be, *w = 60 mm.*

So, the shear lag width, *bs = 60 mm.*

- The average length of weld along the direction of load L_c=75 mm

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

$$= 1.4 - 0.076 \times \frac{60}{75} \times \frac{60}{6} \times \frac{250}{410} = 1.029$$

$$\beta \leq \frac{f_u \gamma_{m0}}{f_y \gamma_{m1}} \quad \text{and} \quad \geq 0.7$$

$$\frac{f_u \gamma_{m0}}{f_y \gamma_{m1}} = \frac{410 \times 1.1}{250 \times 1.25} = 1.443$$

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

$$T_{dn} = \frac{0.9 \times 410 \times 522}{1.25} + \frac{1.029 \times 250 \times 342}{1.1} = 234 \times 10^3 \text{ N} = 234 \text{ kN}$$

Alternatively, $T_{dn} = \dfrac{\alpha \times A_n \times f_u}{\gamma_{m1}}$

Here, $A_n = A_{nc} + A_{go} = 522 + 342 = 864 \text{ mm}^2$

Thus, $T_{dn} = \dfrac{0.8 \times 864 \times 410}{1.25} = 226.71 \times 10^3 \text{ N} = 226.7 \text{ kN}$

- ***(iii) Tensile strength governed by block shear:***

Assuming average length of weld on each side as 75 mm, refer code 6.4.2

$A_{vg} = 2 \times 75 \times 10 = 1500\ mm^2$ *[As gusset plate thickness = 10 mm]*

$A_{vn} = 2 \times 75 \times 10 = 1500\ mm^2$

$A_{tg} = 90 \times 10 = 900\ mm^2$

$A_{tn} = 90 \times 10 = 900\ mm^2$

A tension member 3 m long carries a factored tensile load of 200 kN. Design a suitable single angle unequal section when connection is made with (i) 20 mm diameter bolts of grade 4.6 and (ii) fillet weld. Assume longer leg to be connected with plate.

- **Solution:**
- **Step 1:**

Approximate gross area required = $A_g = P/(f_y / \gamma_{m0})$

$= 200 \times 10^3/(250/1.1) = 880 \text{ mm}^2$

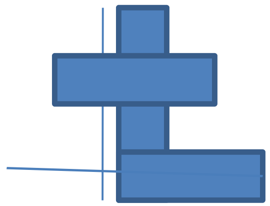
Let use ISA 75 × 50 × 10 with gross area, A_g *as* 1152 mm^2

$A_{nc} = (75 - 10/2 - 22) \times 10 = 480 \text{ mm}^2$

$A_{go} = (50 - 10/2) \times 10 = 450 \text{ mm}^2$

$A_n = 480 + 450 = 930 \text{ mm}^2$

P= Ag*fy/1.1



Design strength due to yielding of gross section

$$T_{dg} = \frac{f_y \times A_g}{\gamma_{m0}} = \frac{250 \times 1152}{1.1} = 261.8 \times 10^3 \text{ N} = 261.8 \text{ kN} > 200 \text{ kN};$$

so ok.

Step 2:

a) For bolt connection

b) Cross sectional area of 20 mm diameter bolt= 0.78 × π/4 × 20^2 = 245 mm^2.

Shear strength of M20 bolt in single shear

$$\frac{\frac{f_{ub}}{\sqrt{3}}\left(n_n A_{nb} + n_s A_{sb}\right)}{\gamma_{mb}} \qquad \frac{(400/\sqrt{3}) \times (1 \times 245)}{1.25}$$

$$= 45.3 \times 10^3 \text{ N} = 45.3 \text{ kN}$$

Bearing strength of bolts = 2.5 × $d \times t \times k_b \times f_{ub}/\gamma_{mb}$

Here, thickness of the angle is 10 mm;

Let the edge distance = 30 mm and pitch = 50 mm

Thus, k_b = lesser of [30/(3 × 22), [50/(3 × 22) - 0.25], 400/410, 1] = 0.454

Bearing strength of bolt = 2.5 × 20 × 10 × 0.454 × 400/1.25 = 72.64 × 10^3 N = 72.64 kN,

So bolt value = 45.3 kN

- **Step 3:**

No. of bolts required = 200/45.3 = 4.4

Thus, use 5 bolts of 20 mm diameter in one line at pitch of 50 mm and edge distance of 30 mm.

Step 4:

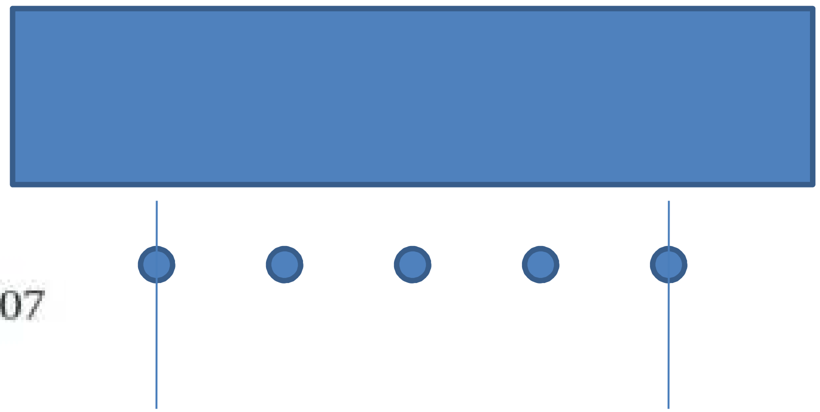
- ***Design strength governed by rupture of net section***
- Here, length of outstanding leg is: *w = 50 mm and* w_1 *= 40 mm.*
- So the shear lag width, *bs = w +* w_1 *– t = 50 + 40 – 10 = 80 mm.*
- Distance between end bolts , *Lc = 4 × 50 = 200 mm.*

$$\beta = 1.4 - 0.076\frac{b_s}{L_c}\times\frac{w}{t}\times\frac{f_y}{f_u} = 1.4 - 0.076\frac{80}{200}\times\frac{50}{10}\times\frac{250}{410} = 1.307$$

Again, $\beta \le \frac{f_u \gamma_{m0}}{f_y \gamma_{m1}}$ and ≥ 0.7

$$\frac{f_u \gamma_{m0}}{f_y \gamma_{m1}} = \frac{410 \times 1.1}{250 \times 1.25} = 1.443$$

