COMMON STEEL STRUCTURES:

1. Roof Trusses
2. Crane or gantry girder
3. Stanchion
4. Transmission towers (space truss)
5. Plate girder
6. Water tanks, Chimneys etc.,

ADVANTAGES OF STEEL:-

- It has high strength per unit mass
- The size of steel elements are lesser resulting in space savings an aesthetic view
- It has assured quality and high durability
- Speed of Construction
- It can be strengthened any later time.
- Easy dismantling of steel structures is possible (Mainly by using bolted connection)'
- The material is reusable
- If the joints are taken care of, it has good resistance against water and gas.

DISADVANTAGES OF STEELN NSTRUCTURES:-

- It is susceptible to corrosion
- Maintenance cost is significant (frequent painting is read to prevent corrosion)
- Steel members are costly (Initial cost)

TYPES OF STEEL:-

- Steel is an alloy, of iron & carbon
- The small percentage of manganese, sulpher, phosphorous, copper & nickel for added to steel to improve the properties of structural steel.
- Increasing the qty of carbon & magnese imparts of high tensile strength but lower ductility.
- Welding is easier in case of ductile steel and ductile steel performs better in case of lateral loads.
- Chrome & nickel impart corrosion resistance property to steel.
- It also resist high temperature.

PROPERTIES OF STRUCTURAL STEEL:-

- The structural steel is classified as mild steel and high tensile steel.
- Standard quality steel (IS 226-1975) is classified under grade E250 & E350 where, 250 & 350 are the yield stress of steel.
- High tensile steel (Weldable quality Steel) is designated as E410 & E450. where 410&450 are tensile stress of steel as given in IS2062.

1. PHYSICAL PROPERTIES:-

Irrespective of the grade the physical properties of steel is given below (cls 2.2.4 I.S 800-2007)

- Unit mass of steel = $7860 kg/m^3$
- Modulus of elasticity, E = $2 \times 10^5 N/mm^2$
- Poissons ratio, μ = 0.3
- Co-eff of thermal expansion α = $12 \times 10^{-6} /°c$
- Modulus of rigidity, G = $0.76 \times 10^5 N/mm^2$

2. MECHANICAL PROPERTIES:-

The mechanical properties of structural steel is w.r.to the yield stress & ultimate stress of the steel sections conforming to IS 2062.
Ex: E250 grade of steel - yield stress 250 N/mm^2
Ultimate stress 410 N/mm^2

The mechanical properties of all the grades a given in table 1:1 of IS 800-2007

WORKING STRESS METHOD OF DESIGN:-

- Previously working stress method was used for steel design as per I.S.800 1984, here F.O.S is applied only for a material and no F.O.S for load.
- Since more economy in design was regd and to take care of serviceability criteria (deflection and cracks) limit start design was introduced IS 800-2007
- The F.O.S for material supplied is applied in the permissible stress (material)

For the various internal forces as given below.

1. Permissible stress in axial tension.
2. Section II of I.S 800-2007 comprises of the W.S.M of design.
3. The code aspects the use of W.S.M of design in places, where L.S.M of design cannot be used as per clause 5.1.2 the design requirements of for any structure is given.

LIMIT STATE METHOD OF DESIGN:- [CIS 5.2 I.S 800-2007]

- Limit state are the states beyond which the structures on longer satisfies state of strength.
- Limit state of serviceability.

LIMIT STATE OF STRENGTH:-

This limit state is prescribe to avoid the collapse of the structure which may endanger the safety of life and property and includes.

1. Loss of equilibrium of structure
2. Loss of stability of structure
3. Failure by excessive deformation
4. Fracture due to fatigue
5. Brittle fracture, these are maintain.

The limit state of strength found for members in tension and compression, flexure and shear.

LIMIT STATE OF SERVICEABILITY:-

The limit state of serviceability includes

1. The deformation & deflection adversely affecting the appearance (or) effective use of the structure (or) cause improper functioning of equipments (or) services (or) causing damage to finishes.
2. Vibrations in structures (or) any part of its component limiting its functioned effectiveness.
3. Repairable damage (or) crack due to fatigue.
4. Corrosion
5. Fire

LOADS ON STRUCTURES:-

1. **DEAD LOAD:** [I.S. 875 Part-I]

Dead loads are the permanent loads acting on the structure including the self wt of the section.

2. **LIVE LOAD:** [I.S. 875 Part-II]

It is an imposed load in structure due to people, furniture, movable objects etc.

Based on utility of the structure the values are given in [I.S 875 Part-II]

Example:-

For Residential Buildings – 2 KN/m^2

For Commercial Buildings – 3 KN/m^2

3. Wind Load [I.S 875 Part-III]
4. Snow Load [I.S 875 Part-IV]
5. Seismic Load (or) Earth quake Load [I.S 1893-2002]
6. Accidental Loads
7. Errection Loads
8. Crane Loads

CHARACTERISTICS OF LOAD:-

It is designed as the action of the load which are not expected more than five percentage probability during the life of the structure.

1. Partial safety factor for loads for limit state 'γf' is given in table 4 [I.S 800-2007]
2. Partial safety factor for material is given in table 5 [I.S 800-2007]

DESIGN STRENGTH:-

The uncertainties to be considered in the strength value for design for

1. Possibilities of deviation of material strength from the characteristic values.
2. Possibilities of unfavorable varities of member sizes.
3. Possibilities of unfavorable reduction in member strength during fabrication.
4. Uncertainity in calculation of strength.

I.S 800 recommands the reduction is strength of the material based on the partial safety factors for the material as given in table 5 of IS 800-2007

Deflection limity in order ro prevent damage to finishes, deflection check is done for the load combinations with partial.

Safety given in table 4 and the limiting deflection factor given in table 6 IS 800-2007

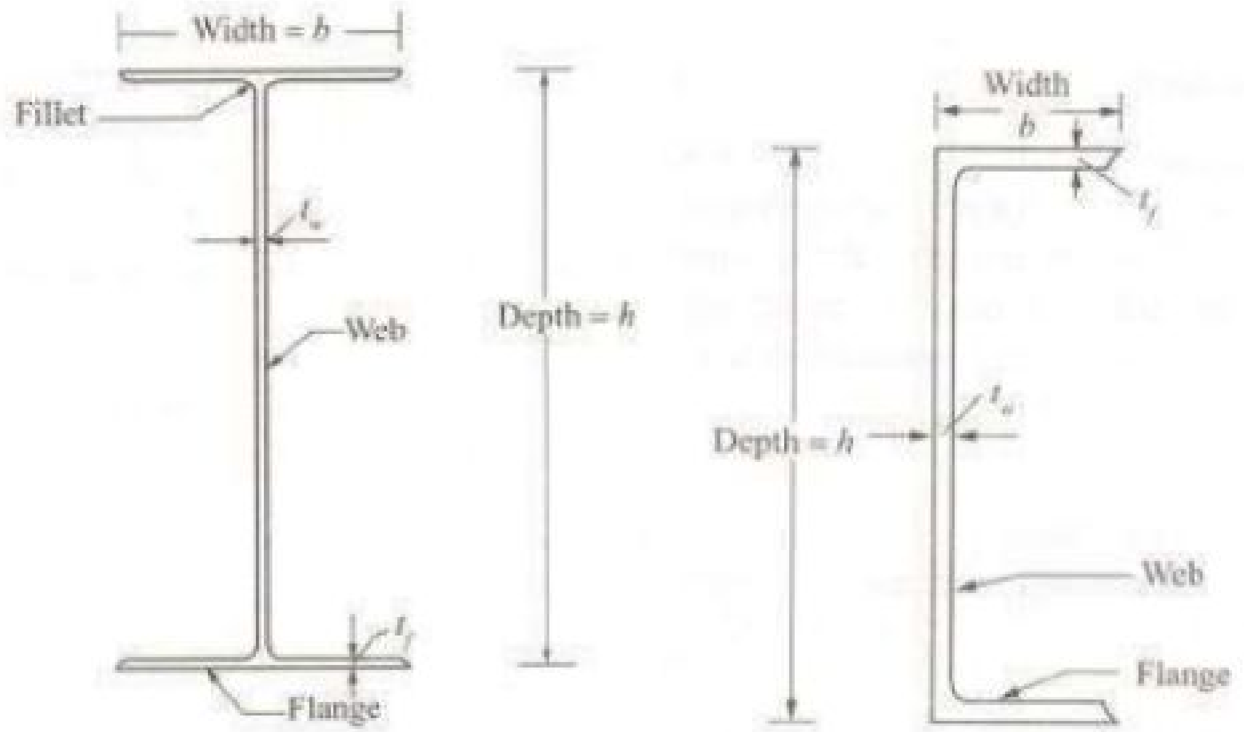
OTHER SERVICEABILITY LIMITS:-

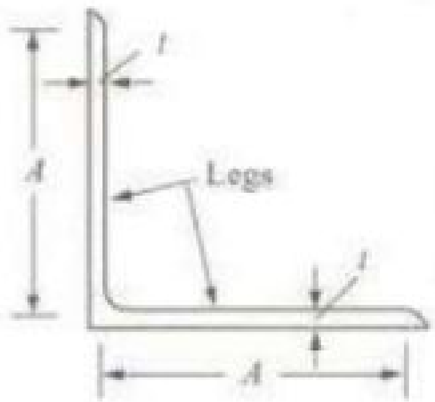
- Vibration Limit
- The flows which are subjected to vibration (supporting machineries) or to be checked for vibration under dynamic loads annex C IS 800-2007 gives the set of guide lines to take care of vibration limits.
- During concentration the following factors affects the durability of steel structure
    1. Environment
    2. Degree of exposure
    3. Shape of the member & structural detail

FIRE RESISTANCE:-

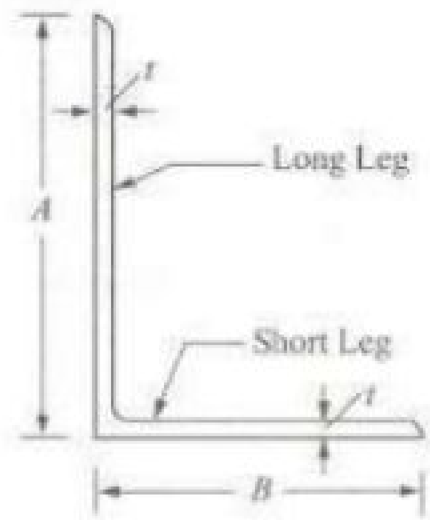
- Fire resistance level [FRL] specified in terms of limit depending upon the purpose for which the structure is used and the time taken to evacuate in case of fire.
- Section 16 of IS 800-2007 deals with fire resistance.
- In addition to the above the stability of structure to be checks due to over turning sliding or uplifts under factored load.
- The structure should also be stiff against sway and fatigue also
- The designer has to ascertain all the limit states are not exceeded.

SOME STRUCTURAL STEEL SECTIONS:

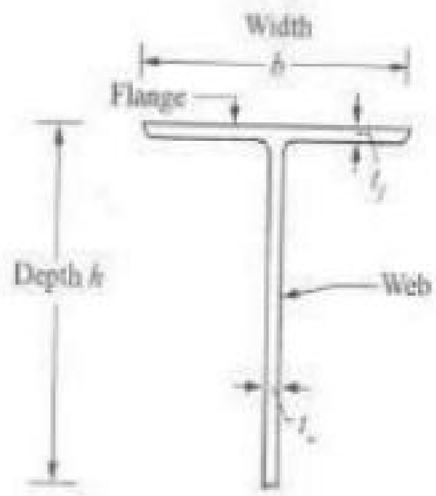




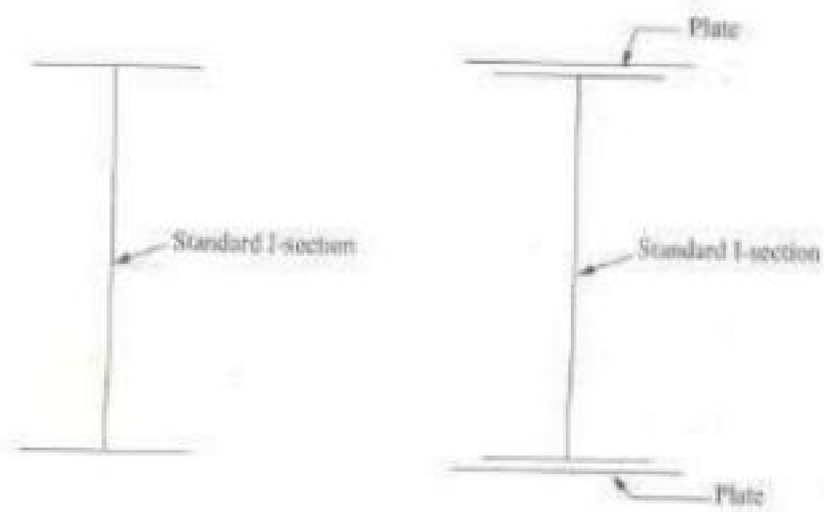
(a) Rolled steel equal angle.



(b) Rolled steel unequal angle.



Rolled steel T-section.

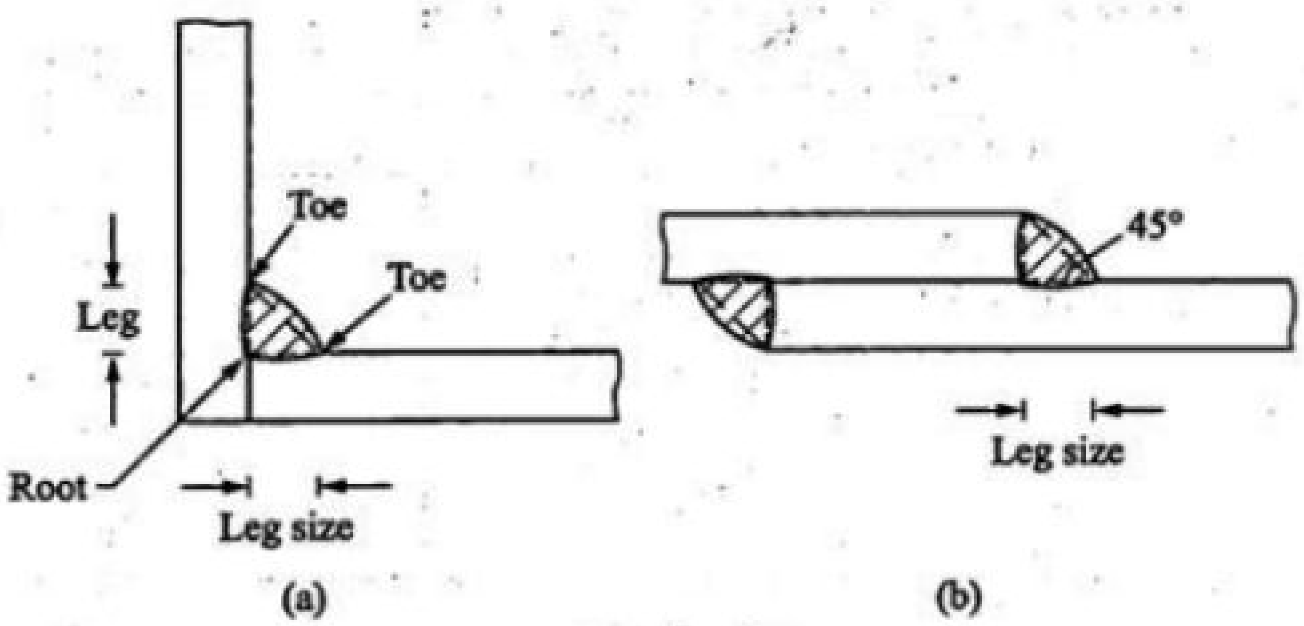


Standard section and built up section.

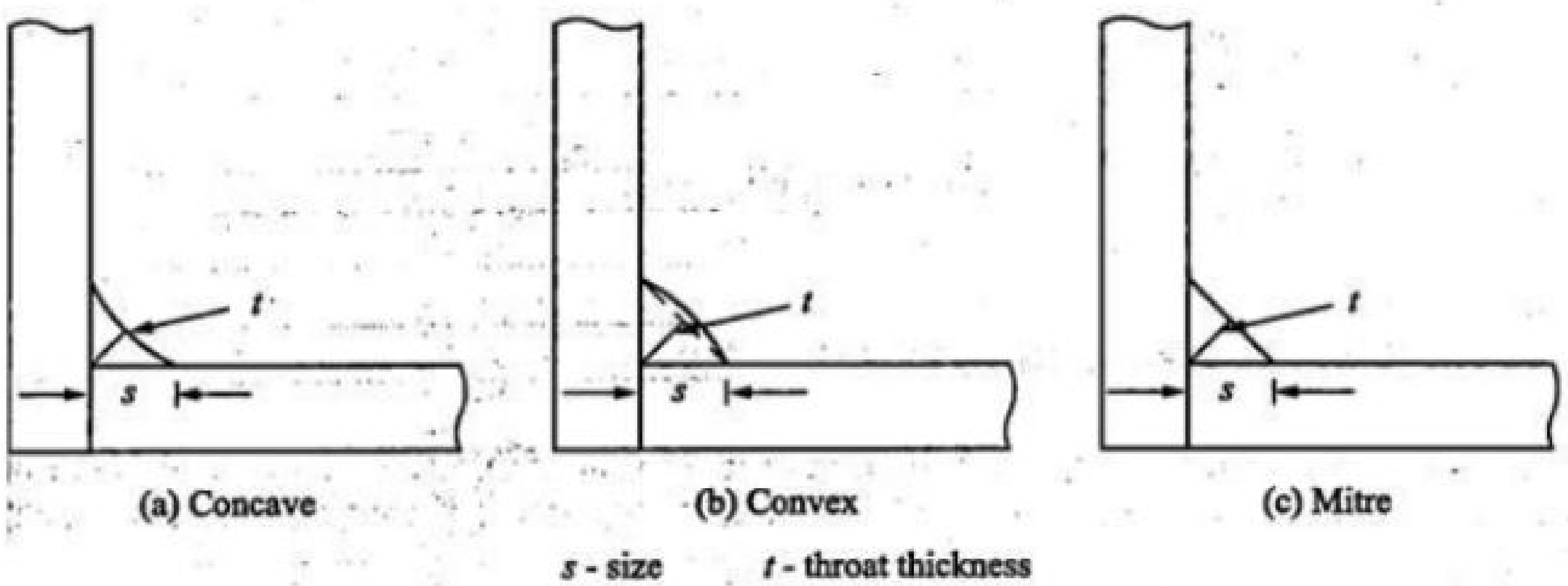
WELDED CONNECTIONS:- [IS 10.5, IS 800-2007]

Welded connections are advantageous in most of the cases, since

(i) Self wt. reduces due to absence of guest plates, connecting angles etc.
(ii) The connection is rigid.
(iii) The process is quicker
(iv) Asthetic appearance is good.
(v) Relatively lesser
(vi) Welded connections are air tight & water tight
(vii) Welded connections are preferable for trusses with circular c/s.



Typical fillet welds.



Types of fillet welds.

DISADVANTAGES OF WELDED CONNECTIONS:-

a) Due to uneven heating & cooling members are likely to distart.
b) There is possibility of brittle fracture at the welded joint.
c) A welded connection fails earlier than a bolted connection, due to fatigue.
d) Inspection of welded its is difficult and expensive.
e) Highly skilled labour is regd. For weld.
f) Proper welding in the field condition is required.

Types of Welds:-

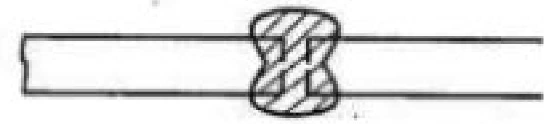
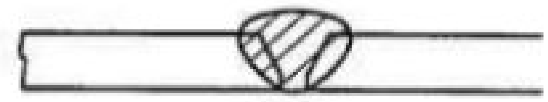
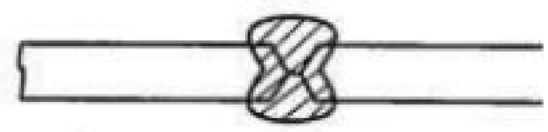
i) Lap weld
ii) Butt weld
iii) Slot weld
iv) Plug weld

(i) Lap weld:-

(ii) Butt weld:-

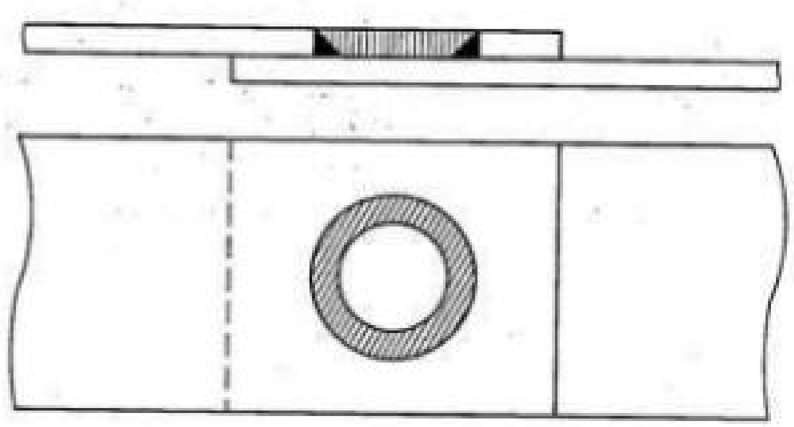
1. Single square butt weld
2. Double square butt weld
3. Single 'V' butt weld
4. Double 'V' butt weld
5. Single 'U' butt weld
6. Double 'U' butt weld
7. 'J' Butt weld

Types of butt weld

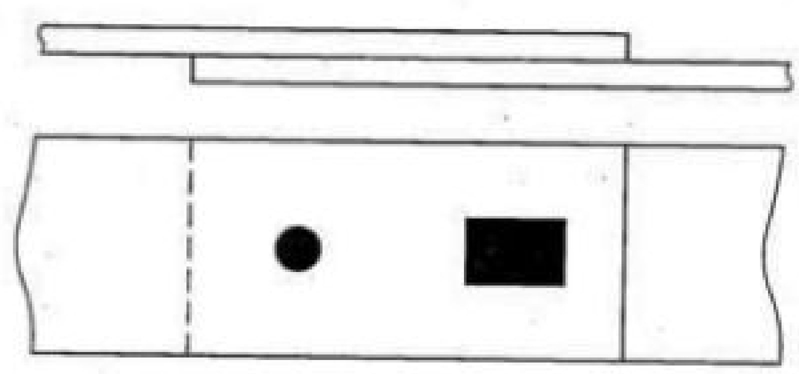
Sl. No.	Type of Butt Weld	Sketch
(a)	Square butt weld, on one side	
(b)	Square butt weld, both sides	
(c)	Single V butt joint	
(d)	Double V-butt joint	
(e)	Single U butt joint	
(f)	Single J-butt joint	
(g)	Single bevel butt joint	

Note: Similarly there can be double U, double J and double bevel butt joints.

