# UNIT-I

## INTRODUCTION

Generally the design of a building consists of two Parts.

1) Functional Design → Planning the building to its requirements Lighting, Ventilation, aesthetic view
2) Structural Design
↓
Proportioning various elements of the building such as loads acting on it are transfered safely to the ground and at the same time unneccessary excess material not used.

→ For transfering load to the ground, various materials like

Asbestos sheets
Tiles
Bricks
Cement concrete
Reinforced concrete
Steel
Aluminium

} are used.

→ However main body of the present day structure consists of RCC and (&) steel

## Common steel structures

Steel has high strength per unit mass ∴ Hence it is used in constructing Large column free structures.

1) Roof trusses for Factories, cinemahalls, auditoriums etc.
2) Trussed bents, crane girders, columns etc; industrial Structures
3) Roof trusses and columns to cover platforms in railway stations and busstations

4) Single layer & double layer domes for exhibition halls indoor stadiums etc. ②

5) plate girder & Truss bridges for roads & railways

6) Transmission towers for microwave & electric power

7) Watertanks

8) chimneys.

Advantages of steel structures

→ High tensile strength.

→ Good quality and high durability

→ Speed of construction ~~is~~

→ It can be strethened at any time, just welding.

→ By using bolted connections can be dismantled easily

→ material reusuable

Disadvantages

1) Corrosion

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

3) Steel members are costly.

## Types of structural steel

→ Steel is an alloy of iron and carbon by adding small percentage of manganese, sulphur, phosphorous, chrome nickel and copper. Special properties can be imparted to iron and a variety of steels can be produced.

The effect of different chemical constituents on steel are generally

i) Increased quantity of carbon and manganese imparts higher tensile strength.

ii) Increased sulphur and phosphorous beyond 0.06% imparts brittleness, and fatigue strength

iii) Chrome and nickel imparts corrosion resistance to steel

iv) Addition of a small quantity of copper also increases the resistance to corrosion.

## Types of structural steel

1) IS 226 (Standard quality)

2) IS 2062 (fusion welding quality)

3) IS 961 (High Tensile steel)

4) IS 1977 (Ordinary quality)

5) IS 8500 (Medium & high strength qualities)

### 1) IS 226

→ Most commonly used for general construction purposes of buildings, bridges, towers etc,

→ Riveting, bolting can be done for all thickness but welding is permitted for thickness ≤ 20mm only

→ Carbon content : 0.23 to 0.25%

→ Elongation : 23%.

→ Designated as Fe 410- cu-S (or) fe410-S, 410 refers to ultimate tensile strength of 410 Mpa (410 N/mm²)

→ Also known as Grade E250 steel in which 250 refers to yield strength.

2) IS2062

→ Steel commonly used for General construction Purpose. Particularly suitable for structure subjected to Dynamic loads and impact loads such as bridges, girders.

→ Designated as fe 410-WA, Fe410-WB, Fe410-WC

→ suitable for welding of all thickness

→ Carbon content : 0.20% - 0.25%

→ Elongation = 23%

3) IS 961 (High tensile steel)

→ Greater strength & atmospheric corrosion resistance

→ Fe 570-HT for structure with fabrication by methods other than fusion welding

→ Fe 540-W-HT: for structures where fusion welding is involved

→ Carbon content = 0.27% for fe 570-HT

→ Elongation = 20%

4) ordinary quality steel is not supported by test result may be permitted to use for unimportant members.

→ Mild steel (standard quality) and High tensile steel of weldable quality are considered for design.

## Rolled steel sections

Like concrete, steel sections of any shape and size Cannot be cast on site, since steel needs very high temperature to melt it and roll into required shape. User has to cut them to the req. length and use required sections for the steel framework. Many Steel sections are readily available in market and are frequent in demand such steel sections are known as "Regular steel sections".

→ some steel sections are not in use commonly, but the steel mills can roll them if orders are placed. Such steel sections are known as special sections

## Various type of rolled steel sections

i) Rolled steel I - sections (Beam sections)

ii) " " channel sections.

iii) " " Angle "

iv) " Tee "

v) " Bars

vi) " Tubes

vii) " plates

viii) " Flats

ix) " sheets & strips.

## i) Rolled steel I sections:

a) ISJB - Indian standard Junior beams

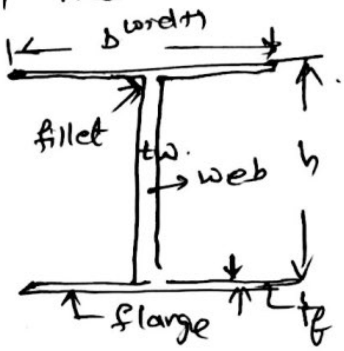
b) ISLB - " Light "

c) ISMB - " medium "

d) ISWB - Indian standard wide flange beams
e) ISHB " Heavy beams

→ These sections are designated, the series they belong to followed by depth (in mm) and wt per metre run.

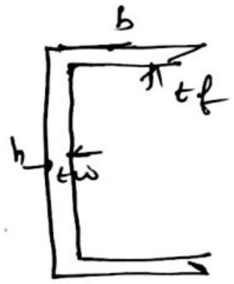
Ex: JSMB 500 @ 0.852 KN/m
ISWB 600 @ 1.423 KN/m
ISHB 450 @ 0.855 KN/m
ISHB 450 @ 0.907 KN/m
ISWB 600 @ 1.312 KN/m



Rolled steel I section

ii) Rolled steel channel sections

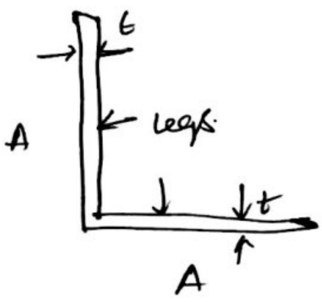
a) ISJC
b) ISLC
c) ISMC
d) ISSC - Indian standard Special channel



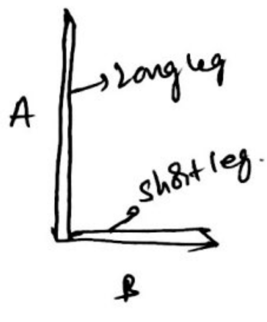

→ These are designated as same as ISMC 300 @ 0.351 KN/m.

iii) Rolled steel angle sections

a) ISA - Indian standard Equal angle
b) ISA - " " unequal "



equal angle



unequal angle

ISA 150, 150, 12mm thick
JSA 150x150x12.

ISA 150, 115, 10mm th
JSA 150x115x10.

iv) Rolled Steel T section

1) ISNT
- Rolled Normal T section
2) ISDT
- Rolled Deep legged T sect
3) ISLT - Light weight T bars
4) ISMT - medium " "
5) IS HT - H-sections bars

Ex: ISNT 125 @ 274 N/m

## Properties of Structural Steel

The properties of steel required for Engineering design may be classified as

i) physical properties

ii) mechanical properties

### i) Physical properties

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

b) $E = 2\times10^5\ N/mm^2$ c) $\mu = 0.3$ (0.25 to 0.3) d) $G = 0.769\times10^5\ N/mm^2$

e) coefficient of thermal expansion, $\alpha_t = 12\times10^{-6}/^\circ C$

### ii) Mechanical properties

a) yield stress $f_y$

b) Tensile & ultimate stress $f_u$

c) Maximum percentage elongation on a standard guage Length

d) Notch toughness.

### Mechanical properties of steel confirming to IS code 2062

| S.NO | Grade/classification | ($f_y$) yield stress ($N/mm^2$) t<20mm | t=20-40mm | t>40mm | UTS ($f_u$) | % Elongation |
|---|---|---|---|---|---|---|
| 1) | E250(Fe410WA) | 250 | 240 | 230 | 410 | 23 |
| 2) | E250(Fe410W)B | 250 | 240 | 230 | 410 | 23 |
| 3) | E250(Fe410W) C | 250 | 240 | 230 | 410 | 23 |
| 4) | E300(Fe440) | 300 | 290 | 280 | 440 | 22 |
| 5) | E350(Fe490) | 350 | 330 | 320 | 490 | 22 |
| 6) | E410(Fe540) | 410 | 390 | 380 | 540 | 20 |
| 7) | E450(Fe570)D | 450 | 430 | 420 | 570 | 20 |
| 8) | E450(Fe590) E | 450 | 430 | 420 | 590 | 20 |

Loads Various Loads act on a structure classified as

a) Dead loads (DL) b) Imposed loads (IL) (c) Wind loads (WL)

d) Earthquake Loads (EL) (e) Erection Loads (ER) f) Accidental loads (AL)

g) Secondary effects

(a) Dead Loads

→ Include weight of all permanent construction.

→ In building weight of roofs, floors, floor finishes, wall, beams, columns, footing, architectural finishing etc; constitute dead load.

→ These loads may be assesed by estimating the quantity of each material and then multiplying it with unit wt.

