LECTURER NOTES ON STRUCTURAL DESIGN-II

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Chapter-1

Introduction

1.1. common steel str., Adv. & Disadvantage of steel structure.

1.2. Types of steel, properties of structural steel.

1.3. Rolled steel sections special consideration in steel design

1.4. Loads & load combinations.

1.5. Structural Analysis & Design philosophy.

1.6. Brief review of principle of LSM.

1.1 common steel str. Adv. & Disadv. of steel structure.

Steel as a building material has been used extensively in various types of str.

→ same of example are high rise building skeleton industrial buildings, transmission to wers railway bridges over head tanks, chimneys, bunkers & silos storing bulk material crane girders, military for protect people from falling (bomb)

Advantage of steel str :-

1. It can resist heavy load due to high strength.
2. It can be conveniently handled & transported to its wt.
3. It has long life.
4. property do not change with time.
5. It is a ductile material so does not fail suddenly & it give warning before failure.
6. steel can under go large deformation.
7. It can be effect at a faster rate.
8. It has highest screep value.
9. It improve as stretic view as the size of sp element is small

10. Material is highly durable.
11. Material is reusable.
12. Better quality control.

Disadvantage

1. It need fire proof treatment which increase cost
2. It require frequent painting when exposed env.
   so cost increase.
3. skilled labour required.
4. Higher cost of construction
5. It is susceptible to corrosion.

1.2 Types of steel.

Steel is an alloy of iron & carbon. Different types of steel are:- High carbon steel, Mild steel, Medium carbon steel, Alluminium steel, etc high tensile steel etc. high tensile steel etc.

Properties of structural steel:

mainly classified as → 1. Mechanical properties
2. physical properties.

Physical properties :- clause - 2.2.4 :-

$\rho = 7850 \ kg/m^3$, $E = 2.1 \times 10^5 \ N/mm^2$ or 2×10^5

$\mu = 0.3$, $G = 0.769 \times 10^5 \ N/mm^2$ $\alpha_t = 12 \times 10^{-6} \ /^\circ C$

* Mechanical properties :-

Yield stress $f_y = 220 - 540 \ N/mm^2$

ultimate tensile strength $= 1.2 \ f_y = f_u$

% Elongation $= 20$

* cost iron - 2% carbon

carbon → highest → wrought

Fe 410 = main tensile strength of steel is 410 N/mm^2

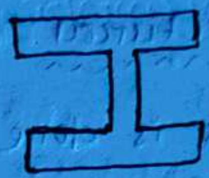
Pure - (5%

1.3 Rolled steel section :-

Steel cannot be cast any shape & size on site as it needs very temp. to may & roll in to reqd shape.
various types of rolled steel section are :-

1. Rolled steel I section 

I section



2. Rolled steel channel section C
3. Rolled steel Angle section L
4. Rolled steel Tee section T
5. Rolled steel Tube O

Special considerection in steel design :- places, bars, flats

Following special considerations are reqd. in steel design,
→ size & shape, buckling, minimum thickness, connection design

Buckling :-

For same load cross-section area for steel < concrete as after ~~~~ higher permissible stress.
→ steel structures are more slender so the compⁿ members in steel str. are liable to buckling.

Min thickness :-

Provision due to corrosion if very thin str. then small amount of corrosion may result in a large % redⁿ in effective area.

a) If fully accessible for clearing & painting = 6 mm
b) If not accessible for clearing & painting = 8 mm
Not for rolled section.

Connection :- There are 3 types of connection commonly used

i) Rivetted connection
ii) Bolted.
iii) Welded.

1.4 Loads & load combination:-

Following are the various types of loads:-

1) DL. 2. LL 3. WL 4. seismic load IS SL 6. water current load 7. impact load 8. Temp. & correction effects.

Load combⁿ are recomanded by IS 875- DL, DL + IL, DL + WL + DL + IL, + WL/EL

1.5 Structural analysis & design philosophy:-

Analysis is done to find internal forces developed in the member is code permits the following methods of analysis

a) Elastic analysis (b) plastic analysis (c) Advanced analysis d) Dynamic analysis.

Design philosophy:-

The aim of design is to decide shape, size & connection details of member.

with an appropriate degree of safety the str. should

a) sustain all loads expected on it.

b) sustain deformations during & after construction

~~c) show sustain~~

c) should have aquedate durability.

d) should have resistance to misuse & fire

→ Design philosophy are:- 1) WSM (2) LSM (3) ULD ultimate load design.

1.6 Brief Review of Principles of LSM

It is the comprehensive Method which will take care of both strength & service eability requirement.

→ Limit states are the states beyond which structure no longer satisfied performance requirements.

→ The various limit states to be considered in design are:-

(a) Limit state of strength.

(b) Limit ~~state~~ state of serveceability.

Limit state of strength

It includes

i) Loss of eqⁿ of whole or part of the structure.

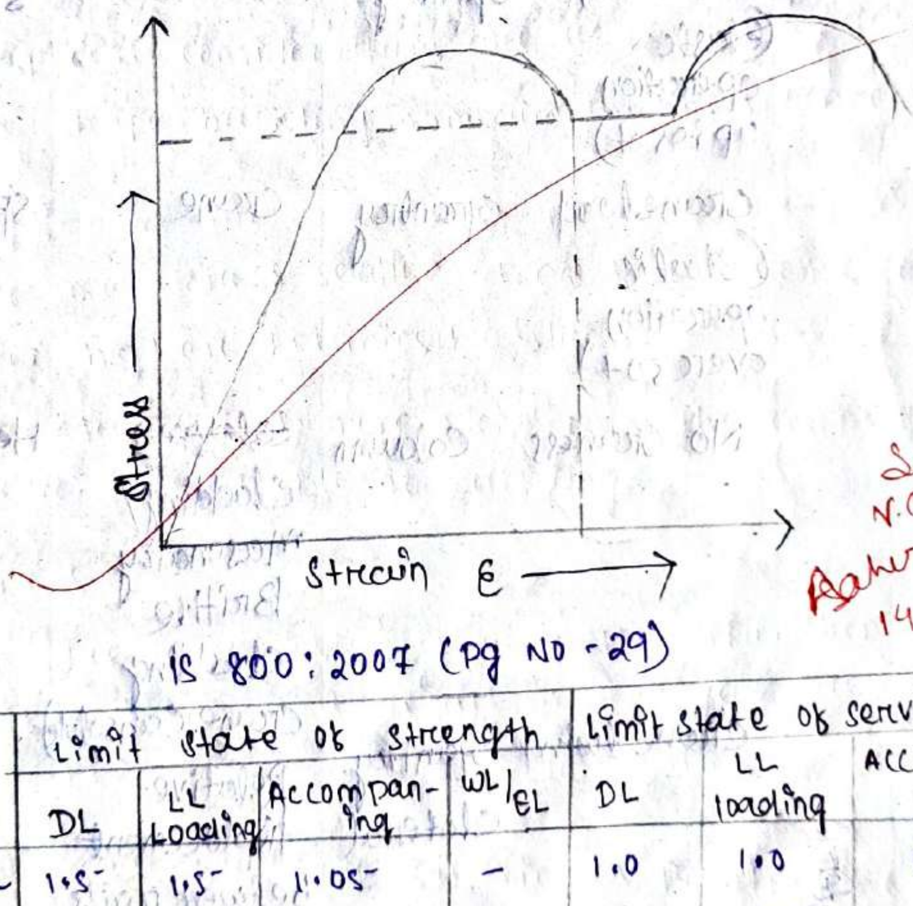
ii) Loss of stability of structure as a whole or part of it.

iii) Failure by encessive deformation.

iv) Fracture due to fatigue.

v) Brittle fracture.

~~Stres~~ Stress strain curve of steel



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IS 800: 2007 (Pg No - 29)

Cominatiion	Limit state of strength				Limit state of servicieability			
	DL	LL Loading	Accompan-ing	WL/EL	DL	LL loading	Accompan-ing	
1) DL + LL + CL	1.5	1.5	1.05	–	1.0	1.0	1.0	–
2) DL + LL + CL + WL/EL	1.2	1.2	1.05	0.6	1.6	0.8	0.8	0.8
3) DL + LL + WL + EL	1.2 1.5 (0.9)	1.2	0.53	1.2 1.5				
4) DL + ER	1.2 (0.9)	1.20	–	–	1.0	–	–	1.0
5) DL + LL + AL	(0.9) 1.0	0.35	0.35	–	–	–	–	–

DL → 85 KN × 1.2 = 400   45 × 0.53 = 45

LL → 48 KN × 1.2 = 200   645 KN → Load

WL → 30 KN × 1.2

Types of Building (1)	Deflection (2)	Deflection Limits Design load (3)	member (4)	Supporting (5)	Max^m Deflection (6)
Industrial Buildings	Vertical	Live load/ wind load	Purlines & Girts	Elastic cladding	span/150
				Brittle cladding	span/180
		Live load	simple span	Elastic cladding	″/240
		Live load	cantilever span	Brittle ″	″/300
		Live load/ wind load	Rafter Supporting	Elastic ″	″/120
				Brittle ″	″/150
				Profiled metal sheeting	″/180
				Plastered Sheeting	″/240
		crane load (Manual oper-ation)	Gantry	→ crane	span/500
		Crane load (Elastic operation up to 50t)	Gantry	crane	span/750
		Crane load (Elastic operation over 50t)	Gantry	crane	span/1000
		No craness	Coloumn	Elastic Cladding/ Masonarly Brittle Cladding	Height/150
	Lateral	Crane +wind	Gantry (lateral)	Crane (absolute)	span/400
				Relative displacement between rails Supporting crane	10mm
				Gantry Elastic cladding pendet operated)	Height/200
				Gantry (Brittle cladding Pendent operated	Height/400

Live load floor & Roof	Elements susceptible to cracking	span/300
	Elements susceptible to cracking	span/360
Live load cantilever	Element not susceptible to cracking	span/150
	Elements not susceptible to cracking	span/180
wind Building	Elastic cladding	Height/300
wind inter storey drift	Brittle cladding	storey height/300

Table 5 partical safety factor for materials, γ_m

(Clause 5.4.1)

Pg no - 30

	Definition	partical safety factor	
(i)	Resistance, governed by yielding γ_{m0}	1.10	
(ii)	Resistance of member to buckling γ_{m0}	1.10	
(iii)	Resistance, governed by ultimate stress, γ_m	1.25	
		shop fabrications	Field fabrication
(iv)	Resistance of connection:-		
	a) Bolts - friction type, γ_{mf}	1.25	1.25
	b) Bolts - bearing type, γ_{mb}	1.25	1.25
	c) Rivets γ_{mn}	1.25	1.25
	d) welds, γ_{mw}	1.25	1.50

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Deflection

The deflection under serviceability loads of a building or a building component should not impair the strength of the structure or components or cause damage to finishing. Deflections are to be checked for the most adverse but realistic combination of services loads and their arrangement by elastic analysis using a load factor of 1.0. Table 6 gives recommended limits of deflections for certain structural members and systems. Circumstances may arise where greater or lesser values would be more appropriate depending upon the nature of material in element to be supported (vulnerable to cracking or not) and intended use of the structure as required by client.

Bolted connection

Classification of Bolt:-

1) Bolts used in steel structure are 3 types.

a) Black bolt

b) Turned & fitted bolt

c) High strength friction grip bolt (HSFG)

2) The international standards for designation of bolts in India is given by grade x.y

3) In this x indicates (→) $\frac{1}{10}$ th of the minimum ultimate tensile strength in kgf/mm2

y → $\frac{1}{10}$ th of the ratio of the yield stress to ultimate stress in percentage.

4) For ex:- Grade 4.6 means bolt will have a minimum ultimate strength of 40 kgf/mm2 & min^m yield strength of 0.6 times 40 i.e. 24 kgf/mm2

Q) Grade 5.2

$x = \frac{1}{10} \times$ ultimate tensile strength

$\Rightarrow 5 = \frac{1}{10} \times$ ultimate tensile strength

$\Rightarrow$ ultimate = 50 kgf/mm2

$$\text{yield strength } (y) = \frac{\frac{1}{10}\left(\frac{\text{yield strength}}{\text{ultimate strength}}\right)}{100}$$

$$2 = \frac{1}{10}\left(\frac{\text{yield strength}}{50}\right)$$

$$\Rightarrow 20 = \frac{\text{yield}}{50}$$

$$\Rightarrow \text{yield} = \frac{1000}{100} = 10 \text{ kgf/mm2}$$

4) Grade 4.9

$x = \frac{1}{10} \times$ ultimate tensile strength

$\Rightarrow 4 = \frac{1}{10} \times$ ultimate tensile strength

$\Rightarrow$ ultimate = 40 kgf × mm2

yield strength (y) = $\frac{\frac{1}{10}\left(\frac{\text{yield strength}}{\text{ultimate strength}}\right)}{100}$

$\Rightarrow 9 = \frac{1}{10}\left(\frac{\text{yield strength}}{40}\right)$

$90 = \frac{\text{yield}}{40}$

$\Rightarrow$ yield $= \frac{3600}{100} = 36$ kgf/mm2

Advantages of Bolts:-

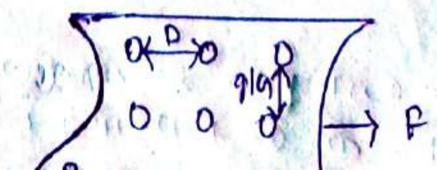
a) Making joints noise less
b) Do not need skilled abour.
c) Need less labour.
d) Structure can be put to use immediately.
e) working area required in the field is less.
f) Number of bolts require is less

Disadvantages of bolts

a) Tensile strength is reduce due to stress concentration & reduction of the area at root of the thread.
b) Due to vibration bolts are likely to loosen so the safety of structure is in danger.

Pitch of Bolts

p = pitch distance
G = Gauge distance
e = End distance
e' = Edge "



Pitch:-

It is the centre spacing of the bolt in a row along the direction of load.

Gauge dist.:-

It is the dist. betn centre of bolts of adjacent rows and it acts at right angle to the direction of load.

Edge distances:-

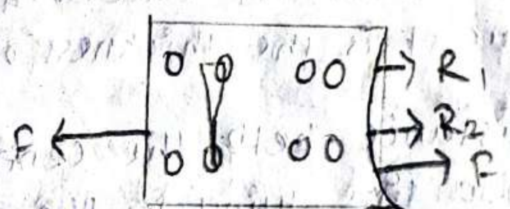
It is the dist. of centre of bolt whole from the adjacent edge of plate (perpendicular to load)

End distance

It is the dist. of nearest bolt whole from the end of plate (along the direction of load)

Staggered distance (s):-

It is the centre distance of staggered bolts along the direction of load.



Specification for spacing And Edge distance OF Bolt hole

Minm spacing

Pg no - 73

The distance betn centre of fasteners shall not be less than 2.5 time the nomical dia. of the fasterener.

Table 19 clearances for fastener Holes
(clause 10.2.1)

Table 19 Clearances for Fastener Holes (Clause 10.2.1)

Sl No	Nominal size of fastener, d mm	Size of the Hole = Nominal diameter of the fastener + Clearances mm			
		Standard clearance in Diameter and width of slot	Over size Clearance in diameter	Clearance in the length of the slot	
				Short slot	Long slot
(1)	(2)	(3)	(4)	(5)	(6)
i)	12 - 14	1.0	3.0	4.0	2.5 d
ii)	16 - 22	2.0	4.0	6.0	2.5 d
iii)	24	2.0	6.0	8.0	2.5 d
iv)	Larger than 24	3.0	8.0	10.0	2.5 d

Max^m Spacing :-

→ The distance betn the centres of any two adjacent fasteners shall not exceed 32 t or 300 mm whichever is less, where t is the thickness of the thinner plate.

→ The distance betn the centres of two adjacent fasteners (Pitch) in a line lying in the direction of stress shall not exceed 16t or 200 mm, whichever is less in tension members where t is the thickness of the thinner plate.

→ In the case of compression members where in forces are transferred through butting faces, this distance shall not exceed 4.5 times the diameter of the fasteners for a distance equal to 1.5 times the width of the member from the butting faces.