(iii) Slot & Plug weld:-

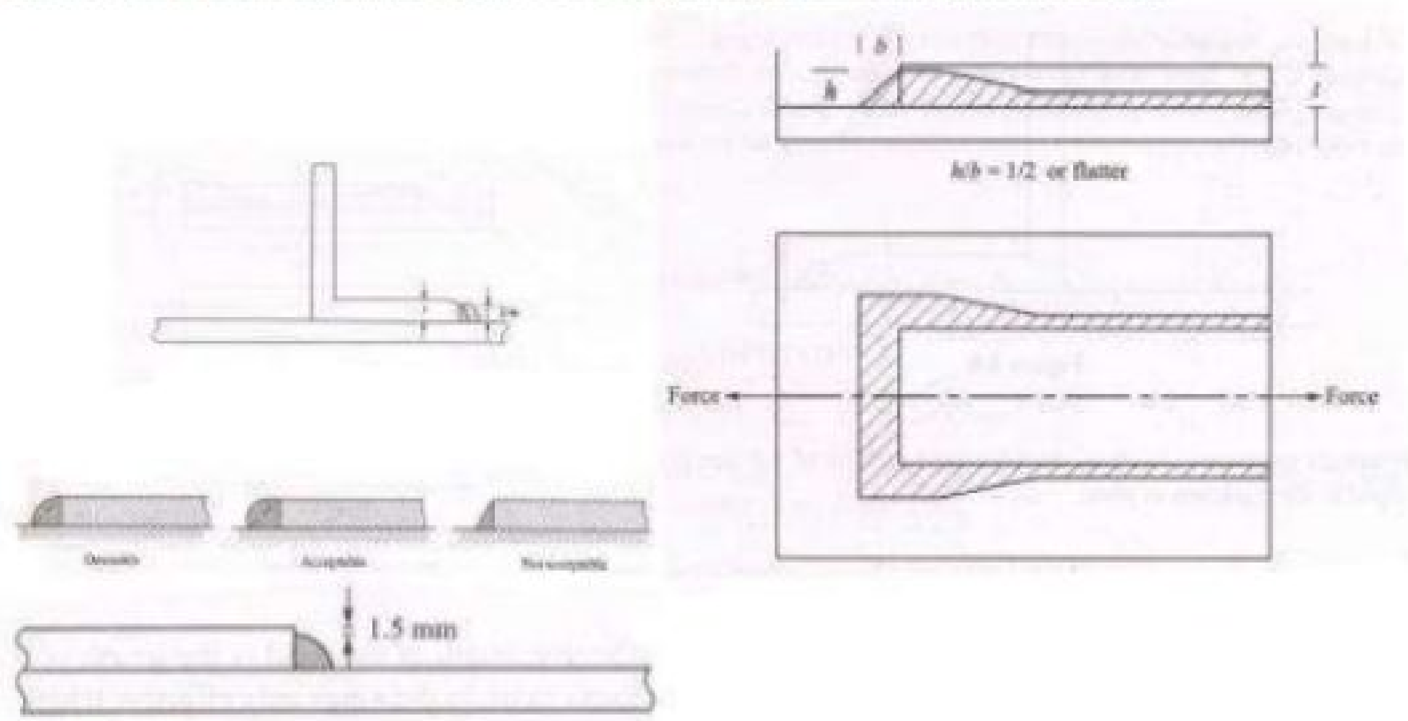


Slot weld.



Plug welds.

I.S. 800-2007 PROVISIONS FOR WELDED CONNECTION:-



1. Butt weld:-
   - The size of weld is specified by effective throat tks.
   - In case of complete penetration butt weld, it is taken as tks of the thinner part jt.
   - Double U & Double V Double J type butt welds are regarded as complete penetration butt welds.
   - For incomplete penetration butt welds, is taken as 5/8t
   - The eff. Length of butt weld is taken as length of full size weld.
   - The mini. Length of butt weld shall be 4 times the size of weld.

2. Filled weld:-

(a) Size of fillet weld:-
   - The size of normal fillet weld is taken as mini weld leg size
   - For deep penetration weld with penetration not less than 2.4mm, the size of weld is mini. leg size + 2.4mm

(b) Mini size of weld is 3mm. for plates of tks 10 to 20mm, min size is 5mm, for 20 to 32mm plates min size is 6mm & greater than 32mm plates min size is 8mm

(c) Eff. Threat tks:-
   - It shall not be less than 3mm and shall not exceed 0.7t [upto 90°] where, t=tks of thinner plate.

(d) Eff. Length:-
   - The eff. Length of the weld is the length of weld for which the specified size and throat tks exist.
   - The welding length provided is equal to the eff. Length t twice the size of weld

$$L = l_{eff} + 2s$$

   - Eff. Length should not be less than 4 times the size of weld.

(e) The min. lap should be 4 times the tks of thinner part jt (or) 40mm whichever is more.

3. Slot & Plug weld:-
   - For slot weld the length of weld is along the perimeter of the cut portion. [Circumference if the cut is circular]
   - For plug weld, the eff. Area is taken as the nominal area of the hole cut in the parent member.

Design stresses in weld:-

1. Butt weld:-

   Butt weld shall be treated as parent metal with tks equal to the throat tks and stresses not exceeding those permitted in parent.

2. For fillet, slot and plug weld:-

   The design strength is based throat area [strength of weld =fwd x lwx t]

   The design strength is given by $f_{wd} = f_{wn} / \gamma_{mw}$

Where,

$f_{wn} = fu/\sqrt{3}$

fu => smaller of the ultimate stress the weld (or) of the parent metal.

NOTE:-

The eff. Throat tks in case the angle varies (or) angle of fusion face varies a modification factor 'k' as given in table IS 800-2007 and cls 10.5.3.2

Each fillet weld normal to the direction force shall be of unequal size with the tks is not less than 0.5t.

The reduction in design stress for long is as per cls 10.5.7.3 IS 800-2007

$$\beta_{lw} = 1.2 - \frac{0.2lj}{150tt} \leq 1.0$$

lj => length of the jt in the direction of force transfer

tt => throat size of the weld

1. A 18mm tk plate is joint to a 16mm plate by 200mm long [effective] butt weld. Determine the strength of the joint, if

   (i) A double 'V' butt joint is provided

   (ii) A single "V' butt joint is provided

Assume the grade Fe410 for the plates and for the welds which are shop welded

Given Data:-

L_e= 200mm

Grade of plate Fe=410

Fu = 410 N/mm^2

Weld:- Shop welded

Sln:-

(i) Double 'V' butt joint:-

Strength of weld = Design stress of weld X Eff. Area
= fwd x lw x t

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

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

For double 'V' butt joint complete penetration of the takes place.

$$f_{wn} = 236.71 N/mm^2$$

$$f_{wd} = \frac{236.71}{1.25}$$

$$f_{wd} = 189.368 N/mm^2$$

Strength of weld = 189.368 x 200 x 16
= 605.977 KN

(ii) Single 'V' butt joint:-

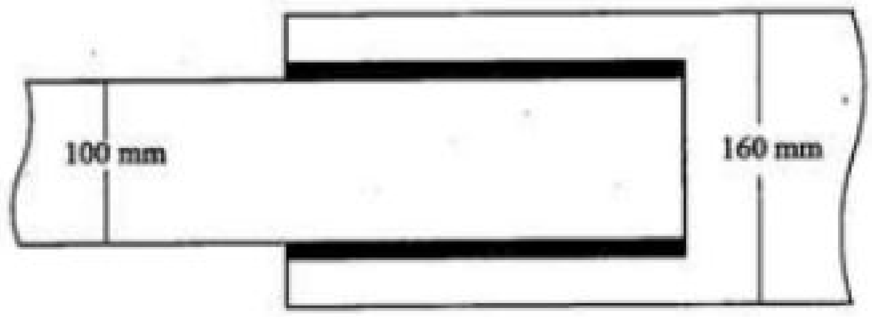
t = 5/8 s [Incomplete Penetration]
= 5/8 x 16
t = 10mm

$$\text{Strength of weld} = \frac{fu/\sqrt{3}}{\gamma_{mw}} \times lw \times t$$

= 189.368 x 200 x 10

Strength of weld = 378.74 KN

2. Design a suitable longitudinal fillet weld to connect the plates as shown in fig. The pull to be transmitted is equal to the full strength of the small plate. Given the plates are 12mm tk, grade of plates is Fe410 and welding is made in the factories.



Given:-

TKS of Plate = 12mm
Grade of Plate Fe410
$fu = 410 N/mm^2$
$fy = 250 N/mm^2$

The strength of the weld is equated to the design strength of the smaller plate.

Design strength of weld = $\frac{fu/\sqrt{3}}{\gamma_{mw}} \times lw \times t$

Mini size of weld = 5mm [from Table-21 pg.No:78]

Maxi size of weld = t_p- 1.5 = 12-1.5 = 10.5mm

Assume the size of weld = 10mm

t = 0.7s

= 0.7 x 10

t = 7mm

Strength of smaller plate [yielding criteria] = $\frac{fuAg}{\gamma_o}$

Where, $\gamma_o = 1.1$

Strength of smaller plate (yielding criteria) = $\frac{250 \times 1200}{1.1}$

= 272.72 KN

Strength of weld = $\frac{fu/\sqrt{3}}{\gamma_{mw}} \times lw \times t$

$272.72 \times 10^3 = \frac{410/\sqrt{3}}{1.25} \times lw \times 7$

$lw = 205.7\ mm \simeq 205\ mm$

Provide an over lap of 105mm.

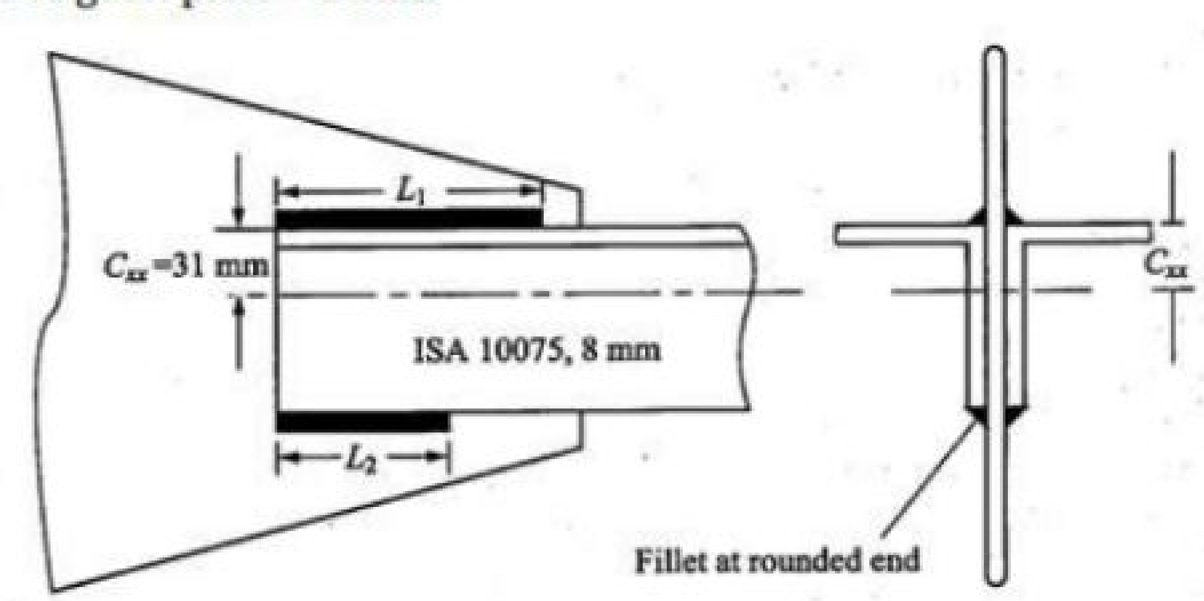
3. A tie member of a roof truss consist of 2Nos of ISA 100x75x8mm. The angles are connected to either side of a 10mm tk guset plate and the member is subjected to a working pull of 300KN. Design the welded connection. Assume the connections are made in the shop.

Given Data:-

Working load = 300KN

2 ISA 100x75x8mm

Tks of guset plate = 10mm



Sln:-

Factored load = 1.5x300

= 450 KN

Each ISA 100x75x8mm takes 450/2 = 225 KN

Min. size of weld = 3mm [From table-21 IS 800-2007]

Max. size of weld = 8-1.5

= 6.5mm

Also, max. size of weld (rounded edger) = 3/4 x 8 = 6mm

Throat tks, t = 0.7 x S [∵ Angle of fusion = 90^0]

= 0.7 x 6

t = 4.2mm

Strength of weld = Design stress of weld x Eff. Area

$$= \frac{fu/\sqrt{3}}{\gamma_{mw}} \times lw \times t$$

$$225 \times 10^3 = \frac{410/\sqrt{3}}{1.25} \times lw \times 4.2$$

$$\because lw = 282.89\,mm \simeq 283\,mm$$

Since the C.G of angle section does not lie at the centre of the connected leg, the weld length at top & bottom need to be such that the C.G of weld.

C.G of angle ISA 100x75x8 = 31mm from the outstanding leg.

<u>To find C.G of weld:-</u>

Let L1 = length of weld @ top

L2 = length of weld @ bottom

∴ For the C.G of the weld to lie at 31mm from the outstanding leg

$$\therefore L_1 \times 31 = L_2(100-31)$$

$$L_1 = 2.23 l_2$$

$$L_1 \times L_2 = 283$$

$$2.23 l_2 + l_2 = 283$$

$$3.23 L_2 = 283$$

L_2 = 87.62mm $¿ 90\,mm$

$L_1 = 195.39\,mm \simeq 200\,mm$

Provide 200mm length of weld @ the top and 90mm length of weld @ the bottom

∴ The min. over lap length is required 200mm

<u>NOTE:-</u>

In case the length of weld is limited, (length of overlap) end fillet weld can be provided which should also satisfy the condition C.G of weld = C.G of member.

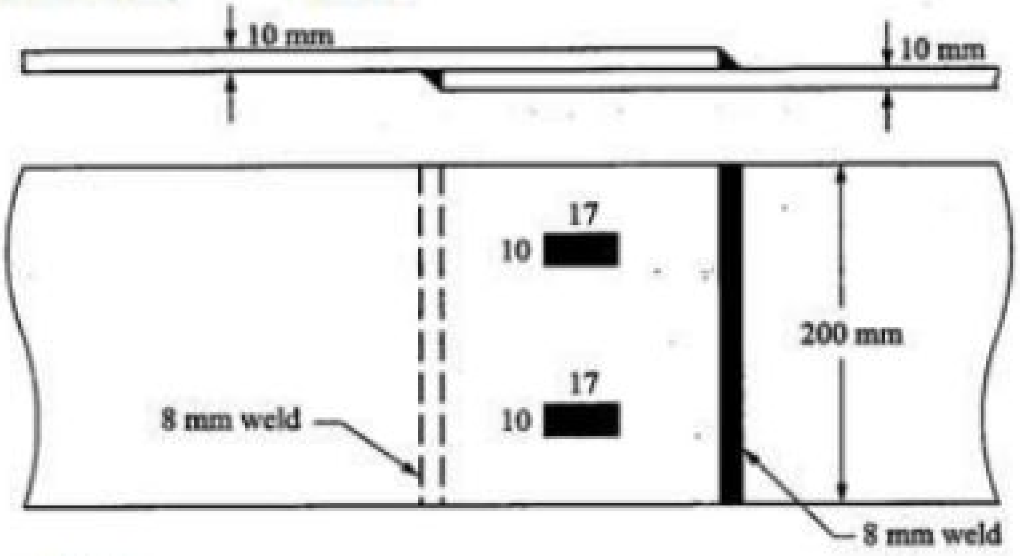
4. Design the welded connection to connect 2 plates of width 200mm & tks 10mm for 100% efficiency.

Given:-

Width of plate = 200mm

Tks of plate = 10mm

Efficiency = 100%



Sln:-

1. <u>Strength of the solid plate:-</u>

$$= \frac{f_y A_g}{\gamma_0}$$

$$= \frac{250\times200\times10}{1.1}$$

$= 454.5$ KN

Mini. Size of weld = 3mm

Maxi. Size of weld = 10 -1.5=8.5mm

Assume size of weld as 8mm 78.5mm

Strength of the weld = Design stress of weld x Eff. Area

$$= \frac{f_u/\sqrt{3}}{\gamma_{mw}}\times l_w \times t$$

$$454.5\times10^3 = \frac{410/\sqrt{3}}{1.25}\times l_w\times0.7\times8$$

lw = 428.6mm

Total length available for weld l = 200+200

l = 400mm

Eff. Length available for weld, lew = l-2.5 x 2

= 400-2x8x2

lew = 368mm

To find strength of weld for 368mm:-

∴ End filled weld is provided for left = 368mm

∴ Design strength of weld for end fillet

$$= \frac{fu/\sqrt{3}}{\gamma_{mw}} \times lw \times t$$

$$= \frac{410/\sqrt{3}}{1.25} \times 368 \times 5.6$$

Design strength of weld for end filled = 390.2KN

Strength of weld reqd. = 454.5-390.2 = 64.3KN

Additional weld is reqd. for this additional weld strength. Here slot weld or plug weld may be provided.

Provided plug weld, Area of plug weld read is,

$$\text{Area of plug weld reqd} = \frac{AdditionalStrengthreqd.}{DesignStressofweld}$$

$$= \frac{64.3 \times 10^3}{\frac{410/\sqrt{3}}{1.25}}$$

$$Aw = 339.5\,mm^2$$

∴ Provide one side of 10mm with 2 rectangular plug welds.

le of oneside = 10mm

∴ other side = 2(10)x = 339.54

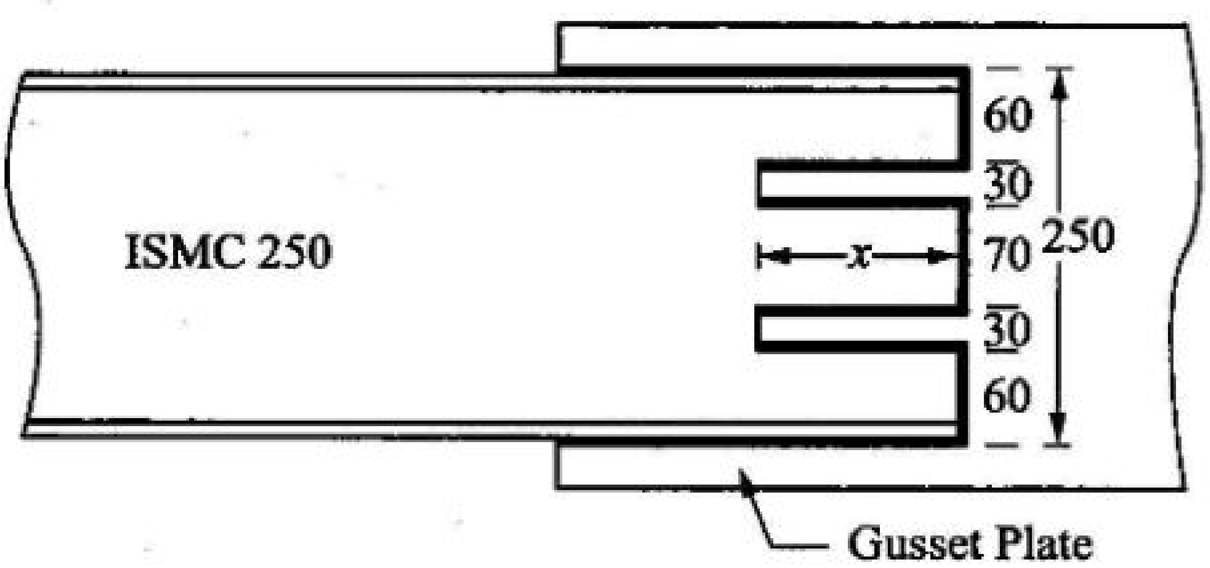
∴ x = 16.97mm

The channels are connected on either side a 12mm tk gusset plate. Design the welded joint to develop full strength of the tie member. The overlap is limited to 400mm.

Given Data:-

Tks of the plate = 12mm

Tie member = IS MC 250(2Nos)



Sln:-

ISMC 250 – Properties:-

$A = 3867mm^2$

$t_f = 14.1mm$

$t_w = 7.1mm$

<u>Strength of solid Plate:-</u>

Strength of solid plate [channel] = $\frac{f_y A_g}{\gamma_o}$

$= \frac{250 \times 3867}{1.1}$

= 878.86KN

∴ Strength of weld read = 878.86KN

Mini size of weld = 3mm [from table-21]

Maxi size of weld = 7.1-1.5=5.6mm

∴ Provide size of weld S = 4mm

∴ Throat tks, t = 0.7 x S

= 0.7 x 4

t = 2.8mm

Strength of weld = $\frac{fu/\sqrt{3}}{\gamma_{mw}} \times lw \times t$

$878.86 \times 10^3 = \frac{410/\sqrt{3}}{1.25} \times lw \times 2.8$

lw = 1657.48mm

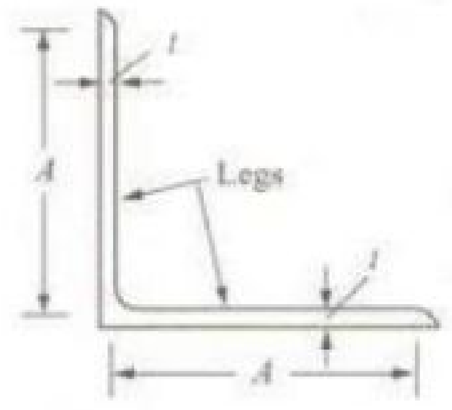
The available length along sides & end = 400+250+400 = 1050mm

[Since overlap is limited to 400mm]

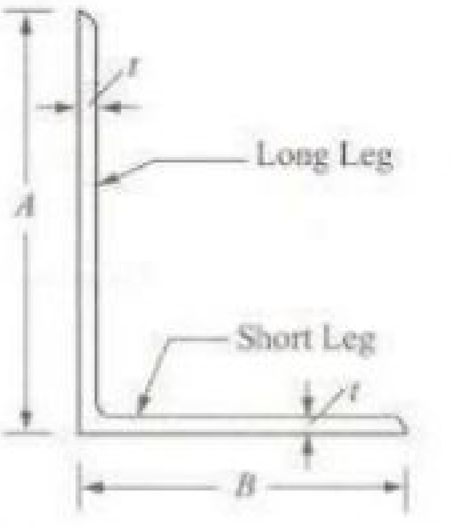
[Either plug weld or slot weld can be provided]

Assuming 2 nos of 30mm wide to be provided along the end of the channel at equal spacing.