→ The unit wt of various materials are

| | |
|---|---|
| Asbestos sheets | $0.13 \, kN/m^2$ |
| mangalore tiles | $0.785 \, kN/m^2$ |
| plain concrete | $24 \, kN/m^3$ |
| RCC. | $25 \, kN/m^3$ |
| steel | $78.5 \, kN/m^3$ |

(b) Imposed loads

i) Live load
ii) crane Load
iii) Snow load
iv) Dust load
v) Hydrostatic & earth pressure
vi) Impact load
vii) Horizontal loads on parapets

(c) Wind loads The force exerted by the horizontal component of wind is to be considered in design of buildings, towers etc;

→ The wind force depends upon the velocity of wind, shape, size and location of a building

d) Earthquake Loads

→ Cause movement of foundation of structures
→ Due to inertia additional forces develop on super structure.
→ The total vibration caused by earthquake may be resolved into three mutually perpendicular directions, 1 vertical & 2 Horizontal directions
→ To calculate seismic forces they are two methods
i) Seismic coefficient method
ii) Response spectrum method.

(e) Erection loads

→ Prefabricated & precast members are subjected to diff types of supports and different types of loads during erection compared to types of supports and types of loads after erection
→ It is responsibility of the engineer to see that the structure & part of structure donot fail during erection
→ Dead load, wind load and Imposed live load during erections shall be considered along with special erection loads

f) Accidental loads

i) Impact & collision
ii) Explosions
iii) Fire

} The probability of occurence of such Loads may be quite less.

g) Secondary effects

→ Differential settlement of foundations
→ " shortening : columns
→ Eccentric connections.

## Load combinations

Combination of Loads is necessary to ensure the safety and economy in design.

The Load combinations recommeded are

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

DL = Dead load
IL = Imposed load
TL = Temp load
EL = Earthquake "
WL = wind load

## Design philosophy

The aim of design is to decide shape, size and connection details of members so that the structure being designed will perform satisfactorily during its intended life.

Methods are

i) Working stress method
ii) ultimate Load design "
iii) Limit state method

### ① Working stress method

→ oldest systematic analytical design method

→ In this stress strain relationship is considered linear till the yield stress

→ Ratio of yield stress to working stress = f.s

→ Permissible stress = $\frac{\text{Yield stress}}{F.S}$

→ The following load combinations are considered and increase of permissible stress by 33% is permitted.

Stress due to DL + LL ≤ permissible stress

Stress due to DL + WL ≤ permissible stress

Stress due to DL + LL + WL ≤ 1.33 permissible stress

Limitations of WSM

→ It gives the impression that factor of safety ~~times~~ times the working load is the failure load, which is not true

→ gives uneconomical sections.

Advantages of WSM

1) Simple 2) Reasonably reliable 3) As ~~working~~ working stresses are low, serviceability requirements are satisfied automatically

ii) Ultimate Load method

→ The limitation of working stress method to asses actual load carrying capacity, made researchers to develop ultimate Load method, which also known as 'Load factor method'.

→ when applied to steel structure referred as plastic design method.

→ In this safety measures are introduced by suggesting a load factor ie, $\frac{\text{Design load}}{\text{working load}}$

Suggest load factors

| | |
|---|---|
| 1) Dead load | 1.7 |
| 2) DL + IL | 1.7 |
| 3) D.L + W.L + seismic load | 1.7 |
| 4) D.L + I.L + W.L + seis load | 1.3 |

ii) ~~Limit state~~ design



Advantages

1) Redistribution of internal forces is accounted

2) varied selection of load factors allows

Disadvantages

1) Does not guarntee serviceability performance.

iii) Limit state design

→ Comprehensive method which will take care of both strength and serviceability requirements.

→ Aim of a design is to see that the structure built is safe and serves the purposes for which it is build.

→ In this method of design various limiting conditions are fixed to consider a structure as fit

Deflection Limits

→ These are specified from consideration that excess deformations do not cause damage to finishing.

Analysis: Determination of axial force, shear force, BM, Torsional moments } acting on diff. members of structure due to applied loads & load combinations

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Design: selection of shape and size of a member and connection details of various members (beam to beam, beam to column, column to foundation) to resists all forces and moments determined in analysis safely and economically

## Permissible stresses (or) Allowable stresses

It is the ratio of yield stress to ~~working stress~~ Factor of safety.

IS: 800: 1984 specifies the permissible stress in its various Sections:

a) Permissible avg. shear stress $= 0.40 f_y$

b) Permissible max. shear " $= 0.45 f_y$

c) " axial tensile " $= 0.60 f_y$

d) " bending " " $= 0.66 f_y$

e) " bending compressive " $= 0.66 f_y$

f) " bearing stress $= 0.75 f_y$

g) combined bearing and bending " $= 0.90 f_y$

## CONNECTIONS

The various types of connections used for connecting structural members are

1) Riveted connections
2) Bolted "
3) Pin "
4) Welded "

## (1) RIVETED CONNECTIONS

→ In the riveted connections, rivets are used.

→ A piece of round steel forged in place to connect two or more than two steel members is known as Rivet.

→ Rivets for structural members are manufactured from mild steel and High tensile rivet bars.

**Types of Bolted Joints**

If the load line is assumed to pass through the CG of the bolt group then there are two types of bolted joints viz. lap joint and butt joint. The other case ie when the Join line does not pass through the CG of the bolt group gives rise to eccentric connections.

(a) **Lap joint**

- Here the two members to be connected are overlapped and jointed as shown in Fig 1.1(a).
- Fig. 1.1 (b and c) shows respectively the single bolted lap joint and the double bolted lap joint. The load lines in the two members of lap joint do not coincide and hence lap joint has an eccentricity leading to the formation of an undesirable couple there by inducing tension in bolt which may lead to failure of joint as shown in Fig. 1.1(d).
- Due to this eccentricity only, the stresses are distributed unevenly across the contact area between the bolts arid members to be connected
- Cl. 10.5.1.2 of IS 800:2007 states that minimum length of lap shall not be less than four times the thickness of thinner part being jointed or 40 mm, whichever is more.

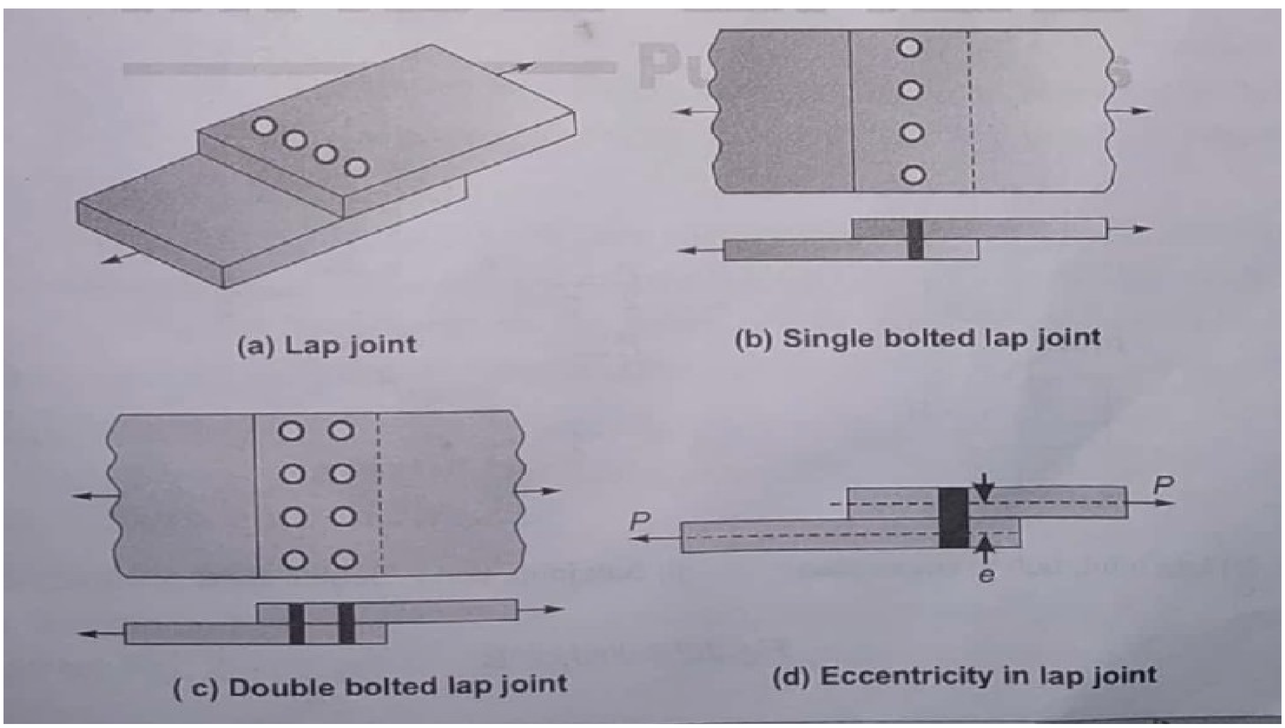
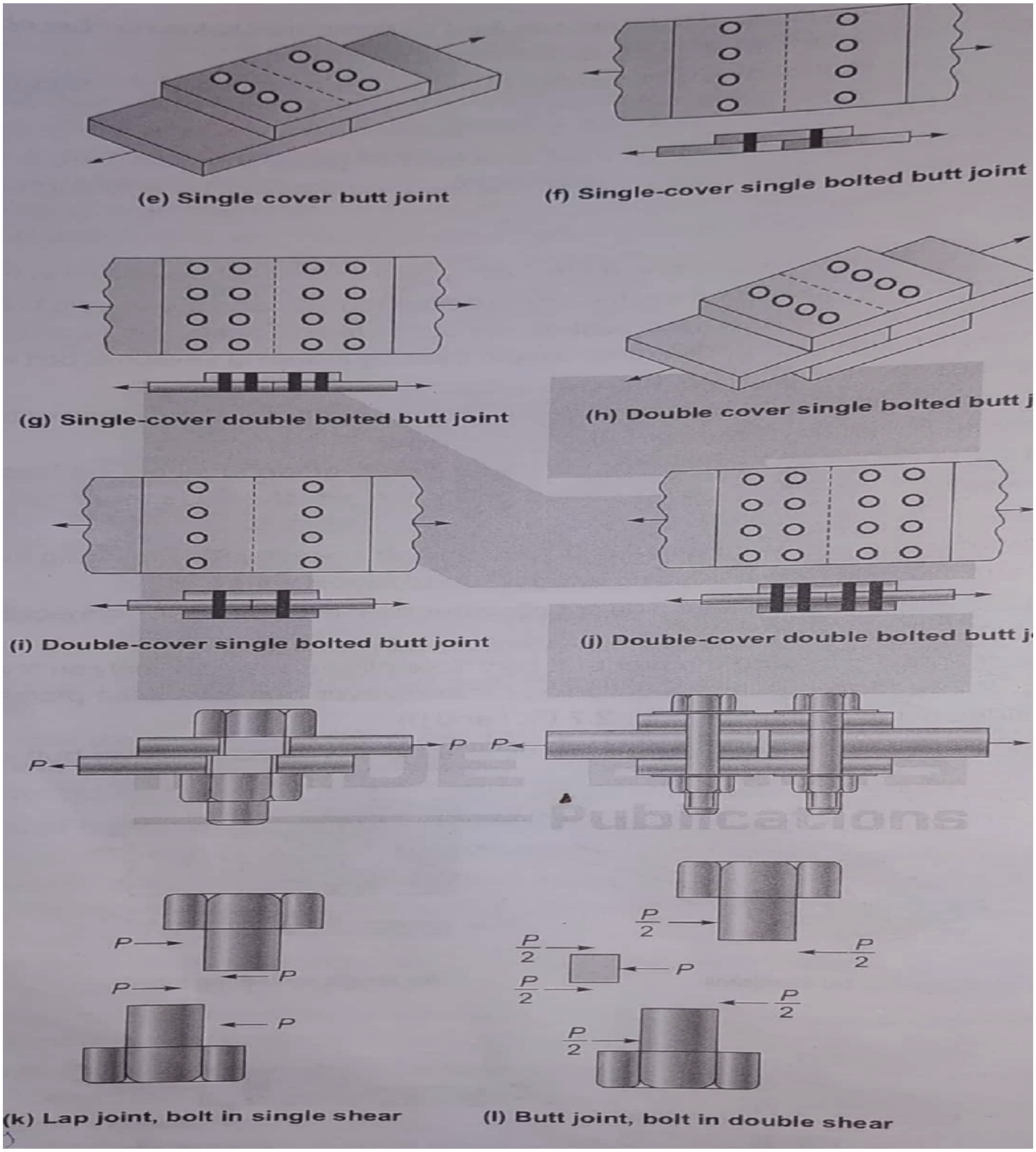