→ The distance between the centres of any two consecutive fasteners in a line adjacent and parallel to an edge of an outside plate shall not exceed 100 mm plus 4t or 200 mm whichever is less, in compression and tension members; where t is the thickness of the thinner outside plate

→ The distance between the centres of any two consecutive fasteners in a line adjacent and parallel to an edge of an outside plate shall not exceed 100mm plus 4t or 200 mm

→ When fasteners are staggered at equal intervals and the gauge does not exceed 75 mm, the spacing specified in 10.2.3.2 and 10.2.3.3 betn centres of fasteners may be increased by 50% subject to the maxm spacing specified in 10.2.3.1.

Edge and End distance:-

→ The edge distance is the distance at right angles to the direction ~~end distance is the distance in the~~ of stress from the centre of hole to the adjacent edge. The end distance is the distance in the direction of stress from the centre of a hole to the end of the element

→ In slotted holes, the edge and end distances should be measured from the edge or end of the material to the centre of its end radius or the centre line of the slot, whichever is smaller in oversize holes, the edge and end distances should be taken as the distances should be taken as the distance from the relevent edge/end plus half the diameter of the standard clearance hole corresponding to the fasten, less the nominal diameter of the oversize hole.

→ The minm edge and end distances from the centre of any hole to the nearest edge of a plate shall not be less than 1.7 times the hole diameter in case of sheared or hand flame cut edges; and 1.5 times the hole diameter in case of rolled machine flame cut, sawn and planed edges.

→ The maxm edge distance to the nearest line of fasteners from an edge of any un-stiffened part should not exceed 12tε, where ε = (250/fy) and t is the thickness of the thinner outer plate.

→ This would not apply to fasteners interconnecting the of back to back tension members where the max^{m} edge distance shall not exceed 40mm plus 4t where t is the thickness of thinner connected plate.

Taking fasteners :-

→ In case of members covered under 10.2.4.3 when the max^{m} distance betn centres of two adjacent fasteners as specified in 10.2.4.3 is exceeded taking fasteners subjected to calculated stress shall be used.

→ Taking fasteners shall have spacing in a line not exceeding 32 times the thikness of the thinner outside plate or 300mm whichever is less.

→ Where the plates are exposed to the weather the spacing in line shall not exceed 16 times the thickness of the thinner outside plate or 200mm, whichever is less. In both cases the distance betn the lines of fasteners shall not be greater than the respective pitches.

1Q) What is the minimum spacing of bolt whose diameter is 17 mm?

Soln:- Nominal Dia = 17 mm

minimum spacing = 2.5 × Nominal dia

= 2.5 × 17

= 42.5 mm (Ans)

2Q) What is the maximum spacing of fastenerce it thickness thinner plate is 16 mm?

Soln:- Give t = 16 mm

32 t = 32 × 16

= 512 mm

max^{m} spacing = less.

3Q) What is the minm edge distance of bolt whose diameter is 15 mm, in rolled edges?

Solⁿ:- Nominal dia = 15 mm

Hole dia = 15 + 1

∴ min^m edge distance = 1.5 × hole dia

= 1.5 × 16

= 240 mm

4Q) What is the maximum edge distance of fastener w.r.t plate of thickness 19 mm.

($f_y = 250\ N/mm^2$)

Solⁿ:- Thickness of thinner plate = 19 mm

$f_y = 250\ N/mm^2$

max^m edge distance = 12 + ε (Cl 10.2.4.3)

$$\varepsilon = \left(\frac{250}{f_y}\right)^{1/2} = \left(\frac{250}{250}\right)^{1/2}$$

$$= (1)^{1/2}$$

$$= 1$$

max^m edge distance = 12 + ε

= 12 × 19 × 1 = 228 m

7) A part from require bolts from the consideration of design forces additional bolt are called tacking fasteners.

a) Edge distance of taking fastener should be provided at 32t or 300 mm, whichever is less when plates are not exposed to weather.

b) At 16t or 210 mm whichever is less → when plates are exposed to weather.

8) In case of member made up of two flash is cade 2.5.4 angles, channels, tees taking reveals are provided along the length to connect components.

(i) Not exceeding 1000 mm → If it is a tension member it

(ii) Not exceeding 600 mm → If it is a compression member

min^m spacing table - 19

max^m

Types of joint :-

Depending upon arrangement of bolts & place there are two types of joint in bolted connection.

1) Lap joint.

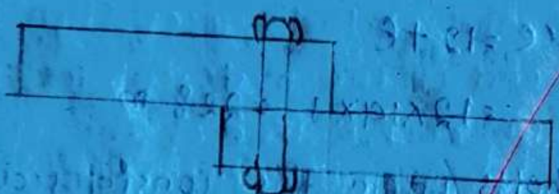
2) Butt joint.

1) Lap joint:-

It is the simplest type of joint in this the place tobe connected lap one another.

2) Butt joint :-

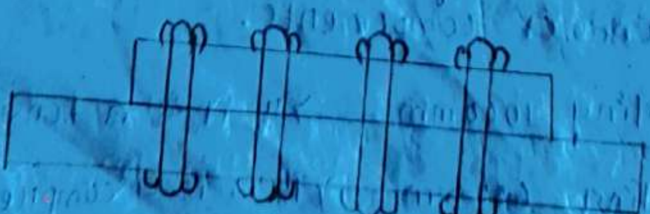
In this type of connection two main place plates and the connection is made providing a single cover plate connected to main plate or by double cover plates.



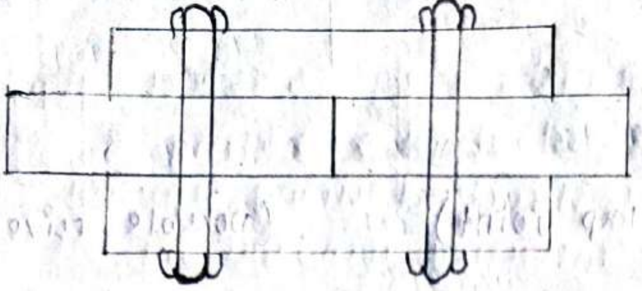
(a) single Bolted

(b) Double bolted

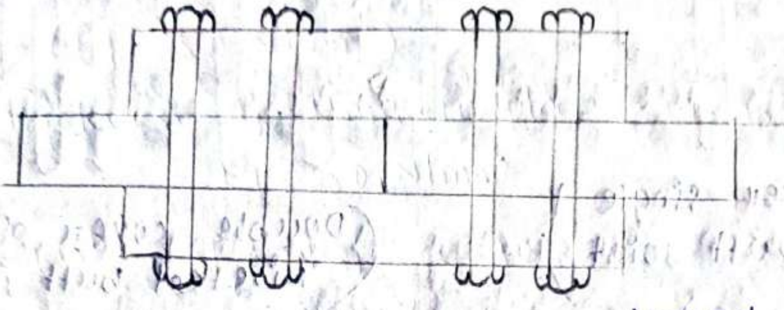
(a) single cover single bolted



(b) ~~stape~~ single cover ~~dob~~ double bolted



(c) Double cover single bolted



(d) Double cover double bolted

Assumption in Design of bearing Bolt :-

1) Friction between place are negligible.

2) Shear is uniform over the cross section of bolt.

3) Distribution of stress on the place between the bolt holes is uniform.

4) Bending stress developed in the bolts is neglected.

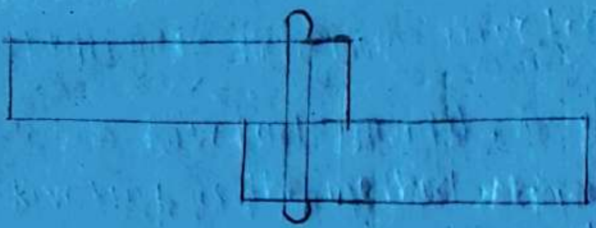
Riveted joint :-

i) Riventing is the method of joining together the structural steel componet by inserting metal its called rivets.

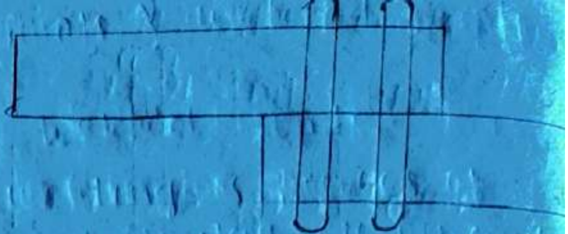
ii) Based on the pattern of riveted joint the riveted connection may be two types.

1) Lap joint — Single riveted lap joint.
— Double riveted lap joint.

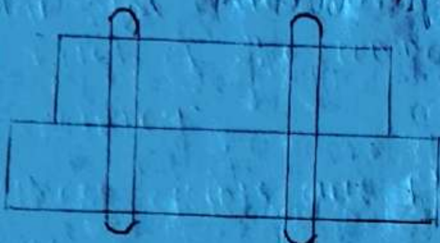
2) Butt joint — Single cover butt joint
— Single cover double riveted butt joint
— Single cover single riveted butt joint.
— Double cover butt joint
— Double cover double riveted butt joint
— Double cover single riveted butt joint



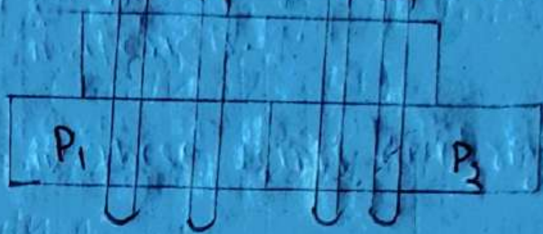
(single riveted lap joint)



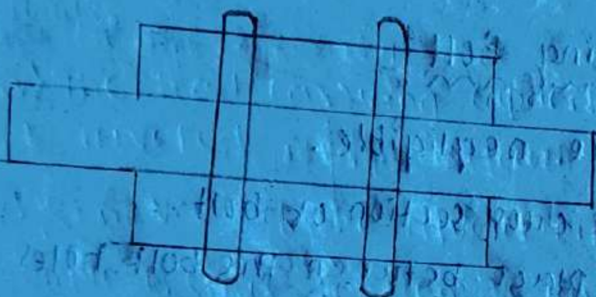
(Double riveted lap joint)



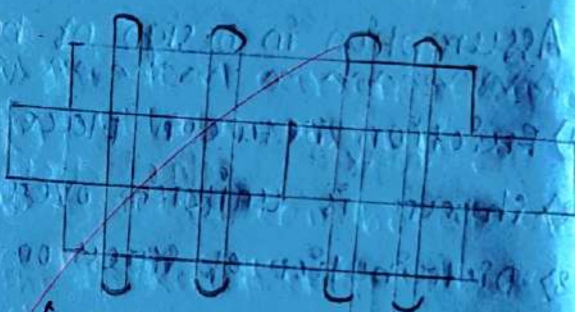
(single cover single riveted butt joint)



(Double cover double riveted butt joint)



(Double cover single riveted butt joint)



(Double cover double riveted butt joint)

Welded connection:-

i) welded is the process of joining similar or non-similar member by the application of heat fusion with or without pressure by using filler material.

ii) The fusion betn metals are made by no. of method. The method include electric arc welding & oxyacetelene gas

iii) In arc welding the ends of member to be connected are heated by an electric arc. The molten weld gets diposited between them and allowed to cool.

In oxyacetelene gas the heat require for fusion metal is done by burning oxygen &

acetylene get released through nozzle with a high pressure.

Advantages of welded joint:-

i) welded joints are economical from the points of view of cost of ~~labour~~ labour & material.

ii) The efficiency of the welded joints is 100% as compare an efficiency of 75% to 90% incase of riveted joints.

iii) The fabrication of a complicated structure is easier by welded connection.

iv) The welding provides very rigid joint.

v) The welding work is done more quicker than the riveting work.

Disadvantages of welded joint:-

i) No provision for expansion and contraction & contraction is kept in welded connection & therefore, there is possibility of cracks developing in such structures.

ii) Due to uneven heating & cooling of the members during welding the members may distort resulting in additional stress.

iii) on account of extreme heat, fatigue may occur during welding.

iv) There is possibility of brittle failure during welding.

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Difference between Riveted joints & welded joints

Riveted joints	Welded joints
i) Riveting is the method of joining joining together structural steel componets by inserting metal pins called rivets in to the holes of the componets to be jointed.	i) welding is process of joining similar or non-similar members by the application of heat with or with out pressure by using filler materials.
ii) These joints are flexible joints	ii) These joints are rigid joints
iii) The time taken for riveting is high compaired to welded joints.	iii) welding is quicker process
iv) The load carrying capacity of riveted joints is low.	iv) The load carrying capacity of welded joints is high.
v) Removal of the rivet pin is complex. since to remove rivet large holes are made in the members	v) A weld can be cut with a torch causing mineral damage

Strength of plate in a joint :-

1) shear capacity of bolt :- (pg no - 75) cl no - 10.3.3

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

V_{dsb} = Design strength of bolt.

V_{nsb} = Nominal shear capacity of bolt.

γ_{mb} = partial safety factor of bolt.

(Table no- 5 of IS - 800)

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

fu = ultimate tensile strength of bolt.

nn = No of shear plane with threads intercepting the shear plane

ns = No of shear planes without through intercepting the shear plane.

Asb = Nominal plain shank area of bolt

Anb - Net shear area of bolt at threads may be taken as the area corresponding to root dia threads.

Lap joints:-

when the length of the joints. lj of a splice or end connection is a compression or tension element containing more than two bolts (that is the distance betn the 1st & last rows of bolts in the joint, measured in the direction of the load transfer exceeds 15 d in the direction of the load the nominal shear capacity Vdb shall be reduced by the factor βlj given by

$\beta_{lj} = 1.075 - l_j/(200 d)$ but $0.75 \le \beta_{lj} \le 1.0$

$= 1.075 - 0.005 (lj/d)$

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where

d = Nominal diameter of the fastener

Note = This provision does not apply when the distribution of shear over the length of joint is uniform as in the connection of web of a section to the flanges.

Large grip lengths :-

When the grip length, l_g (equal to the total thickness of the connected plates) exceeds 5 times the diameter, d of the bolts, the design shear capacity shall be reduced by a factor β_{lg}, given by :

$$\boxed{\beta_{lg} = 8d/(3d+l_g) = 8/(3+l_g/d)}$$

β_{lg} shall not be more than β_{lj} given in 10.3.3.1. The grip length, l_g shall in no case be greater than 8d.

Packing Plates

The design shear capacity of bolts carrying shear through a packing plate in excess of 6 mm shall be decreased by a factor β_{pk} given by :-

$$\boxed{\beta_{pk} = (1 - 0.0125\, t_{pk})}$$

Where,

t_{pk} = thickness of the thicker packing, in mm.

Bearing capacity of the bolt :-

Pg no - 75

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

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

Where,

V_{npb} = nominal bearing strength of a bolt

$= 2.5\, k_b\, d\, t\, f_u$

Where ;

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

e, p = end and pitch distances of the fastener along bearing direction.

d_0 = diameter of the hole ;

f_{ub}, f_u = ultimate tensile stress of the bolt and the ultimate tensile stress of the plate respectively.

d = nominal diameter of the bolt and

t - summation of the thicknesses of the connected plates experiencing bearing stress in the same direction, or if the bolt are counter sunk, the thickness of the plate minus one half of the depth of countersinking.

The bearing resistance (in the direction normal to the slots in slotted holes), of bolts in holes other than standard clearance holes may be reduced by multiplying the bearing resistance obtained as above, V_{npb}, by the factors given below:-

(a) over size and short slotted holes - 0.7, and

b) long slotted holes - 0.5.

NOTE :- The block shear of the edge distance due to bearing force may be checked as given in 6.4.

Tension capacity :-

A bolt subjected to a factored tensile force, T_b shall satisfy :-

$$\boxed{T_b \le T_{db}}$$

where, $\boxed{T_{db} = T_{nb} / \gamma_{mb}}$

T_{nb} = nominal tensile capacity of the bolt calculated as :-

$$\boxed{0.90 f_{ub} A_n < f_{yb} A_{sb} (\gamma_{mb} / \gamma_{mo})}$$

where,

f_{ub} = ultimate tensile stress of the bolt.

f_{yb} = yield stress of the bolt.

A_n = Net tensile stress area as specified in the appropriate Indian standard (for bolts where the tensile stress area is not defined A_n shall be taken as the area at the bottom of the threads) and

A_{sb} - shank area of the bolt.