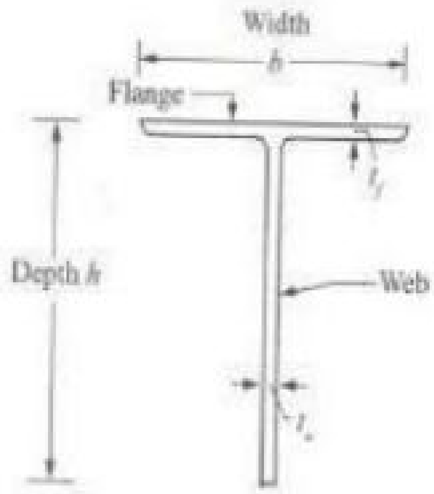
∴ Reqd length of slot = $\frac{1657-1050+2\times4}{4}$



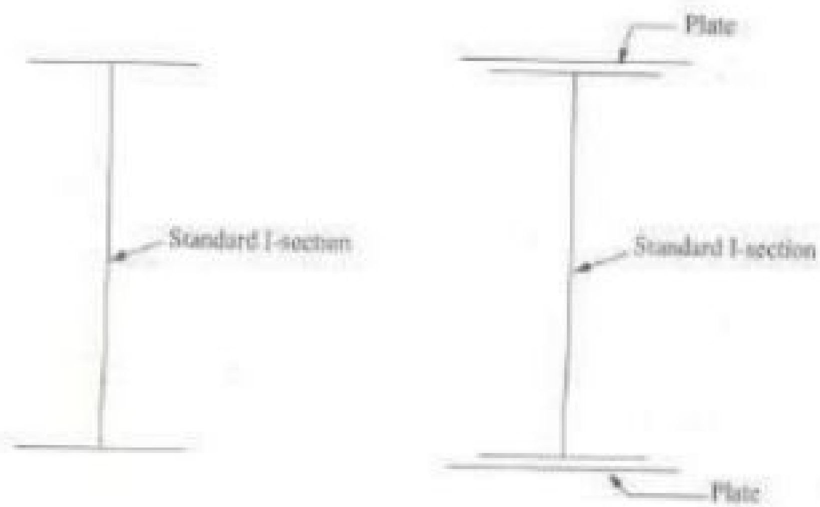
(a) Rolled steel equal angle.



(b) Rolled steel unequal angle.



Rolled steel T-section.

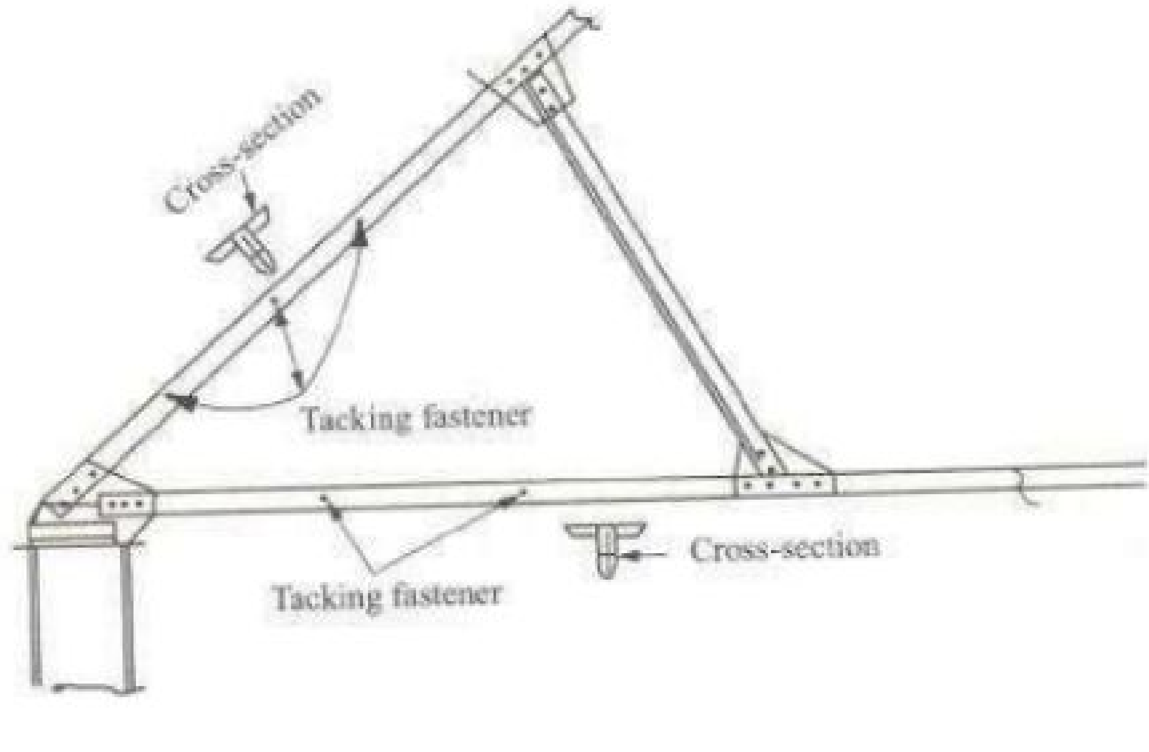


Standard section and built up section.

DESIGN OF CONNECTIONS:- [Section-10 IS 800-2007]

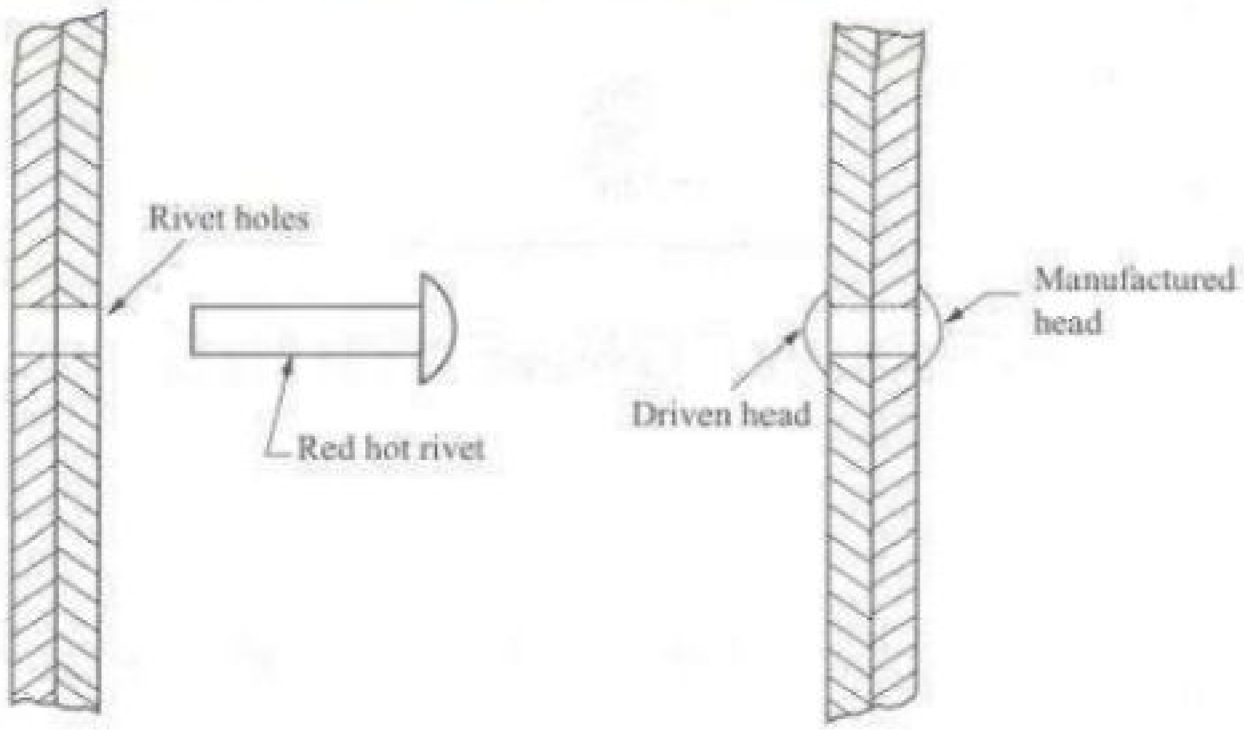
The possible connections in steel designs are

1. Riveted connections
2. Bolted Connections
3. Welded Connections



1. <u>Riveted Connections:-</u>

Riveted connections are used because rigid connection are establish since there was lot of disadvantages in riveted connection.



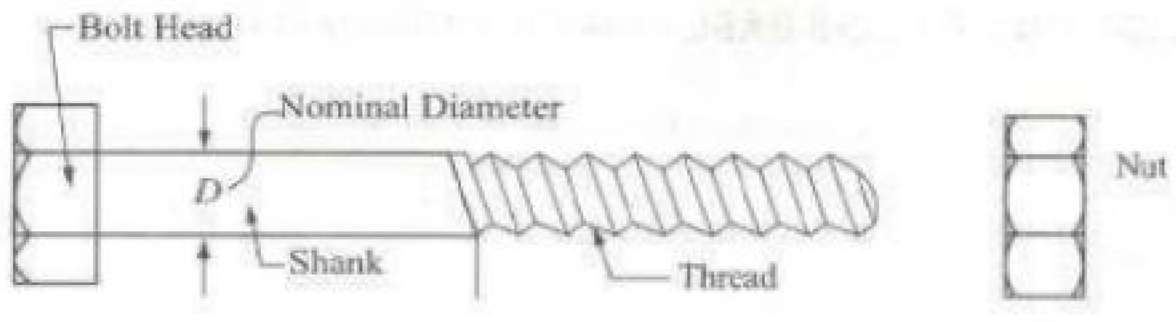
Riveting.

DISADVANTAGES:-

1. Requirements of skilled labour
2. Cost increased due to defective rivets, the connections are later preferred.
3. Noise Pollution.

TYPES OF BOLT CONNECTIONS:-

1. Bearing type bolts
   a) Unfinished [d+mm] } M.S. Steel
   b) Finished [d+1.2mm] } M.S. Steel
2. Friction type bolts → above Fe415 steel



Bolt and nut.

a) <u>Block Bolts:-</u> [Unfinished Bolts]

- These bolts are made from mild steel with square or hexagonal heads.
- The nominal dia(d) available are 12,16,20,22,24,27,30 & 36 mm designated as M16 M20 etc.,
- As the shank is unfinished, there is no contact with the members at the entire shown of contact surface.
- Joints remain quite loose result into large deflections & loosening of nuts in course of time.

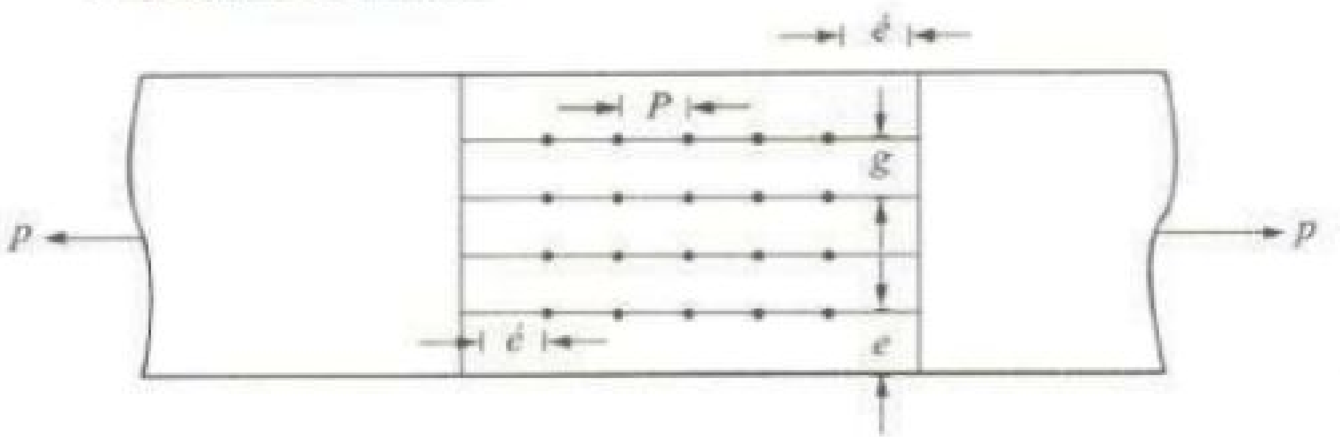
Generally the dia of bolt hole is 1.5mm to 2m larger than the nominal dia of shank.

b) <u>Finished Bolts:-</u> [Turned Bolts]

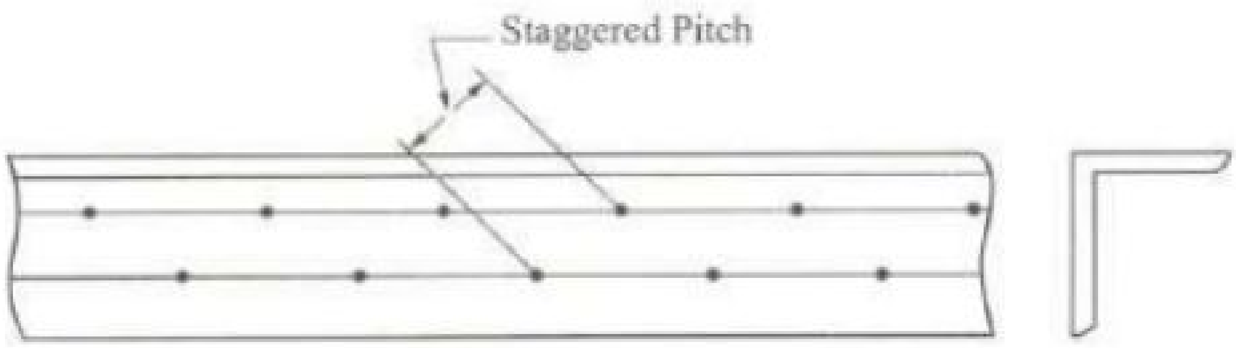
- These bolts are made from M.S.steel formed from hexagonal rods which are finished by turning to a circular shape within the bolts hole.
- The actual dimension of the bolt holes are kept 1.2 to 1.3mm larger than the nominal dia. Where the blot hole is kept 1.5mm larger than 'd'
- Here aligning the bolt holes needs special care.

<u>TERMINOLOGY:-</u>

1. Pitch [C/c distance b/w the bolt holes along the direction of load]
2. Gauge [C/c distance b/w the bolt holes to the direction of load]
3. Edge distance
4. End distance
5. Staggered distance



Pitch, gauge distance and edge distance.



Bolt distance in staggered bolts.

<u>IS 800-2007 SPECIFICATIONS:-</u> [Section-10] Table-73

1. For Spacing [cls 10:2]
   a) Pitch P shall not be less than 2.5d where, d-nominal dia of bolt
   b) In case of tension member P shal not be more than 16t (or) 200mm
   c) In case of comp. member P > 12t (or) 200mm where, t – tks of thinnest member
   d) In case staggered pitch, the pitch may be increased by 50% value specified provided the gauge distance less than 75mm
   e) In case of butt joint max pitch is restricted to 4.5d for a distance 1.5 times a width of plat from the butting surface.
   f) Gauge length (g) should not be more than 100+4t (or) 200mm whichever less.

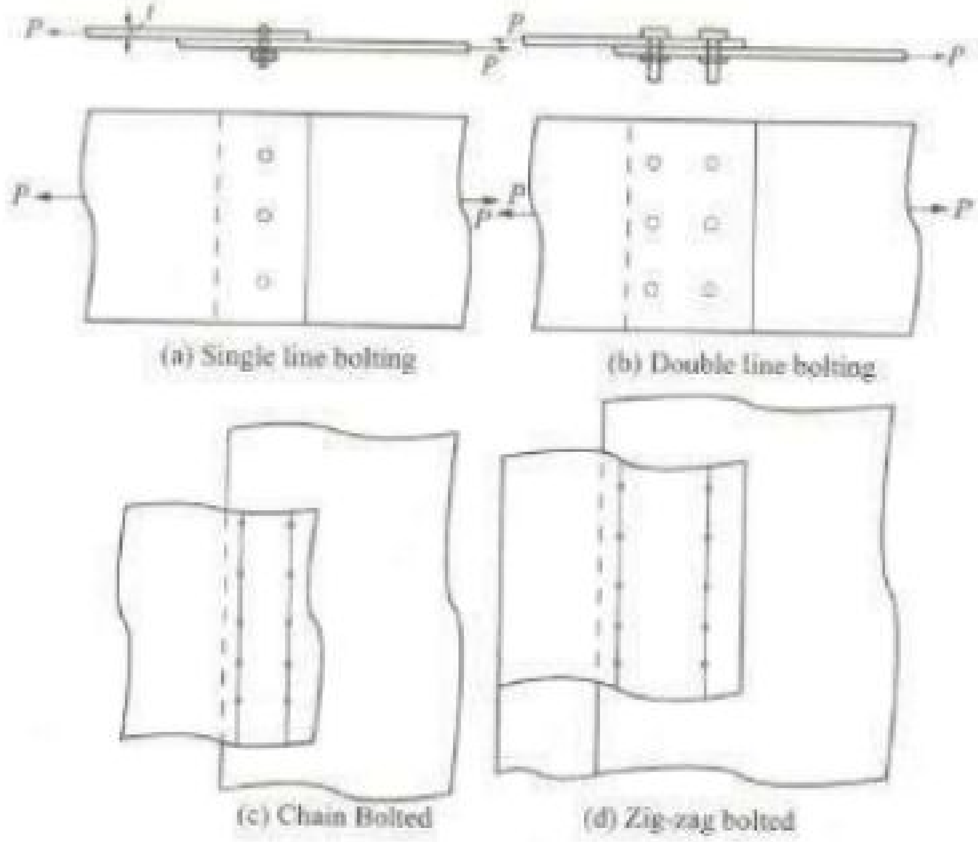
2. Edge Distance [cls 10:2:4]

   Mini edge distance shall not be less than
   (i) 1.7 times the hole dia in case of hand flame edges.
   (ii) 1.5 times hole dia in case of machine flame cut.
   (iii) Maxi. Edge distance should not exceed 16t ∑ where ∑= √250/fy=∑

   Also max edge distance should not exceed 40+ 4t

Types of Bolted Connections:-

1. Lap Joint
2. Butt Joint
   a) Single cover butt joint, b) Double cover butt joint

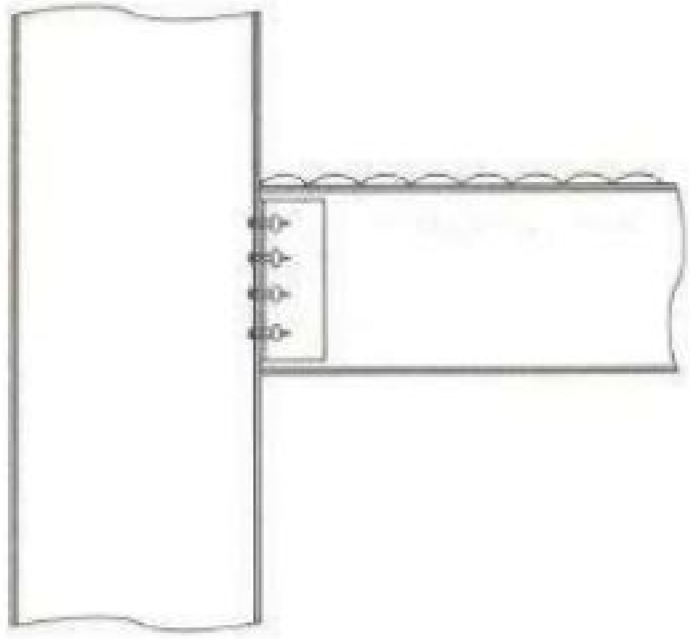


Types of lap joints.

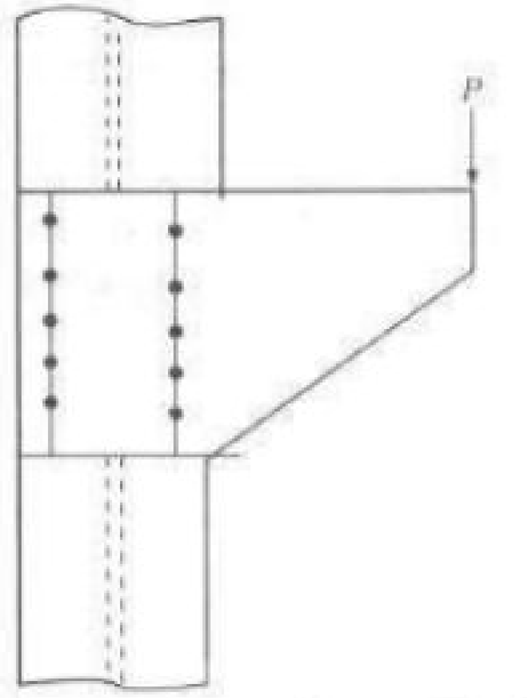
Lap joint is established by overlapping one plate to the other. Butt joint is made by placing the two plates to butt [edges facing each other] and connection

Internal forces on bolts:-

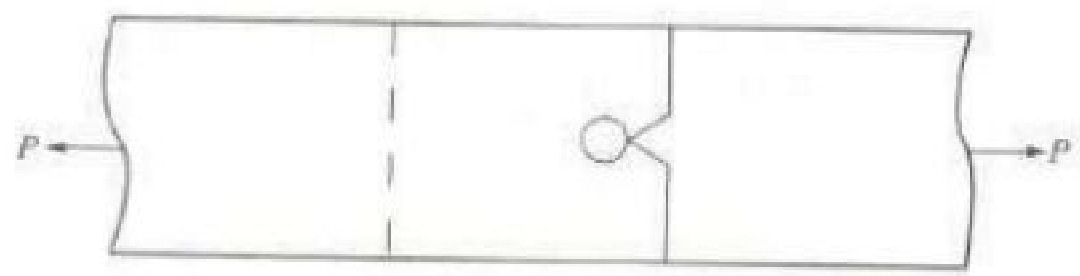
1. Single shear
2. Double shear
3. Pure tension
4. Pure moment
5. Shear & moment in the plane of connection
6. Shear & tension



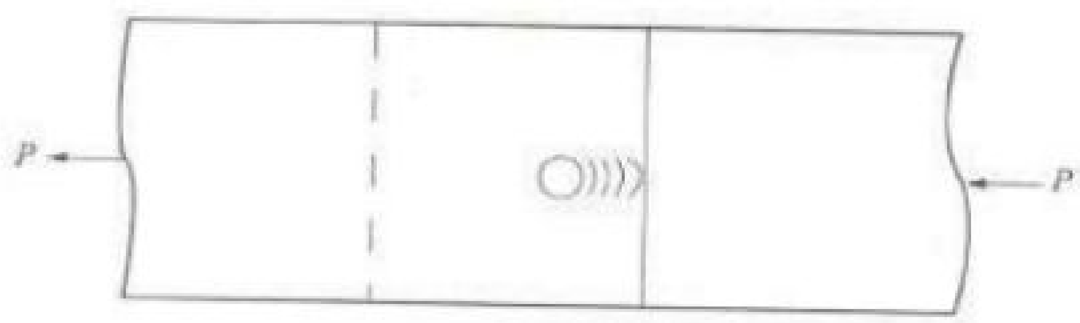
(e) Bolts subject to shear and tension



(f) Joint subject to shear and moment in its plane



Bursting or shearing of plates.