**Fig 1.1 lap joint**

(b) **Butt joint**

- Here the two members to be connected are placed end to end thereby bringing the load lines in the two members in one line and reducing eccentricity to almost zero.
- Additional cover plate(s) on either side or bath sides can be provided to connect the main plates as shown in Fig. 1.1 (e and h).
- The butt joint is called as single cover butt joint if the plate is provided only on one side of the main plate (Fig. 1.1 (e, f and g) and is called as double cover butt joint if the plates are provided on both the sides of the main plate (Fig1.1 (h, i and j).
- Fig. 1.1 (k ard I) shows the transfer of forces in lap joint and double cover butt joint respectively




1.1 butt joint

**Failure of Bolted Joints**

**(a) Shear failure of bolts**: When plates slip due to the applied forces, shear stresses are generated. It may be possible that maximum factored shear force exceeds the shear capacity of the bolt. Shear failure of bolt takes place at bolt shear plane. However the bolt may fail in single shear or double shear as shown in Fig. 1.2(a).

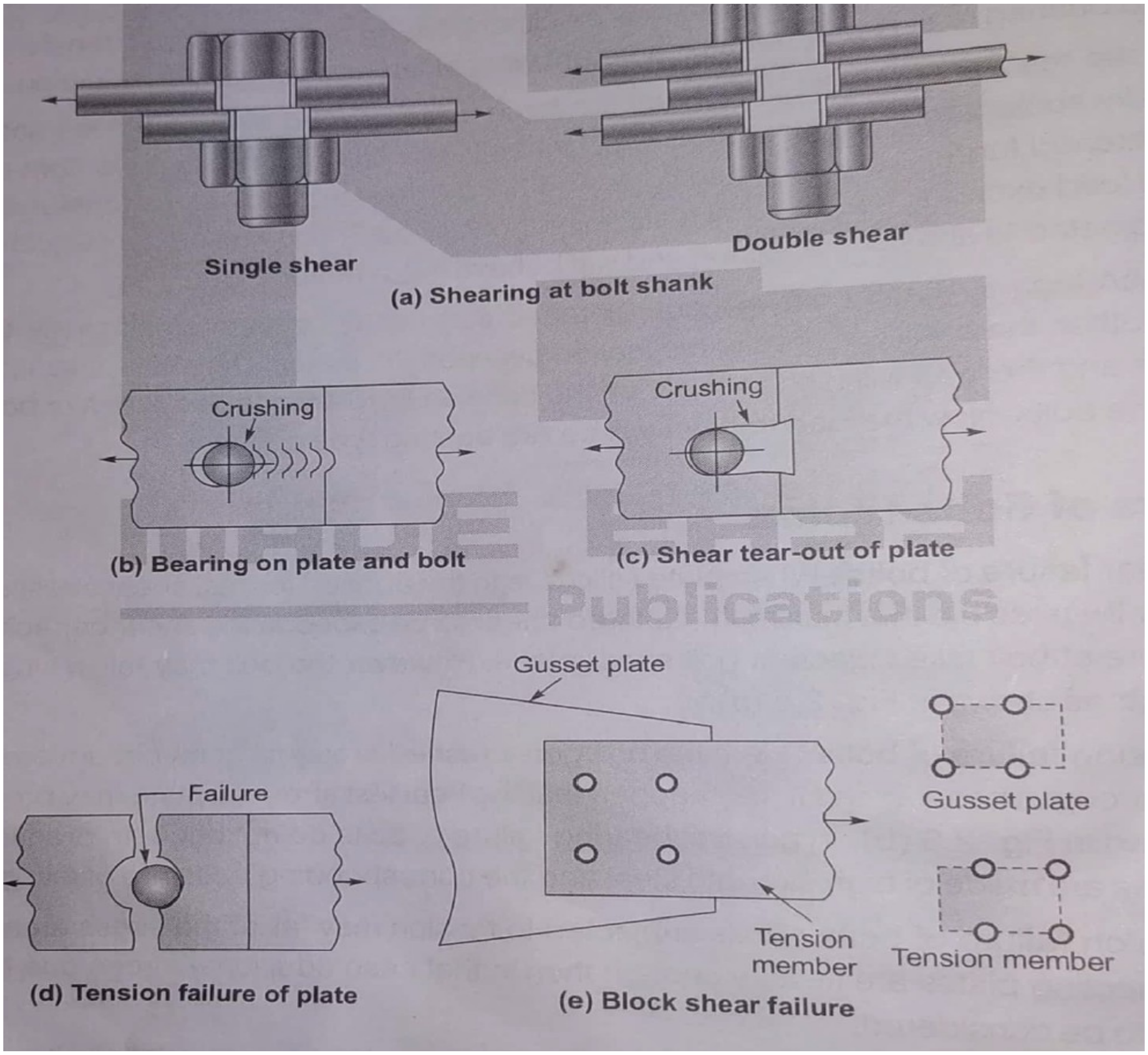
**(b) Bearing failure of bolts :** Here the bolt gets crushed around a semi-circumference The plate may be strong in bearing and it may happen that the heaviest stressed plate may press the bolt shank as shown in Fig. 1.2 (b). In general bearing failure of bolts do not occur in practice except when the plates are made of high strength steel and the corresporiding bolts are of low grade steel.

**(c) Tension failure of bolts :** Bolts subjected to tension may fail at the stress area: In case any of the connecting plates are flexible enough then in that case additional forces due to prying action has also to be considered.

**(d) Tension/tearing failure of plates :** Tension Failure of plates occurs when bolts are stronger than the plates, Tension on both the gross area (i.e yielding) and the net effective area (i.e rupture) must be considered. Fig1.2 (d) shows the tension failure of plate in rupture.

**(e) bearing failure of plates:** When ordinary bolts are subjected to shear forces then sip takes place and bolts come in contact with the plates, It may be quite possible that plate may get crushed mule Prate material is weaker than the bolt material as shown in Fig. 2.8 (b). This bearing failure gets Complicated further due to the presence of nearby bolt or nearness of an edge in the load direction. he bearing strength gets affected by bolt spacing and the edge distance. One of the possible mode of failure resulting from too much bearing is the shear tear-out at the end of the connected, member as shown in Fig. 1.2 (c).