Q) Calculate the design strength of a tension member due to yielding of gross section for a plate 200mm width & 10mm thickness take $f_y = 250\ N/mm^2$

Given data:-

plate width = 200mm

Thickness = 10 mm

$f_y = 250\ N/mm^2$

∴ As per IS 800 : 2007 cl no 6.2 (pg no - 32)

design strength of plate due to yielding

$T_{dg} = A_g f_y / \gamma_{m0}$

A_g = Gross Area of cross - section

$A_g = 200 \times 10 = 200\ mm^2$

γ_{m0} : partial safety factor for failure in tension by yielding (Table - 5 pg no - 30)

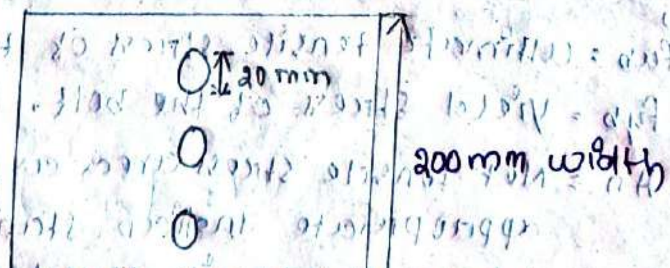
$\gamma_{m0} = 1.10$

$$\therefore T_{dg} = \frac{2000 \times 250}{1.10}$$

$= 454545.45\ N$

$= 454.54\ kN$

Q) Calculate design strength of plate of size 200 mm width & 10mm thickness which has 3 holes of a diameter 20mm as shown in figure $f_y = 250\ N/mm^2$ & $f_u = 410\ N/mm^2$



Given data

width = 200mm

Thickness = 10mm

No. of holes = 3 no

Dia of hole = 20 mm

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

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

∴ As per IS 800 : 2007 clause No 6.2 (pg no - 32)

$A_g = 200 \times 10 = 2000 \, mm^2$

∴ $\gamma_{m0} = 1.10$ (Table no 5 pg no - 30)

$$T_{dg} = \frac{200 \times 250}{1.10}$$

$= 454545.45 \, N$

$= 454.54 \, kN$ (Ans)

∴ Design strength of plate due to Rupture.

$$T_{dn} = \frac{0.9 \, A_n f_u}{\gamma_{m1}}$$ (pg no - 32)

An = Net effective area of member given by cross - section

$$A_n = \left(b - n d_n + \sum_i \frac{p_{si}^2}{4 g_i}\right) t$$

$= (b - n d_n) t$

$A_n = (200 - 3 \times 20) \times 10$

$= 1400 \, mm^2$

$\gamma_{m1} = 1.25$

$$T_{dn} = \frac{0.9 \times 1400 \times 410}{1.25}$$

$= 413280 \, N$

$T_{dn} = 413.28 \, kN$

∴ Design strength of plate is least of Tdg and Tdn

∴ Design strength = 443.28 kN (Ans)

Q) Calculate the design strength of a tension member due to yielding as gross for a plate of 500 mm width and 50mm thickness Take $f_y = 250\ N/mm^2$

Given data

width = 500 mm

thickness = 50 mm

$f_y = 250\ N/mm^2$

∴ As per IS 800:2007 clause NO-6.9 (pg no

$A_g = 500 \times 50$

$= 25000\ mm^2$

$\gamma_{m0} = 1.10$ (Table no 5) (pg no 30)

$$T_{dg} = \frac{25000 \times 250}{1.10}$$

$= 5681818.182\ N$

$= 568.18\ kN$ (ans)

A bolt required to resist both design shear force (V_{sd}) and design tensile force ( ) at the same time shall satisfy :-

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

where,

V_{sb} = Factored shear force acting on the bolt

V_{db} = design shear capacity

T_b = Factored tensile force acting on the bolt and

Tdb = design Tension capacity.

Q) Determine the design strength of plate of size 160mm width and 8 mm thickness connected to a 10 mm thick gusset plate using 16 mm bolts a show in fig. If the yield stress and ultimate stress are $f_y = 250$ N/mm² and $f_u = 410$ m/p

Sol^n :- Given data

Sol^n :- Given data

width = 160 mm

Thickness plate = 8 mm

Thickness of Gusset plate = 10 mm

bolt dia = 16 mm

$f_y = 250$ mpa | N/mm²

$f_u = 410$ mpa | N/mm²

$A_g = 160 \times 8$

$= 1280$ mm

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

$$= \frac{1280 \times 250}{1.10}$$

$= 290909.0$ N

$= 290.91$ kN

$$A_n = \left(b - nd_n + \sum_i \frac{p s_i^2}{4 g_i}\right) t$$

$= (160 - 2 \times 18) \times 8$

$= 992 \ mm^2$

$$= \frac{0.9 \times 992 \times 410}{1.25}$$

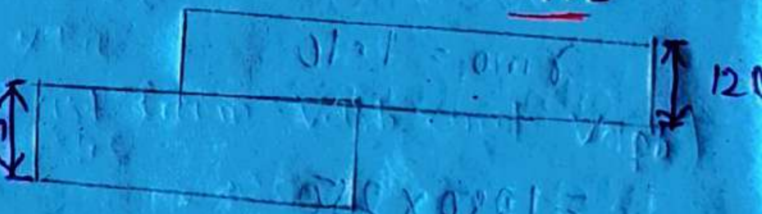
$= 292.83$ KN

$\therefore$ design strength $= 290.9$ KN

Q) Determine the design strength of plate size 160 mm width and 8 mm thickness connected to a 10mm thick gusset plate using 16 mm bolts as show in fig. If the yield stress and ultimate stress are 250 mpa and 410 mpa.

strength = 240.88 KN (Ans)

Q) Design a lap joint betn two plate as shown in figure. so as to transmit a factor load of 70 K/N using M16 bolts of grade 4.6 ultimate stress 410



Given data :-

dia of bolt = 16 mm (d)

grade = 4.6

factored load = 70 KN

(t) Thickness of plate = 12 mm

(least t from 20 mm and 12 mm)

Ultimate stress f_u = 40 kgf/mm^2

= 400 N/mm^2

yield stress = 24 kgf/mm2

= 240 N/mm2

$$\boxed{\text{shear capacity of bolt } V_{dsb} = \left(\frac{V_{nsb}}{\gamma_{mb}}\right)}$$

IS - 800 : 2007 (Pg no - 75)

V_{nsb} = Nominal shear capacity of bolt

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

* Nominal dia of bolt = bolt dia.
* Gross dia of bolt = bolt hole dia.

bolt hole diameter (d_0) : 16 mm + 2 mm
= 18 mm

minimum edge distance (e) = 1.5 × hole diameter
= 1.5 × 18 = 27 mm

minimum pitch = 2.5 × 16
= 40 mm

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

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

$$= 0.78 \times \frac{\pi}{4} \times 16^2$$

d = Dia of bolt
= 16 mm

$$= 157 \text{ mm}^2$$

* In lap joint case n_s is without threads is 0
* n_n without treads is = 1

$$V_{nsb} = \frac{400}{\sqrt{3}} (1 \times 157 + 0)$$

= 36257
= 36.257 KN

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

$\gamma_{mb} = 1.25$ (Table NO - 5)

$$= \frac{36.257}{1.25} = 29 \text{ KN}$$

Bearing capacity of bolt :- (pg 75)

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

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

k_b is smaller of $\frac{e}{3d_0} = \frac{27}{3 \times 18} = 0.50$

$$\frac{p}{3d_0} - 0.25 = \frac{40}{3\times 18} = 0.74$$

$$= 0.74 - 0.25 = 0.49$$

$$= \frac{f_{ub}}{f_u} = \frac{400}{410} = 1.0$$

1.0

$$= K_b = 0.49$$

$$V_{npd} = 2.5 \times 0.49 \times 16 \times 1.2 \times 400$$

$$= 9408 \text{ N}$$

$$= 94.08 \text{ kN}$$

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

$$= \frac{94.08}{1.25}$$

$$= 75.264 \text{ kN}$$

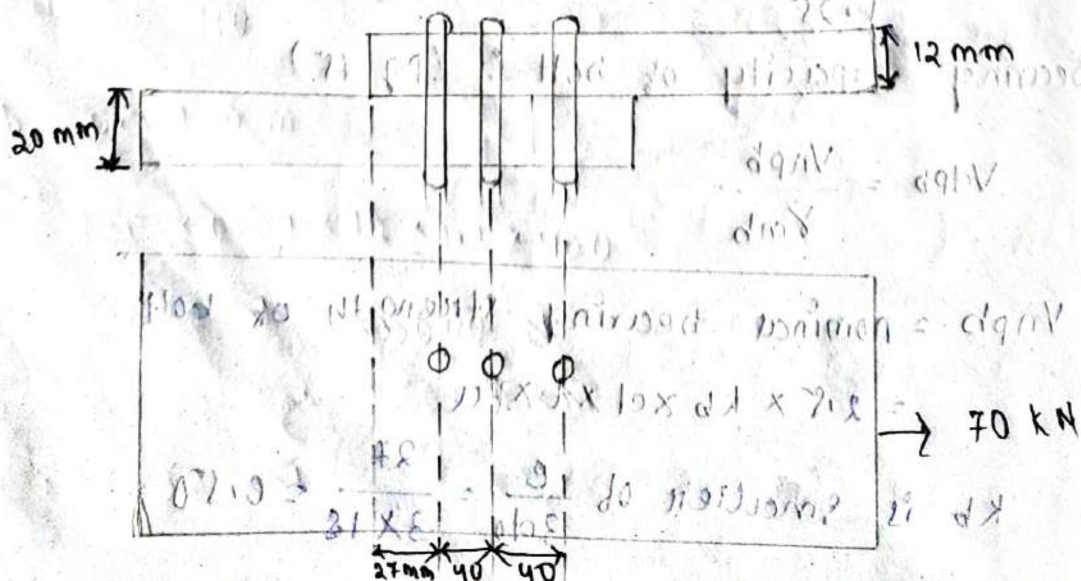
∴ Thus, $V_{dpb} \geq V_{dsb}$ (ok)

Bolt value = 29 kN (least from V_{dsb} and V_{dpb})

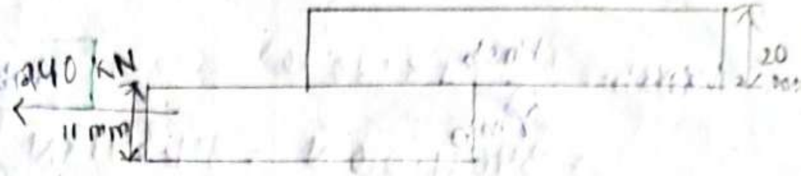
Req no of bolt :-

$$\frac{\text{factor load}}{\text{bolt value}} = \frac{70}{29} = 2.41 \simeq 3 \text{ bolt}$$

> P.S.F = only load then 1.5 put
> concrete ~~partial~~ safety factor = 1.5
> partial

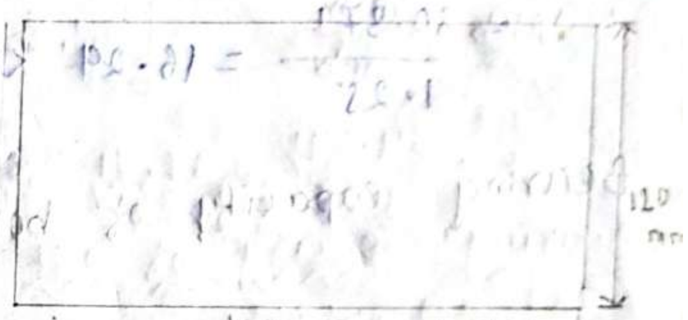


Q) Design a lap joint between two plates each of width 120 mm, if thickness of one plate is 20 mm and the other is 11 mm. The joint has to transfer a design load of 240 kN, the plates are of Fe 410 grade use bearing type bolts of property class 4.6.



Sol^n

Given data



Thickness of plate = 11 mm

Grade = 4.6

dia of bolt = 12 mm

4.6 yield stress = 240 N/mm2

(fub) ultimate stress = 400 N/mm2

factored load = 1.5 × 240

= 360 KN

fe = 410

bolt hole diameter = 12 + 1 mm

= 13 mm

minimum edge distance = 1.5 × hole diameter

= 1.5 × 13

= 19.5

minimum pitch = 2.5 × 12

= 30 mm

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

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

= 88.21

$$V_{nsb} = \frac{400}{\sqrt{3}}(1 \times 88.21 + 0)$$

$$= 20371.22 \text{ N}$$

$$= 20.371 \text{ kN}$$

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}}$$ $\boxed{\gamma_{mb} = 1.25 \quad \text{Table no - 5}}$

$$= \frac{20.871}{1.25} = 16.29 \text{ kN}$$

Bearing capacity of bolt: $\boxed{\text{Pg no 175}}$

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

V_{dpb} - nominal bearing strength of bolt

$$= 2.5 \times k_b \times d \times t \times f_u$$

k_b is smaller of $\frac{e}{3d_0} = \frac{19.5}{3 \times 13} = 0.5$

$$= \frac{p}{3d_0} - 0.25 = \frac{30}{3 \times 13} = 0.78$$

$$= 0.78 - 0.25 = 0.51 \text{ mm}$$

$$\frac{f_{ub}}{f_u} = \frac{400}{410} = 1.0$$

$$K_b = 0.5 \text{ mm}$$

$$V_{npd} = 2.5 \times 0.51 \times 12 \times 1.1 \times 400$$

$$= 6600 \text{ N}$$

$$= 66.00 \text{ kN}$$

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

$$= \frac{66.00}{1.25} = 52.8 \text{ KN}$$

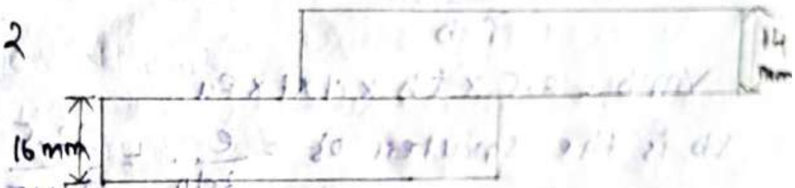
$\therefore$ Thus, $V_{dpb} > V_{dsb}$ (ok)

Bolt value = 16.29 kN

Req no of bolt :-

$$\frac{\text{factor load}}{\text{bolt value}} = \frac{360}{16.29} = 22.09 \simeq 23 \text{ bolt}$$

Q) Design a lap joint between two plates as shown in below figure so as to transmite a load of 90 kN using M14 bolt of grade 5.2



Soln Given datas

Thickness of plate = 14 mm

Grade = 5.2

Dia of bolt = 14 mm

Ultimate stress f_u = 50 kgf/mm2

= 500 N/mm2

yield stress = 10 kgf/mm2

= 100 N/mm2

bolt hole diameter = 14+1 mm

= 15 mm

minimum edge distance = 1.5 × hole diameter

= 1.5 × 15 mm

= 22.5 mm

minimum picth = 2.5 × 14 mm

= 35 mm

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

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

$= 0.78 \times \frac{\pi}{4} \times (14)^2$

$= 120 \ mm^2$

$n_n = 1$

$\therefore V_{nsb} = \frac{500}{\sqrt{3}} (1 \times 120 + 0)$

$= 34641.016 \ N = 34.641 \ kN$

$\therefore V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}}$ $\quad \because \gamma_{mb} = 1.25$ (Table no - 5, Pg no - 30)

$= \frac{34.641}{1.25} = 27.71 \ kN$

Bearing capacity of bolt (Pg no - 75 cl no : 10.3.4)

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

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

k_b is the smaller of $= \frac{e}{3d_0} = \frac{22.5}{3 \times 15} = 0.5$

$\frac{p}{3d_0} - 0.25 = \frac{35}{3 \times 15} - 0.25$

$= 0.527$

$\frac{f_{ub}}{f_u} = \frac{500}{}$

Types of welded joints

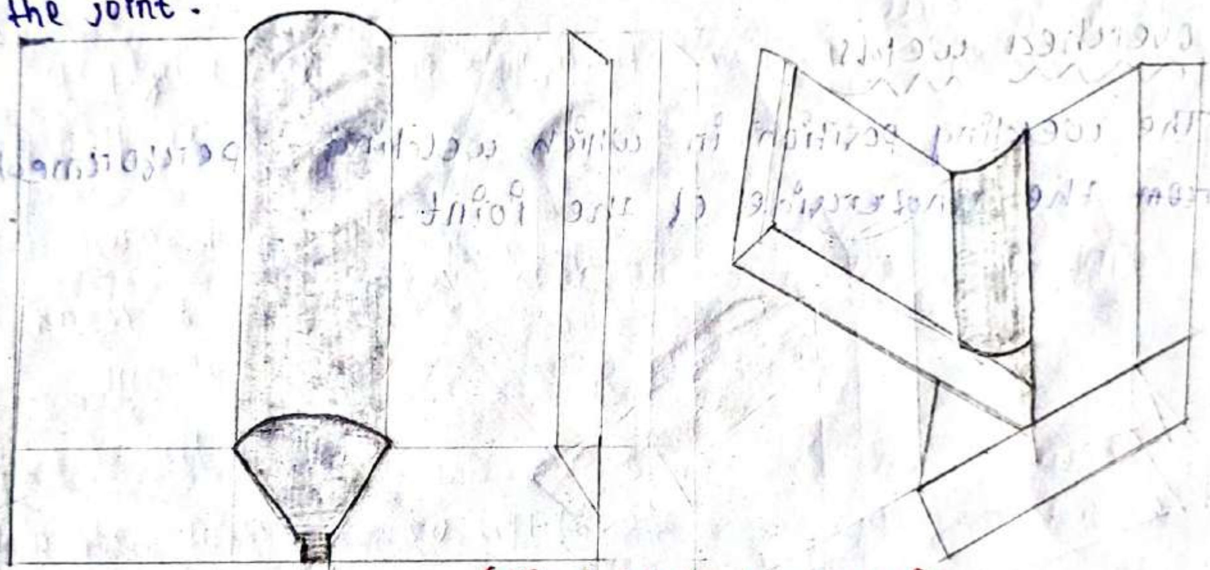
Weld forms based on their position :-

→ Based on the position of the welds, the welds are classified as.