Crushing of plates.

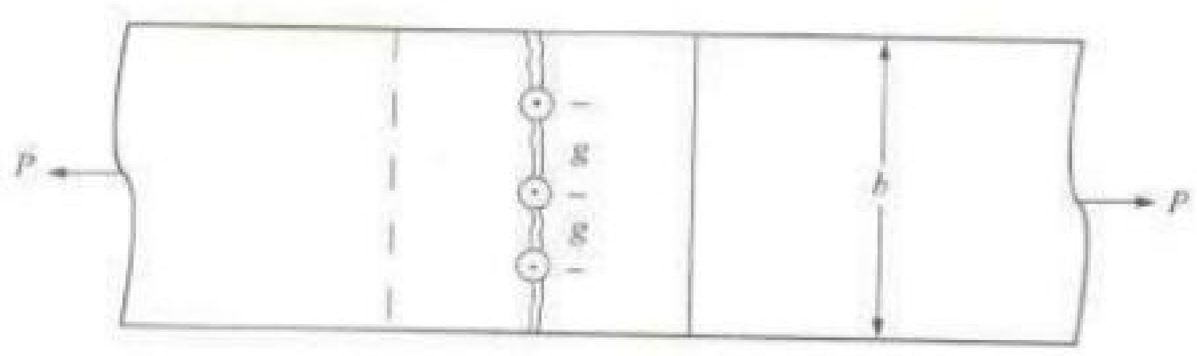
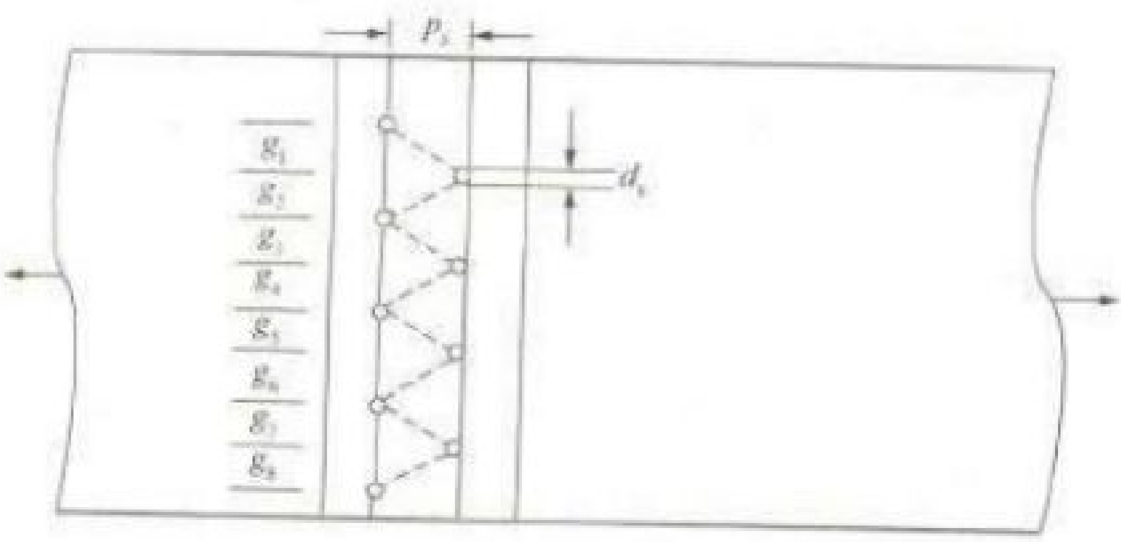


Figure 3.14 Rupture of plate.



Joint with staggered pitch.

Assumptions in design of bearing bolts:-

- Friction b/w plates negligible
- Shear is uniform over the c/s of the bolt.
- Distribution of truss on the plates b/w the bolt hole is uniform.
- Bolts in a groove subjected to direct loads, share the load equally.
- Bending stress developed in the holes is neglected.

Design Strength of Plates:-

Plates may fail due to

- Fupture of Plate [tearing]
- Crushing of Plate
- Bursting or shearing of Plates

Bursting & Crushing of Plates are avoided by providing mini edge distance.

The design tensile strength of plate half the thinnest plate against rupture is given by $T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}}$

Where,

'γ_{m1}' is partial safety factor for failure for ultimate stress 1.25 [Table-5] 'fu' is ultimate stress of the material [Refer table -1]

'An' is net effective area of the plate at he critical section given by

(i) $A_n = [b - nd_n]t$

(ii) $A_n = \left[b - nd_o + \sum_{i=i} \frac{P_{si}^2}{4g}\right]t$

Where,

(i) for single line for bolts
(ii) for staggered pitch of bolts

Here, 'b' is width of plate
't' is tks of plate (thinner plate)
'd_o' is dia of bolt hole.
'g' is the gauge length b/w bolt holes
'n' is no.of bolt holes in critical section
'p_s' is staggered pitch length b/w lines of bolt holes.
'I' is the subscribe for summation of all inclined legs.

Design Strength of Bearing Bolts:- [cls 10:3]

The design strength of bearing bolts under shear in the least of

(i) Shear Capacity
(ii) Bearing Capacity

(i) Shear capacity of bolts:- [cls 10:3:3] IS 800-2007

Shear strength of bolts $V_{dsp} = \frac{V_{nsb}}{\gamma_{mb}}$

$$V_{nsb} = \frac{fu}{\sqrt{3}}\left[n_n A_{nb} + n_s A_{sb}\right]$$

Where,

f_x => Ultimate tensile strength of bolt.
n_n => No. of shear planes with threads = 1
n_s => No. of shear planes without threads intercepting the shear plan
A_{nsb} => Net shear area of a bolt at threads
A_{sb} => Nominal plan shank area of the plane

$$A_{nb} = \frac{\pi}{4}[d - 0.9382]^2$$

For ISO threads = 0.78 $\frac{\pi d^2}{4}$

Reduction factor for shear capacity of bolts :-

The code such as the use of reduction factors for shear the following situation

(i) If the joint is too long [cls 10:33.1 IS 800-2007]
(ii) If the distance b/w the first & the lost hole in the joint exceeds 1.5d, the shear capacity 'V_{db}' shall be reduced by the factor β_{ij} is given by

$$\beta_{ij} = 1.075 - 0.005\ \frac{lj}{d}$$

Limit of 0.75 ¿ β_{ij} ¿ 1.0

(iii) If the crip length is large [cls 10:33.2 IS 800-2007]
If the total tks of connected plates exceeds 5 times the dia of bolt. The reduction factor for large gauge length is given by.

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

(iv) Reduction factor if packing plates are used
[cls 10:33.3 IS 800-2007] if packing plates of tks more than 6mm are used in the joint R.F β_{pk} is given by

$$\beta_{pk} = 1\text{-}0.0125 \text{ tpk}$$

(v) Thus the capacity of bolt in shear is

$$V_{nsb} = \frac{fu}{\sqrt{3}}\left[n_n A_{nb} + n_s . A_{sb}\right] \beta_{lj} x \beta_{lg} x \beta_{pk}$$

(ii) Bearing Capacity of Bolts:- [cls 10.3.1 IS 800-2007]

The design bearing strength of the bolt is $V_{dbp} = \frac{V_{nbp}}{\gamma_{mb}}$

Where,

V_{nbp} = 2.5kb dt fu
V_{nbp} => Nominal bearing strength of bolt
K_b => Smaller value of least of

$$\frac{e}{3d_o}, \frac{p}{3d_o} - 0.25, \frac{f_{ub}}{f_u}, 1.0$$

E => End distance of the bolt
P => Pitch distance
d_o => Dia of bolt hole
d => Nominal dia of bolt
t => Sumation of tks of connecting plates experiencing bearing streis in same direction
fub => Ultimate tensile stress of the bolt
fu => Ultimate tensile stress of plate

EFFICIENCY THE JOINT:-

It is defined as ratio of strength of the joint to strength of the solid plate.

$$\eta = \frac{\text{strength of joint}}{\text{strength of solid plate}} \times 100$$

DESIGN PROCEDURE:-

- Determine the design force [factored] acting on the joint.
- The dia of bolt is assumed.
- Strength of connections is found based on the strength of plate @ critical section and strength of bolt in shear & bearing.
- The design strength is ensure to be not less than the design action.
- Efficiency of the connection is found based on the strength of solid plate.

NOTE:-

Strength of solid plate in yielding is less than that of tearing (rupture) of the solid plate.

For a example: Considering M.S. steel where

$fy = 250\, N/mm^2 \wedge fu = 410\, N/mm^2$

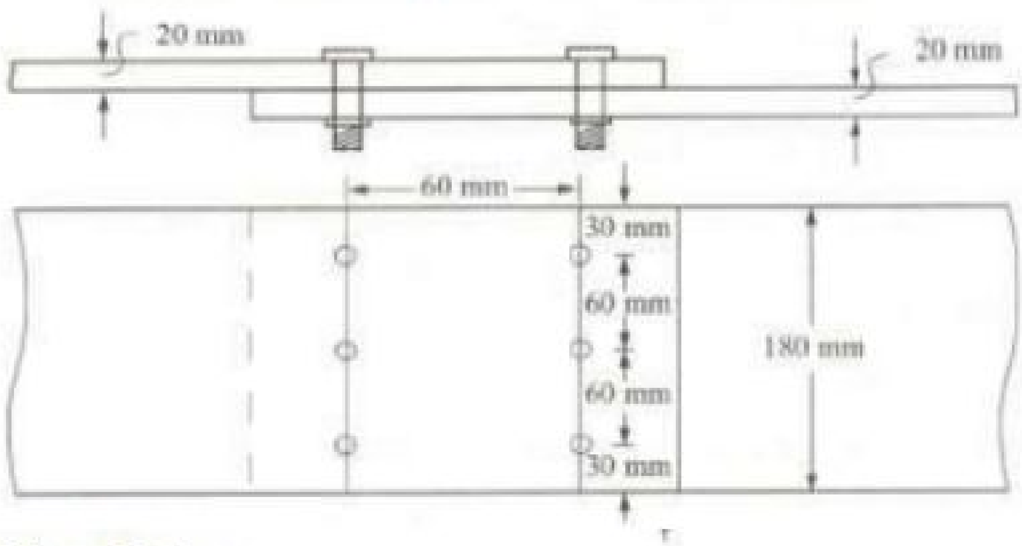
Design strength of solid plate

(i) In yielding $= \frac{250}{1.1} = 227.27\, N/mm^2$

(ii) In rupture $= \frac{0.9 x 410}{1.25} = 295.2\, N/mm^2$

∴ Strength of solid plate is govern by strength in yielding.

1. Find the efficiency of the lap joint shown in fig. given M20 bolt of grade 4.6 and plate of grade Fe410 [E250] are used.



Given Data:-

t = 20mm

Bolt:- M20

Grade 4.6 => fu = 400N/mm^2

fy = 250N/mm^2

Plate:-

Fe 410 [E250]

Fu = 410 N/mm^2

Fy = 250 N/mm^2 [Table 1 – I.S 800 – 2007]

Efficiency of the joint $= \frac{\text{strength of joint}}{\text{strength of solid plate}} \times 100$

Strength of connection is least of strength of plate at critical section and strength of bolt in shear & bearing.

Strength of plate @ the joint:-

Tensile force $T_{dn}=\frac{0.9\,Anfu}{\gamma_{ml}}$

$A_n=(b-nd_o)t$

ps = 0 [∵ Bolts are on a straight line]

= (180 – 3x22) 20 [∵ d_o = 20+2=22]

An = 2280mm^2

γ_{ml} = 1.25 [from table 5- I.S 800-2007]

[d_o = Dia of bolt hole = 20+2=22mm]

$$T_{dn} = \frac{0.9 \times 2280 \times 410}{1.25}$$

T_{dn} = 673.056 KN

Strength of bolts:- [cls 10.3.3 IS 800-2007]

(i) Strength of bolt in shear $V_{dsb}=\frac{V_{nsb}}{\gamma_{mb}}$

$$V_{nsb}=\frac{fu}{\sqrt{3}}[N_nA_{nb}+N_sA_{sb}]$$

N_n = No.of shear planes @ the thread = 1

N_s = No.of shear planes @ shank [N_s = 0 for lap jt N_s = 1 for D.C.B.J]

$A_{nb}=0.78\times\frac{\pi d^2}{4}$ [This formula for ISO thread]

$= 0.78\times\frac{\pi\times2^2}{4}$

A_{nb} = 245mm^2

$V_{nsb}=\frac{400}{\sqrt{3}}[1\times245\times6]$

$V_{nsb}=339.481\,KN$ ∵ $V_{dsb}=\frac{339.48}{1.25}$

V_{dsb} = 271.58 KN

(ii) Strength of bolt in bearing: [cls 10.3.4 IS]

Take

$\beta_{lj}=\beta_{lg}=\beta_{pk}=1$

$V_{dbp}=\frac{V_{nbp}}{\gamma_{mb}}$

V_{dbp} = 2.5 kb dt fu

Kb = least of e/3d_o, p/3d_o-0.25, $\frac{fub}{Fu}$, 1.0

E = end distance [centre of the extreme end bolt to the edge ¿ lr to direction of load.

$$k_b \Rightarrow \frac{30}{3\times22}, \frac{60}{3\times22} - 0.25$$

$$k_b \Rightarrow 0.45, 0.659, 0.976, 1$$

Take K_b value of whichever less [$\therefore K_b = 0.45$]

$$V_{nbp} = 2.5\times0.45\times20\times20\times410$$

$$V_{nbp} = 186.3 \text{ KN}$$

$$V_{dbp} = \frac{186.3}{1.25}$$

$$V_{dbp} = 149.04 \text{ KN}$$

$\therefore$ Design strength of bolt = 6 x 149.04

V_{dbp} bolt = 894.24 KN

Design strength of the joint = 271.58 KN

Design strength of jt is the least of strength of joint 673.06 KN, 271.58 KN & 894.24 KN

<u>Strength of Solid Plate:-</u>

Strength of Solid Plate = $\frac{fy \times Ag}{\gamma_{m1}}$

[yielding sides the strength of solid plate]

$$= \frac{250}{1.1}\times180\times20$$

Strength of solid plate = 818.18 KN

$\therefore$ Efficiency of joint $\eta = \frac{271.58}{818.18}\times100$

$$\eta = 33.19$$

2. Find the efficiency of the joint for the above problem if instead of lap joint, a double cover butt joint is provided. Two cover plates each of size 12mm and 6 nos. of bolts are provided on each side.

<u>Given Data:-</u>

[Table 1, I.S 800-2007] [Pg.No.13]

<u>Plate:-</u>

Fe410 [250]

Fu = 410 N/mm^2

Fy = 250 N/mm^2

<u>Bolt:-</u>

M20, Grade 4.6

φ of bolt = 20 mm

fu_b = 400 N/mm^2

fy_b = 240 N/mm^2

The strength of plate at the joints and the strength of bolts in bearing are same as that of the previous problem.

(1) <u>Strength of plate @ the joint:-</u>

$$T_{dn}=\frac{0.9\,Anfu}{\gamma_{ml}}$$

$$A_n=[b-nd_o]t$$

$$=[180-3\times 22]\,20$$

$$A_n = 2280\text{mm}^2$$

$\gamma_{ml}=1.25$ [from tables-5 IS 800-2007 Pg.No:30]

$$d_o = 20+2 = 22$$

$$=\frac{2280\times 0.9\times 410}{1.25}$$

$$T_{dn}=673.056\,KN$$

(2) <u>Strength of bolts:-</u>

(i) <u>Strength of bolt in bearing :</u> (cls 10.3.4 IS 800-2007]

$$V_{dbp}=\frac{V_{nbp}}{\gamma_{mb}}$$

$$V_{nbp}=2.5\,kb.dt.fu$$

$$K_b=\frac{o}{3d_o},\frac{p}{3d_o}-0.25,\frac{fub}{fu},1.0$$

$$=\frac{30}{3\times 22},\frac{60}{3\times 22}-0.25,\frac{400}{410},1.0$$

$$K_b=0.45,0.659,0.976,1$$

Take kb value of whichever is less

$$\because K_b=0.45$$

$$V_{nbp}=2.5\times 0.45\times 20\times 20\times 410$$

$$V_{nbp}=186.3\,KN$$

$$V_{dbp}=\frac{186.3}{1.25}$$

$$V_{dbp}=149.04\,KN$$

$\therefore$ Strength of bolt in bearing = 6 x 149.04

$$V_{dbp}=894.24\,KN$$

(ii) <u>Strength of bolt in shear:-</u> [cls:10.3.3 IS 800-2007]

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

$$V_{nsb}=\frac{fu}{\sqrt{3}}[N_n A_{nb}+N_{sA_{sb}}]$$

∵ Double cover butt jt provided each bolts resists shear along two planes, the section at the root & another section at the shank.

$\because n_n = n_s = 1$ for each bolts

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

$$= \frac{0.78 \times \pi \times 20^2}{4}$$

$$A_{nb} = 245 mm^2$$

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

$$= \frac{\pi \times 20^2}{4}$$

$$A_{sb} = 314.16 mm^2$$

$$\therefore V_{nsb} = \frac{400}{\sqrt{3}}[6 \times 245 + 6 \times 314.16]$$

$$V_{nsb} = 774.8 KN$$

$$\boxed{V_{dsp} = 619.84 KN}$$

Reduction factors $\beta_{lj} = \beta_{lg} = \beta_{pk} = 1$

∴ Design Strength of the joint = 619.84 KN [least of 673 KN, 894.4 KN, 619.84 KN]

<u>Strength of the solid plate:-</u>

Strength of the solid plate = $\frac{f_y A_g}{\gamma_{m1}}$

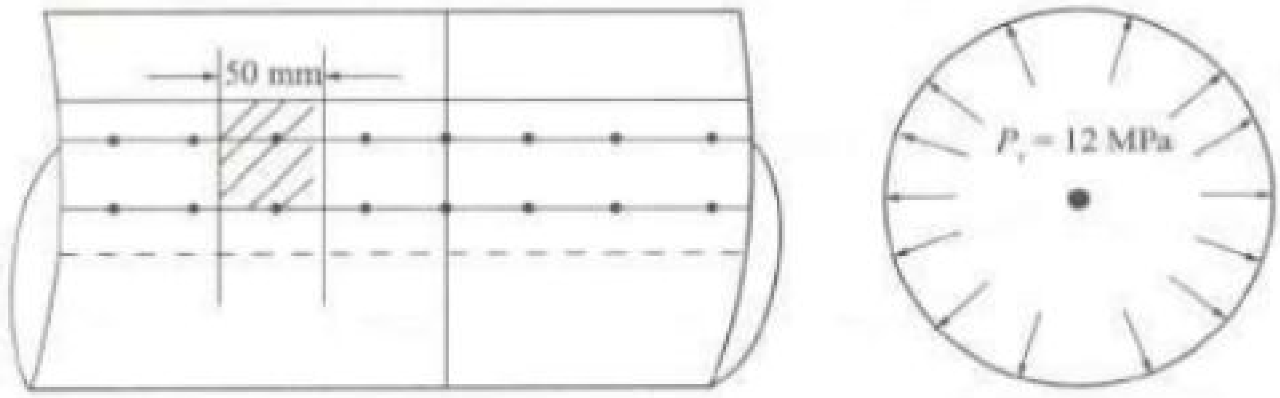
$= \frac{250}{1.1} \times 180 \times 20$ [Tks of thinner plate is the least of sum of cover plate 20(or) 24mm]

Strength of the solid plate = 818.18KN

$$\eta = \frac{619.84}{818.18} \times 100$$

$$\eta = 75.76\%$$

(3) A boiler shell is made up of 14mm tk Fe415 plates. The jt is double bolted lap jt with bolts of grade 4.6 at distances of 500mm. Determine the strength of the jt. Per pitch width for a safe design if the internal dia of the shell is 1m and steam pressure is 12Mpa.



Given:-

Grade 4.6

Bolt:-

$fu_b = 400\ N/mm^2$

$fy_b = 240\ N/mm^2$

Plate:- $fu = 410\ N/mm^2$, $fy = 250\ N/mm^2$

Sln:-

The strength of the plate is check for unit pitch [50mm width]

Strength of Plate @ joint:- [50mm width]

$$T_{dn} = \frac{0.9\,Anfu}{\gamma_{ml}}$$

$$An = [b - nd_o]t$$

Provide 18mm dia of bolt hole.

$$= [50 - 1 \times 18] \times 14$$

$$An = 448\ mm^2$$

$$T_{dn} = \frac{0.9 \times 448 \times 410}{1.25} [\gamma_{ml} \rightarrow table\,5\,IS\,800-2007]$$

$$T_{dn} = 132.25\ KN$$

Strength of bolt:- [50mm width]

The strength of the bolt is found for 1 pitch width in both shear & bearing. For 1 pitch width there are 2 bolts along the line.