**(f) Block shear failure :** Many a times bolts may have been placed at a lesser end-distance than required which may lead to plates to shear out which can in fact be avoided by adherence to edge distance. Fig. 1.2 (e) shows the failure of joint in block shear failure which may occur when a block of a material within the bolted area breaks away from the remaining of the area. This possibility of failure increases if bolts used are of high strength and fewer bolts are used for making the connection. In this type of failure, shear on one plane and tension on perpendicular plane occurs leading to fall of a portion of plate.




**1.2 various types of failure in bolted joints**

**Specifications of bolted joint**

a) **Diameter of bolt holes ($d_o$):**

$d_o$= Nominal diameter of bolt (d)+ 1 mm (for d- 12mm to 14mm)

$d_o$ = Nominal diameter of bolt (d)+ 2mm (for d- 16mm to 24mm)

$d_o$ = Nominal diameter of bolt (d)+ 3mm (for d2 27 mm)

b) Pitch (P) It is distance between centers of two consecutive bolts measured along parallel to direction of force or stress in a member. For wide plates pitch may also defined as the centre to centre distance of bolts measured along the length of member on the connection. When bolts are placed staggered the pitch will be referred to as staggered pitch

1. **Minimum pitch (Pmin) :**

2.5 x Nominal diameter of bolt (2.5d)

2. **Maximum pitch (Pmax)**:
   - 12t or 200mm whichever is less for compression member
   - 16 t or 200mm whichever is less for tension member
   - The distance between the centers of any two consecutive bolts should not exceed 32 t or 300mm whichever is less
   - Maximum gauge should nol more than 100 + 4t or 200 mm whichever is less
   - 32t or 300mm whichever is less for tacking or stitch bolts (when plates are not exposed to weather)
   - 16t or 200mm whichever is less for tacking or stitch holts (when plates are exposed to weather) In case of two lats, angles, channels or lee section maximum pitch of tack bolts (In which tack or stitch bolts are to be provided along length to connectcach of them)
   - Not exceeding 600mm for compression members
   - Not exceeding 1000mm for tension members

**c) Gauge (g)**: It is distance between adjacent bolt lines, or it is centre to centre distance between two consecutive bolts measured along the width of member or connection

**d) End distance(e):** It is the distance from centre of bolt hole to the nearest edge of member or cover plate in the direction of stress or force.

**e) Edge distance:** It is the distance from centre of bolt hole to the nearest edge of member or cover plate at right angle to the direction of stress. n

I. **Minimum edge distance** : Minimum edge distance 1.7 x diameter of hole for sheared or hand flame cut edges
Minimum edge distance =1.7 x diameter of hole in case of rolled machine flame cut edges

II. **Maximum edge distance** 15 x hole diameter incase of rolled, machine flame cut edges (H) Maximum edge distance Maximum edge distance to nearest edge of bolt hole to an edge of un stiffened part should not exceed 250 where t is 12te where e- thickness of thinner outside plate 40mm + 4t, where t is thickness of thinner outside plate (for corrosive Environments)

## Strength and Efficiency of bolted Joints :-

(1) Shear Strength of Bolt :-

The design shear strength of Bolt $V_{dsb}$ as governed shear strength is given by

$$V_{dsb} = V_{nsb}/\gamma_{mb} > V.$$

$$V_{nsb} = \text{nominal Shear Strength of Bolt} = \frac{f_u}{\sqrt{3}}\left[n_n A_{nb} + n_s A_{sb}\right]$$

where $f_u$ = ultimate strength of Bolt

$n_n$ = No. of Shear planes with threads

$n_s$ = No. of shear planes without threads.

$A_{sb}$ = Nominal plain area of bolt

$A_{nb}$ = Net Shear area of the bolt.

The nominal shear capacity of bolts for long Joints, long grip length and with Packing plates, will be less than and is modified as

$$V_{dsb} = \frac{f_u}{\sqrt{3}\gamma_{mb}}\left[n_n A_{nb} + n_s A_{sb}\right] \beta_{lj}\, \beta_{lg}\, \beta_{pkg}$$

Reduction factor for Long Joints ($\beta_{lj}$) :-

If length of Joint is exceeds 15 times bolt diameter the shear capacity reduced.

This reduction factor for long Joints is given by

$$\beta_{lj} = 1.075 - \frac{L_j}{200d} \quad \because 0.75 \le \beta_{lj} \le 1$$

Reduction factor for Large griplength ($\beta_{lg}$) :-

If the total thickness of the connected plates exceeds five times the nominal dia of bolt then shear capacity of the bolt gets reduced by a factor $\beta_{lg}$ which is given by

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

Reduction factor for packing plates ($\beta_{pkg}$) :-

If thickness of packing plate exceeds 6mm then shear capacity of bolts gets reduced. by a factor $\beta_{pkg}$ is given by

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

## Bearing strength of Bolt:-

The design strength of a bolt on any plate, $V_{dpb}$ as governed by bearing is given by

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

$V_{npb}$ = nominal bearing strength of bolt $= 2.5\, k_b\, d\, t\, f_u$

$k_b$ is smaller of $\frac{e}{3d_0}, \frac{p}{3d_0} - 0.25, \frac{f_{ub}}{f_u}, 1.0$

## Tensile strength of Bolt:-

The design tensile strength of Bolt is given by

$$T_{db} = \text{min of} \left[ \frac{0.9 f_{ub} A_{nb}}{\gamma_{mb}}, \frac{f_{yb} A_{sb}}{\gamma_{m0}} \right]$$

$\gamma_{mb} = 1.25$

$\gamma_{m0} = 1.1$

## Tensile strength of plate:-

The design tensile strength of plate is given by

$$T_{dp} = \text{min of} \left\{ \frac{0.9 A_n f_u}{\gamma_{m1}}, \frac{A_g f_y}{\gamma_{m0}} \right]$$

$\gamma_{m1} = 1.25$

$\gamma_{m0} = 1.1$

## Bolt value (or) strength of bolt:-

Bolt value = minimum of $[V_{dsb}, V_{dpb}]$

## Tensile strength of Bolt:-

The design tensile strength of Bolt $T_{db} = \text{min}\left[ \frac{0.9 f_{ub} A_{nb}}{\gamma_{mb}}, \frac{f_{yb} A_{sb}}{\gamma_{m0}} \right]$

$A_{sb}$ = shank area of bolt

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

## Tensile strength of plate

The design strength of solid plate $T_{nd}P = \left[ \frac{0.9 A_n f_u}{\gamma_{m1}}, \frac{A_g f_y}{\gamma_{m0}} \right]$

$$A_n = \left[ B - nd + \sum_{i=1}^{m} \frac{P_i^2}{4g_i} \right] t \quad \text{for staggered bolting}$$

$$A_n = (B - nd)\, t \quad \text{for chain bolting.}$$

Strength of Bolted Joint

Strength of Bolted Joint = min of $[V_{dsb}, V_{dpb}, T_{db}]$

Efficiency of a Bolted Joint :-

It is the ratio of strength of Joint to the strength of the Solid plate

$$\eta = \frac{\text{Strength of bolted Joint}}{\text{Strength of Solid plate}} \times 100$$

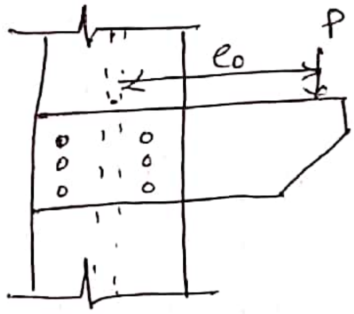
# Design of Bracket Connections:-

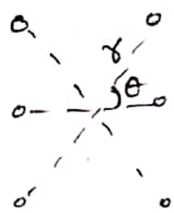
- when the bolts are subjected to direct shear and torque due to shear then the connection is classified as bracket connection type-I
- when the bolts are subjected to shear and tension then the connection is called bracket connection Type-II

## Bracket Connection Type -I.

This occurs when line of action of load is in the plane of connection and the centre of gravity of the connection is the center of rotation. The whole bolt group is subjected to shear and torsion

Assume load on the joint is shared equally by all the bolts and they force in any bolt due to direct load is




$$F_1 = \frac{P}{n}$$

where $n$ = No. of bolts
$P$ = Total load on joint.