Thus, satisfying above criteria, $\beta = 1.307$



$$T_{dn} = \frac{0.9 f_u A_{nc}}{\gamma_{m1}} + \frac{\beta f_y A_{go}}{\gamma_{m0}} \quad ¿ \frac{0.9 \times 410 \times 480}{1.25} + \frac{1.307 \times 250 \times 450}{1.1}$$

$$= 275.37 \times 10^3 \text{ N} = 275.37 \text{ kN}$$

Also, $T_{dn} = \dfrac{\alpha \times A_n \times f_u}{\gamma_{m1}} \quad ¿ \dfrac{0.8 \times 930 \times 410}{1.25} = 244 \times 10^3 \text{ N} = 244 \text{ kN}$

Design strength governed by block shear

$A_{vg} = 10 \times (4 \times 50 + 30) = 2300 \text{ mm}^2$

$A_{vn} = 10 \times (4 \times 50 + 30 – 4.5 \times 22) = 1310 \text{ mm}^2$

$A_{tg} = 10 \times 35 = 350 \text{ mm}^2$

$A_{tn} = 10 \times (35 – 0.5 \times 22) = 240 \text{ mm}^2$

$$T_{db} = \frac{0.9 A_{vn} f_u}{\sqrt{3}\gamma_{m1}} + \frac{A_{tg} f_y}{\gamma_{m0}} \textit{ for tension yield and shear fracture}$$

$$T_{db} = \frac{A_{vg} f_y}{\sqrt{3}\gamma_{m0}} + \frac{0.9 A_{tn} f_u}{\gamma_{m1}} \textit{ for tension fracture and shear yield}$$

$$T_{db} = \frac{0.9 * 1310 * 410}{\sqrt{3} * 1.25} + \frac{350 * 250}{1.1}$$

$$= 302.81\ kN\ for\ tension\ yield\ and\ shear\ fracture$$

$$T_{db} = \frac{2300 * 250}{\sqrt{3} * 1.1} + \frac{0.9 * 240 * 410}{1.25}$$

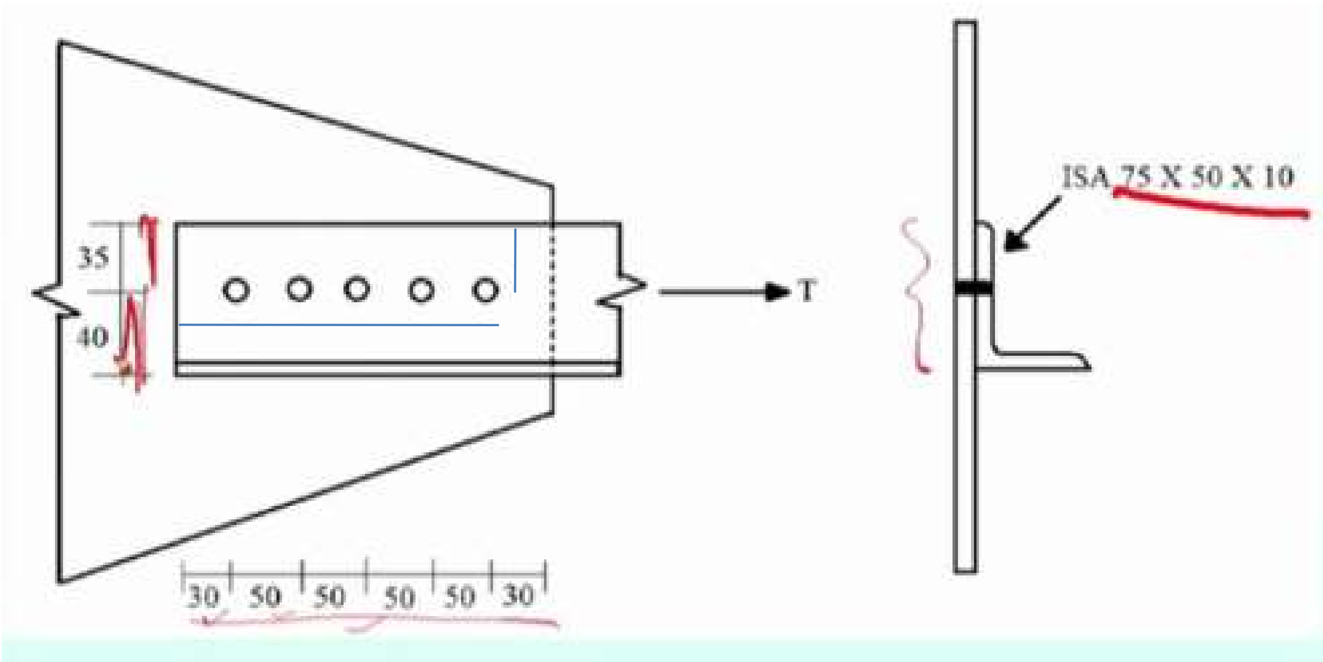
$$= 336.42\ kN for\ tension\ fracture\ \ and\ shear\ yield$$

Out of all these three yielding of gross, rupture of net area and block shear failure lowest is from yielding.

Tensile strength of section=261.8 kN> 200 kN.

Design is safe.

Thus, the design tensile strength of angle = 261.8 kN > 200 kN.
Hence, the selected angle is safe.
Again, the minimum radius of gyration (r_{min}) *of the angle ISA 75×50×10 = 10.6 mm*
The maximum slenderness ratio, $\lambda_{max} = L_{eff} / r_{min} = 3 \times 10^3 / 10.6 = 283 < 350.$
So, the angle is safe.



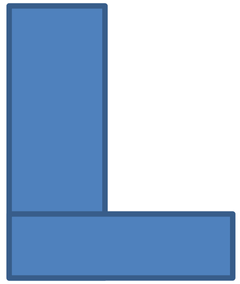
Solution (Connected with fillet weld):

(a) Strength due to yielding of gross section

Approximate area required = $A_g = P/(f_y / \gamma_{m0}) = 200 \times 1000/(250/1.1) = 880\ mm^2$

- Use ISA 75 × 50 × 10 with $A_g = 1152\ mm^2$ and $c_x = 26\ mm$
- $A_{nc} = (75 - 10/2) \times 10 = 700\ mm^2$
- $A_{go} = (50 - 10/2) \times 10 = 450\ mm^2$

$$T_{dg} = \frac{f_y \times A_g}{\gamma_{m0}} = \frac{250 \times 1152}{1.1} = 261.8 \times 10^3\ \text{N} = 261.8\ \text{kN} > 200\ \text{kN};$$



So, section is safe.