* flat welds.
* Horizontal welds.
* vertical welds
* overhead welds

→ flat welds

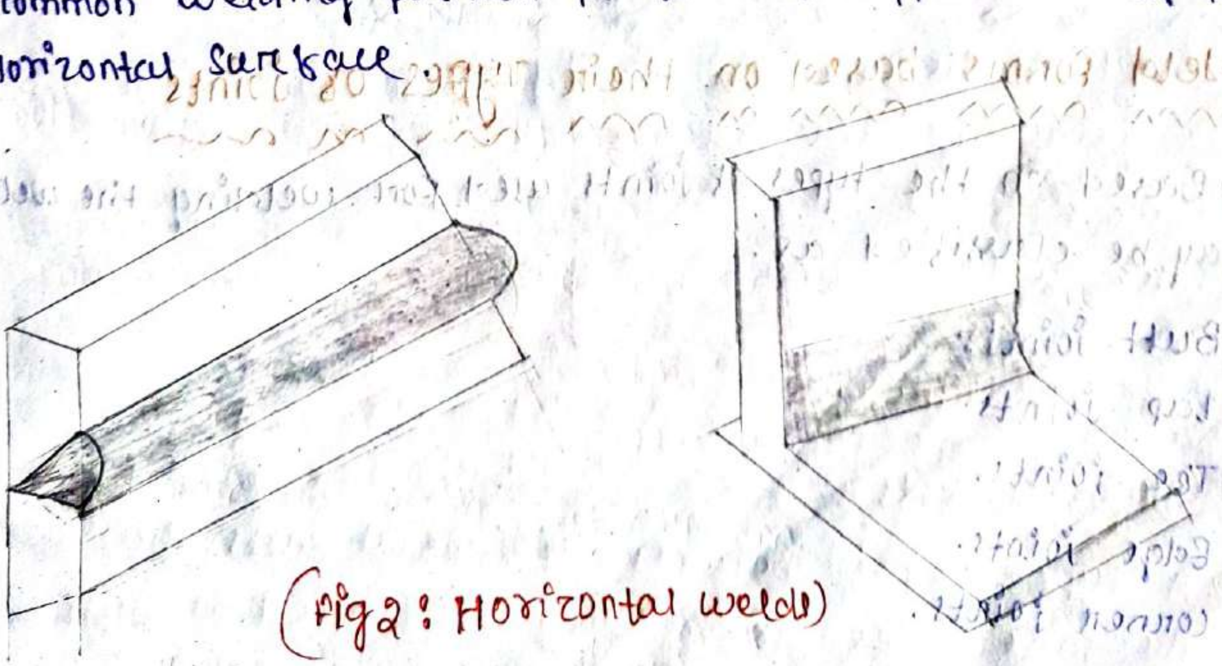
* The welding position used to weld from the upper side of the joint.



(Fig 1 : Flat welds)

→ Horizontal welds :-

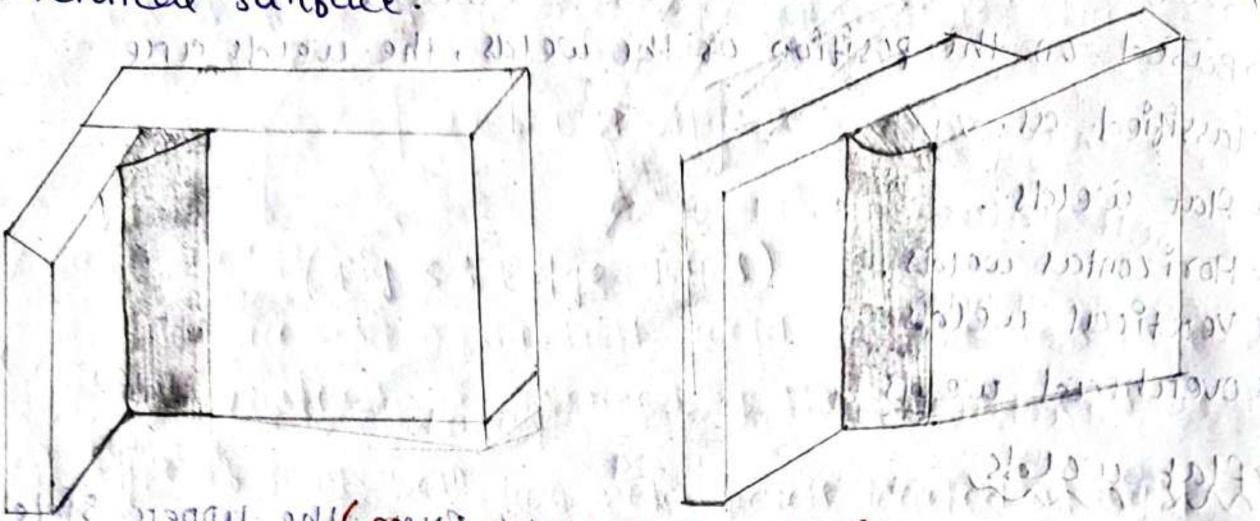
* common welding position performed upper side of the Horizontal surface



(Fig 2 : Horizontal welds)

* Vertical welds

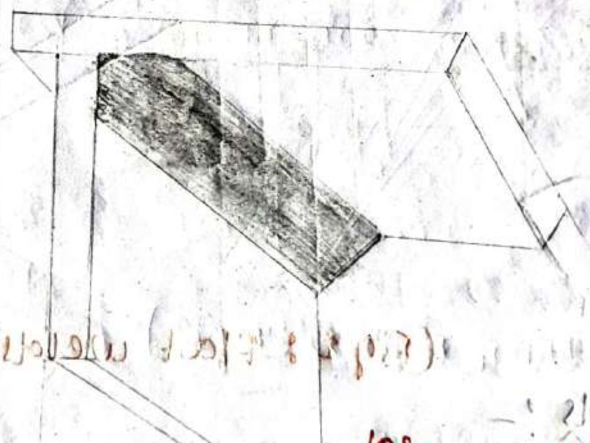
The welding position in which welding is done on a vertical surface.



(Fig. 3: vertical welds)

→ overhead welds

* The welding position in which welding is performed from the underside of the joint.



(Fig 4: overhead welds)

Weld forms based on their types of joints

→ Based on the types of joints used for welding the weld may be classified as.

* Butt joints.
* Lap joints.
* Tee joints.
* Edge joints.
* corner joints.

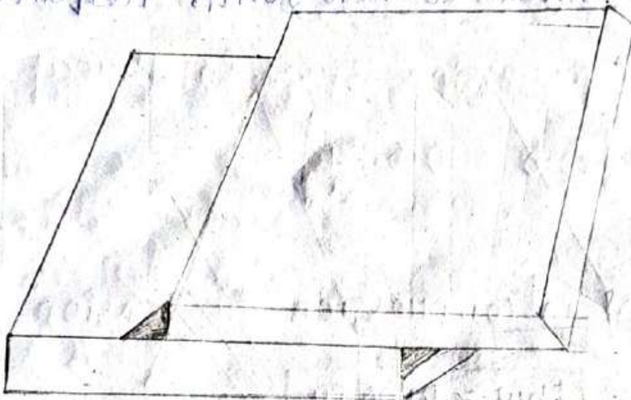
* Butt joints :-

→ A type of joint between two metal parts that lie in the same plane. A butt joint is the most common joint type.

(Fig 5 : Butt joints)

* Lap joints :-

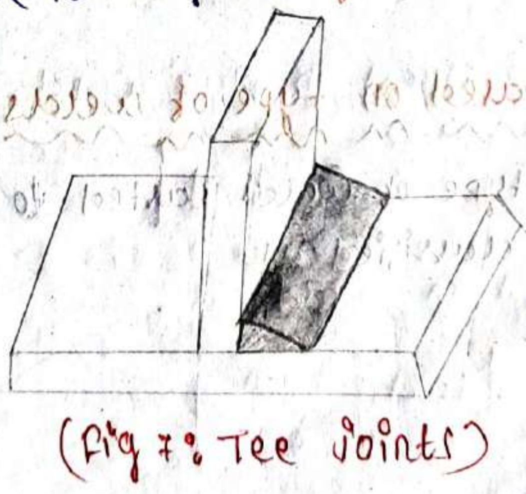
→ A type of joint betn two overlapping metal parts in parallel planes



(Fig 6 : Lap joints)

* Tee joints :-

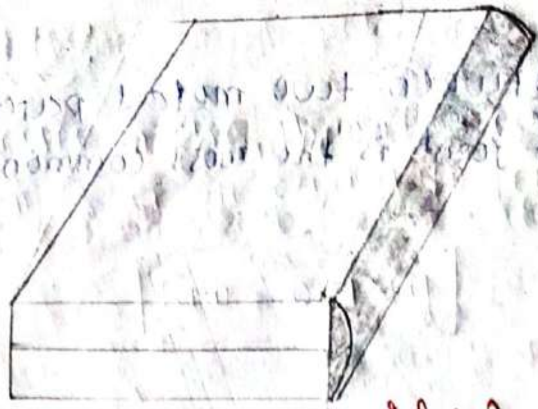
→ A type of joint produced when two metal parts are perpendicular to each other forming the shape of the letter "T"



(Fig 7 : Tee joints)

* Edge joints :-

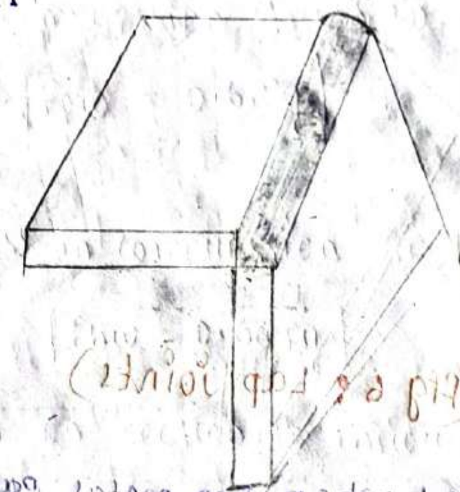
→ A type of joint in which the surface of the two metal parts to be joined are parallel to one another, and the weld is made at their common edges.



(fig 8 : Edge joints)

* Corner joints :-

→ A type of joint betn two metal parts located at right angles to one another. corner joints require large amounts of weld metal.



(fig 9 : corner joints)

Weld forms based on type of welds :-

→ Based on the type of welds casted to join the member. The welds are classified as,

* Groove weld.
* Fillet welds
* Slot welds.
* Plug welds.

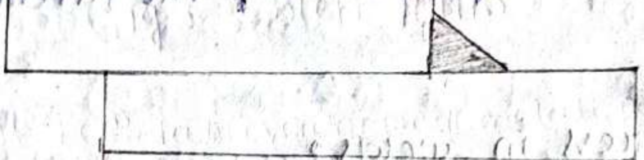
* Groove welds :-

→ A type of joint between two metal parts that lie in the same plane. Groove welds also called as Butt weld

(Fig 10: Groove welds)

* Fillet weld

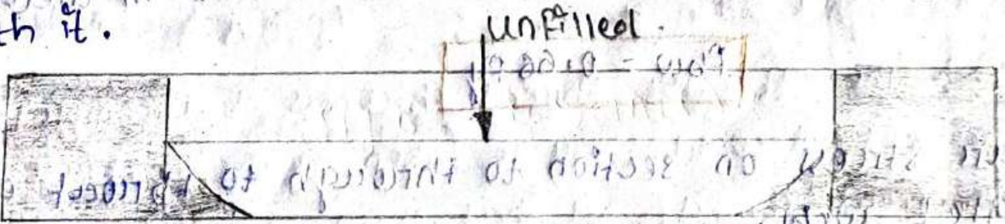
→ A type of weld that is triangular in shape and joins two surfaces at right angles to each other in a lap joint, T-joint or corner joint.



(Fig 11: Fillet weld)

* Slot welds:-

A type of weld made by joining one metal part with an elongated hole to another metal part positioned directly beneath it.



(Fig 12: Slot welds)

* Plug welds

A type of weld made by joining one metal part with a circular hole to another metal part positioned directly beneath it.

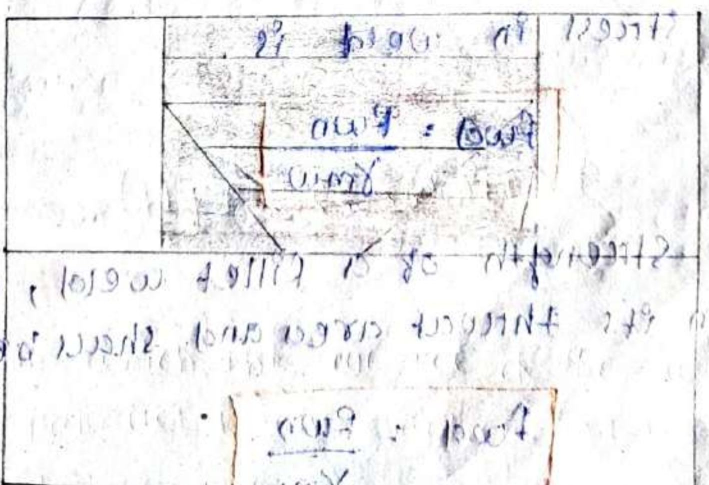


Fig 13: Plug welds

i) Fillet welds :-

Actual stresses in the throat area of fillet welds shall be less than ~~of~~ or equal to permissible stresses.

$$\boxed{f_{aw} = 0.4\,f_y}$$

ii) Butt welds :-

→ Actual stresses in the butt weld shall be less than the permissible stress as governed by the parent metal welded together

Permissible stress in welds :-

→ Tension or compression stress in throat or butt welds

$$\boxed{f_{pw} = 0.6\,f_y}$$

→ Bending stress in compression or tension in welds

$$\boxed{f_{bw} = 0.66\,f_y}$$

→ Shear stress on section to through to throat of butt or fillet weld.

$$\boxed{\tau_w = 0.44\,f_y}$$

→ Permissible stress in plug weld.

$$\boxed{f_{pw} = 0.44\,f_y}$$

→ Design stress in weld is.