Also force in any bolt due to torsion is directly proportion to its distance from the center of rotation of the connection

$$F_2 \propto r$$
$$F_2 = kr \qquad \therefore k = \frac{F_2}{r}$$
$$k = \frac{F_2}{r}$$

Thus Torque about center of rotation of the bolt group $= F_2 r = kr^2$

Total resisting torque $\Sigma kr^2 = \frac{F_2}{r}(\Sigma r^2)$

The torque resisting must be equal to or greater than the torque on the connection i.e

$$M = \frac{F_2}{r}\Sigma r^2$$
$$Pe_o = \frac{F_2}{r}(\Sigma r^2)$$
$$F_2 = \frac{Pe_o r}{\Sigma r^2}$$

This force $F_2$ is maximum when distance $r$ is maximum, let it be $r_n$

$$F_2 = \frac{Pe_o r_n}{\Sigma r^2}$$

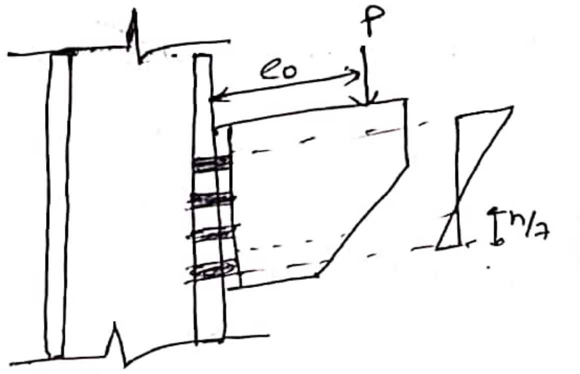
Thus resultant F of the forces $F_1$ and $F_2$ on the critical bolt is

$$F = \sqrt{F_1^2 + F_2^2 + 2F_1F_2\cos\theta}$$

## Type II – Bracket Connection:-

* Here moment in the plane normal to the plane of connection
* The line of action of load doesn't lie in the plane of bolt group.

Also the line of rotation doesn't pass through CG of the bolt group.




The forces acting on bolts are
(i) Direct Shear
(ii) Tension due to moment.

Direct Shear force $F = P/n$

Direct Shear stress $= F/A$

Assume line of action of rotation lies at $h/7$ from the bottom of bracket
h is the distance from bottom of bracket to top most bolt in the connection.

Thus bolt above line of action will experience direct shear and tension due to moment. Bracket below the line of action provides necessary compression.

Tensile strength of $i^{th}$ bolt $T_i \propto y_i$

$$T_i = k y_i$$
$$k = T_i / y_i$$

Moment of resistance due to tensile force in $i^{th}$ bolt

$$m_i = T_i y_i = k y_i^2$$

The total resistance due to tensile force in bolts

$$m' = \Sigma m_i = k \Sigma y_i^2 = \frac{T_i}{y_i} = \Sigma y_i^2$$

$$T_i = \frac{m' y_i}{\Sigma y_i^2}$$

Tensile force in farthest

- For equilibrium Total Compression = Total tension

$$C = T = \frac{m' \Sigma y_i}{\Sigma y_i^2}$$

Total External moment $= m' + C\bar{y}$

$$= m' + \left[\frac{m' \Sigma y_i}{\Sigma y_i^2}\right]\bar{y} \quad \therefore \bar{y} = \frac{2h}{21}$$

$$m = m'\left[1 + \frac{\Sigma y_i}{\Sigma y_i^2} \times \frac{2h}{21}\right]$$

$$m' = m \Big/ \left[1 + \frac{\Sigma y_i}{\Sigma y_i^2} \frac{2h}{21}\right]$$

This above equation gives the moment restrained by the bolts in tension from which maximum tensile force in the extreme bolt $T_{db}$ can be calculated.

$$T_{db} = \sigma_t \times A$$

Check the Connection for Collective effect of Shear & tension

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

where

$V_{sb}$ = Shear force on the bolt

$V_{dsb}$ = Design shear strength of bolt

$T_b$ = Tensile force on bolt

$T_{db}$ = Design tensile strength of bolt

## Welded connections

→ Welding consists of joining two pieces of metal by establishing a metallurgical bond between them.

→ The elements to be connected are brought closer and the metal is melted by means of electric arc or oxy acetylene flame along with weld rod which adds metal to the joint.

∴ When two members are connected by means of welds, such connection is known as welded connection.

## Types of welded connections

Based on position of welds and type of joints, the welds are classified into three categories

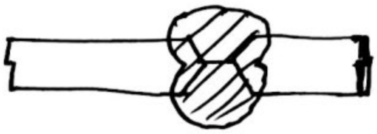
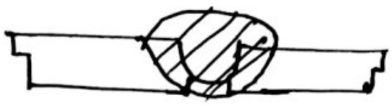
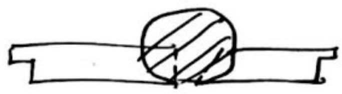
i) Butt weld
ii) Fillet weld
iii) plug & slot weld.

### i) Butt weld

→ It is also known as Groove weld
→ These welds are provided when members to be joined are lined up.
→ These welds are costlier since it requires edge preparation

Depending upon shape of groove, made for welding, butt welds are classified as

i) Square butt weld, on one side
ii) Square " both sides
iii) Single V butt joint
iv) Double " "
v) Single U "
vi) Single J "
vii) Single bevel butt joint

| S.NO. | Type of Butt Joint. | Sketch |
|---|---|---|
| 1) | Square butt weld on one side |  |
| 2) | Square butt weld, both sides |  |
| 3) | Single V butt Joint |  |
| 4) | Double V butt Joint |  |
| 5) | Single U butt Joint |  |
| 6) | Single J butt Joint |  |
| 7) | Single bevel butt Joint |  |

② FILLET WELDS

→ These are provided when two members are to be Jointed in different plane.

→ This situation occurs more frequently.

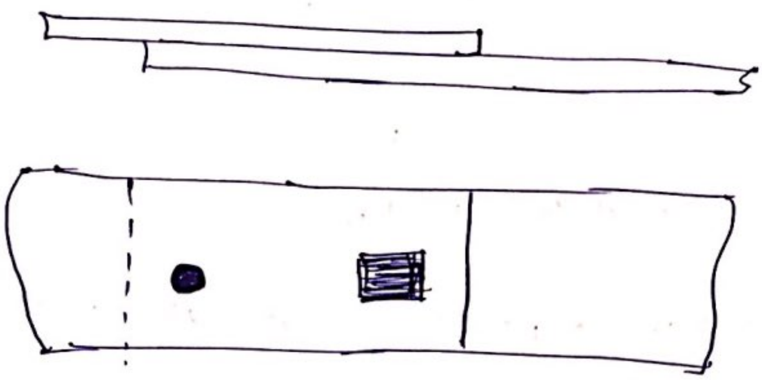
→ fillet welds are more common than butt welds

→ They are easier to make as it requires less surface Preparation

→ But they are not as strong as Groove welds

→ It is a weld of approximately Δlar c/s Joining two surfaces approximately at rt angles to each other in lap Joint, Tee Joint & corner Joint

PLUG WELD : Small holes are made into one plate and is kept over another plate to be connected and then entire hole is filled with filler material.



PLUG WELDS

ix(b)

## ~~A~~dvantages of welded connections

→ Due to absence of gusset plates, connecting angles etc, welded structures are lighter

→ The absence of making holes for fasteners makes welding process quicker

→ Welding is more adaptable than bolting & riveting.
Ex: circular tubes can be easily connected by welding

→ It is possible to achieve 100% efficiency in the joint whereas in bolted connection it can reach a max. of 70-80% only

→ Noise produced in welding is relatively less

→ Good aesthetic appearance

→ These are air tight and water tight. Hence there is less danger of corrosion of steel structures

→ Prefered for making water tanks

→ Joints are rigid

→ Alterations in connections can be easily made in the design of welded connections

## Disadvantages

1) Due to uneven heating and cooling, members are likely to distont in the process of welding
2) There is a greater possibility of brittle fracture in welding
3) A welded joint fails earlier than bolted joint.
4) These are over rigid
5) Proper welding in field conditions is difficult
6) Highly skilled person is required.
7) Inspection of welded joints is difficult and expensive.

## Specifications for Welding

1) Butt weld

→ The size of butt weld shall be specified by effective throat thickness

→ In case of complete penetration butt weld it shall be taken as thickness of thinner part joined.

→ Double U, double V, double J, and double bevel butt welds may be generally regarded as complete penetration butt welds

→ The effective throat thickness in case of incomplete penetration butt weld shall be taken as min. thickness of the weld metal common to the parts joined, excluding reinforcement

→ In absence of data it may be taken as 5/8th of thickness of thinner material

→ The effective length of butt weld shall be taken as the length of full size weld

→ Min. length of butt weld shall be 4 times the size of weld

→ If intermittent butt welding is used, it shall have an effective length not less than 4 times the weld size and space b/w the two welds shall not be more than 16 times the thickness of inner part joined.

2) fillet weld

i) size of fillet weld

a) Size of normal fillet weld shall be taken as minimum weld leg size

b) for deep penetration welds with penetration not less than 2.4mm, size of weld is minimum leg size + 2.4mm

c) for fillet welds made by semi automatic or automatic processes with deep penetration more than 2.4mm,

$$S = \text{min. leg size} + \text{actual penetration}.$$

d) Min. size of fillet weld specified is 3mm. To avoid risk of cracking in the absence of preheating the min. size specified are

| | | | |
|---|---|---|---|
| For less than | 10mm plate | | 3mm |
| " | 10 to 20mm | " | 5mm |
| " | 20 to 32 " | " | 6mm |
| " | 32 to 50 " | " | 8mm |

ii) Effective throat thickness:

→ It shall not be less than 3mm and shall not generally exceed 0.7t, where t is the thickness of thinner plate of elements being welded.