(b) Strength governed by rupture of net section

Here, shear lag width, $b_s = 50$ mm. Assuming average weld length, L_w as 165 mm

$$\beta = 1.4 - 0.076 * (b_s/L_c) * (w/t) * (f_y/f_u) = 1.4 - 0.076 * (50/165) * (50/10) * (250/410) = 1.329$$

$$T_{dn} = 0.9 * f_u * A_{nc}/\gamma_{m1} + \beta f_y * A_{go}/\gamma_{m0} \quad T_{dn} = 0.9 * 410 * 700/1.25 + 1.329 * 250 * 450/1.1 = 342.56\ kN$$

$$A_n = A_{nc} + A_{go} = 700 + 450 = 1150\ mm^2$$

$$T_{dn} = \alpha \times A_n \times f_u/\gamma_{m1} = 0.8 \times 1150 \times 410/1.25 = 301.8\ \text{kN}$$

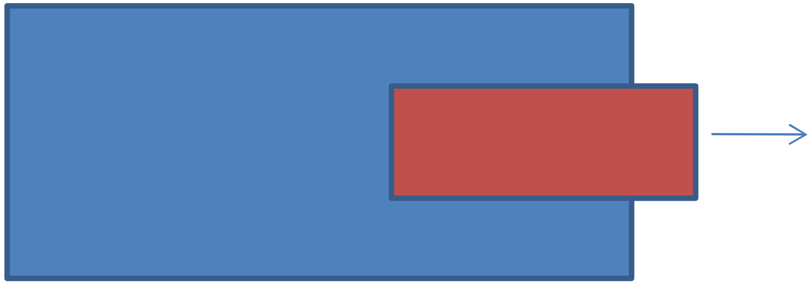
- ***(c) Strength governed by block shear***

A_{vg} = 10 × 165 × 2 = 3300 mm^2

A_{vn} = 10 × 165 × 2 = 3300 mm^2

A_{tg} = 10 × 75 = 750 mm^2

A_{tn} = 10 × 75 = 750 mm^2



$$T_{db1} = \frac{0.9 A_{vn} f_u}{\sqrt{3}\gamma_{m1}} + \frac{f_y A_{tg}}{\gamma_{m0}} = \frac{0.9 \times 410 \times 3300}{\sqrt{3} \times 1.25} + \frac{250 \times 750}{1.1}$$

$$= 732.88 \times 10^3 \text{ N} = 732.88 \text{ kN}$$

$$T_{db2} = \frac{A_{vg} f_y}{\sqrt{3}\gamma_{m0}} + \frac{0.9 f_u A_{tn}}{\gamma_{m1}} = \frac{3300 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 410 \times 750}{1.25}$$

= 654.41 × 10^3 N = 654.41 kN, Thus, *Tdb = 654.41 kN*

Design tensile strength of angle = 261.36 kN > 200 kN, hence safe

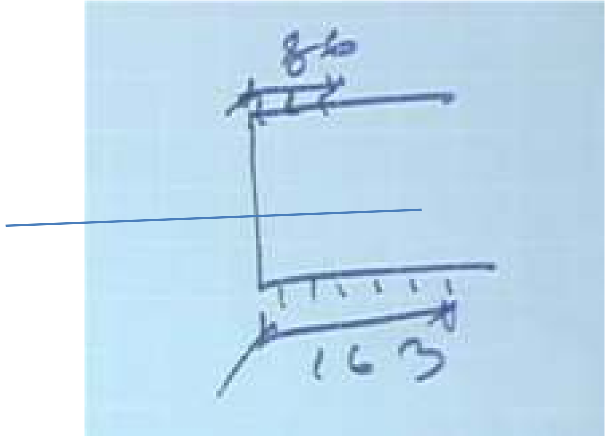
Again, the minimum radius of gyration (*rmin) of the angle ISA 75×50×10 = 10.6 mm*

The maximum slenderness ratio, $\lambda_{max} = L_{eff} / r_{min}$ *= 3 × 103 / 10.6 = 283 < 350.*

Therefore, the angle is safe.

WELD LENGTH:
Force resisted on lower portion of CG=200*(75-26)/75=130.7 kN
Force resisted on upper portion of CG=200*(26)/75=69.3 kN
Taking 6 mm weld=,effective weld thichness=0.707*6=4.24 mm
Length required for lower portion

$$L_{w1} = \frac{P_1}{\left(\frac{t_e f_u}{\sqrt{3}\gamma_{mw}}\right)} = \frac{130.7 * 1000}{\left(\frac{4.24 * 410}{\sqrt{3} * 1.25}\right)} = 163\ mm$$