Strength of bolts in shear:- [cls 10.3.3 IS 800-2007]

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

$$V_{nsb} = \frac{fu}{\sqrt{3}} [n_n A_{nb} + n_s A_{sb}]$$

For lap joint $n_n = 1$

$n_s = 0$

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

$$= \frac{0.78 \times \pi \times 16^2}{4}$$ Assume dia of bolt 16mm for IS

$$A_{nb} = 156.8\,mm^2$$

$$V_{nsb} = \frac{400}{\sqrt{3}}[2 \times 1 \times 156.8]$$

$$V_{nsb} = 72.422\,KN$$

$$V_{dsp} = \frac{72.422}{1.25}$$

$$\boxed{V_{dsp} = 57.94\,KN}$$

(b) <u>Strength of bolt in bearing:</u> [cls 10.3.4 IS 800-2007]

$$V_{dbp} = \frac{V_{nbp}}{\gamma_{mb}}$$

$$V_{nbp} = 2.5\,kb.dt.fu$$

$$K_b = \frac{e}{3d_o}, \frac{e}{3d_o} - 0.25, \frac{fu_b}{fu}, 1.0$$

$$= \frac{54}{3 \times 18}, \frac{50}{3 \times 18} - 0.25, \frac{400}{410}, 1.0$$

$$K_b = 1, 0.676, 0.975, 1.0$$

[∴ e is not given, so it is assume that sufficient edge distance is provided]

Take K_b value whichever is less [$K_b = 0.676$]

$$V_{nbp} = 2.5 \times 0.676 \times 16 \times 14 \times 410$$

$$V_{nbp} = 155.210\,KN$$

$$V_{dbp} = \frac{155.210}{1.25}$$

$$V_{dbp} = 124.16\,KN$$

For 2 bolts $V_{dbp} = 2 \times 124.16$

$$\boxed{V_{dbp} = 248.32\,KN}$$

∴ Design strength of bearing for 50mm width = 248.32 KN

∴ Design strength of the joint per 50mm width is the least of 57.94 KN

132.25 KN

248.32 KN

∴ Design strength of jt/50mm width = 57.94 KN

Check for hoop tension:-

The action of force

$$\text{Hoop tension on shell} = \frac{PD}{2t}$$

Where,

P ⇒ Internal Pressure, D ⇒ Dia of Shell

T ⇒ Tks of the shell

$$= \frac{12\times1000}{2\times14}$$

$$= 428.57\ N/mm^2$$

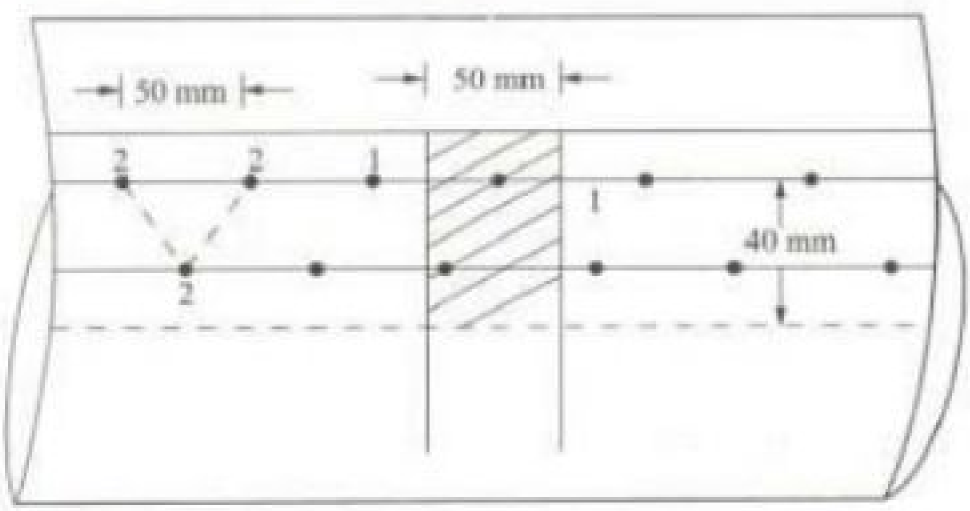
∴ For 50mm width hoop tension = 428.57 x 50 = 21.43 KN

The actual hoop tension acting on shell = 21.43 KN Applying partial safety factor for load as 1.5 factored design load = 21.43 x 1.5

= 32.14 KN < 57.94 km

Hence the connection is safe.

(4) Check the safety of the connection in the above problem, it zig-zag bolts are provided as shown in the fig.



Given Data:-

Bolt : Grade 4.6

$fu_b = 400\ N/mm^2$

$fy_b = 240\ N/mm^2$

Plate:- $fu = 410\ N/mm^2$

$fy = 250\ N/mm^2$

Sln:-

The critical section were tearing of the plate takes place is along section 1-1 and section 2-2.

Providing dia of bolt = 16mm

Dia of bolt hole = 18mm

(i) <u>Net area resisting the tearing force along section -</u>

$$An_1 = (b - nd_o)t$$

$$= [50 - 1 \times 18]\ 14$$

$$An_1 = 448\ mm^2$$

(ii) <u>along section 2-2</u>

$$An_2 = [b - nd_o + \sum \frac{(P_s)^2}{A_g}]t$$

Where

$P_s \Rightarrow 40mm$

$g \Rightarrow 25mm$

$$A_{n2} = \left[50 - 2 \times 18 + \frac{2 \times 40^2}{4 \times 25}\right] 14$$

$$A_{n2} = 644mm^2$$

The least area decides the failure of plate.

∴ Section 1-1 is weaker

<u>Design strength of plate @ the joint:-</u>

$$T = \frac{0.9\ Anfu}{\gamma_m}$$

$$= \frac{0.9 \times 448 \times 410}{1.25}$$

T = 132.25 KN

<u>Design strength of bolt:-</u>

(i) <u>Strength of bolt in shear:-</u> [50mm width]

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

$$V_{nsb} = \frac{fu}{\sqrt{3}}[n_n A_{nb} + n_s A_{sb}]$$

$n_n = 1,\ n_s = 0$

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

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

$$A_{nb} = 156.8\ mm^2$$

$$V_{nsb} = \frac{400}{\sqrt{3}}[2 \times 1 \times 156.8]$$

$$V_{nsb} = 72.42\ KN$$

$$V_{dsp} = \frac{72.42}{1.25}$$

$$V_{dsp} = 57.94\ KN$$

(ii) Strength of bolt in bearing:- [50mm width]

$$V_{dbp} = \frac{V_{nbp}}{\gamma_{mb}}$$

$$V_{nsb} = 2.5\, kb.\, dt.\, fu$$

$$K_b = \frac{e}{3d_o}, \frac{p}{3d_o} - 0.25, \frac{fu_b}{fu}, 1.0$$

Assuming sufficient edge distance is available

$$K_b = \frac{54}{3\times 18}, \frac{50}{3\times 18} - 0.25, \frac{400}{410}, 1.0$$

$$K_b = 1, 0.676, 0.975, 1.0$$

Take K_b value is the least of 0.676

$$V_{nbp} = 2.5\times 0.676\times 16\times 14\times 410$$

$$V_{nbp} = 155.21\, KN$$

$$V_{dbp} = \frac{155.21}{1.25}$$

$$V_{dbp} = 124.17\, KN$$

For 2 bolts $V_{dbp} = 2\times 124.17$

$$V_{dbp} = 248.3\, KN$$

∴ Design strength of bearing for 50mm width = 248.23 KN

Check Hoop Tension:-

Hoop tension on shell = $\frac{PD}{2t}$

$= \frac{1000\times 12}{2\times 14}$

$= 428.57\ N/mm^2$

For 50mm width of hoop tension = 428.57 x 50

= 21.43 KN

The actual hoop tension acting on shell = 21.43 KN

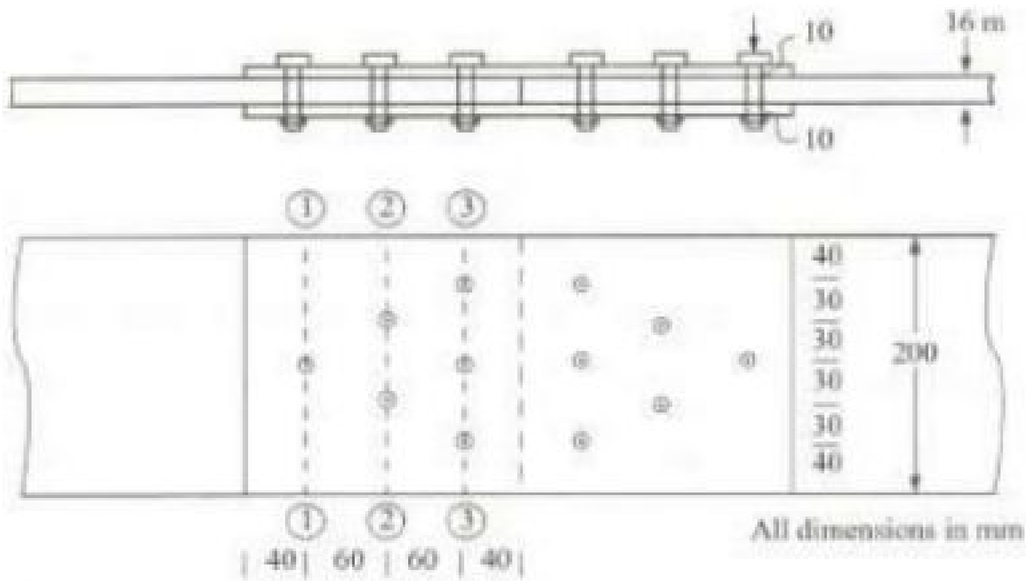
Applying Partial safety factor for load as 1.5 factored design load

= 21.43 x 1.5

= 32.14 KN < 57.94

Hence the connection is safe.

5. Find the bolt value of the connection b/w two plates of tks 16mm which are to be joint using M20 bolts of grade 4.6 by (i) Lap joint (ii) Butt joint [using 10mm cover plates]



Given Data:-

TKS of plate = 16mm

Bolt:-

M20 $\varphi = 20\,mm, fub = 400\,N/mm^2$

Grade 4.6

Sln:-

(i) **LAP JOINT:-**

1. Strength of bolt in shear: [cls 10.3.3 IS800-2007]

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

$$V_{snb} = \frac{fu}{\sqrt{3}}[n_n A_{nb} + n_s A_{sb}]$$

$n_n = 1$, $n_s = 0$

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

$$A_{nb} = 245\ mm^2$$

$$V_{nsb} = \frac{400}{\sqrt{3}}[1 \times 245]$$

$$V_{nsb} = 56.58\ KN$$

$$V_{dsp} = \frac{56.58}{1.25}$$

$$V_{dsp} = 45.26KN$$

2. Strength of bolt in bearing: [cls 10.3.4 IS 800-2007]

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

$$V_{nbp} = 2.5 k_b dt . fu$$
$$K_b = \frac{e}{3d_o}, \frac{p}{3d_o} - 0.25, \frac{fub}{fu}, 1$$

Take

$K_b = 1$

$$V_{nbp} = 2.5 \times 1 \times 20 \times 16 \times 410$$
$$V_{nbp} = 328 \text{ KN}$$
$$V_{dbp} = \frac{328}{1.25}$$
$$V_{dbp} = 262.4 \text{ KN}$$

Design strength of bolt in bearing = 262.4 KN

Design strength of bolt = 45.26 KN [Least Value]

(ii) **BUTT JOINT:-**

1. Strength of the bolt in shear: [cls 10.3.3 IS 800-2007]

$$V_{dsp} = \frac{V_{nsb}}{\gamma_{mb}}$$
$$V_{snb} = \frac{fu}{\sqrt{3}} [n_n A_{nb} + n_s A_{n_{sb}}]$$
$$A_{nb} = \frac{0.78 \times \pi d^2}{4}$$
$$= \frac{0.78 \times \pi \times 20^2}{4}$$
$$A_{nb} = 245 \, mm^2$$
$$A_{nsb} = \frac{\pi d^2}{4}$$
$$= \frac{\pi \times 20^2}{4}$$
$$A_{nsb} = 314.1 \, mm^2$$
$$n_n = 1, n_s = 1$$
$$V_{nsb} = \frac{400}{\sqrt{3}} [1 \times 245 + 1 \times 314.1]$$
$$V_{nsb} = 129.1 \, KN$$
$$V_{dsp} = \frac{129.1}{1.25}$$
$$\boxed{V_{dsp} = 103.28 \, KN}$$

2. <u>Strength of bolt in bearing :</u> [cls 10.3.4 IS 800-2007]

$$V_{dsp} = \frac{V_{nsb}}{\gamma_{mb}}$$
$$V_{nbp} = 2.5 k_b dt . fu$$
$$K_b = \frac{e}{3d_o}, \frac{p}{3d_o} - 0.25, \frac{fub}{fu}, 1$$

Take

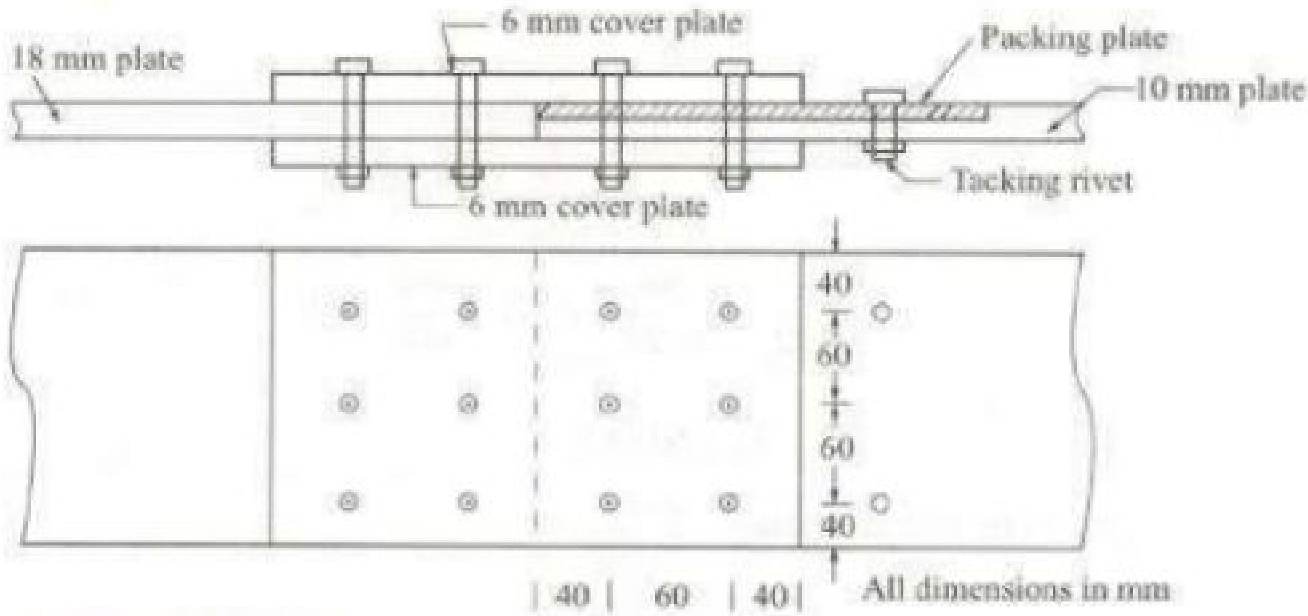
$K_b = 1$ ['t' is least of 16mm (2x10mm)]

$$V_{nbp} = 2.5 \times 1 \times 20 \times 16 \times 410$$
$$V_{nbp} = 328 \text{ KN}$$
$$V_{dbp} = \frac{328}{1.25}$$
$$\boxed{V_{dbp} = 262.4 \text{ KN}}$$

Design strength of bolt in bearing = 262.4 KN

∴ Design strength of bolt = 103.28 KN

5. The above problem find the bolt value for butt joint connection with tks of cover plate as 6mm.



(ii) **<u>BUTT JOINT:-</u>**

1. <u>Strength of the bolt in shear:</u> [cls 10.3.3 IS 800-2007]

$$V_{dsp} = \frac{V_{nsb}}{\gamma_{mb}}$$
$$V_{snb} = \frac{fu}{\sqrt{3}} [n_n A_{nb} + n_s A_{n_{sb}}]$$

$$A_{nb} = \frac{0.78 \times \pi d^2}{4}$$
$$= \frac{0.78 \times \pi \times 20^2}{4}$$
$$A_{nb} = 245\, mm^2$$
$$A_{nsb} = \frac{\pi d^2}{4}$$
$$= \frac{\pi \times 20^2}{4}$$
$$A_{nsb} = 314.1\, mm^2$$
$$n_n = 1, n_s = 1$$
$$V_{nsb} = \frac{400}{\sqrt{3}}[1 \times 245 + 1 \times 314.1]$$
$$V_{nsb} = 129.1\, KN$$
$$V_{dsp} = \frac{129.1}{1.25}$$
$$\boxed{V_{dsp} = 103.28\, KN}$$

Design strength of bolt in shear = 103.28 KN

2. <u>Strength of bolt in bearing :</u> [cls 10.3.4 IS 800-2007]

$$V_{dsp} = \frac{V_{nsb}}{\gamma_{mb}}$$
$$V_{nbp} = 2.5 k_b dt . fu$$
$$K_b = \frac{e}{3d_o}, \frac{p}{3d_o} - 0.25, \frac{fub}{fu}, 1$$

Take

$K_b = 1$ ['t' is least of 16mm (2x6mm)]

$$V_{nbp} = 2.5 \times 1 \times 20 \times 12 \times 410$$
$$V_{nbp} = 246\, KN$$
$$V_{dpb} = \frac{246}{1.25}$$
$$V_{dpb} = 196.8\ KN$$

Design strength of bolt in bearing = 1x196.8 KN

$$\boxed{V_{dpb} = 196.8\ KN}$$

Design strength of bolt = 103.28 KN [Least Value]

6. Design a lap joint b/w two plates each of width 120mm. If the tks of 1 plate is 16mm and tks of other plate is 12mm. The jt has to transfer design load of 160KN. The plates are of grade Fe 410 use bearing type bolts.

Given Data:-

Plate width = 120mm
1 plate tk = 16mm
Other plate tk = 12mm
Plate Grade = Fe 410
Design load = 160KN

Sln:-

Assume dia of bolt as 16mm of grade 4.6
∴ Dia of bolt hole d_o = 18mm
[Refer table-1 IS 800-2007] fu = 400 N/mm^2

No.of bolts required A = $\frac{P}{BoltValue}$

Where,
Bolt value is the least of strength of bolt in single shear & bearing.

Bolt Value:-

(i) Strength of bolt in single shear:- [cls 10.3.3 IS 800-2007]

$$V_{nsb}=\frac{fu}{\sqrt{3}}[n_n A_{nb}+n_s A_{sb}]$$

n_n=1, n_s=0

$$A_{nb}=\frac{0.78\times\pi\times16^2}{4}$$

$$A_{nb}=156.83\,mm^2$$

$$=\frac{400}{\sqrt{3}}[1\times156.83]$$

$$V_{nsb}=36.218\,KN$$

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

$$=\frac{36.218}{1.25} \quad \leftarrow[fromtable-5]$$

$$\boxed{V_{dsp}=28.97\text{ KN}}$$

Design strength of bolt in single shear = 28.97 KN

(ii) Strength of bolt in bearing:- [cls 10.3.4 IS 800-2007]

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

$$V_{nbp}=2.5k_b\,dt.fu$$

Assume the edge distance e = 1.5d, 27 mm, 30mm

p = 2.5d = 40mm

$$K_b=\frac{e}{3d_o},\frac{p}{3d_o}-0.25,\frac{fub}{fu},1$$

$$=\frac{30}{3\times 18},\frac{40}{3\times 18}-0.25,1,1$$

$$= 0.556, 0.491, 1,1$$

Take K_b = 0.491

$$V_{nbp}=2.5\times 0.49\times 16\times 12\times 400$$

$$V_{nbp} = 94.08 \text{ KN}$$

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

$$V_{dbp}=75.26\,KN$$

Design strength of bolt in bearing = 75.26 KN

∴ Design strength of bolt value = 28.97 KN

∴ No.of bolts required n = $\frac{160}{28.97}$

$$n = 5.5 \simeq 6 \text{ Nos.}$$

Providing an edge distance of 30mm for the bolts and providing them in two layers, with edge distance 30mm & pitch 40mm, the length of overlap read for the plates is read 140mm as shown in fig.

Strength of plate @ the joint:-

$$T=\frac{0.9A_n fu}{\gamma_{ml}}$$

$$A_n=[b-nd_o]t$$

$$= [120 - 2 \times 18]12$$

$$A_n=1008\,mm^2$$

$$=\frac{0.9\times 1008\times 410}{1.25}$$

$$T=297.56\,KN>160\,KN$$

Hence the plate is safe against tearing.

6. Design a connection using butt joint for the above problem. [Assume cover plate 6mm of each]

Given Data:-

Plate width = 120mm
1 plate tk = 16mm
Other plate tk = 12mm
Plate Grade = Fe 410
Design load = 160KN

Sln:-

No.of bolts required A = $\frac{P}{BoltValue}$

BOLT VALUE:-

(i) Design strength of the bolt in shear: [cls 10.3.3 IS 800-2007]

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

$$V_{snb} = \frac{fu}{\sqrt{3}}[n_n A_{nb} + n_s A_{n_{sb}}]$$

n_n=1, n_s=0

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

$$A_{nb} = 156.83\,mm^2$$

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

$$A_{sb} = 201\,mm^2$$

$$V_{nsb} = \frac{400}{\sqrt{3}}[1 \times 156.83 + 1 \times 201]$$

= 82.637 KN

$$V_{dsp} = \frac{82.637}{1.25}$$

$$V_{dsp} = 66.109\,KN$$

Design Strength of bolt in bearing :- [cls 10.3.4 IS 800-2007]

$$V_{dbp} = \frac{V_{nbp}}{\gamma_{mb}}$$

$$V_{nbp} = 2.5\,kb.dt.fu$$

$$K_b = \frac{30}{3 \times 18}, \frac{40}{3 \times 18} - 0.25, 1, 1$$

= 0.556, 0.491, 1,1

Take K_b = 0.491

$$V_{nbp} = 2.5 \times 0.491 \times 16 \times 12 \times 401$$
$$V_{nbp} = 96.63 \text{ KN}$$
$$V_{dbp} = \frac{96.63}{1.25}$$
$$V_{dbp} = 77.304\, KN$$

Design strength of bolt value = 66.109 KN

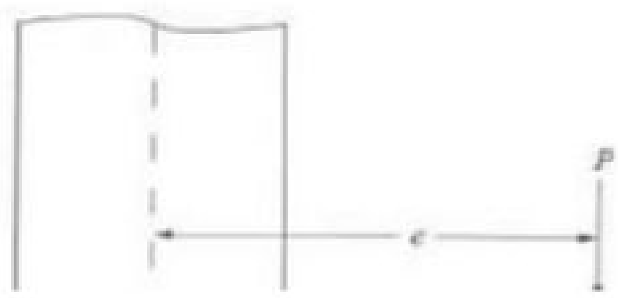
No. of bolts, n = $\frac{160}{66.109}$

= 2.4 $\simeq$ 4Nos.

<u>Strength of Plate @ the joint:-</u>

$$T_{dn} = \frac{0.9 A_n fu}{\gamma_{ml}}$$
$$A_n = [b - nd_o]t$$
$$= [120 - 3 \times 18]12$$
$$A_n = 924\, mm^2$$
$$= \frac{0.9 \times 924 \times 410}{1.25}$$
$$T_{dn} = 272.76 \text{ KN } ¿ 160\, KN$$

∴ The plate is safe against tearing.