If the faces of plates being welded are inclined to each other, the effective throat thickness shall be taken as K times the fillet size

| Angle b/w fusion faces | 60-90° | 91°-100° | 101-106° | 107-113 | 114-120 |
|---|---|---|---|---|---|
| Constant K | 0.7 | 0.65 | 0.60 | 0.55 | 0.5 |

4) Effective length

→ The effective length of weld is the length of weld for which specified size & throat thickness exist.

→ In drawing only effective length is shown.

→ While welding length made is equal to effective length plus twice the size of the weld.

→ Effective length should not be less than 4 times the size of weld.

5) Lap joint

→ Min. lap should be 4 times the thickness of thinner part joined & 40mm whichever is more

→ The length of weld along either edge should not be less than transverse spacing of welds

(6) Intermittent welds

→ Length shall not be less than 4 times the weld size or 40mm whichever is more

→ Min. clear spacing of ~~members~~ intermittent welds shall be 12t for compression joints & 16t for tensile joints, where t is the thickness of thinner plate joined.

(iii) PLUG WELDS

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

(IV) Design stresses in welds

Butt welds

Butt welds shall be treated as parent metal with a thickness equal to throat thickness and the stresses shall not exceed those permitted in the parent metal.

fillet weld, slot & plug welds

Design strength shall be based on its throat area and shall be given by

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

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

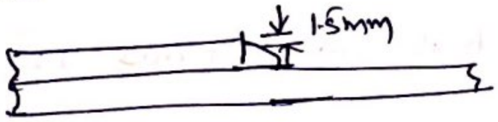
$f_u$ = smaller of the ultimate stress of weld

$\gamma_{mw}$ = 1.25 for shop welds

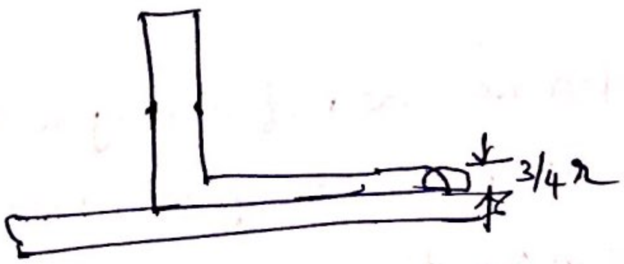
= 1.5 for field welds.

The following provisions are made in the code for fillet welds applied to the edge of a plate

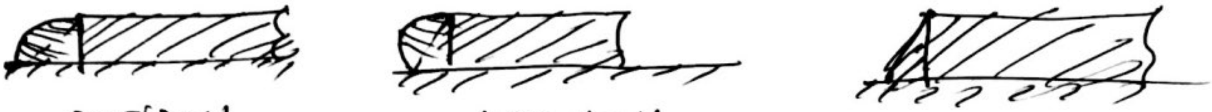
1) If a fillet weld is to the square edge of a part, the specified size of weld should generally be atleast 1.5mm less than edge thickness



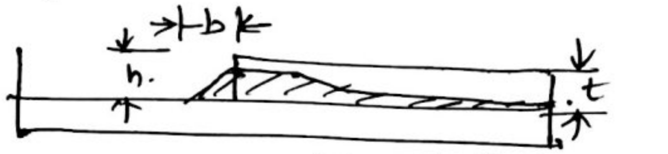

2) If a fillet weld is to the rounded toe of a rolled section, the specified size of a weld should generally not exceed 3/4th of thickness of section @ toe.



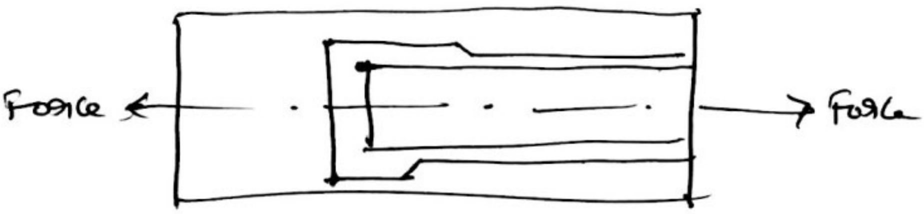

3) In members subject to dynamic loading, the fillet weld shall be of full size with its leg length equal to the thick of plate




4) Each fillet weld normal to the direction of force shall be of unequal size with throat thickness not less than 0.5t as shown in fig. The diff. in the thickness of weld shall be negotiated @ a uniform slope




$h/b = 1/2$ or flatter

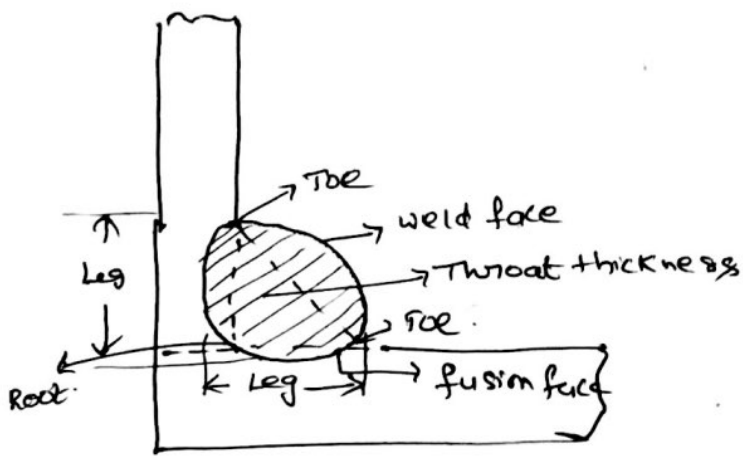



Reduction in design stresses for long Joints

If the length of welded Joint $l_J$ is $> 150t$, where $t$ is throat thickness, the design capacity of weld $f_{wd}$ shall be reduced by factor,

$$\boxed{\beta_{lw} = 1.2 - \frac{0.2\, l_J}{150t} \leq 1.0}$$

## Terms in fillet weld




1) Size of fillet weld (S) The size of normal fillet weld is equal to min. leg length

2) Throat thickness of fillet weld (t)

→ Perpendicular distance from root of fillet weld to the line joining its toes

→ Taken equal to k size of fillet

→ Value of k for diff angles b/w fusion faces is given

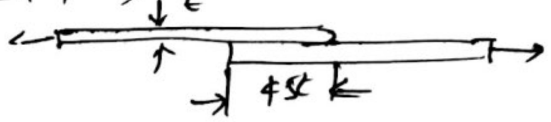
→ Almost in all cases rt angle fillet welds are used $k = 0.707$ (0.7)

$$\therefore t = k \cdot \text{size of fillet weld} = 0.707S$$

3) Effective length of weld → Actual length − 2(weld size)

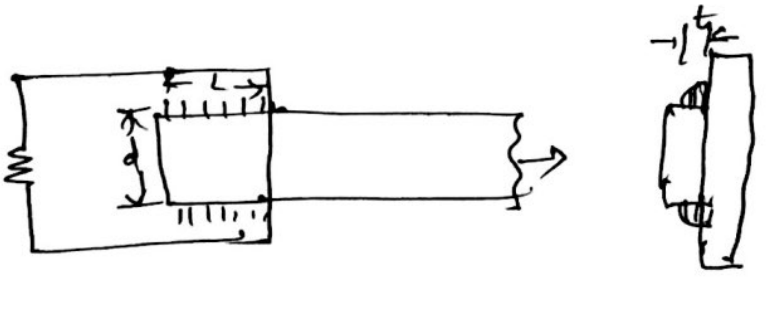
4) overlap: It should not be less than 5 times the thickness of thinner pl

(Lap ≮ 5t)




5) Side fillet

→ In a lap joint made by a side & longitudinal fillet weld, the length of each fillet weld should not be less than the lap distance b/w them.




L = Not less than d

d = Not to exceed 16t

p/o

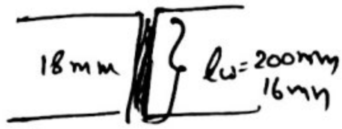
1) An 18mm thick plate is subjected to 16mm thick plate by a 200mm long butt weld. Determine the strength of the joint.

a) if double V-butt weld is used

b) if single V butt weld is used and check for safety of joint if it is subjected to a factored tensile load of 500KN?

Sol Assuming Fe 410 steel (IS 2062)

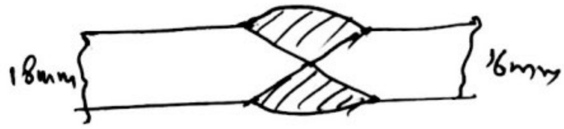
from IS 800:2007 Table ① page 14




$f_y = 250 N/mm^2$

$f_u = 410 N/mm^2$

Assuming shop weld (Table 5 page 30)




$\gamma_{mw} = 1.25$

a) Given that welding is done in the form of double V-butt weld.

As per clause 10.5.3.3 (page 78) IS 800:2007

for complete penetration (double V butt weld) the effective throat thickness is taken as thickness of thinner part joined.