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

* Design strength of a fillet weld, f_{wd} shall be based on its throat area and shall be given by

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

where,

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

f_u = smaller of the ultimate stress of the weld or of the parent metal and.

γ_{mw} = partial safety factor (see Table-5)

Size of weld :- 10.5.2 (pg no. 78)

10.5.2.1 :-
The size of ~~norml~~ normal fillets shall be taken as the min^m 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.

10.5.2.2 :-
For fillet welds made by semi-automatic or automatic processes, where the depth of penetration is considerably in excess of 2.4 mm, the size shall be taken considering actual depth of penetration subject to agreement betn the purchaser and the contractor.

10.5.2.3 :-
The size of fillet welds shall not be less than 3 mm. The min^m size of the first run or of a single run fillet weld shall be as given in Table 21, to avoid the risk of cracking in the absence of preheating.

10.5.2.4 :-
The size of butt weld shall be specified by the effective throat thickness.

Effective throat Thickness :- 10.5.3 (pg no 78)

10.5.3.1 :-
The effective throat thickness of a fillet weld shall not be less than 3 mm and shall generally not exceed 0.7 t or 1.0 t under special circumstances, where t is the thickness of the thinner plate of elements being welded.

Table 21 minimum size of first Run or of a single Run fillet weld

(clause 10.5.2.3)

Sl NO	Thickness of Thicker part mm Over	Thickness of Thicker part mm Up to and Including	minimum size mm
(1)	(2)	(3)	(4)
i)	–	10	3
ii)	10	20	5
iii)	20	32	6
iv)	32	50	8 of first run 10 for minimum size of weld

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

10.5.3.3:- The effective throat thickness of a complete penetratio 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.

Table 22 values of k for Different Angles Between fusion faces.

(clause 10.5.3.2)

Angle Between fusion faces	60°-90°	91°-100°	101°-106°	107°-113°	114°-1
constant, k	0.70	0.65	0.60	0.55	0.50

10.5.4.3 :- The effective area of a plug weld shall be considered as the nominal area of the hole in the plane of the faying surface. These welds shall not be designed to carry stresses.

10.5.4.4 :- If the maximum length l_j of the side welds transferring shear along its length exceeds 150 times the throat size of the weld ip the reduction in weld strength as per the long joint (see 10.5.7.3) should be considered. For flange to web connection where the welds are loaded for the full length the above limitation would not apply.

10.5.5 Intermittent welds

10.5.5.1 :- Unless otherwise specified, the intermittent fillet welding shall have an effective length of not less than four times the weld size, with a minimum of 40 mm.

10.5.5.2 :- The clear spacing between the effective lengths of intermittent fillet weld shall not exceed 12 and 16 times the thickness of thinner plate joined, for compression and tension joint respectively, and in no case be more than 200 mm.

10.5.5.3 :- Unless otherwise specified, the intermittent butt weld shall have an effective length of not less than four times the weld size and the longitudinal space between the effective length of welds shall not be more than 16 times the thickness of the thinner part joined. The intermittent welds shall not be used in positions subject to dynamic, repetitive and alternating stresses.

10.5.6 Weld types and Quality :-

For the purpose of this code, weld shall be fillet, butt slot or plug or compound welds. Welding electrodes shall conform to IS 814.

Procedure for design of fillet weld :-

Given data :-

Size of plate (length x thickness)

Tensile stress in N/mm^2

Load or tension on weld in KN

Ultimate stress in weld N/mm^2

Step - 1 :- Strength of weld :- $\left(\frac{F}{A}\right)$

* If tension force or load is given then strength of weld (Given load or tension force)

* If tensile stress is given, strength of plate = (Area of plate x tension stress)

* If permissible stress is given (cl = 11.6.2.4) assume permissible stress as tensile stress.

Step-2 :- Design strength weld :- (B.O.S)

* Design strength of weld = 1.5 x strength of weld

Step-3 :- Size of weld :-

* If fillet weld is of squre edge, size of weld = Thickness of Plate - 1.5

* If fillet weld is of Rounded edge, then size of weld = $\frac{3}{4}$ x thickness of plate.

Step-4 :- Partial safety factor γ_{mw}, (Table no-5, pg no-30)

shop weld → 1.25

field weld → 1.50

Step-5 :- Design stress of weld :-

(cl no - 10.5.7.1.1) (pg no - 79)

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

Step - 6 :- Effective length of weld :-

Design strength = Area of weld × design stress.

or

Design strength = Throat thickness × length of weld × fwd

$\therefore$ Design strength = 0.7 × S × L × fwd

$$\therefore \boxed{\text{effective length i.e } = \frac{\text{Design strength}}{0.7 \times S \times f_{wd}}}$$

In case, weld is done no of sides

$$\boxed{\text{length of each side} = \frac{\text{effective length}}{\text{No. of sides}}}$$

* minimum over lap length

= $\boxed{\text{length of fillet weld} + 2\ \text{size}}$

Q) A flat of size 120 mm × 8 mm carrying a load of 130 KN is to be connected at its end with a gusset plate by side fillet weld in the workshop. Determine the minm length of over lap required if the ultimate stress of the fillet weld is 330 N/mm2

Given data

length = 120 mm

Thickness = 8 mm

ultimate stress, fu = 330 N/mm2

Load of flat = 130 KN

Step - 1 :- Strength of plate = 130 KN.

Step - 2 :- Design Strength of weld = 1.5 × Strength of weld

= 1.5 × 130

= 195 KN

Step - 3 = Size of weld = Thickness of plate − 1.5

= 8 − 1.5 = 6.5 mm

Adopt = 7 mm weld size

Step - 4 = Partial safety factor :-

γ_{mw} = 1.25 (IS 800 : 2007, Table No - 5)

Step - 5 = Design stress of weld :-

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

$= \frac{330}{\sqrt{3}} = 190.52$ ~~KN~~ N/mm²

$\frac{f_{wn}}{\gamma_{mw}} = \frac{190.52}{1.25}$

= 152.416 ~~KN~~ N/mm²

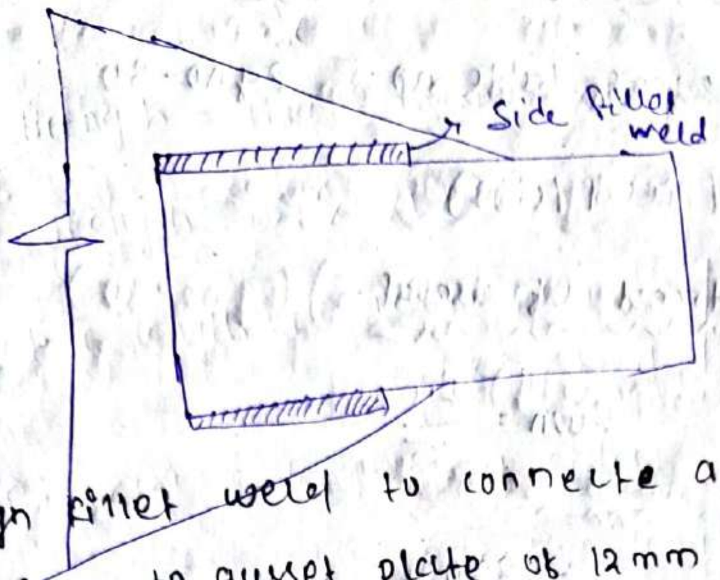
Step-6 Effective length of weld.

Effective length = $\frac{\text{Design Strength}}{0.7 \times S \times f_{wd}}$

$= \frac{195 \times 10^3}{0.7 \times 7 \times 152.416} = 261.10$

= Length of each side = Effective length / No. of sides ÷ 2

=



Q) Design fillet weld to connecte a plate of 100 mm x 8 mm to gusset plate of 12 mm thickness tensile stress in the plate is 150 N/mm² & the ultimate stress in weld is 330 N/mm² Assume the connection are made at site / field.

Given data

plate = 100 mm x 8 mm

Gusset stress in plate = 150 N/mm²

ultimate stress in weld = 330 N/mm²

Step-1 Strength of weld :-

Strength of weld = Area of plate x Tensile stress

= (100 x 8) x 150

= 120000 N

= 120 KN.

Step-2 Design strength of weld

Design strength = F.O.S x Strength of weld

= 1.5 x 120 = 180 KN

Step-3 Thickness of the plate :-

size of weld = Thickness of the plate - 1.5

= 8 - 1.5

= 6.5 .

adopt 6mm size of fillet weld

Step-4 :- partial safety factor γ_{mw}

As per IS 800 : 2007, table no-5, pg no-30

$\gamma_{mw} = 1.50$ (Field / site)

Step-5 Design stress of weld

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

$= \frac{190.52}{1.50}$ $= \frac{330}{\sqrt{3}}$

$= 127.01\ N/mm^2$ $= 190.52\ N/mm^2$

Step-6 Effective length of weld :-

Effective length of fillet weld $= \frac{\text{Design strength}}{0.7 \times s \times f_{wd}}$

$= \frac{180 \times 10^3}{0.7 \times 6 \times 127.01}$

$= 337.43$ mm

$\simeq 338$ mm

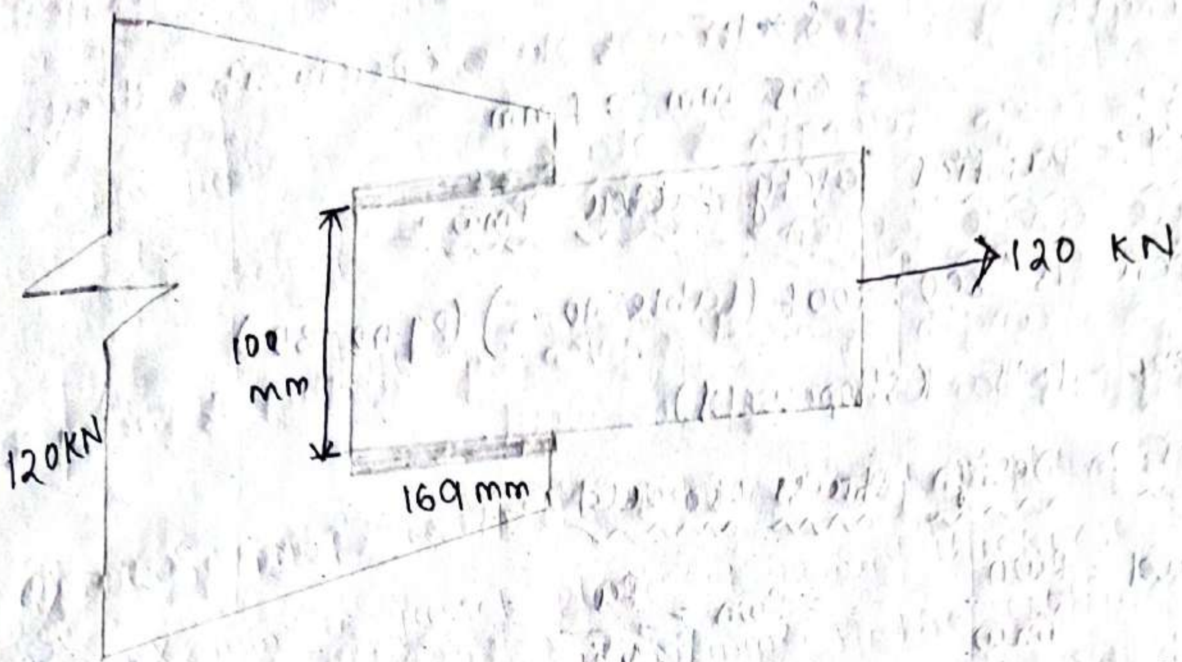
Length of each side $= \frac{\text{effective length}}{\text{No. of sides}}$

$= \frac{338}{2} = 169$ mm

minimum over lap length = length of fillet weld + 2 size

$= 169 + 2 \times 6$

$= 181$ mm



Q) Design fillet weld to connect a plate of 120 mm x 8 mm to gusset plate of 10 mm thickness the permissible stress in the plate is 150 N/mm² & the ultimate stress in the weld is 330 N/mm². Assume the connection are made at workshop.

Given data

Plate = 120 mm x 8 mm

Permissible stress = 150 N/mm²

ultimate stress : 330 N/mm²

Step-1 strength of weld

Strength of weld = Area of plate x Tensile

$= (120 \times 8) \times 150$

$= 144000$ N

$= 144$ KN

Step-2 Design strength of weld

Design strength = F.O.S x strength of weld

$= 1.5 \times 144$

$= 216$ KN

Step-3:- Thickness of the plate:-

Size of weld = Thickness of the plate − 1.5

$= 8 - 1.5$

$= 6.5\ mm \simeq 7\ mm$

Step-4:- Partial safety factor γ_{mw}:-

As per IS 800:2007 (table no-5) (pg no-30)

$\gamma_{mw} = 1.25$ (shop weld)

Step-5:- Design stress of weld:-

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

$= \frac{330}{\sqrt{3}} = 190.52\ N/mm^2$

$\Rightarrow \frac{190.52}{1.25}$

$= 152.42\ N/mm^2$

Step-6:- Effective length of weld:-

effective length of fillet weld $= \frac{\text{Design strength}}{0.7 \times S \times f_{wd}}$

$= \frac{126 \times 10^3}{0.7 \times 7 \times 152.42}$

$= 168.70\ mm$

$\simeq 169\ mm$

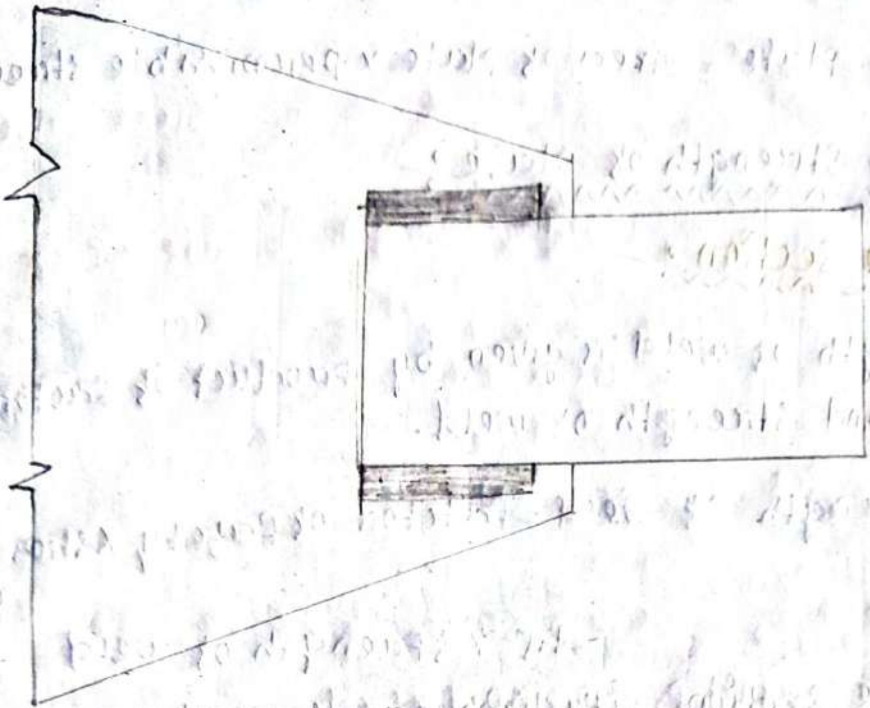
Length of each side $= \frac{\text{effective length}}{\text{No. of sides}}$

$= \frac{169}{2} = 84.5\ mm$

minimum over lap length = length of fillet weld + 2 × size

$= 84.5 + 2 \times 7$

$= 98.5\ mm$



Procedure for design of fillet welded joint for angle section :-

*procedure for design of fillet welded for section for which axial or tension, shear stress and angle size is given.

Given data:-

Angle section size in 'a' mm x 'b' mm x 't' mm

load or tension in kN

shear stress in N/mm2

Step-1:- size of weld:-

Solution:- As per IS 800:2007 Cl-10.5.8.2 for rounded toe rolled section.

Size of welded $= \frac{3}{4} \times$ minimum thickness of plate.

Step-2:- Strength of angle

If tension force or load is given, then

Strength of plate = Given load or tension force

If tensile stress is given, then.

Strength of plate = Area of plate x tension stress

* If permissible stress is given, then as per IS 800:2007 cl 11.6.2.4 assume permissible stress is equal to tensile stress.