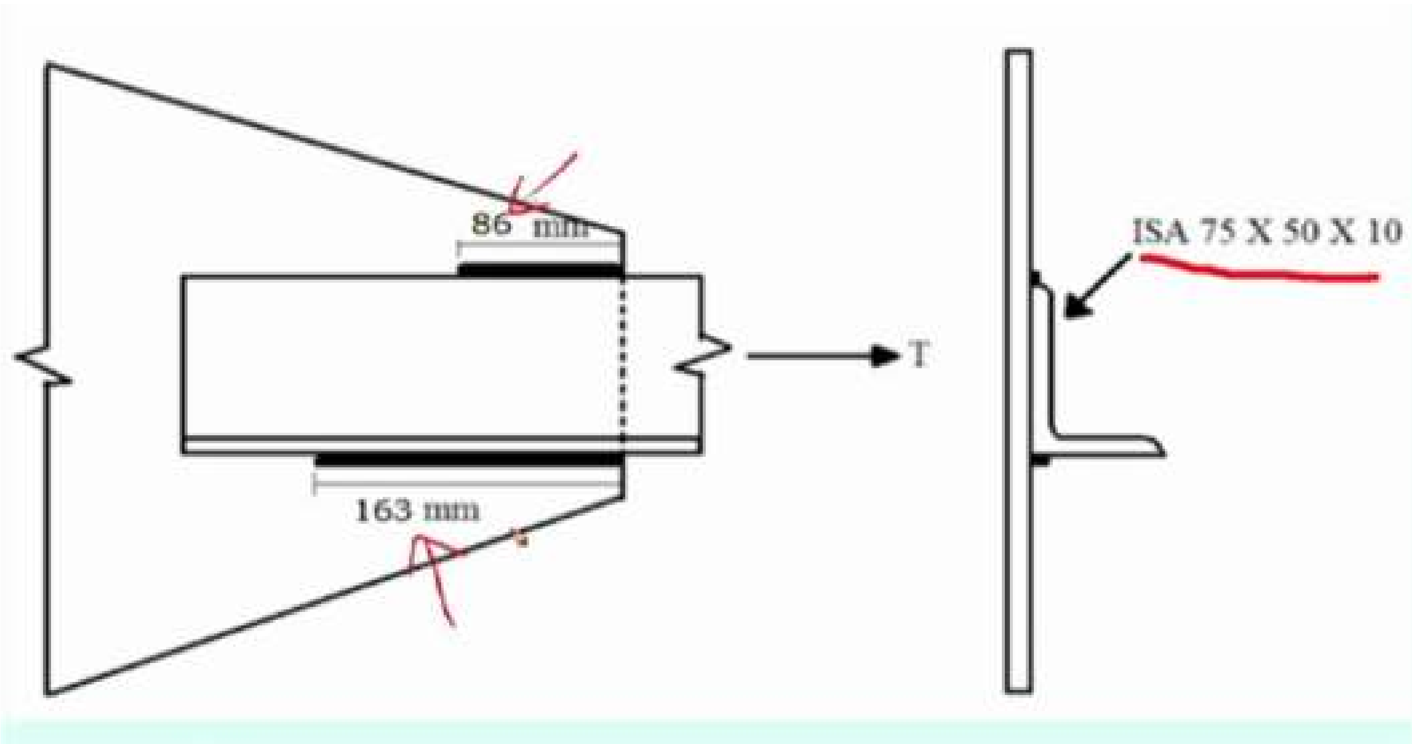


$$L_{w2} = \frac{P_2}{\left(\frac{t_e f_u}{\sqrt{3}\gamma_{mw}}\right)} = \frac{69.3 * 1000}{\left(\frac{4.24 * 410}{\sqrt{3} * 1.25}\right)} = 86\ mm$$

$$\beta = 1.4 - 0.076 \frac{b_s}{L_c} \times \frac{w}{t} \times \frac{f_y}{f_u} \qquad 1.4 - 0.076 \frac{50}{124.5} \times \frac{50}{10} \times \frac{250}{410} \quad = 1.307$$

$$T_{dn} = \frac{0.9 f_u A_{nc}}{\gamma_{m1}} + \frac{\beta f_y A_{go}}{\gamma_{m0}} \qquad \frac{0.9 \times 410 \times 700}{1.25} + \frac{1.307 \times 250 \times 450}{1.1} \quad = 340.31\ kN$$

$$T_{dn} > T_u = 200\ kN$$


86 mm
163 mm
T
ISA 75 X 50 X 10

GUSSET PLATE

Gusset plates are used when more than one member is joined. The size of the gusset plate actually depends on the number of bolts used or weld length of weld used to join the section.

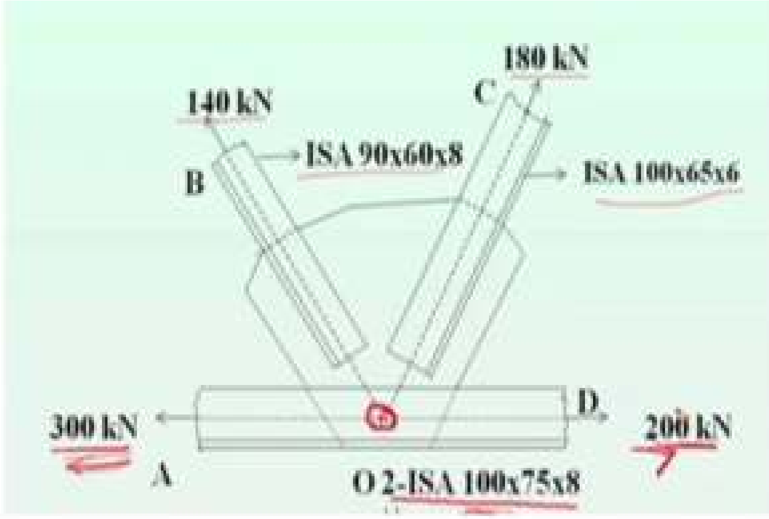
The gusset plate length should be as less as possible so that the material for gusset plate is minimized. Gusset plate thickness should be little more than the member itself.

Design criteria

1. The shape and size of a gusset plate is decided on the basis of direction of members meeting at the joint.
2. The plate size is decided to meet specification of pitch and edge distance.
3. The gusset plates are designed to resist shear mainly shear and direct and flexible stress acting on the critical section.
4. Thickness of gusset plate equal to or higher than the members that have to be connected by the gusset plate that also we have to keep in mind.

Design the gusset plate of thickness 12 mm at the joint O of a lower chord of truss as shown in the figure below. Use M20 grade 4.6 bolts.

- # Solution:



Forces on members OA, OB, OC and OD are:

F_{OA} = *300 kN*

F_{OB} = *140 kN*

F_{OC} = *180 kN*

F_{OD} = *200 kN*

Shear strength of M20 bolts in single shear

$$= \frac{\frac{f_{ub}}{\sqrt{3}}\left(n_n A_{nb} + n_s A_{sb}\right)}{\gamma_{mb}} = \frac{(400/\sqrt{3}) \times (1 \times 245)}{1.25} = 45.3 \text{ kN}$$

Shear strength of M20 bolts in double shear = 45.3 × 2= 90.6 kN.

Assume pitch as 60 mm and edge distance as 40 mm.