$\therefore$ Effective throat thickness = 16mm $\Rightarrow$ $t_t = 16mm$

Effective Length of weld ($l_w$) = 200mm

" Area " = $(l_w \times t_t) = 200 \times 16 = 3200 mm^2$

Design stress of butt weld = $\frac{f_y}{\gamma_{mw}}$ for butt weld

$= \frac{f_u}{\sqrt{3}\,\gamma_{mw}}$ for fillet weld [10.5.7.1.1]

$\therefore$ Design stress of butt weld $= \frac{f_y}{\gamma_{mw}} = \frac{250}{1.25} = 200 N/mm^2$

Design strength of double V-butt weld joint = Effective area × Design stress

$= 3200 \times 200$

$= 640 \times 10^3 N = 640 KN.$

b) Given that welding is done in the form of single v butt weld (Incomplete penetration)

Effective throat thickness ($t_t$) $= \frac{5}{8} \times$ thickness of thinner part joined

(∵ Details about single v-butt weld is not given)

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

∴ Effective area of weld $= l_w \times t_t = 200 \times 10 = 2000 mm^2$

Design strength of single v butt weld joint = Eff. area of weld × Design stress of butt weld

$= 2000 \times 200$

$= 400 KN$

when the joint is subjected to a factored tensile load $T_u = 500kN$

Design strength of double v butt joint $= 640 KN > 500 KN.$

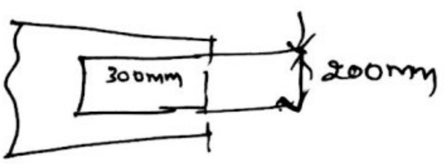
Hence safe

Design strength of single v butt joint $= 400 KN < 500 KN$

Hence not safe.

② A Tie member in a truss girder is 200 × 14mm in size it is welded to a 10mm gusset plate by a fillet weld. The overlap of the member is 300mm & weld size is 6mm. Determine the design strength of the joint if i) welded as shown in fig ii) welded all round?

Sol



For fillet weld, throat thickness $(t_t) = 0.7S$ ↳ size of weld

Here, size of weld = 6mm

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

Design stress of fillet weld $(f_{wd}) = \frac{f_{wn}}{\gamma_{mw}}$ (cl. 10.5.7.1.1) page 79

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

Assuming fe 410 steel (IS 2062) from table-1 page 14,

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

Assuming shop welds from table 5 page 30, $\gamma_{mw} = 1.25$

$$\therefore f_{wd} = \frac{410}{\sqrt{3} \times 1.25}$$

$$f_{wd} = 189.37 N/mm^2$$

from i) Effective length of weld $(l_w) = 300 + 300 = 600mm$

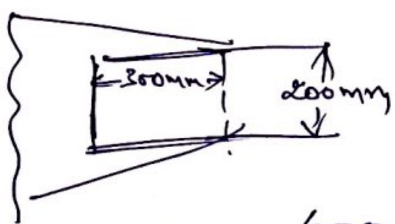
Effective area of weld = $600 \times 4.2 = 2520 mm^2$

Design strength of fillet weld joint = Eff. area × Design stress

$= (600 \times 4.2)(189.37)$

$= 477.21 kN.$

ii) when welding is done all round




Eff. length of weld $(l_w) = (300+300) + (200+200)$

$l_w = 1000mm$

$t_t = 4.2mm$

Eff area of weld $= 1000 \times 4.2 = 4200 mm^2$

Design strength of fillet weld joint = Eff. area × Design stress

$= 4200 \times 189.37 = 795.35 kN.$

3) A circular plate 125mm dia is welded to another plate along its periphery by 6mm fillet welds. find max. twisting moment that can be applied to the plate in plane if

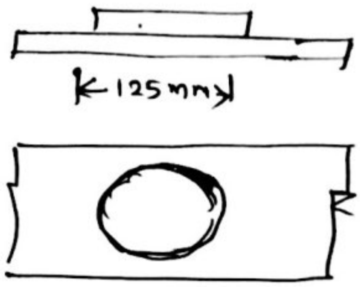
i) Fe 410 steel and shop welding is adopted

ii) Stressing weld is not to exceed $110 N/mm^2$

Sol Given that, fe410 steel is used ⇒ $f_u = 410 N/mm^2$

Shop welding is adopted, $\gamma_{mw} = 1.25$

Design stress of fillet weld, $f_{wd} = \frac{f_u}{\sqrt{3}\gamma_{mw}}$




$= \frac{410}{\sqrt{3}(1.25)}$

$f_{wd} = 189.37 N/mm^2$

Length of weld $(l_w) = \pi d - 2\pi r = \pi (125) \Rightarrow \boxed{l_w = 392.699mm}$

Size of weld (s) = 6mm

Throat thickness $(t_t) = 0.7s$

$= 0.7(6)$

$\boxed{t_t = 4.2mm}$

Effective area of weld $= l_w \times t_t$

$= 392.699 \times 4.2$

$= 1649.336 mm^2$

Design strength of fillet weld Joint $\}$ = Eff. area × Design stress

$= 1649.336 \times 189.37$

$= 312.33 kN$

In plane maximum twisting moment $= 312.33 \times \frac{0.125}{2}$ kNm

$= 19.52 kNm$

ii) Given stress of weld should not be more than $110 N/mm^2$

$$f_{wd} = 110 N/mm^2$$

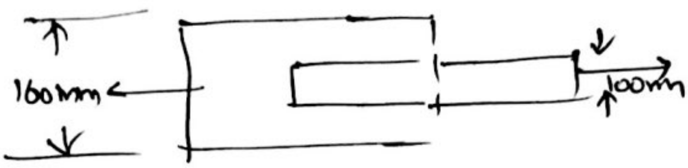
Design strength of filled weld Joint $= 1649.336 \times 110$

$= 181.43 KN$

In plane max twisting moment $= 181.43 \times \frac{0.125}{2} = 11.34 KN \cdot m$

4) Design a suitable longitudinal fillet weld to connect two plates to transmit a pull equal to strength of small plate. plates are 12mm thick, shop welding is adopted. steel is of Fe 410 grade. Welding is done @ sides

Sol




from clause 6.2 page 32 of IS 800:2007

Design strength of member under axial tension $(T_{dg}) = \frac{A_g f_y}{\gamma_{mo}}$

for Fe 410 steel, from table-1 page 14, $f_y = 250 N/mm^2$

From table-5 page 30, $\gamma_{mo} = 1.10$

Gross area of smaller plate $(A_g) = 100 \times 12 = 1200 mm^2$

Strength of smaller plate under tension $(T_{dg}) = \frac{1200 \times 250}{1.1}$

$(T_{dg}) = 272.7 KN$

Size of weld(S)

From 10.5.2.3 cl, of 78 page, $S \nless 3mm$

& Table 21, $S_{min} = 5mm$

From cl 10.5.8.1, page 79; $S_{max} = 12mm - 1.5mm = 10.5\,mm$

Assuming size of weld as 10mm

$S = 10mm$

Effective throat thickness $(t_t) = 0.7S = 0.7(10) = 7mm$

Design stress of fillet weld $(f_{wd}) = \dfrac{f_u}{\sqrt{3}\,\gamma_{mw}}$

For Fe 410 steel, $f_u = 410\,N/mm^2$ ; $\gamma_{mw} = 1.25$

$$f_{wd} = \frac{410}{\sqrt{3}(1.25)} = 189.37\,N/mm^2$$

Design strength of fillet weld = Eff. area of weld × Design stress

= Eff. length × $t_t$ × $f_{wd}$

= $l_w \times t_t \times f_{wd}$ —— (1)

For equilibrium,

Design strength of fillet weld = Design strength of smaller plate

$= 272.7 \times 10^3\,N$ —— (2)

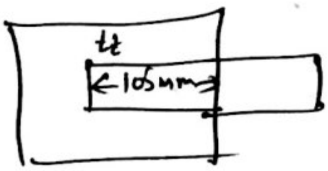
(1) = (2)

$\Rightarrow l_w \times 7 \times 189.37 = 272.7 \times 10^3$

$\boxed{l_w = 205.7mm}$

Length of weld on each side = $\dfrac{205.7}{2} = 102.85mm$

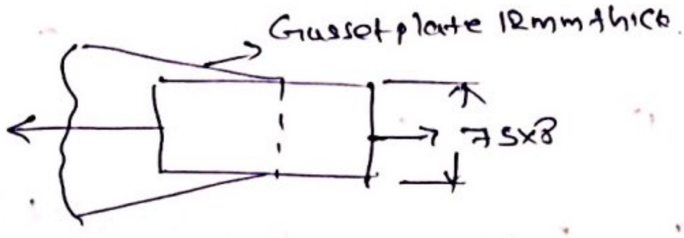
$\simeq 105mm$




5) A tie member 75×8mm is to transmit a factored load of 145 kN. Design a fillet weld and necessary overlap for the cases shown. The tie member is connected to 12mm

thick gusset plate. use Fe410 steel and adopt shop welding

case i) welded all round ii) welded three sides only iii) welded two sides only.