Strength of plate = Area of plate × permissible stress.

Step-3 :- Design strength of plate :-

Single angle section :-

* Design strength of weld is given by product of factor of safety (1.5) and strength of weld.

Design strength of weld = factor of safety × strength of weld

= 1.5 × strength of weld

Double angle section with two equal angles :-

Design strength of one section = Design strength

Step-4 :- partial safety factor γ_{mw} :-

* The partial safety factor γ_{mw} is taken from IS 800:2007 table no-5 (pg no 30)

* For weld section mostly for shop welds safety factor is 1.25 and for field welds is 1.50.

Partial safety factor for materials, γ_m

SL NO	Definition	Partial safety factor	
1	Resistance, governed by yielding, γ_{m0}	1.10	
2	Resistance of member to buckling, γ_{m0}	1.10	
3	Resistance, governed by ultimate stress γ_{m1}	1.25	
4	Resistance of connection	Shop fabrications	Field fabrications
a)	Bolts - friction type, γ_{mf}	1.25	1.25
b)	Bolts - bearing type, γ_{mb}	1.25	1.25
c)	Rivets, γ_{mr}	1.25	1.25
d)	Welds, γ_{mw}	1.25	1.50

Step 5 :- Design stress of the weld :-

→ The design stress of weld can be calculated from the formula from (CL 10.5.7.1.1 of IS 800:2007)

$$\boxed{f_{wd} = \frac{f_{wn}}{\gamma_{mn}}}$$

Where,

f_{wn} = Nominal stress in the weld, $f_{wn} = \frac{f_u}{\sqrt{3}}$

f_u = ultimate stress in the weld

γ_{wn} = partial safety factor (from step 4)

Step 6 :- Design of joint using fillet weld :-

→ For a given section the value of C_{zz} and C_{yy} is taken from the steel table.

For side fillets :-

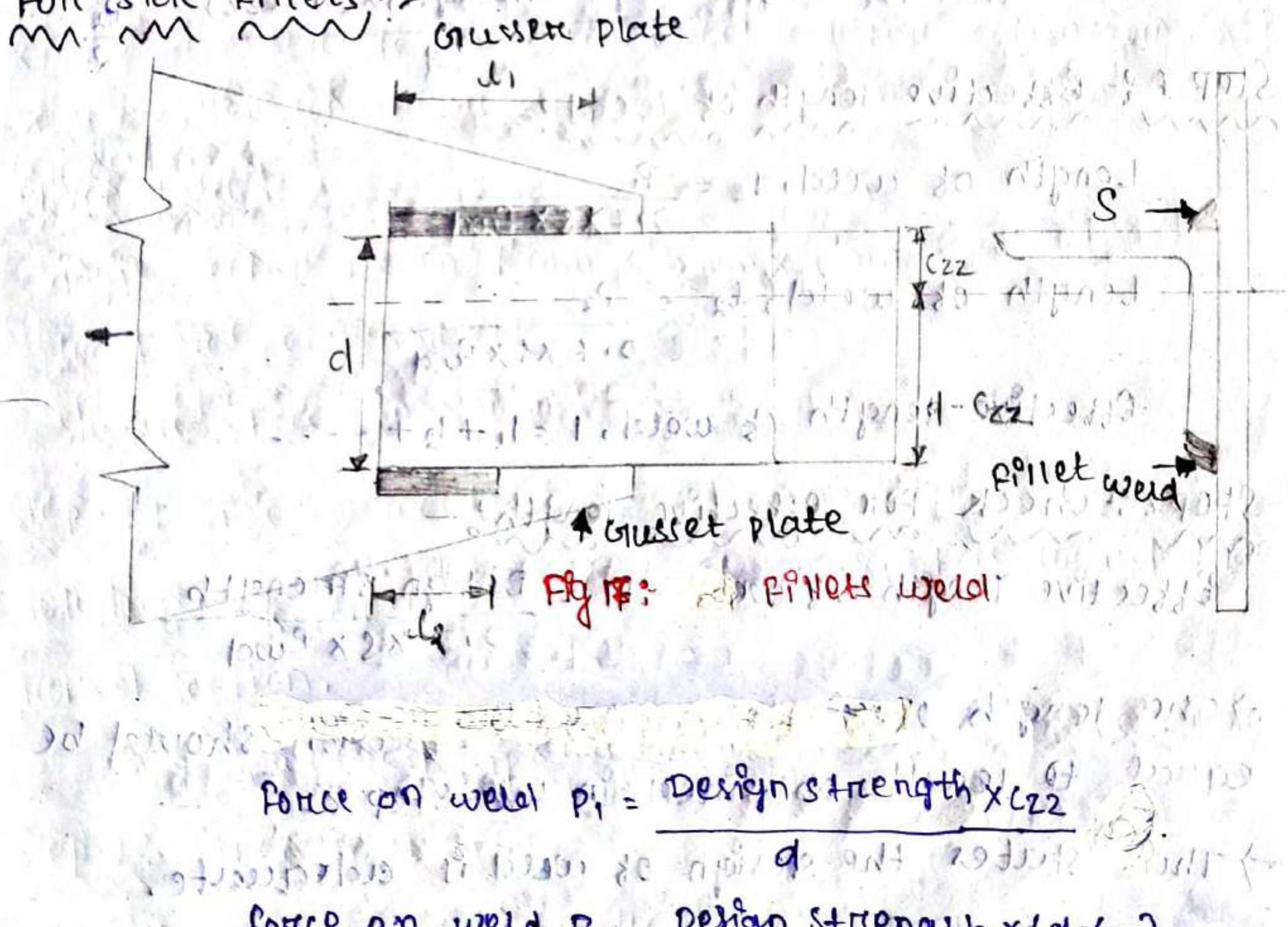


Fig 17 :- Fillets weld

Force on weld $P_1 = \frac{\text{Design strength} \times C_{zz}}{d}$

Force on weld $P_2 = \frac{\text{Design strength} \times (d - C_{zz})}{d}$

For side and end fillets:-

Force on weld $P_2 = f_{wd} \times 0.7 \times s \times d$

Force on weld $P_1 = \frac{\text{Design strength} \times C_{zz}}{d} - \frac{P_2}{2}$

Force on weld $P_3 = \frac{\text{Design strength} \times (d - C_{zz})}{d} - \frac{P_2}{2}$

Step 7:- Effective length of weld:-

Length of weld, $l_1 = \frac{P_1}{0.7 \times s \times f_{wd}}$

Length of weld, $l_2 = \frac{P_2}{0.7 \times s \times f_{wd}}$

Effective length of weld, $l = l_1 + l_2 + \ldots$

Step 8:- Check for effective length:-

Effective length of weld, $L = \frac{\text{Design strength}}{0.7 \times s \times f_{wd}}$

→ The length obtained in above equation should be equal to length obtained in the step 8.

→ Thus states the design of weld is adequate.

ISA = Indian

Q) An angle tie of ISA 150mm x 150 mm x 10 mm, carrying an axial load of 500 kN is to be connected to a 12mm thick gusset plate through its leg by side fillet welds only at site. Design the joint if the ultimate sheare stress in the weld is 410 MPa.

Given data:-

ISA = 150 x 150 x 10mm

Axial load = 500 kN

f_u, ultimate shear stress = 410 MPA

Step 1:- Size of weld:-

Size of weld $= \frac{3}{4} \times$ min^m thickness of plate

$= \frac{3}{4} \times 10$

$= 7.5$ mm

Adopt 7mm size of fillet weld.

Step 2:- Strength of angle:-

Strength of angle plate = 500kN

Step 3:- Design strength of plate:-

Design strength = F.O.S x Strength of weld

$= 1.5 \times 500$

$= 750$

Step 4:- Partial safety factor:-

$\gamma_{mw} = 1.50$ (Table no - 5 pg no - 30)

Step 5:- Design stress of weld:-

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

$f_{wn} = \frac{f_u}{\sqrt{3}} = \frac{410}{\sqrt{3}}$

$= 236.71$

$\gamma_{mw} = 1.50$

$f_{wd} = \frac{236.71}{1.50} = 157.80 \text{ N/mm}^2$

Step-6 Design of Joint using fillet weld :-

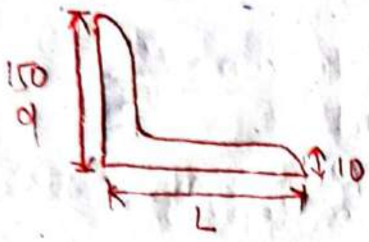
$C_{zz} = 4.06$ cm

$= 40.6$ mm

For given section the value $C_{zz} = 40.6$ mm from (Steel table)

For side fillet :-

Force on weld $P_1 = \dfrac{\text{Design strength of plate} \times C_{zz}}{d}$



$= \dfrac{750 \times 40.6}{150}$

$= 203$ KN

Force on weld $P_2 = \dfrac{\text{Design strength} \times (d - C_{zz})}{d}$

$= \dfrac{750 \times (150 - 40.6)}{150}$

$= 547$ KN

Step-7 Effective length of weld :-

length of weld $l_1 = \dfrac{P_1}{0.7 \times s \times f_{wd}} = \dfrac{203 \times 10^3}{0.7 \times 7 \times 157.80}$

$= 262.53$ mm

length of $l_2 = \dfrac{P_2}{0.7 \times s \times f_{wd}}$

$= \dfrac{547 \times 10^3}{0.7 \times 7 \times 157.80} = 707.43$ mm

effective length of weld $= l_1 + l_2$

$= 262.53 + 707.43$

$= 969.9$ mm

Step-8 check for effective length :-

Effective length of weld :- $\dfrac{\text{Design strength}}{0.7 \times s \times f_{wd}}$

$= \dfrac{750 \times 10^3}{0.7 \times 7 \times 157.80}$

$= 969.9$ mm

∴ Design is ok or adequate

Q) An angle ISA 125 mm x 95 mm x 10 mm carrying a axial load of 220 KN is connected to gusset plate 12 mm thick design the welded conected with side and end fillet if the ultimate shear stress of the weld is 410 MPA assume the connection are made in site.

Given data

ISA = 125 mm x 95 mm x 10 mm

Axial load - 220 KN

Ultimate shear stress = 410 MPA

Step-1 Size of weld :-

Size of weld = $\frac{3}{4}$ x min^m thikness of plate

$= \frac{3}{4} \times 10$

$= 7.5$ mm

Adopt 7 mm size of fillet weld.

Step-2 Strength of angle :-

Strength of angle plate : 220 KN

Step-3 Design strength of plate :-

Design strength = F.O.S x strength of weld

$= 1.5 \times 220$

$= 330$

Step-4 partial safety factor :-

$\gamma_{mw} = 1.50$ (Pg no - 30 table no - 5)

Step-5 Design stress of weld :-

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

$f_{wd} = \frac{236.71}{1.50} = 157.80 \text{ N/mm}^2$

$f_{wd} = \frac{f_u}{\sqrt{3}} = \frac{410}{\sqrt{3}}$

$= 236.71$

$\gamma_{mw} = 1.50$

Step-6 Design of Joint using fillet weld :-

For side fillet and end fillet

$$C_{zz} = 3.88 \text{ cm} = 38.8 \text{ mm}$$

$$P_2 = f_{wd} \times S \times 0.7 \times d$$

$$= 157.80 \times 7 \times 125$$

$$= 96.658 \text{ KN}$$

For side fillet

$$P_1 = \frac{\text{Design strength} \times C_{zz}}{d} - \frac{P_2}{2}$$

$$= \frac{330 \times 38.8}{125} - \frac{96.658}{2}$$

$$= 54.103 \text{ KN}$$

$$P_3 = \frac{\text{Design strength} \times (d - C_{zz})}{d} - \frac{P_2}{2}$$

$$= \frac{330 \times (125 - 38.8)}{125} - \frac{96.658}{2}$$

$$= 179.239 \text{ KN}$$

Step-7 Effective length of weld

$$L_1 = \frac{P_1}{0.7 \times S \times f_{wd}}$$

$$= \frac{54.103 \times 10^3}{0.7 \times 7 \times 157.80}$$

$$= 69.97 \text{ mm}$$

$$l_2 = \frac{P_2}{0.7 \times S \times f_{wd}}$$

$$= \frac{96.658 \times 10^3}{0.7 \times 7 \times 157.80}$$

$$= 125 \text{ mm}$$

$$l_3 = \frac{P_3}{0.7 \times S \times f_{wd}}$$

$$= \frac{179.239 \times 10^3}{0.7 \times 7 \times 157.80}$$

$$= 231.81 \text{ mm}$$

$$l = l_1 + l_2 + l_3$$

$$= 69.97 + 125 + 231.81$$

$$= 426.78 \text{ mm}$$

Step-8 Check for effective length:-

Effective length of weld :- $\frac{\text{Design strength}}{0.7 \times S \times f_{wd}}$

$$= \frac{330 \times 10^3}{0.7 \times 7 \times 157.80}$$

$$= 426.78 \text{ mm}$$

$\therefore$ Design is ok or adequate.

3Q) A tie member of a roof truss consists 2 ISA 100mm X 100mm X 8mm angles are connected by a site fillet to either site 8 mm thick gusset plate and member is subjecte to tension of 400KN. Design the welded connection Assume connection are made at site, and ultimate shear stress in the weld 410 MPa.

Given data

2ISA = 100 mm × 100 mm × 8 mm (The given section is double angle section with, two equal angle)

Total load = 400 KN

Ultimate shear stress f_u = 410 MPa = 410 N/mm²

Soln:-

Step-1:- Size of weld

As per IS 800:2007 Cl 10.5.8.2 rounded toe rolled section.

$$\text{Size of weld} = \frac{3}{4} \times 8$$

$$= 6\,mm$$

Step2:- Strength of angle:-

Tension force is given. Hence.

Strength of plate = 400 KN

Step3:- Design strength of plate:-

Design strength of weld = Factor of safety × strength of weld

$$= 1.5 \times 400$$

$$= 600\,KN$$

Double angle section with two equal angles:-

$$\text{Design strength of one-section} = \frac{600}{2} = 300\,KN$$

Step4:- partial safety factor γ_{mw}

$$\gamma_{mw} = 1.50$$

Step5:- Design stress of the weld:-

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

$$= \frac{236.71}{1.5} \qquad = \frac{410}{\sqrt{3}} = 236.71\,N/mm^2$$

$$= 157.8\,N/mm^2$$

Step-6:- Design of joint using fillet weld:-

$$C_{zz} = 2.76\,cm$$

$$= 27.6\,mm$$

For a given section the value of C_{zz} = 27.6 from (steel table)

For side fillet :-

Force on weld $P_1 = \frac{\text{Design strength of plate} \times C_{zz}}{d}$

$= \frac{300 \times 27.6}{100} = 82.8$ kN

$P_2 = \frac{300 \times (100 - 27.6)}{100} = 217.2$ kN

Step-7 Effective length of weld :-

$l_1 = \frac{P_1}{0.7 \times S \times f_{wd}}$

$= \frac{82.8 \times 10^3}{0.7 \times 6 \times 157.8} = 124.93$ mm

$l_2 = \frac{217.2 \times 10^3}{0.7 \times 6 \times 157.8} = 327.72$ mm.

$l = l_1 + l_2$

$= 124.93 + 327.72$

$= 452.65$ mm

Effective length of an both side $= 2 \times 452.65$

$= 905.3$ mm

Step-8 :- Check for effective length

Effective length of weld $l = \frac{600 \times 10^3}{0.7 \times 6 \times 157.8}$

$= 905.305$ mm

∴ Design is adequate.

Strength of welded joints :-

→ The strength of the weld joint depends on either tensile or shear, or a combination of both.

→ The direction of the weld joints decides the design stress acting on them.

→ Let's consider lap joint welding first. A lap joint is about joining the pieces after overlapping them, welding along the edge.

Procedure for calculating strength of weld :-

→ In problems based on calculation of strength of weld, safe load and tensile stress. The size of weld, the length of weld and ultimate shear stress in the weld will be given.

Given data :-

Size of weld in mm

Effective length of weld L in mm

Ultimate stress f_u in the weld in N/mm²

Procedure :-

Step 1 :- Throat thickness of weld :-

The throat thickness of weld is given as,

$$t = K \times s$$

where,

K = coefficient constant.