- **Member OB:**

k_b = *40/(3 × 22), 60/(3 × 22) - 0.25, 400/410, 1 = 0.606*

Bearing strength of bolts on 8 mm thick angles,

= $2.5 \times d \times t \times k_b \times f_{ub}/\gamma_{mb}$

= 2.5 × 20 × 8 × 0.606 × 400/1.25 = 77.57 × 103 N = 77.57 kN

Strength of angle per pitch length

$$T_{dn} = \frac{0.9 \times f_u \times A_n}{\gamma_{m1}} \quad \frac{0.9 \times 410 \times [(60-22) \times 8]}{1.25}$$

= 89.74 kN

So the bolt value = 45.3 kN (lesser of 45.3, 77.57 and 89.74)

No. of bolt required = 140/45.3 = 3.1 ≈ 4

The length of gusset plate = 3 × 60 + 2 × 40 = 260 mm

- **Member OC:**

Here, the value of *kb will be same as derived for member OB as pitch and edge are same.*

Bearing strength of bolts on 6 mm thick angles = 2.5 × *d* × *t* × *kb* × *fub/γmb*

= 2.5 × 20 × 6 × 0.606 × 400/1.25 = 58.18 kN

Strength of angle per pitch length $= T_{dn} = \frac{0.9 \times f_u \times A_n}{\gamma_{m1}} \quad \frac{0.9 \times 410 \times [(60-22) \times 6]}{1.25}$

$$= 67.31 \times 10^3 \text{ N} = 67.31 \text{ kN}$$

The bolt value = 45.3 kN (lesser of 45.3, 58.18 and 67.31)

No. of bolt required = 180/45.3 = 3.97 ≈ 4

Length of gusset plate = 3×60 + 2×40 = 260 mm

- **Member AD:**
- Net force in member AD = 300 – 200 = 100 kN
- Bearing strength of bolts on 12 mm thick gusset plate

= 2.5 × *d* × *t* × *kb* × *fub*/*γmb*

= 2.5 × 20 × 12 × 0.606 × 400/1.25 = 116.352 kN

Strength of angle per pitch length = $T_{dn} = 0.9 \times f_u \times A_n / \gamma_{m1}$

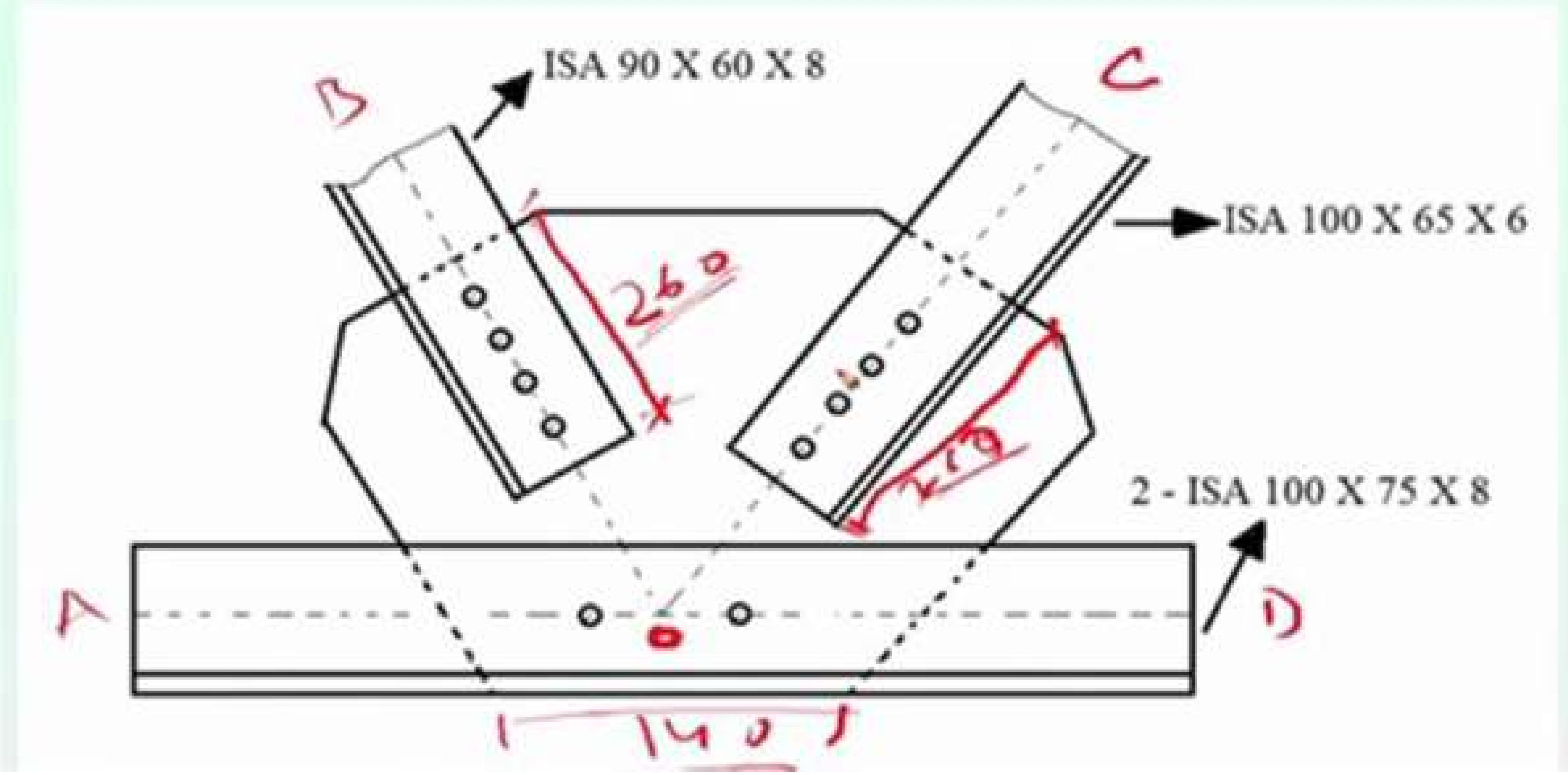
*=0.9*410*[(60-22)*16]/1.25=179.5 kN*

So the bolt value = 90.6 kN (*i.e., lesser of 90.6 kN, 116.4 kN and 179.5 kN)*

No. of bolts required = 100/90.6 = 1.1 ≈ 2

Length of gusset plate = 60 + 2 × 40 = 140 mm

Bolt arrangements are shown in following figure.