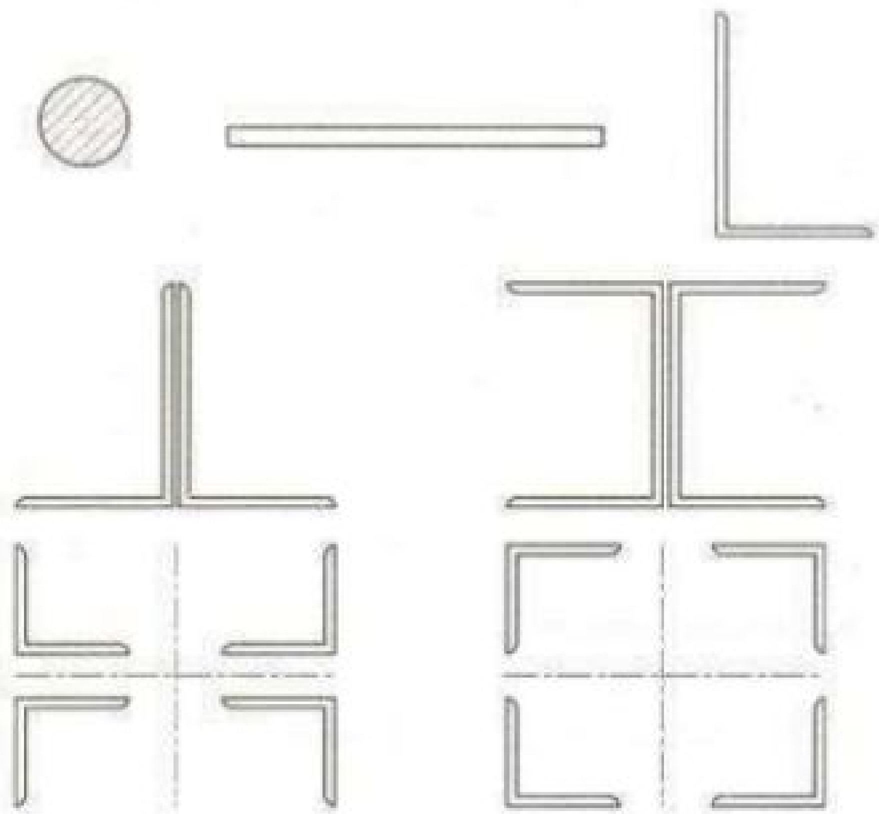


Tension Members:-

Tension members are design to satisfy the design strength of the member against

(i) Yielding of gross section
(ii) Rupture of critical section
(iii) Block shear @ end of connection

Generally tension members are known as tie member.



The various shapes of tension members are solid circular sections, plates, angles, channels, I-sections, T-section & built-up section.

1. Design strength of Tension Members are due to yielding:- [cls 6.2 IS 800-2007]

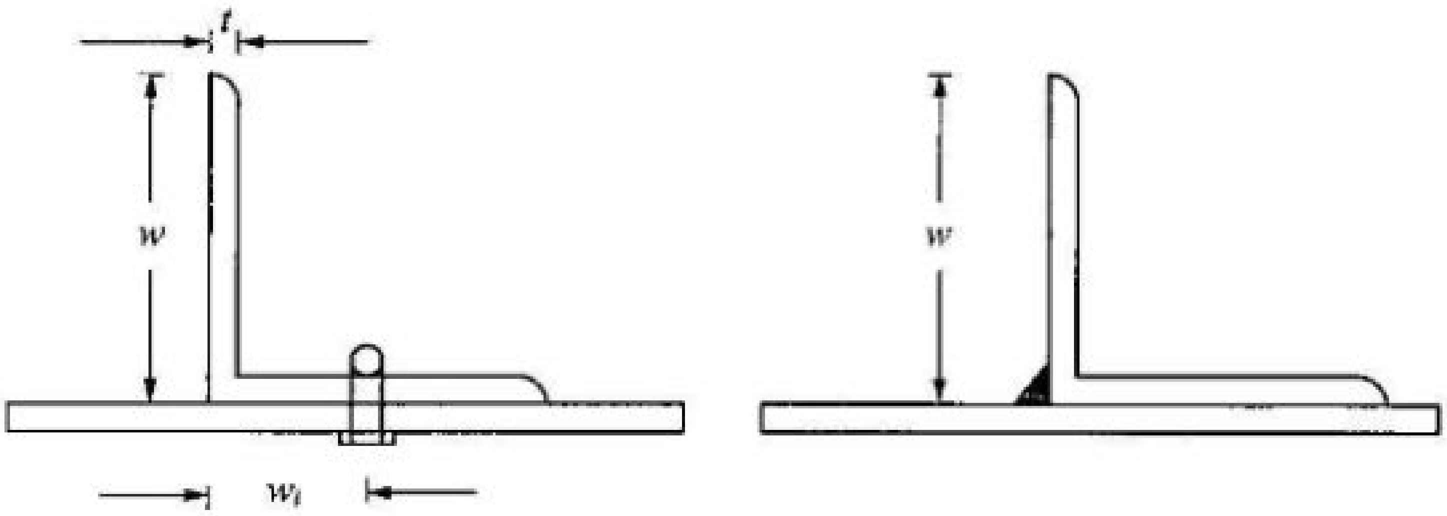
$$T_{dg} = \frac{A_g f_y}{\gamma_{mo}}$$

2. Design strength due to rupture of critical section:- [cls 6.3 IS 800-2007] (Ultimate)

$$T_{dn} = \frac{0.9\, Anfu}{\gamma_{ml}}$$

Where,

$$An = [b - nd_n + \sum_i \frac{p_{si}}{u_{gi}}]\, t$$



$b_s = w + w_l - t$ $b_s = w$

a) For Angular section design strength of rupture:-

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

Where,

$$\beta = 1.4 - 0.076\left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{l_c}\right) \leq \left(\frac{f_u\gamma_{mo}}{f_y\gamma_{ml}}\right) \geq 0.7$$

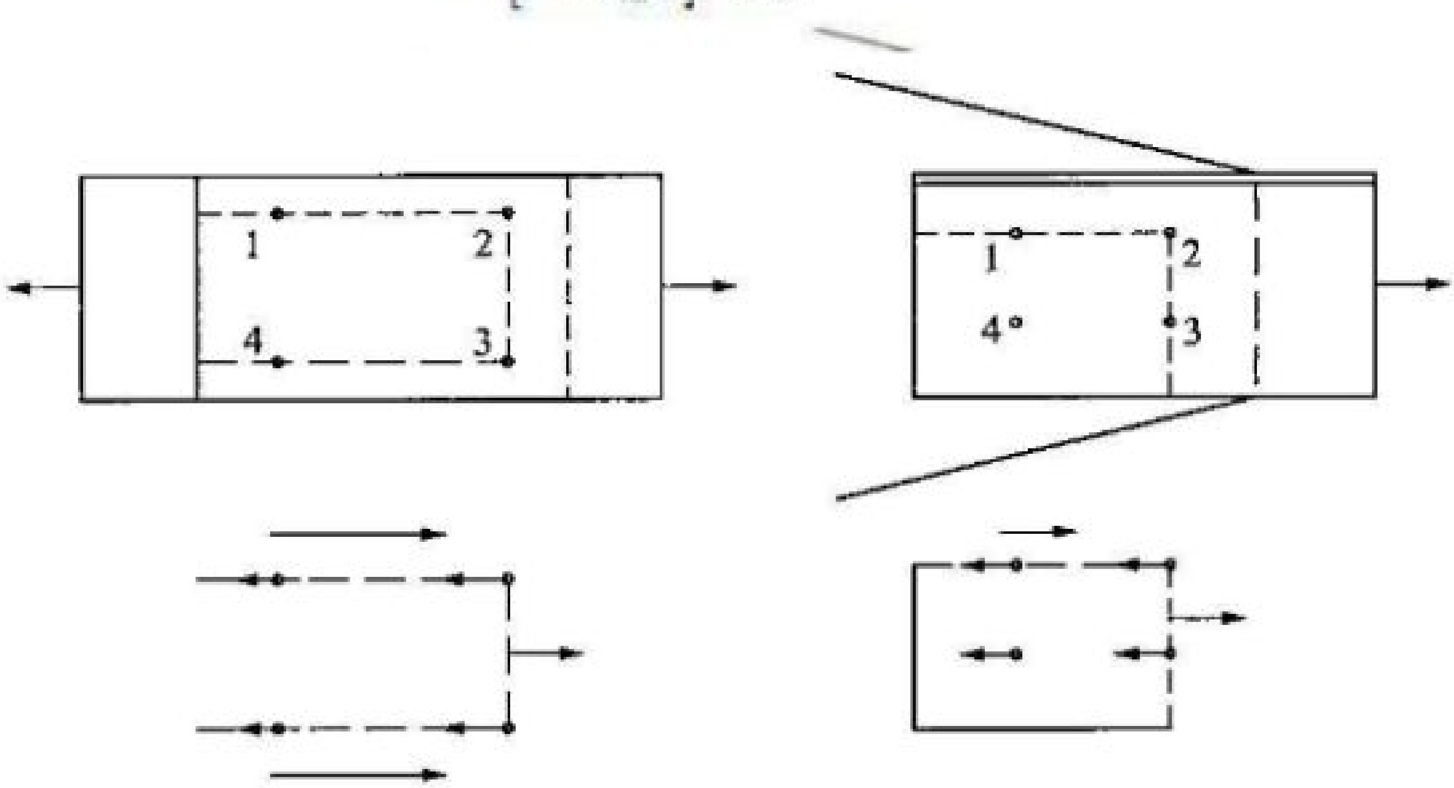
3. Design strength of member due to block shear failure @ the end connection:- [cls 6.4 IS 800-2007]

a) Bolted Connections:- [cls 6.4.1]

$$T_{db} = \left[\frac{A_{vg}f_y}{\sqrt{3}\gamma_{mo}}\right] + \frac{0.9A_{tn}f_u}{\gamma_{ml}}$$

(or)

$$T_{db} = \left[\frac{0.9A_{vn}f_u}{\sqrt{3}\gamma_{ml}}\right] + \frac{A_{tg}f_y}{\gamma_{mo}}$$



(a) (b)

Prepared by Er.Durga M

b) Welded Connection:-

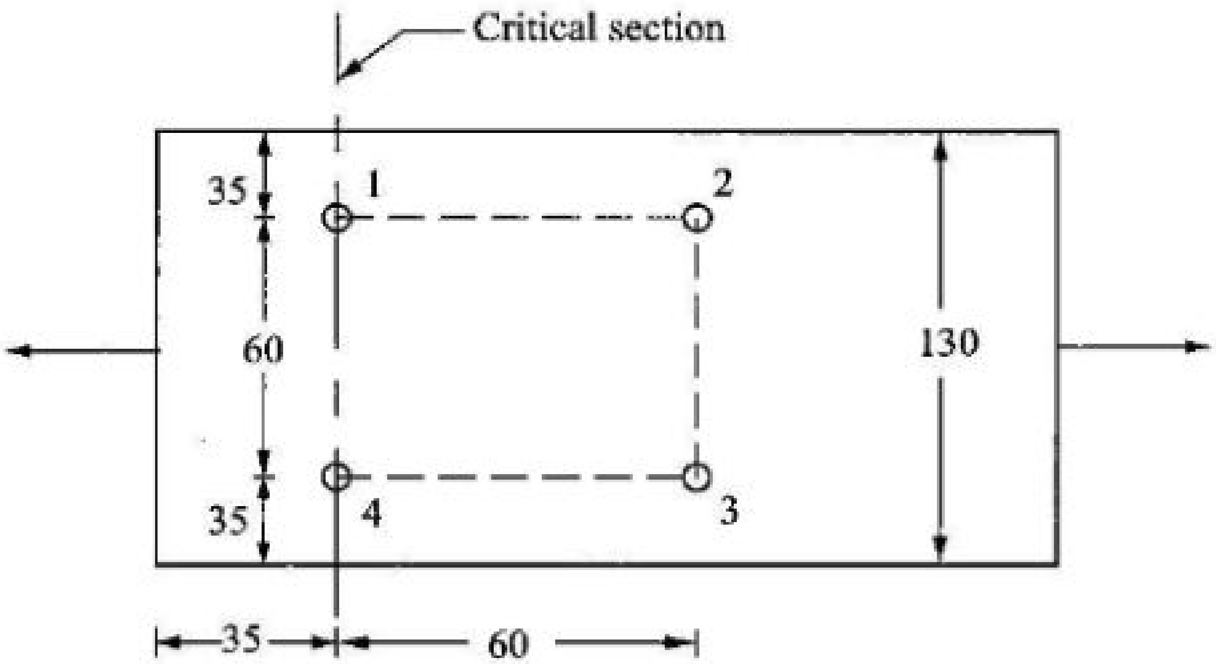
Appropriate length of member is considered around the end weld.

Preliminary section:- [cls 6.3.3 IS 800-2007]

Preliminary section is assumed from the relation is based on

$$T_{dn}=\frac{\alpha A_n f_u}{\gamma_{ml}}$$

1. Determine the design tensile strength of the plate of size 200x12mm with holes having bolts of dia 16mm (M16). The grade of steel used is Fe410.



Given:-

Size of plate = 200mm x 12mm
Dia of bolt = 16mm
Grade Fe410 => fu = 410 N/mm²
fy = 250 N/mm²

Sln:-

1. Design strength due to yielding:- [cls 6.2 IS 800-2007]

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

$$Ag = 130x12 = 1560mm^2$$

$$= \frac{1560\times250}{1.1\Rightarrow(table\,5)}$$

$$T_{dg}= 354.5\ KN$$

2. Design strength of plate @ rupture: [Along critical section] [cls 6.3 IS 800-2007]

The critical section is along the line having 2 bolts

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

$$A_n=[b-nd_n]t$$

n = 2

b = 130mm

= [130-2x18]12

$$A_n = 1128mm^2$$

$$= \frac{0.9\times1128\times410}{1.25\Rightarrow[Table-5]}$$

$T_{dn} = 332.9$ KN

3. Design strength due to block shear:- [cls 6.4 IS 800-2007]

$$T_{db}=\left[\frac{A_{vg}f_y}{\sqrt{3}\gamma_{mo}}\right]+\left[\frac{0.9A_{tn}f_u}{\gamma_{ml}}\right]$$

(or)

$$T_{dn}=\left[\frac{0.9A_{vn}f_u}{\sqrt{3}\gamma_{mn}}\right]+\left[\frac{A_{tg}f_y}{\gamma_{mo}}\right]$$

Where,

Section considered for A_{vg} is (e+n'p) x t

Section considered for A_{tg} is (n'g) x t

Section considered for A_{vn} & A_{tn} is the net area after detecting the bolt hole.

A_{vg} = (35+60)12= 1140mm^2

A_{tg} = 60x12 = 720mm^2

A_{vn} = [35+60-18] x 12 = 924mm^2

A_{tn} = [60-18] 12 = 504mm^2

$$T_{db_1}=\left[\frac{1140\times250}{\sqrt{3}\times1.1}\right]+\left[\frac{0.9\times504\times410}{1.25}\right]$$

T_{db_1} = 298.36 KN

(or)

$$T_{db}=\left[\frac{0.9\times924\times410}{\sqrt{3}\times1.25}\right]+\left[\frac{720\times250}{1.1}\right]$$

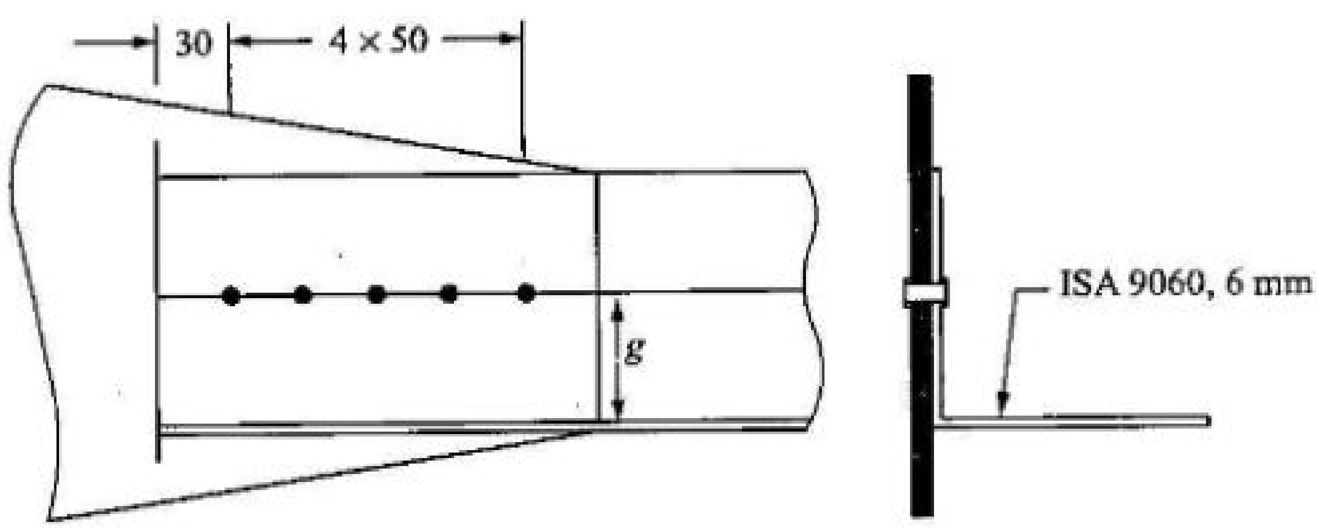
T_{db_2}=321.12 KN

∴ The least of the above 4 strength value is the design strength of the plate.

∴ Design strength of the plate = 298.36 KN

1. A single unequal angle ISA 90x60x6mm is connected to a 10mm tk gusset plate at the ends with 5 Nos of 16mm dia bolts to transfer tension. Determine the design tensile strength of the angle if the gusset is connected to the 90mm leg.

Given:-



g =50 mm, if 90 mm leg is connected
=30 mm, if 60 mm leg is connected

Unequal angle = ISA 90x60x6
Tks of gusset plate = 10mm
φ of bolt = 16mm
Nos of bolt = 5 Nos.

Sln:-

1. Design strength of angle in yielding:- [cls 6.2 IS 800-2007]

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

$$A_g=\left[\left(90-\frac{6}{2}\right)+\left(60-\frac{6}{2}\right)\right]\times 6$$

$$A_g=864\text{mm}^2$$

$$=\frac{864\times 250}{1.1}$$

$$T_{dg}=196.36\,KN$$

2. Design strength of angle against rupture [cls 6.3.3 IS 800-2007]

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

$$\beta=1.4-0.076\left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L_c}\right)\leq\frac{f_u\gamma_{mo}}{f_y\gamma_{ml}}\geq 0.7$$

Where,

A_n=> Net area of the connected leg = (90-6/2-18) 6
A_n=414mm^2
A_g=> Gross area of the outstanding leg = (60-6/2) x 6
A_g = 342mm^2
w => outstanding leg width = 60mm
t => tks of angle = 6mm
b_s=> shear lag width = w+w_1-t
Assume, w_1=90/2=45mm $\simeq$ 50mm
Provide w_1 = 50mm
b_s=60+50-6

b_s = 104mm
L_c=> length of the end connection = 90mm

$$=\left[1.4-0.076\times\frac{60}{6}\times\frac{250}{410}\times\frac{104}{90}\right]$$

$$¿\frac{410\times 1.1}{250\times 1.25}\geq 0.7$$

$$\beta=0.864\leq 1.44\geq 0.7$$

Which is true [0.7 ¿ 0.864 ¿ 1.44]

$$\therefore\beta=0.864$$

$$T_{dn}=\frac{0.9\times 414\times 410}{1.25}+\frac{0.864\times 342\times 250}{1.1}$$

T_{dn} = 189.369 KN

3. Design strength of plate against block shear of end connection: [cls 6.4 IS 800-2007]

$$T_{db}=\frac{A_{vg}f_y}{\sqrt{3}\gamma_{mo}}+\frac{0.9A_{tn}f_u}{\gamma_{ml}}$$

(or)

$$T_{db}=\frac{0.9A_{vn}f_u}{\sqrt{3}\gamma_{ml}}+\frac{A_{tg}f_y}{\gamma_{mo}}$$

Where,

A_{vg}= (30+4x50)6=1380mm²

A_{vn}= [230-(4.5x18)]6=894mm²

Block shear failure takes place along line 1 to 3

'A_{vg}' is found along line 1-2

A_{vg} = (30+4x50)6=1380mm²

A_{tg} is taken along line 2-3

$$A_{tn}=\left[40-\frac{1}{2}\times 18\right]6$$

A_{tn} = 186mm²

$$T_{db}=\frac{1380\times 250}{\sqrt{3}\times 1.1}+\frac{0.9\times 186\times 410}{1.25}$$

$$T_{db}=235.98\,KN$$

(or)

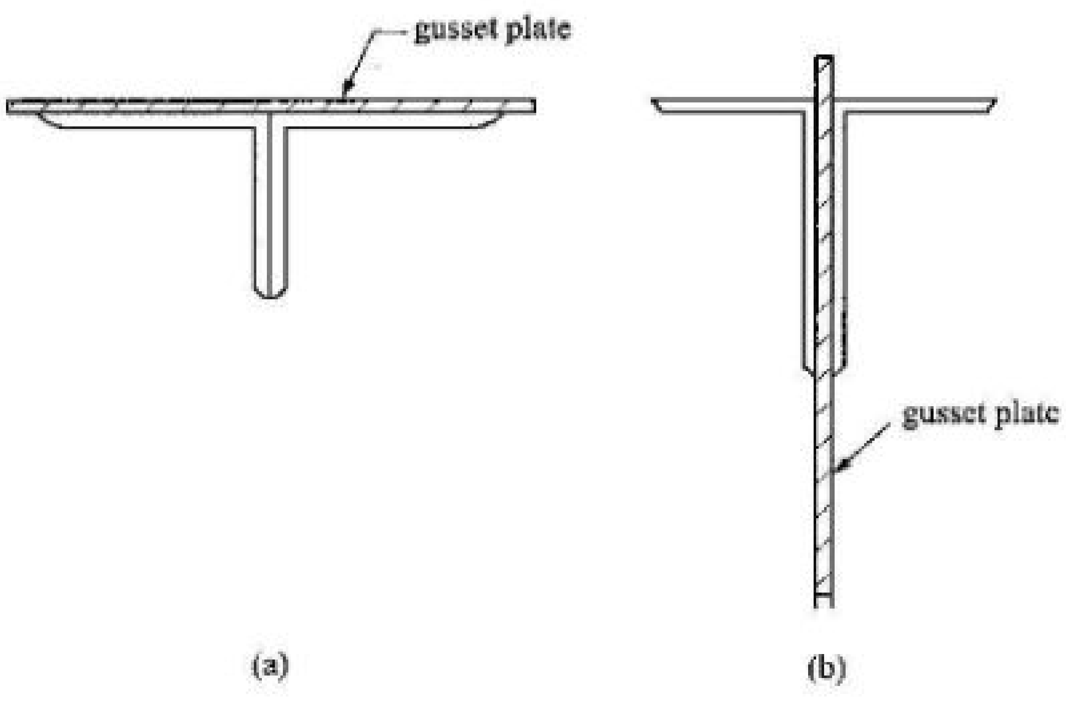
$$T_{db}=\frac{0.9\times 894\times 410}{\sqrt{3}\times 1.25}+\frac{240\times 250}{1.1}$$

T_{db} = 206.9 KN

∴ The least of strength of section in yielding, rupture and block shear is the design strength of the section.