S = size of weld.

→ The K value is taken from IS 800 : 2007 table 22 for corresponding angle between fusion face.

→ If angle between fusion face is not given then the angle is taken as 90°.

K	0.7	0.65	0.60	0.55	0.50

Step 2: Area of weld:

The area of section is calculated by

$A = l \times t$

where,

l = Effective length.

t = Throat thickness of weld.

Step 3 :- partial safety factor γ_{mw} :-

→ The partial safety factor γ_{mw} is take from IS 800:2007 Table no-5

→ For weld section mostly for shop welds safety factor is 1.25 and for field welds is 1.50

Partial safety factor for materials, γ_m

Sl. No	Definition	Partial safety factor	
1	Resistance, governed by yielding, γ_{m0}	1.10	
2	Resistance of member to buckling, γ_{m0}	1.10	
3	Resistance governed by ultimate stress, γ_{m1}	1.25	
4	Resistance of connection		
		Shop Fabrications	Field Fabrications
a	Bolts - friction type, γ_{mf}	1.25	1.25
b	Bolts - bearing type, γ_{mb}	1.25	1.25
c	Rivets, γ_{mr}	1.25	1.25
d	welds, γ_{mw}	1.25	1.50

Step 4:- Design stress of the weld:-

→ The design stress of weld can be calculated from the formula from CL 10.5.7.1.1 of IS 800:2007

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

Where,

f_{wn} = Nominal stress in the weld, $f_{wn} = \frac{f_u}{\sqrt{3}}$

f_u = ultimate stress in the weld

γ_{mn} = partial safety factor (From step 4)

Step 5 :- Design strength of weld

→ The design strength of weld is given by the product of area of weld and design stress of weld.

Design strength = $A \times f_{wd}$

where, A = area of weld (From step 3)

f_{wd} = Design stress of weld (From step 5)

Step 6 :- Safe load :-

→ In case the problem is given to find the safe load of the weld.

$$\text{Safe load} = \frac{\text{Design strength}}{\text{Partial safety factor}}$$

Step 7 :- Tensile stress :-

→ In case the problem is given to find the tensile stress of the section.

$$\text{Tensile stress} = \frac{\text{Design strength}}{\text{Area of Plate}}$$

Q) Calculate the design strength of welded joint if the size of weld is 7mm & its effective length is 230mm. The ultimate stress 410 N/mm². Assume the connection are made in workshop.

Given data:-

Size of weld = 7mm

L = 230 mm

Step-1 Throat thickness weld

Throat thickness $\boxed{t = k \times s}$

Table No-22 (pg-78), k = 0.70

k = 0.70 × 7 = 4.9 mm [If in Question not given]

Step-2 Area of weld:-

$A = l \times t$

$= 230 \times 4.9 = 1127 \text{ mm}^2$

Step-3 partial safety factor

$\gamma_{mw} = 1.25$ (Table no-5 pg no-30)

Step-4 Design stress of weld :-

$f_{wd} = \frac{f_{wn}}{\gamma_{mw}}$ $f_{wn} = \frac{f_u}{\sqrt{3}} = \frac{410}{\sqrt{3}} = 236.71 \text{ N/mm}^2$

$\frac{236.71}{1.25} = 189.368 \simeq 189.37 \text{ N/mm}^2$

Step-5 Design strength weld

Design strength = $A \times f_{wd}$

$= 1127 \times 189.37$

$= 213419.99$ N

$= 213.42$ kN

Q) Find the safe load that can be transmitted to fillet welded joint of plates of size 8mm × 230mm & 10mm × 230 mm is casted in the workshop. The size of weld is 5mm & length of weld is 212mm. The ultimate stress in the weld is 410 N/mm². Find the tensile stress on the Thinner plate.

Step-1 Throat thickness

$t = K \times S$

$= 0.7 \times 5 = 3.5$

Step-2 Area of weld;

$A = l_w \times t$

$= 212 \times 3.5 = 742\ mm^2$

Step-3

P.S.f

$\gamma_{mw} = 1.25$

Step-4 Design stress of weld

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

$\therefore f_{wn} = \frac{f_u}{\sqrt{3}} = \frac{410}{\sqrt{3}} = 236.71\ N/mm^2$

$\therefore f_{wd} = \frac{236.71}{1.25} = 189.368\ N/mm^2$

Step-5 Design Strength of weld

$\therefore$ Design Strength $= A \times f_{wd}$

$= 742 \times 189.368$

$= 140511.016\ N/mm^2$

Step-6 Safe Load $= \frac{\text{Design Strength}}{\text{P.S.f}}$

$= \frac{140511.016}{1.25}$

$= 112408.9\ N$

$= 112.41\ kN$ CAM

Step-7 Tensile Stress $= \frac{\text{Design Strength}}{\text{Area of plate}}$

$= \frac{140511.016}{742}$

$=$ N/mm²

Modes of failure :-

i) yielding.

ii) Rupture

iii) Block shear failure

Q) calculate the design strength of a tension member due to yielding of Gross section for a plate of 200mm width and 10mm thickness ~~tak~~ take $f_y = 250\ N/mm^2$ [Inter

Given data :-

width of plate : 200 mm

thickness = 10 mm

$f_y = 250\ N/mm^2$

Design strength of Tension member due to yielding

$$\boxed{T_{dg} = \frac{A_g f_y}{\gamma_{m0}}}$$ (pg no - 32)

$\gamma_{m0} = 1.10$ (Table No - 5) (pg no - 30)

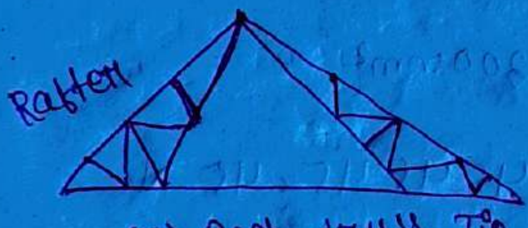
$A_g = b \times t = 200 \times 10 = 200\ mm^2$

$$T_{dg} = \frac{2000 \times 250}{1.10} = 454545.45\ N$$

$= 454.54\ kN$

Tension member :-

The structural element which is subjected to the direct axial tensile loads, that tend to elongate the members is called tension members.



(a) Roof truss Tie

Table 3 maximum values of effective (pg no-20)

Slenderness ratio :-

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

$$\boxed{\text{Slenderness ratio} = \frac{le}{r} = \frac{KL}{r}}$$

where,

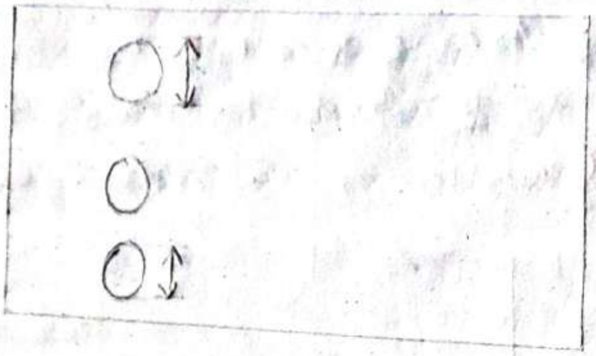
L = Actual length of compression member

le = KL = Effective length

r = Appropriate radius of gyration

Q) calculate design strength of a plate of size 200mm width and 10mm thickness which as three holes of diameter 20 mm as shown in figure. If f_y = 250 N/mm2 and f_u = 410 N/mm2

L5



Given data

width of plate = 200mm

thickness = 10mm

dia of hole = 20mm

f_y = 250 N/mm2

f_u = 410 N/mm2

Sol^n Design strength of plate due to yielding:-

A_g = 200×10 = 2000 mm2

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

γ_{m0} = 1.10

$$= \frac{2000 \times 250}{1.10}$$

$$= 454545.45 \text{ N}$$

T_{dg} = 454.54 kN

Design strength of plate due to Rupture:-

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

f_u = 410 N/mm2

γ_{m1} = 1.25 (Table no5)

A_n = Net effective area of cross section :-

$$A_n = \left[b - nd_n + \sum_i \frac{\cancel{p_{si}^2}\,0}{\cancel{4g_i}}\right]t$$

$$= (b - nd_n)t$$

$$= (200 - 3\times 20)10 = 1400\ mm^2$$

$$T_{dn} = \frac{0.9\times 1400\times 410}{1.25}$$

$$= 413280\ N$$

$$= 413.28\ KN$$

$\therefore$ Design strength of plate is least from T_{dg} & T_{dn}

$\therefore$ Design strength = 413.28 KN (Ans)

<u>Internal Question & Answer</u>

1)(a) Find yield strength of grade 5.4 bolted connection?

<u>Soln</u>:- 5.4

$x = \frac{1}{10}\times$ Ultimate tensile strength

$5 = \frac{1}{10}\times$ Ultimate tensile strength

$\Rightarrow$ Ultimate = 50 kgf/mm²

Yield strength $= \frac{1}{10}\left(\frac{\text{Yield strength}}{\text{Ultimate strength}}\right)$

$$\Rightarrow 4 = \frac{1}{10}\left(\frac{\text{Yield}}{50}\right)$$

$$\Rightarrow \text{Yield} = \frac{2000}{100} = 20\ kg/mm^2$$

Ultimate strength stress, $f_u = 50\ kg/mm^2$

$= 500\ N/mm^2$

Net effective area of cross section

$$A_n = \left[b - nd_h + \Sigma \frac{P_{si}^2}{4g_i}\right]t$$

$n = 5$

$$A_n = [300 - 5 \times 22] \times 20$$

$A_n = 3800 \, mm^2$

$$T_{dn} = \frac{0.9 \times 3800 \times 410}{1.25}$$

$= 1121760 \, N$

$T_{dn} = 1121.76 \, kN$

∴ Design strength of plate is least T_{dg} and T_{dn}

∴ Design strength = 1121.76 kN (Ans)

Chapter - 4

Compression member

→ Structural member which is subjected to compressive forces along its axis is called compression member.

→ In R.C.C compression member is called as coloumn while in steel structure compression member as called stanchion.

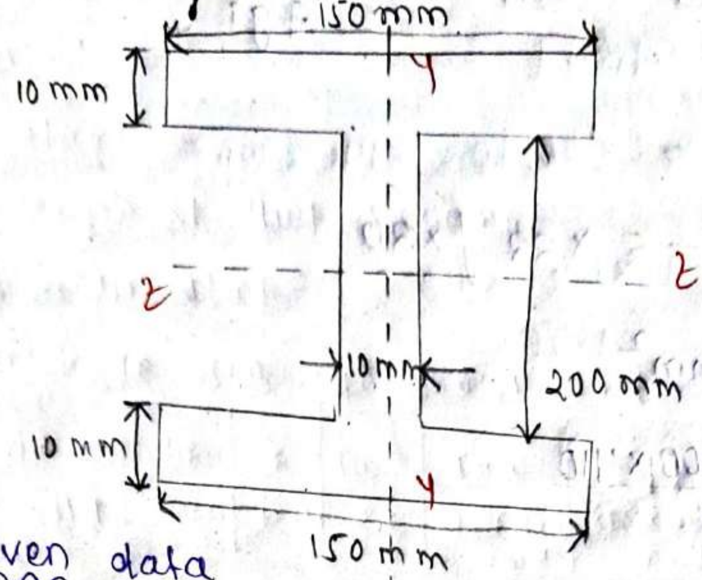
Table - 10 Buckling class of cross-section.

At oned end

Translation → moving in horizontal & vertical

Re-strained → Not moving.

Q) What is slenderness ratio for given I section which acts as the compression member whose ends are fixed. The length of the compression is 3 mt :



Soln :- Given data

Length of member L = 3 mt

The ends are fixed

moment of inertias Izz and Iyy

Formula $\frac{bd^3}{12}$ = $I_{zz} = \frac{150 \times 220^3}{12} - 2 \times \frac{70 \times 200^3}{12}$

$= 133.1 \times 10^6 - 93.33 \times 10^6$

$I_{zz} = 39.77 \times 10^6 \, mm^4$

$\frac{db^3}{12}$ $I_{yy} = \frac{220 \times 150^3}{12} - 2 \times \frac{200 \times 70^3}{12}$

$= 61.78 \times 10^6 - 11.43 \times 10^6$

$I_{yy} = 50.44 \times 10^6 \, mm^4$

Radius of gyration r_{zz} and r_{yy} is, $r_{xx} = \sqrt{\frac{I_{xx}}{A}}$

$A = (150 \times 220) - 2 \times (70 \times 200)$

$= 5000 \, mm^2$

$r_{xx} = \sqrt{\frac{39766666.7}{5000}} = 89.18 \, mm$

$r_{yy} = \sqrt{\frac{I_{yy}}{A}} = \sqrt{\frac{50441666.7}{5000}}$

$= 100.43 \, mm$

Radius of gyration, = smaller of r_{xx} & r_{yy}

= 89.18 mm

Slenderness Ratio

$$\lambda = \frac{K.L}{r} = \frac{0.65\,l}{89.18} = \frac{0.65 \times 300}{89.18}$$

$$= 21.86$$

Q) Determine the design compressive strength of single ISLB 450 at 641 N/m when it is used as coloumn of effective length 4mt. The yield stress of steel is 300 N/mm2

Given data

$f_y = 300\ N/mm^2$

Effective length $KL = 4mt = 4000mm$

ISLB at = 641 N/mt

From steel table ISLB 450 @ 641 N/mt

(h)	A(mm²)	bf	tf	tw	Ixx	Iyy	Iz	Iy
ISLB 450	8314	170	13.4	8.6	27536.1×10^4 mm4	853×10^4 mm4	182	32

As per table No-10

$$\frac{Depth}{width} = \frac{h}{bf} = \frac{450}{170} = 2.65 > 1.2$$

$$tf = 13.4 \leq 40\ mm$$

∴ The buckling of section 'a' is about z-z axis and 'b' about y-y axis.

Effective slenderness Ratio

$$\frac{KL}{r_{zz}} = \frac{4000}{182} = 21.97\ mm$$

$$\frac{KL}{r_{yy}} = \frac{4000}{32} = 125\ mm$$

Design compressive stress (fcd)

Table No-9(a)

$f_y = 300\ N/mm^2$

$21.97 \begin{cases} 20 \longrightarrow 270\ MPa \\ 30 \longrightarrow 262 \end{cases}$

$$fcd = 270 - \left(\frac{270-262}{30-20}\right)(21.97-20)$$

$$= 268.42\ N/mm^2$$

Design compressive stress

$Pd = Ae \times fcd$

$= 8314 \times$ (least of stress in zz and yy axis)

$= 8314 \times 89.58$

Q) Determine the design axially loaded & capacity of the column ISHB 300 at 577 N/mt if the length of the column is 3mt ~~mt~~ and both end are flange take $f_y = 250\ N/mm^2$

Given data

$f_y = 250\ N/mm^2$

Length $KL = 3mt = 3000\ mm$

ISHB at $= 577\ N/mt$

From steel table ISHB 300@ 577 N/mt

	$A\ mm^2$	b_f	t_f	t_w	I_{xx}	I_{yy}	r_{zz}	r_{yy}
ISHB 300	7485	250	10.6	7.6	12545.2×10^4	2193.6×10^4	129.5	54.1

As per table No-10

$\frac{h}{b_f} = \frac{300}{250}$ 1.2 $\leq$ 1.2

$t_f = 10.6 <$ ~~40 mm~~ 100mm

∴ The buckling of section 'b' is about z-z axis and 'c' about y-y axis

Effective slenderness ratio

$f_{cd} = \frac{KL}{r_{zz}} = \frac{3000}{129.5} = 23.16$

$f_{cd} = \frac{KL}{r_{yy}} = \frac{3000}{54.1} = 55.45$

Design compressive stress (f_{cd})

Table No-9(a)

$f_y = 250\ N/mm^2$

$20 \longrightarrow 226$ Mpa

$30 \longrightarrow 220$

$f_{cd} = 226 - \left(\frac{226-220}{30-20}\right)(23.16 - 20)$

$f_{cd} = 224.104\ N/mm^2$

Design compressive stress

$P_d = A_e \times f_{cd}$

$= 7485 \times$ (least of stress in zz and yy axis)

$= 7485 \times 224.104$

$= 1677418.44$ N~~/mm~~

$= 1677.418$ kN (Ans)

Q) A tension member consist of a flat plate 100mm x 8mm which is connected to a gusset plate 10mm thick by 2 Nos of 16mm dia bolts is steel of grade Fe 410 & bearing bolts of grade 4.6 are used in the workshop. Determine the strength of plate against yielding, Rupture block shear, Also determine the maximum load the joint can carry safety ?