ISA 90 X 60 X 8
B
C
ISA 100 X 65 X 6
260
2 - ISA 100 X 75 X 8
A
D
140

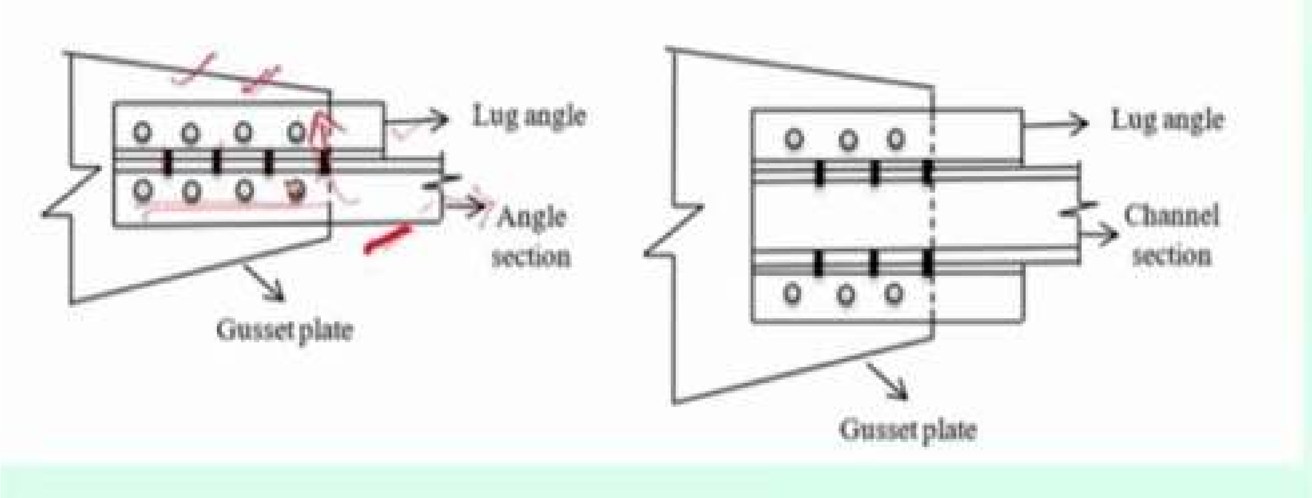
Lug Angles

Lug Angle is an angle with short length which is required to share the load of the main angle. Sometimes the main angle carries a huge amount of load and to make connections of this type of angle section with the main member, we need a large number of bolts.

Use a large number of bolt or large length of weld, size of the gusset plate become very high, so it is uneconomical.

To reduce the length of the joint or the size of the joint to a certain amount sometimes we provide lug angles at the beginning of the joint to share the load from main angle to the gusset plate.

So basically lug angle is connected with the outstanding leg of the main angle and some percentage of load main angle is transferred to the lug angle and then that load again is transferred to the gusset plate through connected leg of the lug angle.



FOR ANGLE SECTION

clause 10.12, IS 800:2007, general procedure of design of lug angles are provided..

Design will be two type one with angle section, another with channel section.

If main member is an angle section, then join lug angle to the outstanding leg of the main angle. If unequal angle is used the load gets distributed in the ratio of gross areas of connected leg to the outstanding leg.

Lug angle and their connections to gusset or other supporting member shall be capable of developing strength not less than 20 percent in excess of force in outstanding leg of main members.

The attachment of lug angle to the main angle shall be capable of developing strength not less than 40 percent in excess of the force in outstanding leg angle,

FOR CHANNEL SECTION

If the main member is channel section, then we should remember that the lug angle, as far as possible should be disposed symmetrically with respect to the section of the member. So lug angle should be provided at the top and bottom in a symmetric way.

The lug angle and their connection to gusset or other supporting member shall be capable of developing strength of not less than 10 percent excess of the force in flange of the channel.

And similarly attachment of the lug angle to the members shall be capable of developing strength of not less than 20 percent in excess of that force. So this is what we have to keep in mind while designing the lug angle.

Minimum two bolts, rivets or equivalent weld length be used for attaching lug angles to gusset plates or other supporting member.

The effective connections of the lug angles should as far as possible be terminated at the end of the member connected.

The fastening of lug angle to the member shall preferably start in advance of direct connection of member to gusset or other supporting members, so these things we have to keep in mind when we are going to design this lug angle.

A tension member carrying a factored tensile load of 180 kN has to convert through a gusset plate of 10 mm thick using 16 mm diameter of ordinary bolt of grade 4.6. The available length of the gusset plate for making connection is 250 mm. Design the member & its connection. Also design the lug angle if required.

Solution:

Gross area required Ag= $T/(f_y/\gamma_{m0})$= 180×$10^3/(250/1.1)$= 792 mm^2

Select angle ISA 75 × 75 × 6 with $Ag = 866\ mm^2$ *and* $r_{min} = 14.6\ mm$

$A_{nc} = (75 - 6/2 - 18) \times 6 = 324\ mm2$

$A_{go} = (75 - 6/2) \times 6 = 432\ mm2$

$A_n = 324 + 432 = 756\ mm^2$

Strength governed due to rupture of net section

$T_{dn} = \alpha \times A_n \times f_u/\gamma_{m1}$= 0.8×756×410/1.25 =198.4 × 10^3 N = 198.4 kN > 180 kN

Hence, the chosen section is safe.

End connection

Strength of M16 bolts in single shear,

$$V_{dsb} = \frac{f_u}{1.25 * \sqrt{3}}(n_n A_{nb} + n_s A_{sb}) b_{lj} b_{lg} b_{pkg}$$

$$V_{dsb} = \frac{400}{1.25 * \sqrt{3}}(1 * 157) = 29\ kN$$

Assume pitch as 40 mm and edge distance as 30 mm.

k_b = *least of [30/(3 × 18), {40/(3 × 18) - 0.25}, 400/410, 1]= 0.49*

Bearing strength of bolts on 8 mm thick angles,= $2.5 \times d \times t \times k_b \times f_{ub}/\gamma_{mb}$

$= 2.5 \times 16 \times 6 \times 0.49 \times 400/1.25 = 37.63 \times 10^3$ N = 37.63 kN

Therefore, bolt value = 29.0 kN

No. of bolts required = 180/29.0 = 6.2 ≈ 7

Length of gusset plate = 6 × 40 + 2 × 30 = 300 mm > 250 mm, therefore lug angle should be used.