Sol

Size of fillet weld

From cl. 10.5.2.3/78, $S \nless 3mm$

Table 21, $S_{min} = 5mm$ } 12mm thick gusset plate

from cl. 10.5.8.1/79, $S_{max} = 8-1.5 = 6.5mm$ } 8mm thick section

Assuming size of weld as 6mm ⇒ $S = 6mm$

Now, effective throat thickness $(t_t) = 0.7S = 0.7 \times 6 = 4.2mm$

Given that Fe410 steel is used, from table-1 $f_u = 410 N/mm^2$

As shop welding is adopted, $\gamma_{mw} = 1.25$

∴ Design stress of fillet weld $(f_{wd}) = \frac{f_u}{\sqrt{3}\,\gamma_{mw}}$

$= \frac{410}{\sqrt{3}(1.25)} = 189.37 \; N/mm^2$

Design strength of weld = Eff. length × $t_t$ × $f_{wd}$

$= l_w \times t_t \times f_{wd}$

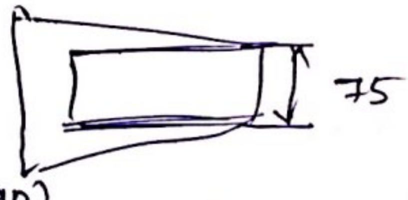
we have factored load $(T_u) = 145kN$

For equilibrium, $l_w \times 4.2 \times 189.37 = 145 \times 10^3$

∴ $\boxed{l_w = 182.3mm}$

Case i) welding is done all round

from fig; Length of weld $(l_w) = 75 + 75 + 2(\text{overlap})$




⇒ $182.3 = 150 + 2(\text{overlap})$

overlap length = 16.1 mm

As per cl. 10.5.1.2/78, Minimum overlap ≮ 4x thickness of thinner part

(or) 4x8 = 32mm

40mm (whichever is more)

∴ Provide overlap min as 40mm

Length of weld provided = 75+75+40+40 = 230mm

Case ii) Welded three sides

length of weld ($l_w$) = 75+2(overlap)

182.3 = 75+2(overlap)


overlap distance = 53.65mm

Minimum overlap is 40mm Hence ok.

Case iii) welded two sides

From fig; Length of weld ($l_w$) = 2(overlap)


182.3 = 2(overlap)

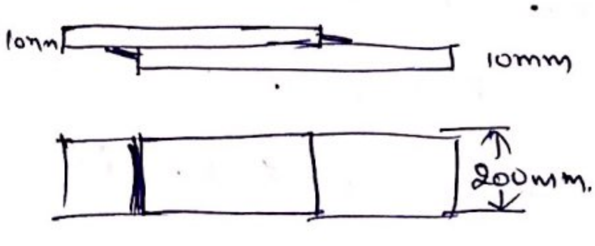
overlap distance = 91.15mm (ok)

Min. overlap is 40mm

Design strength- Efficiency of Joint

5) Design a welded connection to connect two plates of width 200mm & thickness 10mm for 100% efficiency?

Sol




Given data, thickness of plates = 10mm

width of plate = 200mm

from cl. $\frac{62}{32}$, Design strength of plate ($T_{dg}$) = $\frac{A_g f_y}{\gamma_{mo}}$.

$A_g$ = 200x10 = 2000mm$^2$

Assuming Fe 410 steel, $f_y = 250\ N/mm^2$ ; $f_u = 410 N/mm^2$

Table-5, $\gamma_{m0} = 1.10$ $\therefore T_{dg} = \frac{A_g f_y}{\gamma_{m0}} = \frac{200 \times 250}{1.1} = 454.5 \times 10^3 N.$

Design of fillet weld

Size of fillet weld (s)

From cl. 10.5.2.3/78, $s \nless 3mm$

Table 21, $S_{min} = 5mm$

From cl. 10.5.8.1/79, $S_{max} = 10 - 1.5 = 8.5mm$

Adopt size of fillet weld as 8mm $\Rightarrow$ $S = 8mm$

Throat thickness $(t_t) = 0.7S = 0.7 \times 8 = 5.6mm$

Now, Design stress of fillet weld $(f_{wd}) = \frac{f_u}{\sqrt{3}\gamma_{mw}} = \frac{410}{\sqrt{3}(1.25)}$
$= 189.37 N/mm^2$

from fig, effective Length of weld $= 200 + 200$
$= 400mm$

Design strength of fillet weld = Eff. area x Design stress

$= l_w \times t_t \times f_{wd}$

$= 400 \times 5.6 \times 189.37$

$= 424.18 \times 10^3 N$

But the plates are subjected to $454.5 \times 10^3 N$. Hence length of weld should be (↑) to resist an extra force of $(454.5 - 424.18)\ kN = 30.32\ kN$

providing slot weld from cl. 10.5.7.1.3, the design stress of slot weld is given by $f_{wd} = \frac{f_u}{\sqrt{3}\gamma_{mw}} = \frac{410}{\sqrt{3}(1.25)} = 189.37 N/mm^2$

Design strength of slot weld = Eff area of slot weld x $f_{wd}$ = $30.32 \times 10^3$

$\Rightarrow$ Eff area of slot weld $= \frac{30.32 \times 10^3}{189.37}$

$= 1601 mm^2$

Providing two slots, Area of each slot = $80.05 mm^2$

Provide $85 \times 10mm$ as each slot

Clear distance b/w the holes of the slot > 2t (or) 25mm

Here t = thickness of plate in which slot is provided.

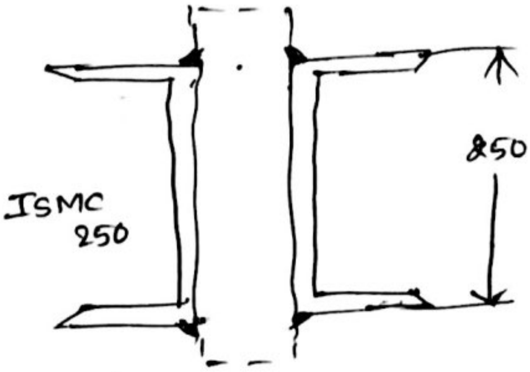
7) A tie member consisting of two ISMC 250 channels connected on either sides of a 12mm gusset plate. Design the welded joint to develop full strength of the tie. overlap is limited to 300mm shop welding is done?

Sol from steel tables,

$t_w = 7.1mm$

$t_f = 14.1mm$

$A_g = 3867 mm^2$




Design strength of each ISMC 250

$$(T_{dg}) = \frac{A_g f_y}{\gamma_{m0}}$$

Table 5

Assuming Fe410, Steel, $f_y = 250 N/mm^2$, $f_u = 410 N/mm^2$ ; $\gamma_{m0} = 1.1$

$$\therefore T_{dg} = \frac{3867 \times 250}{1.1} = 878.86 \times 10^3 N$$

Size of weld (s)

From Cl. $\frac{10.5.2.3}{78}$, $s \nless 3mm$ Table 21, $S_{min} = 5mm$

From cl. $\frac{10.5.8.1}{79}$ $S_{max} = 12 - 1.5 = 10.5mm$

Adopt size of weld as 6mm

Throat thickness $(t_t) = 0.7s = 0.7(6) = 4.2mm$

Design stress of fillet weld $(f_{wd}) = \frac{f_u}{\sqrt{3}\gamma_{mw}} = \frac{410}{\sqrt{3}(1.25)} = 189.37 N/mm^2$

Scanned by CamScanner

Design strength of fillet weld $= l_w \times t_t \times f_{wd}$

$= l_w \times 4.2 \times 189.37$

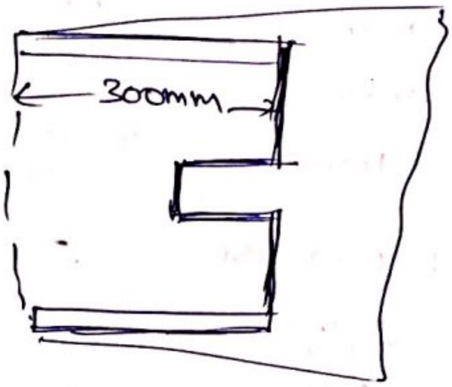
for equilibrium $l_w \times 4.2 \times 189.37 = 878.86 \times 10^3$

$l_w = 1104.99 \simeq 1105\,mm$

Welding on 3 sides with a restricted overlap of 300mm, the extra length of weld to be provided is $1105 - 300 - 300 - 250 = 255\,mm$

$l_w = \frac{255}{2} = 127.5\,mm \simeq 130\,mm.$

Provide a slot of length 130mm as shown and provide 6mm fillet weld as shown



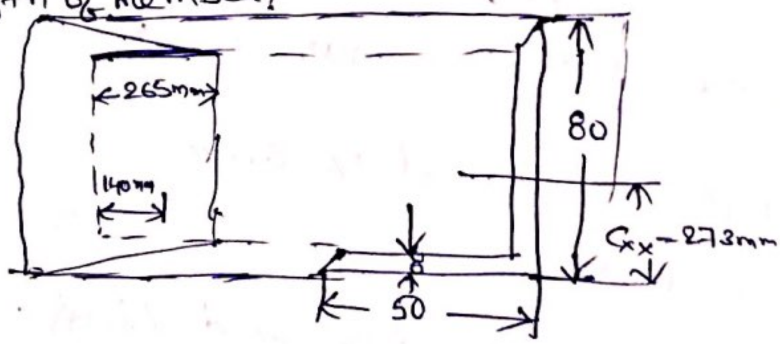

$\therefore$ length of weld provided on one side of gusset plate $\Big\} = 300 + 130 + 130 + 300 + 250$

$= 1110\,mm > 1104\,mm$

Hence ok

Q) A Tie member consists of ISA 80 50, 8mm (fe410) is welded to a 12mm thick gusset plate. Design welds to transmit loads equal to design strength of member?

Sol




12mm thick gusset plate