∴ Design strength of the section = 189.369 KN

3. Find the design strength if the 60mm side is connected to the gusset plate as in the above problem.



Sln:-

Here the 60mm side is connected to gusset plate.

∴ Assume the line of bolts to be placed at a distance 60/2 = 30mm

1. Design strength of angle in yielding:- [cls 6.2 IS 800-2007]

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

$$A_g=\left[\left(90-\frac{6}{2}\right)+\left(60-\frac{6}{2}\right)\right]\times 6$$

$$A_g = 864\text{mm}^2$$

$$= \frac{864\times 250}{1.1}$$

$$\boxed{T_{dg} = 196.36\text{ KN}}$$

2. Design strength of angle against rupture [cls 6.3.3 IS 800-2007]

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

$$\beta=1.4-0.076\left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L_c}\right)\leq\frac{f_u\gamma_{mo}}{f_y\gamma_{ml}}\geq 0.7$$

Where, [cls 6.3.3 IS 800-2007]

$$A_{nc}=\left(60-\frac{6}{2}-18\right)6$$

$$= 234\text{mm}^2$$

$$A_g=\left(90-\frac{6}{2}\right)\times 6$$

$$= 522\text{mm}^2$$

w => 90mm

t => 6mm

$b_s = w+w_1-t$

$= 90+30-6=114$mm

$b_s = 114$mm

$L_c = 60$mm

$$\beta=1.4-0.076\left(\frac{90}{6}\right)\left(\frac{250}{410}\right)\left(\frac{114}{60}\right)\leq\left(\frac{410\times 1.1}{250\times 1.25}\right)\geq 0.7$$

$$\beta=0.079\leq 1.44\geq 0.7$$

Max. limit for β is $\frac{f_u\gamma_{mo}}{f_y\gamma_{ml}}$

∴ Provide $\beta = 0.7$

$$\therefore\ T_{dn}=\frac{0.9\times 234\times 410}{1.25}+\frac{0.7\times 522\times 250}{1.1}$$

$$\boxed{T_{dn} = 152.12\text{ KN}}$$

3. Design strength of plate against block shear:- [cls 6.4 IS 800-2007]

$$T_{db}=\frac{A_{vg}f_y}{\sqrt{3}\gamma_{mo}}+\frac{0.9A_{tn}f_u}{\gamma_{ml}}$$

(or)

$$T_{db}=\frac{0.9A_{vn}f_u}{\sqrt{3}\gamma_{ml}}+\frac{A_{tg}f_y}{\gamma_{mo}}$$

Where,

$$A_{vg}=[30+(4\times 50)]6$$

$$A_g=1380\,mm^2$$

$$A_{tg}=30\times 6=180\,mm^2$$

$$A_{tn}=\left[30-\frac{18}{2}\right]\times 6$$

$$A_{tn}=126mm^2$$

$$A_{vn}=[230-(4.5\times 18)]6$$

$$A_{vn}=894\,mm^2$$

$$T_{db}=\frac{1380\times 250}{\sqrt{3}\times 1.1}+\frac{0.9\times 126\times 410}{1.25}$$

$$\boxed{T_{db}=218.27\text{ KN}}$$

$$T_{db}=\frac{0.9\times 894\times 410}{\sqrt{3}\times 1.25}+\frac{180\times 250}{1.1}$$

$$\boxed{T_{db}=193.27\text{ KN}}$$

∴ Design strength of the section = 152.12 KN

DESIGN OF TENSION MEMBER:-

Design Procedure:-

1. Find the reqd gross area to carry the factored load considering the strength at yielding.

$$\boxed{A_g=\frac{1.1T_u}{f_y}}$$

2. Select suitable section depending upon the type of structure & location of member such that the gross area is 25 to 40% [generally 30%] more than 'A_g' calculated.

3. Determine the no. of bolts are length of weld reqd and arrange them appropriately. [design of connection]

4. Find the strength of the assumed section considering
   (i) Strength of section in yielding of gross area
   (ii) Strength of section in rupture of critical section.
   (iii) Strength of section against block shear at the end of connection.
5. The strength of section obtained [Design strength of section] should be more than a factored tensile force ting on the section. If not, the section has to be revised and redesign the section.
6. The slenderness ratio has to be check for the tension member, as per table-3, IS 800-2007 [Pg.No:20]

Slenderness ratio, $\lambda = \frac{l_{eff}}{\gamma_{min}}$

Where,

γ_{min} => The least of γ_{xx} & γ_{yy} of the section. [from steel table]

1. Design a single angle section for tension member of a roof truss to carry a factored load of 225KN. The member is subjected to possible reversal of stress due to the action of wind. The length of the member is 3m. Use 20mm shop bolts of grade 4.6 for the connection.

Given:-

T_u= 225 KN
d = 20mm

$f_y = 400 \, N/mm^2$

Grade 4.6 => $f_y = 250 \, N/mm^2$

$d_o = 22 \, mm$

Sln:-

$n = \frac{T_u}{v}$

Required area, $A_g = \frac{1.1T_u}{f_y}$

$= \frac{1.1 \times 225 \times 10^3}{250}$

$A_g = 990 \, mm^2$

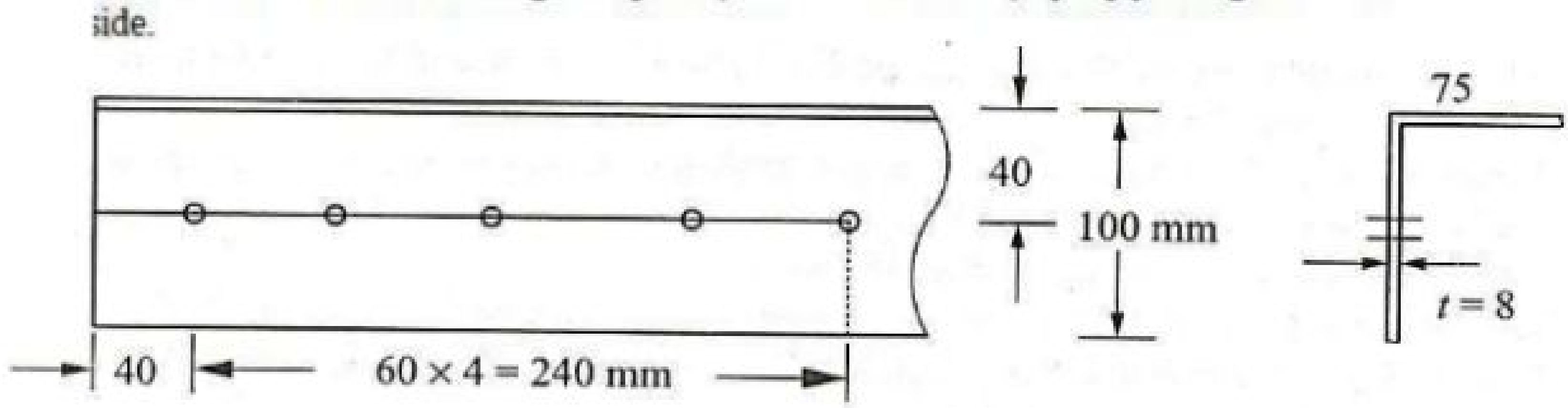
To select ISA 100x75x8mm

$A_g = 1336 \, mm^2$ [from steel table]

γ_{xx}=31.4mm γ_{yy} = 21.8mm

γ_{min} = 21.8mm

$\lambda = 21.8$ is connected to the gusset plate (assumed tks 10mm) by lap jt along the 100mm side.



BOLT VALUE:-[M20]

(i) Strength of bolt in single shear:- [cls 10.3.3 IS 800-2007]

$$V_{dsp} = \frac{V_{nsp}}{\gamma_{mb}}$$

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

$$V_{dsp} = \frac{f_u}{\sqrt{3}} \frac{(n_n A_{nb} + n_s A_{sb})}{\gamma_{mb}} \leftarrow n_s = 0 \quad f \rightarrow ss$$

$$= \frac{400}{\sqrt{3}}\left[1 \times \frac{\pi \times 20^2 \times 0.78}{4}\right]$$

1.25Table5 – IS 800-2007

$$\boxed{V_{dsp} = 45.27\ KN}$$

(ii) Strength of the bolt in bearing:- [cls 10.3.4 IS 800-2007]

$$V_{dsp} = \frac{V_{nbp}}{\gamma_{mb}} = \frac{2.5 k_b d_t f_u}{\gamma_{mb}}$$

$$k_b = \frac{e}{3d_o}, \frac{p}{3d_o} - 0.25, \frac{f_{ub}}{f_u}, 1$$

Assume, $e = 1.5\, d_o = 1.5 \times 22 = 33mm$ ¿ 40mm

$p = 2.5d = 2.5 \times 20 = 50mm$ ¿ 60mm

$$k_b \Rightarrow \frac{40}{3 \times 22}, \frac{60}{3 \times 22} - 0.25, \frac{400}{410}, 1$$

$$= 0.606, 0.659, 0.975, 1$$

$\therefore$ $k_b = 0.606$ (least value)

$$V_{dbp} = \frac{2.5 \times 0.606 \times 20 \times 8 \times 410}{1.25}$$

$$\boxed{V_{dbp} = 79.5\ KN}$$

$\therefore$ Design strength of bolt value = 45.27 KN

$\therefore$ No. of bolts, $n = \frac{T_u}{V}$

$$= \frac{225}{45.27}$$

$= 4.97$ ¿ 5 Nos.

$\therefore$ Provide 5 Nos of 20mm dia bolts pitch 60mm and the edge distance 40mm.

Check for strength of section:-

1. Strength of section against yielding:- [cls 6.2 IS 800-2007]

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

$$A_g = \left[\left(100 - \frac{8}{2}\right) + \left(75 - \frac{8}{2}\right)\right] \times 8$$

$$A_g = 1336\, mm^2$$

$$= \frac{1336 \times 250}{1.1}$$

$$\boxed{T_{dg} = 303.636\ KN}$$

2. Design strength of the section against rupture:- [cls 6.3.3 IS 800-2007]

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

Where,

$$\beta=1.4-0.076\left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L_c}\right)\leq\frac{f_u\gamma_{mo}}{f_y\gamma_{ml}}\geq0.7$$

$$A_{nc}=\left[100-22-\frac{8}{2}\right]8$$

$$A_{nc}=592\,mm^2$$

$$A_{go}=\left(75-\frac{8}{2}\right)8$$

$$A_{go}=568\,mm^2$$

w = 75mm

t = 50mm

$b_s = w+w_1-t$

= 75+50-8

b_s = 117mm

L_c = 100mm

$$\beta=1.4-0.076\left(\frac{75}{8}\right)\left(\frac{250}{410}\right)\left(\frac{117}{100}\right)\leq\left(\frac{410\times1.1}{250\times1.25}\right)\geq0.7$$

$$=0.89\leq1.44\geq0.7$$

$$\beta=0.07\leq0.89\leq1.44$$

$$\therefore\beta=0.89$$

$$T_{dn}=\frac{0.9\times592\times410}{1.25}+\frac{0.89\times568\times250}{1.1}$$

$$\boxed{T_{dn}=289.6\text{ KN}}$$

> 225 KN

3. Design strength of plate against block shear:- [cls 6.4 IS 800-2007]

$$T_{db}=\frac{A_{vg}f_y}{\sqrt{3}\gamma_{mo}}+\frac{0.9A_{tn}f_u}{\gamma_{ml}}$$

(or)

$$T_{db}=\frac{0.9A_{vn}f_u}{\sqrt{3}\gamma_{ml}}+\frac{A_{tg}f_y}{\gamma_{mo}}$$

Where,

$$A_{vg}=[40+(4\times60)]\times8$$

$$A_{vg}=2240\,mm^2$$

$$A_{tg}=50\times8$$

$$A_{tg}=400\,mm^2$$

$$A_{vn}=[280-(4.5\times22)]8$$

$$A_{vn}=1448\,mm^2$$

$$A_{tn}=\left[50-\frac{22}{2}\right]\times8$$

$$A_{tn}=312\,mm^2$$

$$T_{db_1}=\left[\frac{2240\times250}{\sqrt{3}\times1.1}\right]+\left[\frac{0.9\times312\times410}{1.25}\right]$$

$$\boxed{T_{db_1}=386.026\text{ KN}}$$

$$T_{db}=\left[\frac{0.9\times1448\times410}{\sqrt{3}\times1.25}\right]+\left[\frac{400\times250}{1.1}\right]$$

$$\boxed{T_{db_2}=337.697\ KN}$$

The above 2 values of strength against block shear 337.697 KN > 225KN

The strength of the section against yielding, rupture & block shear are greater than the external load of 225KN.

∴ The assume section ISA 100x75x8mm is safe.

2. Solve the above problem using angle section on opposite sides of gusset plate

Given:-

T_u= 225KN

d = 20mm

d_o= 22mm

Grade 4.6 => f_u= 400 N/mm²

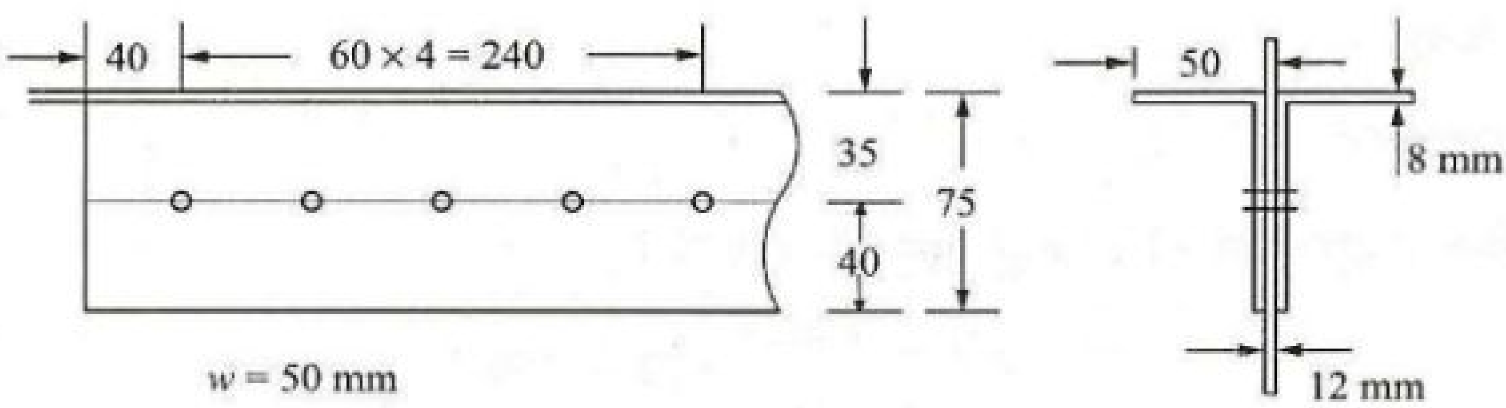
f_y= 250 N/mm²

$$\lambda act=\frac{l_{ef}}{\gamma_{min}}$$

$\lambda max = 350$ *table* 3

$\lambda act < \lambda max$

Sln:-



$w = 50$ mm

$b_s = 50 + 30 - 10 = 70$ mm

To find A_g:-

$$A_g=\frac{1.1T_u}{f_y}$$

$$=\frac{1.1\times225\times10^3}{250}$$

$$A_g=990\,mm^2$$

Area each angle reqd = 990/2 = 495mm²

∴ Select the section from steel table having area 30% more than 495mm²

Try ISA 70x70x5mm

A_s=667mm² [from steel table]

$\gamma_{xx} = 21.5mm$

$\gamma_{yy} = 21.5mm$

Bolt Value:- [M20]

(i) Strength of bolt in double shear:-

Assuming gusset plate of tks = 10mm

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

$$V_{nsp} = \frac{f_u}{\sqrt{3}}\left[n_n A_{nb} + n_s A_{sb}\right]$$

$$n_n = n_s = 1$$

$$A_{nb} = \frac{0.78 \times \pi \times 20^2}{4}, A_{sb} = \frac{\pi \times 20^2}{4} = 314.16mm^2$$

$$V_{dsb} = \frac{\frac{400}{\sqrt{3}}\left[1 \times 245 + 1 \times 314.16\right]}{1.25}$$

$$\boxed{V_{dsb} = 103.3\,KN}$$

(ii) Strength of bolt in bearing:-

$$V_{dbp} = \frac{V_{nbp}}{\gamma_{mb}}$$

$$= \frac{2.5 k_b d_t f_u}{\gamma_{mb}}$$

Assume e=1.5 d_o= 33mm ¿ 40mm

p =2.5d = 50mm ¿ 60mm

$$k_b = \frac{40}{3 \times 22}, \frac{60}{3 \times 22} - 0.25, \frac{400}{410}, 1$$

= 0.606, 0.659, 0.975, 1

∴ Take k_b = 0.606 [least value]

$$V_{dbp} = \frac{2.5 \times 0.606 \times 20 \times 10 \times 410}{1.25}$$

$$\boxed{V_{dbp} = 99.38KN}$$

∴ Design strength of bolt value = 99.38KN

∴ No. of bolts = 225/99.38

= 2.26 ¿ 3 Nos.

∴ Provide 3 nos. of 20mm bolts for pitch 60mm & tks edge distance 40mm.

Check for strength of section:-

1. Strength of section against yielding:- [cls 6.2 IS 800-2007]

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

$$A_g=\left[\left[\left(70-\frac{5}{2}\right)+\left(70-\frac{5}{2}\right)\right]\times 5\right]\times 2$$

$$A_g=1334\,mm^2$$

$$=\frac{1334\times 250}{1.1}$$

$$\boxed{T_{dg}=303.18\,KN} > 225KN$$

2. Strength of section against rupture:- [cls 6.3.3 IS 800-2007]

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

Where,

$$\beta=1.4-0.076\left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L_c}\right)\leq\frac{f_u\gamma_{mo}}{f_y\gamma_{ml}}\geq 0.7$$

$$A_{nc}=\left[70-22-\frac{5}{2}\right]\times 5\times 2$$

$$A_{nc}=456\,mm^2$$

$$A_{go}=\left(70-\frac{5}{2}\right)\times 5\times 2$$

$$A_{go}=675\,mm^2$$

w = 70mm

w_1= 35mm ¿ 40mm

b_s= w+w_1-t

= 70+40-10

=100

$$\beta=1.4-0.076\left(\frac{70}{100}\right)\left(\frac{250}{410}\right)\left(\frac{100}{160}\right)\leq\left(\frac{410\times 1.1}{250\times 1.25}\right)\geq 0.7$$

$$=1.89\leq 1.44\geq 0.7$$

$$\beta=0.7\leq 1.19\leq 1.44$$

$$\therefore \beta=1.19$$

$$T_{dn}=\frac{0.9\times 456\times 410}{1.25}+\frac{1.19\times 675\times 250}{1.1}$$

$$\boxed{T_{dn}=316.87\,KN}$$

> 225 KN

3. Design strength of plate against block shear: [cls 6.4 IS 800-2007]

$$T_{db}=\frac{A_{vg}f_y}{\sqrt{3}\gamma_{mo}}+\frac{0.9A_{tn}f_u}{\gamma_{ml}}$$

(or)

$$T_{db}=\frac{0.9A_{vn}f_u}{\sqrt{3}\gamma_{ml}}+\frac{A_{tg}f_y}{\gamma_{mo}}$$

A_{vg} & A_{vn} are found along section 1-2 and
A_{tg} & A_{tn} are found along section 2-3

$$A_{vg} = [(40+2(60))5]2$$
$$= 1600mm^2$$
$$A_{vn} = [(40+2(60)-2.5(22))5]2$$
$$= 1050mm^2$$
$$A_{tg} = 30\times5\times2 = 300\,mm^2$$
$$A_{tn} = [(30-0.5(22))5]2$$
$$= 190mm^2$$
$$T_{db_1} = \frac{1600\times250}{\sqrt{3}\times1.1} + \frac{0.9\times190\times410}{1.25}$$

$$\boxed{T_{db_1} = 266.03\ KN}$$

For 2 angles

$$T_{db_2} = \frac{0.9\times1050\times410}{\sqrt{3}\times1.25} + \frac{300\times250}{1.1}$$

$$\boxed{T_{db_2} = 247.137\ KN}$$

For 2 angles

T_{db}= 247.137KN [least value of these two]

Hence 2 nos. of ISA 70x70x5mm is safe against yielding, rupture & block shear conditions.

TENSION SPLICE:-

- When a single piece of reqd length is not available, for a tension member, splice plates are used to transverse the reqd tension force from 1 piece to another.
- The strength of the splice plates & the bolts connecting them should have strength atleast equal to a design load.

1. Design a splice to connect a plate of size 300x20mm width a plate of size 300x10mm. The design load is 500KN. Use 20mm block bolts fabricated in the shop. Provide a double cover butt joint with tks of cover as 10mm.

Given:-

1. Plate of size = 300x20mm
2. Plate of size = 300x10mm

Tks of cover plate = 6mm

d = 20mm

d_o =22mm

Design load = 500KN

Sln:-

Since plates have varying tks need to be provided packing plate is reqd to provide the two cover plates.

The bolts are under double shear.