- ***Lug Angle***

Gross area of connected leg, $A_{gc} = (75 - 6/2) \times 6 = 432\ mm^2$

Gross area of outstanding leg, $A_{go} = (75 - 6/2) \times 6 = 432\ mm^2$

If main member is an angle section, then join lug angle to the outstanding leg of the main angle. If unequal angle is used the load gets distributed in the ratio of gross areas of connected leg to the outstanding leg.

Load on outstanding leg of main angle = 180 × 432/(432 + 432)= 90 kN

Load on lug angle = 1.2×90 kN = 108 kN [**Cl. 10.12.2, IS 800-2007]**

Net area required for the lug angle $= T/(f_y/\gamma_{m0}) = 108 \times 10^3/(250/1.1) = 475\ \text{mm}^2$

Select ISA 60 × 60 × 5 as lug angle with $A_g = 575\ mm^2$

Let assume that the section is weakened by one row of 16 mm diameter bolt.

So, the net area available = 575 – 18×5 = 485 mm^2

Connection of lug angle with gusset plate

No. of bolts required = 108/29.0 = 3.72 ≈ 4

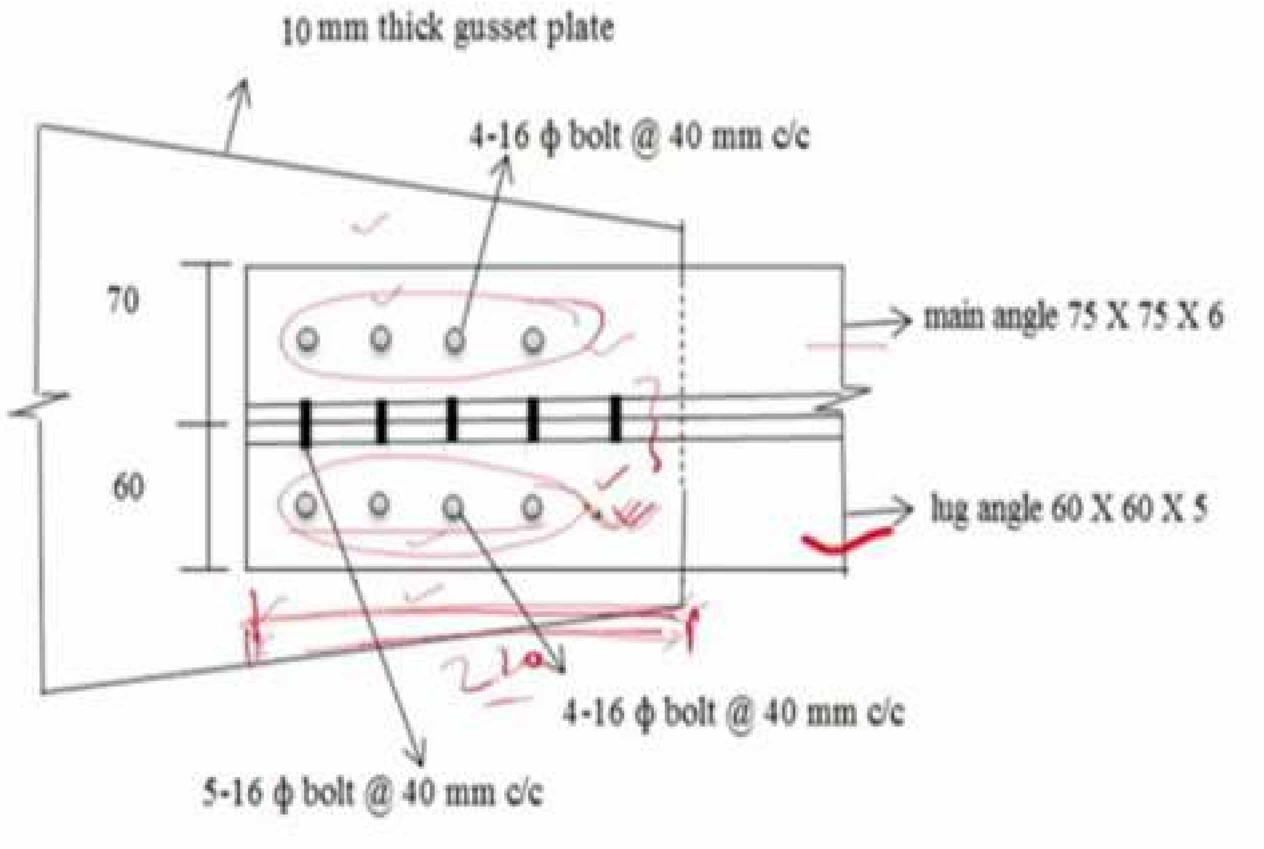
Length of gusset plate = 3 × 40 + 2 × 30 = 180 mm

Connection of lug angle to main angle

No. of bolts required to connect outstanding leg of two angles =1.4× 90/29.0 = 4.34 ≈ 5

Length of gusset plate = 4 × 40 + 2 × 30 = 220 mm.

The arrangement of bolts for connecting lug angle is shown in following Figure



10 mm thick gusset plate
4-16 ϕ bolt @ 40 mm c/c
70
main angle 75 X 75 X 6
60
lug angle 60 X 60 X 5
210
4-16 ϕ bolt @ 40 mm c/c
5-16 ϕ bolt @ 40 mm c/c

SPLICES IN TENSION MEMBERS

Splices are used if the available length is less than the required length of a tension member. When the single piece of tension member of requisite length is not available then we may have to connect with another piece of member with the use of splices.

When two different types of tension members are joined with the use of splices, we may face different type of problem like if we do not join two sides properly then eccentricity of the joint will come into picture and because of eccentricity moment will come into picture.

While joining with splices, avoid ecentricity.

when tension members of dissimilar thickness are to be connected, packing or filler plates are introduced and if thickness is in excess of 6 mm shall be decreased by a factor as given by

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

Where, t_{pk} = thickness of the packing plate in mm

As per IS specification, the splice connection should be designed for a force of at least 0.3 times the member design capacity in tension or design action whichever is more

Design a tension splice to connect two tension member plates of size 200 X 10 and 220 X 12. The member is subjected to a factored tensile force of 280 kN. Use M20 grade 4.6 ordinary bolts for the connection.