Given data :-

$F_e = 410 \; N/mm^2$

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

Breadth = 100mm

Thickness = 8 mm

16 mm dia bolt $d = 16mm$

Bolt hole dia $d_h = 16 + 2 = 18mm$

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

Strength of plate due to yielding

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

$\gamma_{m0} = 1.10$ [From IS code 800 : 2007 table no-5 pg no-30]

$A_g = B \times T$

$= 100 \times 8 = 800 \; mm^2$

$$T_{dg} = \frac{800 \times 250}{1.10}$$

$= 181818.18 \; N = 181.81 \; kN$

design strength due to rupture

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

$$A_n = \left[b - nd_n + \sum_i \frac{\cancel{p_{si}^2}^{\,0}}{4g_i}\right]t$$

$= [b - nd_n]t$

$= [100 - 2 \times 18] \times 8$

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

$$= \frac{0.9 \times 512 \times 410}{1.25}$$

$$= 151142.4 \text{ N}$$

$$= 151.14 \text{ kN}$$

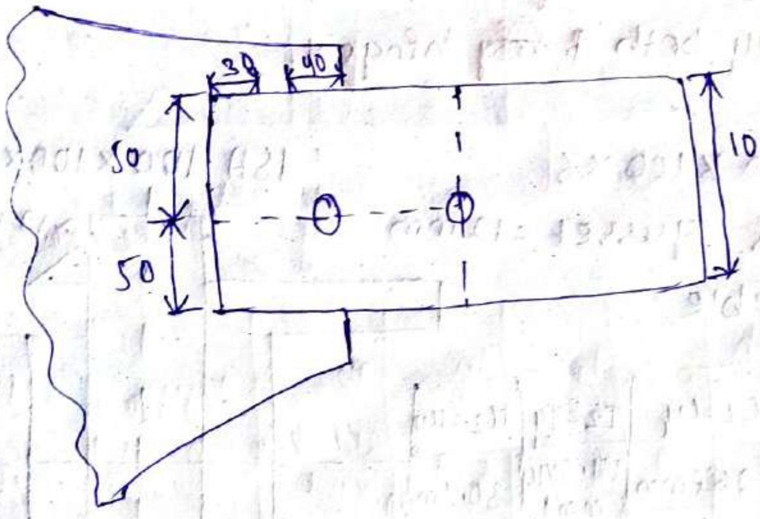
Design strength due to block shear

$$T_{db} = \left[\frac{A_{vg} f_y}{(\sqrt{3} \gamma_{m0})} + \frac{0.9 A_{tn} f_u}{\gamma_{m1}}\right]$$

$$= \left[\frac{0.9 \, A_{vn} f_y}{\sqrt{3} \, \gamma_{m1}} + \frac{A_{tg} f_y}{\gamma_{m0}}\right]$$

A_{vg} :- Minimum gross area, in shear

A_{vn} :- min^m Net area in shear



$A_{vg} = (40 + 30) \times 8 = 560 \text{ mm}^2$

$A_{vn} = (70 - 18 - \frac{18}{2}) \times 8 = 344 \text{ mm}^2$

A_{tg} = min^m gross area in tension

A_{tn} = min^m Net area in tension

$A_{tg} = 50 \times 8 = 400 \text{ mm}^2$

$A_{tn} = (50 - \frac{1}{2} \times 18) \times 8 = 328 \text{ mm}^2$

$T_{db} = 170306.5434 \text{ N} = 170.30 \text{ kN}$

or

$$\left[\frac{0.9 \, A_{vn} f_u}{\sqrt{3} \, \gamma_{m1}} + \frac{A_{tg} \times f_y}{\gamma_{m0}}\right]$$

$$\left[\frac{0.9 \times 344 \times 410}{\sqrt{3} \times 1.25} + \frac{400 \times 250}{1.10}\right]$$

$= 149538.3179\ N = 149.53\ kN$

<u>Strength of plate</u>

= min^m (yielding, Rupture, block shear)

= yielding :- 181.81 kN. Rupture :- 151.14 kN

Block shear :- (149.54 kN)

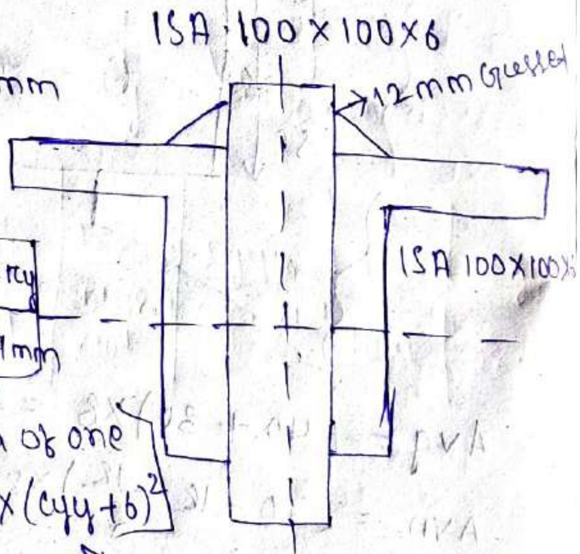
= 149.54 kN (Ans)

Q) In a truss of struct of 3mt long consist of two angle ISA = 100×100×6 mm. Find the slenderness ratio of the member. If the angles are connected on both of 12mm gusset by both of ~~body~~ hinged.

Two ISA 100×100×6

Thickness of gusset = 12 mm

From steel table



ISA	t	A	$C_z = C_y$	$I_z = I_y$	$r_z = r_y$
100×100	6.0mm	1167 mm²	26.7mm	111.3×10⁴ mm⁴	30.9mm

$$I_{yy} = 2\left[I_{yy} \text{ of one angle} + \text{Area of one angle} \times (c_{yy}+6)^2\right]$$

$$= 2\left[111.3\times10^4 + 1167 \times (26.7+6)^2\right]$$

$= 4721723\ mm^4$

$$r_{yy} = \sqrt{\frac{I_{yy}}{A}} = \sqrt{\frac{4721723}{2\times1167}}$$

$= 44.98\ mm$

∴ Radius of gyration is the least of r_{zz} & r_{yy}

Slenderness ratio

$$\lambda = \frac{KL}{r} \quad r = 30.90\ mm$$

$= \frac{l}{r} = \frac{3000}{30}$

$= 97.1$ (Ans)

Design strength

Common hot rolled & built-up steel member used for carrying axial compression usal fail by flexural buckling. The buckling strength of these members is affected by residual stress, initial bow & ~~accidentes~~ by accidental eccentricities of load. To account for all these factors the strength of members subjected to axial compression is defined by buckling class a, b, c ~~&~~ or d as given table ~~by~~ 7

The design compressive strength Pd, of a member is given by

$$P < P_d$$

where $P_d = A_e f_{cd}$

where, A_e = effective sectional area as defined 7.3.2 &

f_{cd} : design compressive stress obtained as per 7.1.2.1

The design compressive stress f_{cd} of axially loaded compression member shall be calculated using the following eqn

$$f_{cd} = \frac{f_y / \gamma_{m0}}{\phi + [\phi^2 - \lambda^2]^{0.5}} = f_y / \gamma_{m0} \leq f_y / \gamma_{m0}$$

where,

$$\phi = 0.5 [1 + \alpha(\lambda - 0.2) + \lambda^2]$$

λ = non-dimensional effective slenderness ratio

$$= \sqrt{f_f / f_{cc}} = \sqrt{f_y \left(\frac{KL}{r}\right)^2 / \pi^2 E}$$

f_{cc} = Euler buckling stress $= \frac{\pi^2 E}{(KL/r)^2}$

where,

KL/r = effective slenderness ratio or ratio of effective length KL to appropriate radius of gyration, r

α = imperfection factor given in table 7

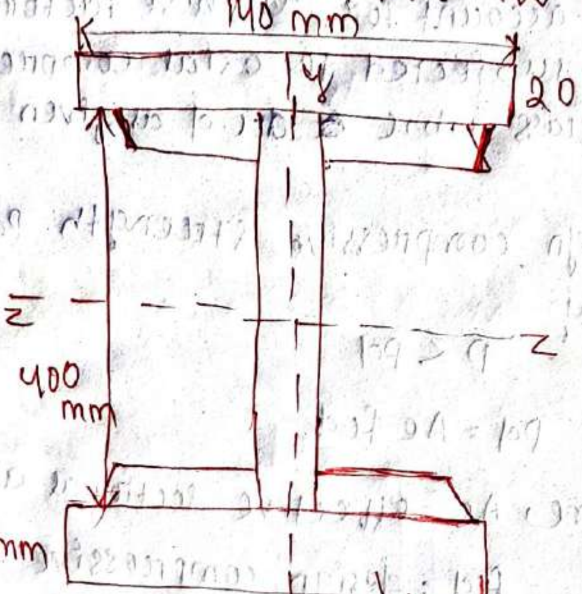
λ = stress reduction factor for different buckling class slenderness ration & yield stress.

$$\frac{1}{\left[\phi + (\phi^2 - \lambda^2)^{0.5}\right]}$$

λ_{mo} = partial safety factor for material strength

Q) Determine the design strength of the column section given in the figure. If its actual length is 4.5 m & its one end may be fixed & other end is hinged. Grade of steel is Fe415

Built up section



Steel table

	A (mm²)	Depth	bf(mm)	tf (mm)	tw (mm)	Izz(mm⁴)	Iyy(mm⁴)	Iz(mm)	Iy(mm)
ISMB	7846	400	140	16.0	8.9	20458.4 X10⁴	622.1X10⁴	161.5	28.2

$$\therefore \frac{h}{bf} = \frac{400}{140} = 2.85 > 1.2$$

$$\therefore t_f = 16 \leq 40\ mm$$

∴ Hence it belongs to buckling class 'a' about z-z axis & buckling class 'b' about y-y axis

Parallel axis theorem

$$I_{zz} = I_{AB} + A \times (\bar{x})^2$$

$$I_{zz} = 20458.4 \times 10^4 + 2 \times 300 \times 20 + (200+10)^2$$

$$= 733784000\ mm^4$$

$$I_{yy} = 622.1 \times 10^4 + 2 \times \frac{20 \times 300^3}{12} \left[\frac{db^3}{12}\right]$$

$$= 96221000\ mm^4$$

$I_{zz} > I_{yy}$

$$\therefore r_{yy} = \sqrt{\frac{I_{yy}}{A}} = \sqrt{\frac{96221000}{19846}} = 69.63 \text{ mm}$$

$$A_e = 2 \times 300 \times 20 + 7846 = 19846$$

Slenderness ratio $= \frac{Kl}{r} = \frac{0.8l}{r}$ (Table no - 11 Pg no - 45)

$$= \frac{0.8 \times 4500}{69.63} = 51.70$$

As per table no - 9b

$$\begin{array}{ll} 50 & - 194 \\ 51.70 \\ 60 & - 181 \end{array}$$

$$f_{cd} = 194 - \left(\frac{194 - 181}{60 - 50}\right)(51.70 - 50)$$

$$= 191.7 \text{ N/mm}^2$$

Design compressive strength

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

$$= 19846 \times 191.7 = 3804478.2 \text{ N}$$

$$= 3804.47 \text{ KN}$$

Q) Design a steel coloumn ~~compression member or strut~~ using a single rolled I-section to carry an axial load of 800 KN. Both end of the column are restrained against rotation and translation, The actual length of the column is 8 mt & the yield stress (fy) of steel is 280 mpa.

Given data

Axial load = 800KN

Length = 8

Both end fixed

$f_y = 280$ mpa

Sol^n:- Design load = f.o.s × axial load

$$= 1.5 \times 800 = 1200 \text{ KN}$$

Assume, design compressive stress $f_{cd} = 0.45 \times f_y$

$$= 0.45 \times 280$$

$$= 126 \text{ N/mm}^2$$

1) Design a simply supported beam of effective span 1 mt carrying a factored ~~load~~. ~~load~~ concentrated load of 360 kN mid span?

$f_y = 250$ mpa

Exam

Given data

$l_{eff} = 1$ mt

Factored load = 360 KN

Factored bending moment $= \frac{Wl}{4} = \frac{360 \times 1}{4} = 90$ KN/mt

$= 90 \times 10^6$ N/mm

Plastic modulus, $M = \frac{Z_p \times f_y}{\gamma_{m0}}$

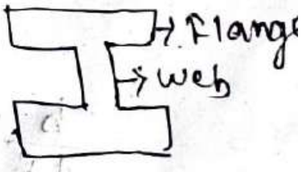
$Z_p = \frac{M \times \gamma_{m0}}{f_y}$ $\gamma_{m0} = 1.1$

Z_p = unit (mm^3)

$Z_p = \frac{(90 \times 10^6) \times 1.1}{250}$

$= 39600$ mm3

$= 396 \times 10^3$ mm^3



Select section ISMB 250, $Z_p = 410.5 \times 10^3$ mm^3

ISMB	weight(N) per mt	A	D	b_f	t_f	t_w	Plastic modulus (Z_{xx})
250	365.9	4755	250	125	12.5	6.9	410.5

∴ Depth of web, $d = D - 2 \times t_f$

$= 250 - 2 \times 12.5$

$= 225$ mm

∴ Self weight of beam = 0.365 KN/mt

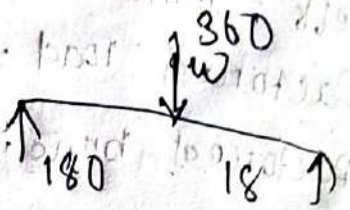
∴ Factored weight of beam = $0.365 \times 1.5 = 0.548$ kN/mt

∴ Additional factored moment due to self weight

$= \frac{Wl^2}{8} = \frac{0.548 \times 1^2}{8} = 0.069$ KN/mt

Total factored moment, $M = 90 + 0.069$
$= 90.069$ kN/mt

$\therefore$ Factored shear force due to self wt. $= \frac{Wl}{4} = \frac{0.548 \times 1}{4} = 0.137$ kN



Total factored

$SF = \frac{360}{2} + 0.137 = 180.187$ kN

<u>Sectional classification</u>

$$\varepsilon = \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{250}} = 1$$

$$\frac{b}{t_f} = \frac{125}{12.5} = 10 \quad \therefore \frac{b}{t_f} = 10\varepsilon < 10.5\varepsilon \text{ (class-2)}$$

$$\frac{d}{t_w} = \frac{225}{6.9} = 32.60 \quad \therefore \frac{d}{t_w} = 32.60\varepsilon < 105\varepsilon \text{ (class-2)}$$

$\therefore$ It is classified as compact (class-2) section.

<u>Shear capacity of the section</u>

$$V_d = \frac{f_y}{\sqrt{3}} \times \frac{1}{\gamma_{m0}} \times h \times t_w$$

$\gamma_{m0} = 1.10$

$$= \frac{250}{\sqrt{3}} \times \frac{1}{1.1} \times 250 \times 6.9 = 226347.5 \text{ N}$$
$$= 226.35 \text{ kN}$$

$\therefore$ Section is adequate to resist shear

$0.6 V_d = 0.6 \times 226.35$
$= 135.81$ kN

moment capacity of section

$M_d = M_{dv}$

$$= \frac{Z_e \times f_y}{\gamma_{m0}} = \frac{410.5 \times 10^3 \times 250}{1.1} = 93295454.55 \text{ N mm}$$

$= 93.29$ KN.mt

> 90.069 KN.mt

$\therefore$ This section is adequate to resist moment

Check for deflection

$$\delta_{maxm} = \frac{Wl^3}{48EI}$$

$$= \frac{(360 \times 10^3) \times (1000)^3}{48 \times (2 \times 10^5) \times (5131.6 \times 10^4)}$$

$= 0.731$ mm $< \frac{1000}{250} = 4$ (ok)

deflection $= \frac{L}{d} =$