Solution:

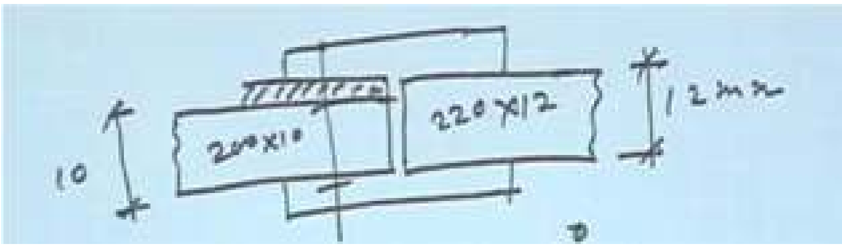
Splice will be provided in both sides of the tension members. Therefore, bolt value needs to be calculated for double shear.

Shear strength of M20 bolts in double shear

$$= \frac{\frac{f_u}{\sqrt{3}}\left(n_n A_{nb} + n_s A_{sb}\right)}{\gamma_{mb}} = \frac{(400/\sqrt{3}) \times (2 \times 245)}{1.25} = 90.6 \times 10^3 \text{ N} = 90.6 \text{ kN}$$

Assume pitch as 50 mm and edge distance as 30 mm.

k_b = *30/(3 × 22), 50/(3 × 22) - 0.25, 400/410, 1 = 0.454*



Bearing strength of bolts on 10 mm thick plate = $2.5 \times d \times t \times kb \times fub/\gamma mb$ = $2.5 \times 20 \times 10 \times 0.454 \times 400/1.25 = 72.64 \times 103$ N= 72.64 kN

So the bolt value = 72.64 kN

No. of bolts required = 280/72.64 = 3.85 ≈ 4

Thickness of packing required = 12 – 10 = 2 mm

Since the thickness of packing is less than 6 mm, no additional bolt will be necessary to connect it with the plate.

Thus, 4 nos. of bolts will be required on the splice with a pitch of 50 mm as shown in the figure below.

Check for strength at critical section:

Strength of main plate at critical section = $T_{dn}=0.9\times f_u\times A_n/\gamma_{m1}$

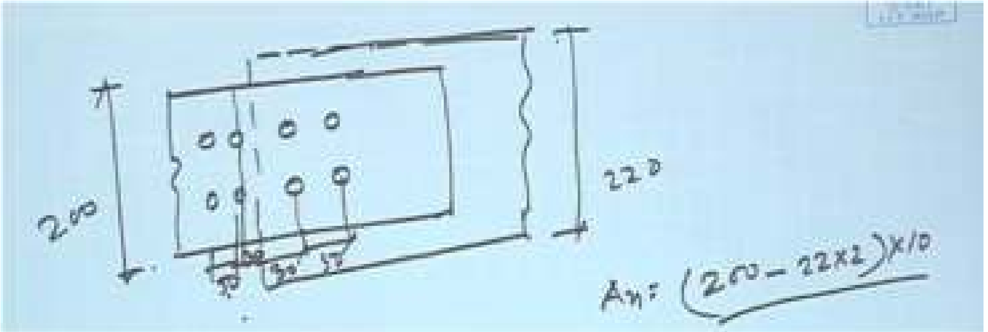
$=0.9\times 410\times[(200-22\times 2)\times 10]/1.25 = 460.5 \times 103$ N = 460.5 kN > 280 kN

Thus the section is OK.

Design strength due to yielding of gross section:

$T_{dg}= f_y\times Ag/\gamma_{m0} = 250\times 200\times 10/1.1 = 454.5 \times 103$ N = 454.5 kN > 280 kN.

So the design tensile strength of the member will be 454.5 kN.



Let the thickness of splice plate is *t.*

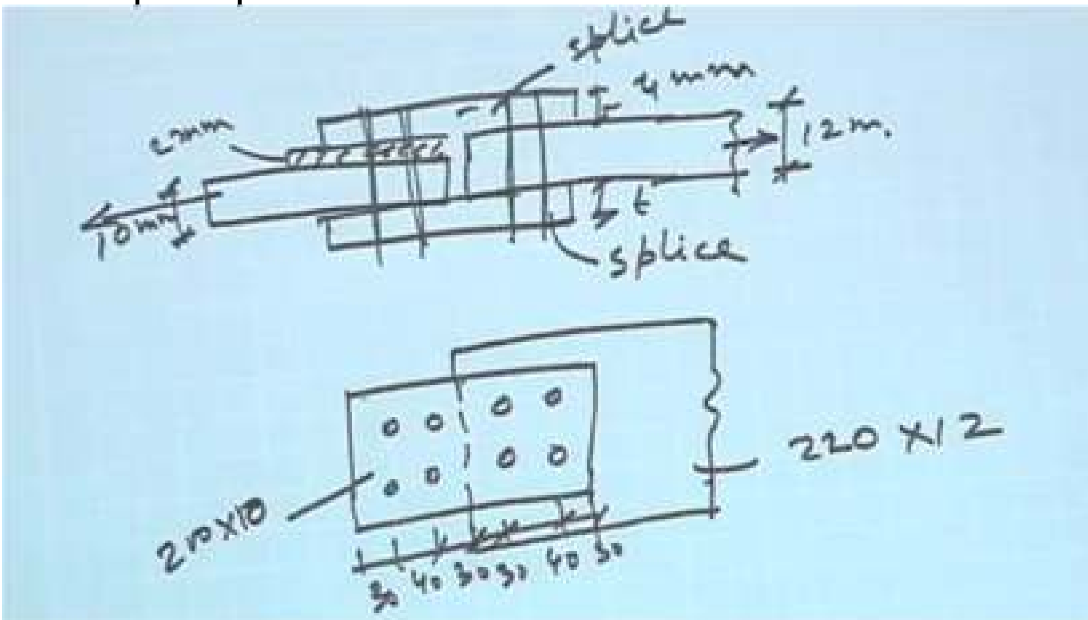
Thus the strength of splice plate will be=$0.9 \times 410 \times [(200-22 \times 2) \times t \times 2]/1.25$

=. $92.1024t \times 10^3\ N = 92.1024t\ kN$

The splice will be designed for $0.3 \times 454.5\ kN = 136\ kN$ *or the factored tensile load of* 280 kN which ever is more.

Thus the thickness of the splice plate will be: $t = 280/92.1024 = 3.04$ mm

Let use 4 mm thick splice plate on both side of the member.



The splice plate is provided in such a way that it is becoming symmetric, so we are maintaining the symmetric of the section so that the eccentricity does not develop in the member.