1. Strength of bolt in double shear:- [cls 10.3.3 IS 800-2007]

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

$$V_{nsp}=\frac{f_u}{\sqrt{3}}[n_n A_{nb}+n_s A_{sb}]$$

$$n_n=n_s=1$$

$$A_{nb}=\frac{0.78\times\pi\times 20^2}{4}=245\,mm^2$$

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

$$\beta_{pk}=[1-0.0125\,tpk]$$

$$=[1-[0.0125\times 10]]$$

$$\beta_{pk}=0.875$$

$$V_{nsb}=\frac{400}{\sqrt{3}}[1\times 245+1\times 314.16]\times 0.875$$

$$=112.99\text{ KN}$$

$$V_{dsb}=\frac{112.99}{1.25}$$

$$V_{dsb}=90.392\text{ KN}$$

2. Strength of bolt in bearing:- [cls 10.3.4 IS 800-2007]

$$V_{dbp}=\frac{V_{nbp}}{\gamma_{mb}}$$

$$=2.5k_b d_t f_u$$

Assume e=1.5 d_o= 33mm ¿ 40mm

p =2.5d = 50mm ¿ 60mm

$$k_b=\frac{40}{3\times 22},\frac{60}{3\times 22}-0.25,\frac{400}{410},1$$

= 0.606, 0.659, 0.975, 1

∴ Take k_b = 0.606 [least value]

$$V_{dbp}=2.5\times 0.606\times 20\times 10\times 410$$

= 124.23 KN

$$V_{dbp}=99.38\text{KN}$$

∴ Design strength of bolt value = 90.39 KN

∴ No. of bolts = $\frac{T_u}{v}$

$$=\frac{500}{90.39}$$

$$n=5.5\simeq 6\,Nos.$$

∴ Provide 6 nos. of 20mm bolts on each side

Providing the 6 bolts on each side of the connecting plate, it can be arrange along 2 vertical rows with 3 bolts on each vertical row as shown in fig.

Check for strength of section:-

1. Strength of the plate against yielding:- [cls 6.2 IS 800-2007]

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

$A_g=300\times 10=3000\,mm^2$ [Tks of thinner plate]

$$=\frac{3000\times 250}{1.1}$$

$$T_{dg}=681.81\text{ KN}>500\text{KN}$$

2. Strength of the plate against rupture:- [cls 6.3.1 IS 800-2007]

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

A_n= The critical section where carrying of plate is occurs along the vertical line passing through the 3 bolts.

= [300-3x(22)]x10

A_n = 2340mm²

$$= \frac{0.9 \times 2340 \times 410}{1.25}$$

$$T_{dn} = 690.77\,KN > 500\,KN$$

3. Strength of the plate against block shear:- [cls 6.4 IS 800-2007]

$$T_{db} = \frac{A_{vg} f_y}{\sqrt{3}\gamma_{mo}} + \frac{0.9A_{tn} f_u}{\gamma_{ml}}$$

(or)

$$T_{db} = \frac{0.9A_{vn} f_u}{\sqrt{3}\gamma_{ml}} + \frac{A_{tg} f_y}{\gamma_{mo}}$$

The block shear failure takes place along the lines 1,2,3,4 as shown in fig. [The path of block shear failure is given in fig:7 IS 800-2007]

A_{vg} & A_{vn} are found along section 1-2 and

A_{tg} & A_{tn} are found along section 2-3

$$A_{vg} = [40+60] \times 10 = 1000\,mm^2$$

$$A_{vn} = [(40+60) - 1.5(22)]10$$

= 670mm²

$$A_{tg} = [2 \times 110]10 = 2200\,mm^2$$

$$A_{tn} = [2(110) - 2(22)]10$$

= 1760mm²

$$T_{db_1} = \frac{1000 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 1760 \times 410}{1.25}$$

$$\boxed{T_{db_1} = 650.76\text{ KN}}$$

$$T_{db_2} = \frac{0.9 \times 670 \times 410}{\sqrt{3} \times 1.25} + \frac{2200 \times 250}{1.1}$$

$$T_{db_2} = 614.190\,KN > 500\,KN$$

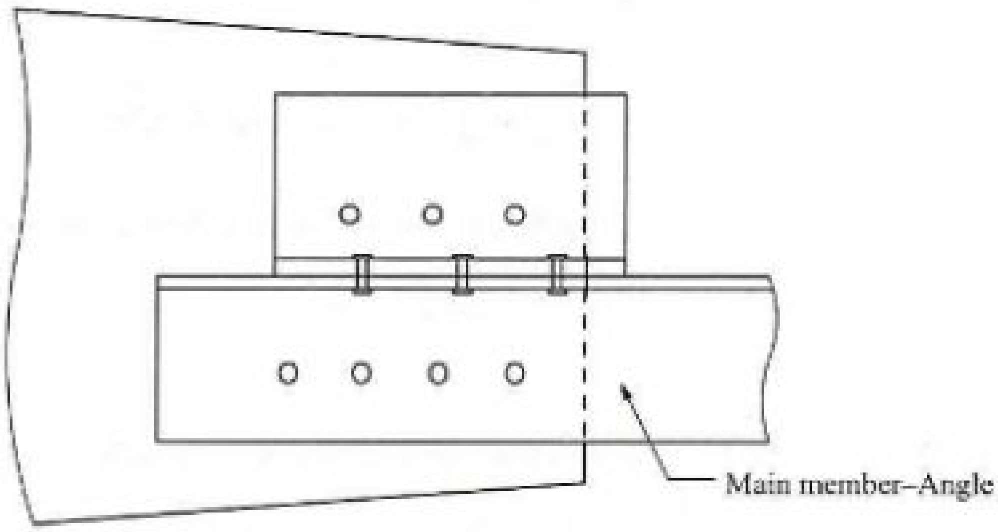
Hence the connection is safe.

LUG ANGLES:-

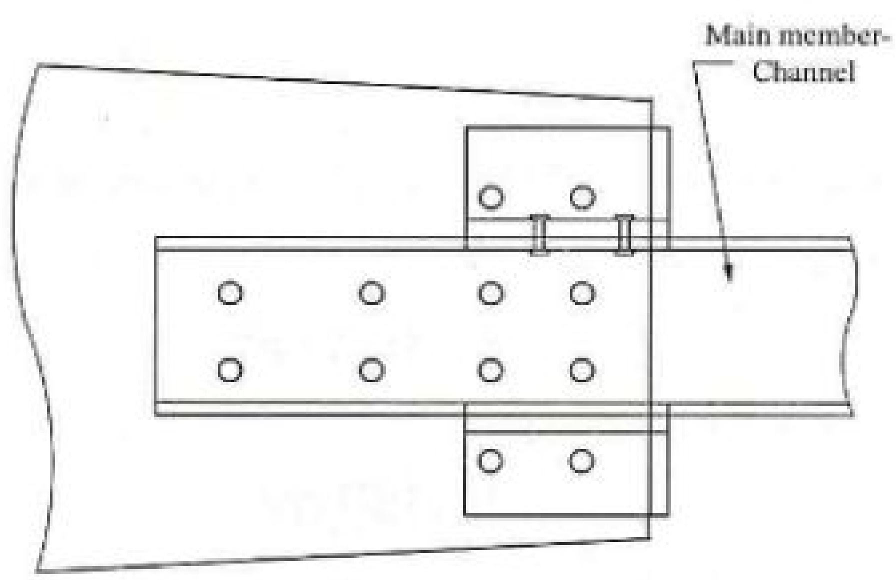
- The length of end connections of heavily loaded tension members may be reduced by using lug angles as shown in fig.
- There is savings in gusset plate but additional cost is incurred from the material of lug angles & the connections for the lug angles.
- The design of tension member with the use of lug angles needs to be check for the load which is share equally by the connected leg and the outstanding leg.

The following guidelines need to be satisfied.

1. The eff. Connection of the lug angle shall as for as possible.
2. It is preferable to start the lug angle in advance of a member connected.
3. A mini of 2 bolts or rivets, are provided.
4. In case of angles, the whole area can be taken rather than the net eff. Area.
5. In case of channels, the lug angles should be placed simitrical and the strength of fasterness connecting lug angle to the gusset be 10% more than the outstanding leg.
   [When main member is a channel]
6. In case of angle [Main member] the above values are 20% & 40% respectively.



(a)



(b)

1. Design a tension member of a roof truss which carries a factored axial tension of 430KN.

Design the connection when

(i) No lug angle is provided
(ii) Lug angle is provided

Hints:-

1. Without lug angle, the connections are designed for 'T_u' and member is check for design strength for 'T_u'.
2. When lug angle is provided, connection in main member is design for '$T_u/2$' and the connection in lug angle is design for '$T_u/2$', where the connection plate & lug angle is increased by 20% and connection b/w lug angle & main plate is increased by 40%

Given:-

$T_u = 430KN$

Sln:-

(i) No lug angle is provided:-

Assume, d = 20mm

d_o=22mm

Tks of gusset plate = 12mm

BOLT VALUE:- [M20]

$$A_g = \frac{1.1T_u}{f_y}$$

$$= \frac{1.1 \times 440 \times 10^3}{250}$$

$$A_g = 1892\,mm^2$$

Select a section from steel table having area 30% more than the reqd area.

Select ISA 110x110x12mm

$$A_g = 2502\,mm^2$$

$$\gamma_{xx} = \gamma_{yy} = 33.4\,mm$$

(i) Strength of bolt in single shear:- [cls 10.3.3 IS 800-2007]

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

$$V_{nsp} = \frac{f_u}{\sqrt{3}}[n_n A_{nb} + n_s A_{sb}]$$

$$n_n = 1, n_s = 0$$

$$A_{nb} = 245\,mm^2$$

$$= \frac{400}{\sqrt{3}}[1 \times 245]$$

$$V_{nsb} = 56.58\,KN$$

$$V_{dsb} = \frac{56.58}{1.25}$$

$$V_{dsb} = 45.264\,KN$$

(ii) Strength of bolt in bearing:- [cls 10.3.4 IS 800-2007]

$$V_{dbp} = \frac{V_{nbp}}{\gamma_{mb}}$$

$V_{nbp} = 2.5 k_b d_t f_u$

Assume e = 40mm

P = 60mm

$K_b = \frac{40}{3\times22}, \frac{60}{3\times22} - 0.25, \frac{400}{410}, 1$

$K_b = 0.606, 0.66, 0.959, 1$

$\therefore$ Take $K_b = 0.606$

$V_{dbp} = \frac{2.5\times0.606\times20\times12\times410}{1.25}$

$V_{dbp} = 119.26$ KN

$\therefore$ Design strength of bolt value = 45.264 KN

$\therefore$ No. of bolts = $\frac{T_u}{v}$

$= \frac{430}{45.264}$

= 9.49 ¿ 10 Nos.

$\therefore$ Provide 10 nos of 20mm dia to bolts edge distance 40mm & pitch of 60mm.

Check for strength of section:-

1. Strength of section against yielding:- [cls 6.2 IS 800-2007]

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

$A_g = \left[\left(110 - {}^{12}\!/_2\right) + \left(110 - {}^{12}\!/_2\right)\right]\times12$

$A_g = 2496 mm^2$

$= \frac{2496\times250}{1.1}$

$T_{dg} = 567.27$ KN > 430 KN

2. Strength of section against rupture:- [cls 6.3.3 IS 800-2007]

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

Where,

$\beta = 1.4 - 0.076\left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L_c}\right) \le \frac{f_u \gamma_{mo}}{f_y \gamma_{ml}} \ge 0.7$

$A_{go} = \left[110 - \frac{12}{2}\right]\times12$

$= 1248 mm^2$

$A = \left[110 - 22 - {}^{12}\!/_2\right]\times12$

$= 984 mm^2$

$w = 110\text{ mm}$

$w_1 = 60\text{mm}$

$b_s = w + w_1 - t$

$= 110+60-12$

$= 158\text{mm}$

$L_c = 580\text{ mm}$

$$\beta = 1.4 - 0.076\left(\frac{110}{12}\right)\left(\frac{250}{410}\right)\left(\frac{158}{580}\right) \leq \left(\frac{410 \times 1.1}{250 \times 1.25}\right) \geq 0.7$$

$= 1.28 \leq 1.44 \geq 0.7$

$\therefore \beta = 1.28$

$$T_{dn} = \frac{0.9 \times 984 \times 410}{1.25} + \frac{1.28 \times 1248 \times 250}{1.1}$$

$\boxed{T_{dn} = 653.53\text{ KN}}$

$> 430\text{ KN}$

3. Strength of the section against block shear:- [cls 6.4.1 IS 800-2007]

$$T_{db} = \frac{A_{vg} f_y}{\sqrt{3}\gamma_{mo}} + \frac{0.9 A_{tn} f_u}{\gamma_{ml}}$$

(or)

$$T_{db} = \frac{0.9 A_{vn} f_u}{\sqrt{3}\gamma_{ml}} + \frac{A_{tg} f_y}{\gamma_{mo}}$$

Where,

$A_{vg} = [40 + (9 \times 60)]12$

$= 6960\text{mm}^2$

$A_{tg} = [50 \times 12]$

$= 600\text{mm}^2$

$A_{vn} = [580 - (9.5 \times 22)]12$

$= 4452\text{mm}^2$

$A_{tn} = \left[50 - \frac{22}{2}\right]12$

$= 468\text{mm}^2$

$$T_{db_1} = \frac{6960 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 468 \times 410}{1.25}$$

$T_{db_1} = 997.5\text{ KN} > 430\text{KN}$

$$T_{db_2} = \frac{0.9 \times 4452 \times 410}{\sqrt{3} \times 1.25} + \frac{600 \times 250}{1.1}$$

$T_{db_2} = 895.13\text{ KN} > 430\text{KN}$

(ii) Lug Angle is Provided:-

- When lug angle is provided the member of bolts reqd for establishing the connection reduces thereby reducing the overall length of overlap.

- The connection b/w main member gusset plate is designed for $^{T_u}/_2$
- The lug angle is designed for a force of $^{T_u}/_2$ [Increased by 30%]
- The connection b/w the main member lug angle is designed for 40% of $^{T_u}/_2$ and connection b/w angle & gusset plate designed for 20% of $^{T_u}/_2$

Connection for Main Member:-

$$n = \frac{^{T_u}/_2}{v} = \frac{215}{45.26} \qquad [\because v = 45.26\,KN]$$

n = 4.75 ¿ 5 Nos

∴ Provide 5 nos of 20mm bolts.

Lug Angles:-

$$A_g = \frac{1.1 \times ^{T_u}/_2 \times 1.3}{f_y} = \frac{1.1 \times 215 \times 1.3 \times 10^3}{250} = 1230\text{mm}^2$$

Try ISA 80x80x12mm

$A_g = 1781\text{mm}^2$

$r_{xx} = r_{yy} = 23.9\,mm$

Connection b/w gusset plate & lug angle:-

$$\text{No. of bolts} = \frac{1.2 \times ^{T_u}/_2}{v} = \frac{1.2 \times 215}{45.26}$$

n = 5.7 ¿ 6 Nos

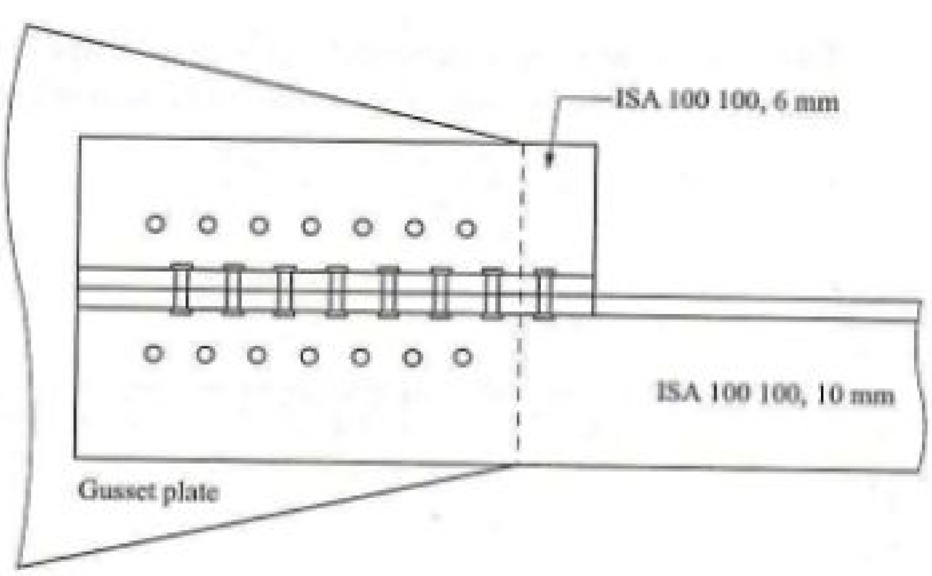
∴ Provide 6 nos of bolts b/w gusset plate and lug angle.

Connection b/w lug angle & main membe:-

$$n = \frac{1.4 \times ^{T_u}/_2}{v} = \frac{1.4 \times 215}{45.26}$$

n = 6.65 ¿ 7 Nos

∴ Provide 7 nos of bolts b/w lug angle and main member.



UNIT – IV Design of Compression Members.

Classes of sections:-

1. Column -> Stanchion
2. Truss -> Strut
3. Beam -> Girder

a) Class 1 [Plastic]:-

Cross sections, which can develop plastic hinges and have the rotation capacity reqd for failure of the structures by formation of plastic mechanism. The width to tks ratio of plate elements shall be less than that specified under class 1 (plastic) in table 21.

b) Class 2 [Compact]:-

Cross-sections which can develop plastic moment of resistance, but have inadequate plastic hinge rotation capacity for formation of plastic mechanism due to local buckling. The width to tks ratio of plate elements shall be less than that specified under class-2 (compact), but greater than that specified under class-1 (Plastic) in table 21.

c) Class 3 [Semi-Compact]:-

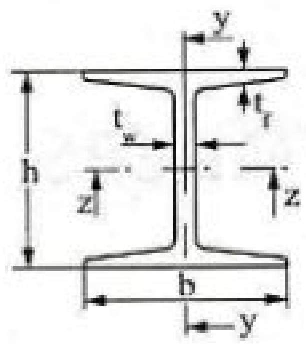
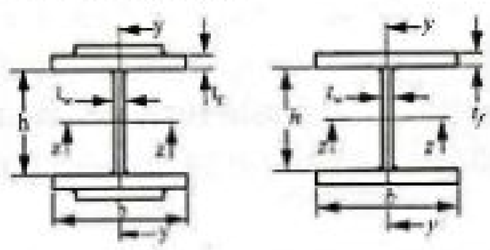
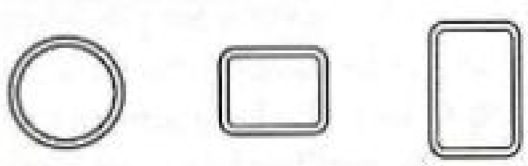
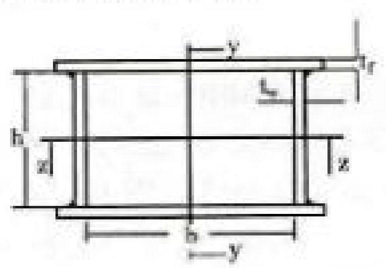
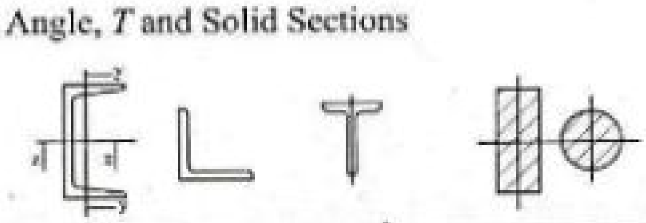
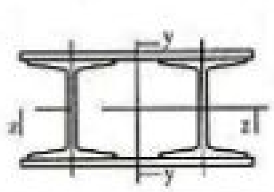
C/S in which the extreme fiber in compression can reach yield stress, but cannot develop the plastic moment of resistance, due to local buckling. The width to tks of plate element shall be less than that specified under class-3 (Semi-Compact) but greater than that specified under class-2 in table-21.

d) Class 4 [Slender]:-

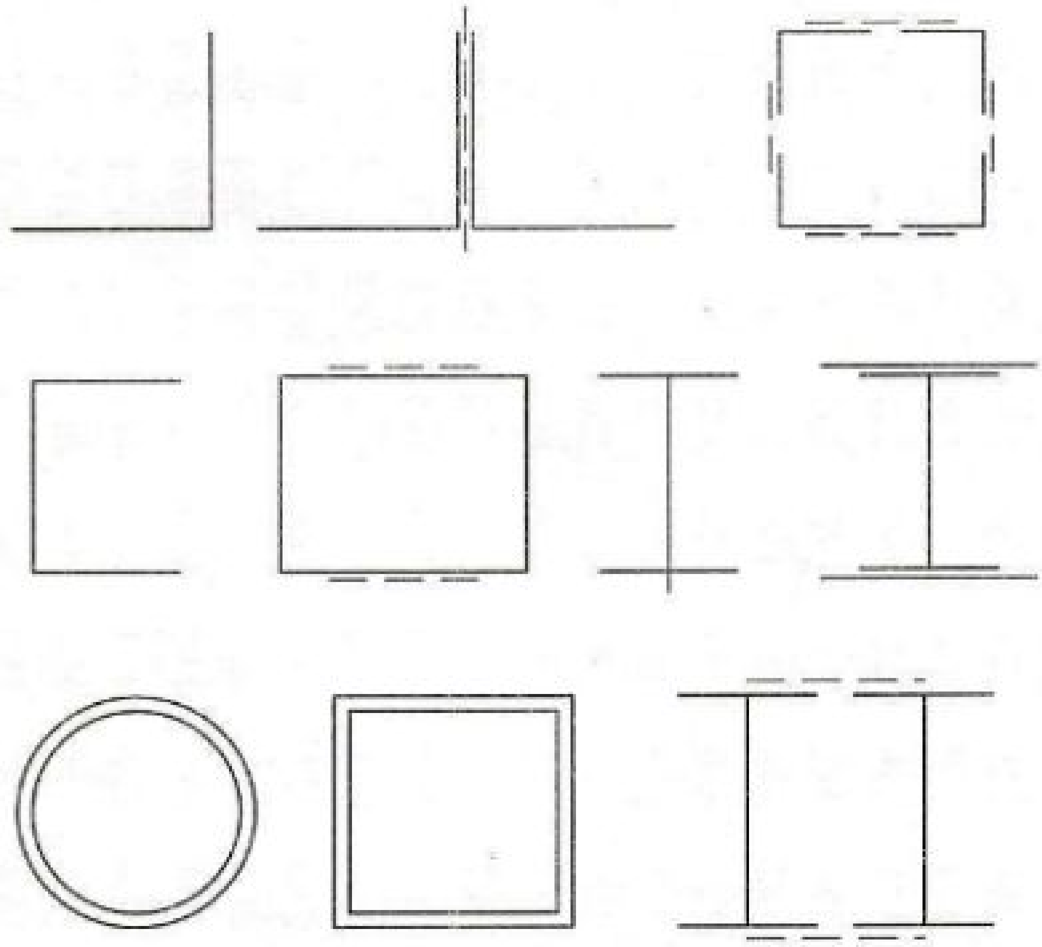
C/S in which the elements buckle locally even before reaching yield stress. The width to tks ratio of plate elements shall be greater than that specified under class-3 in table 21. In such cases, the eff. Sections for design shall be calculated either by following the provisions of IS 801 to account for the Post-local-buckling strength or by deducting width of the compression plate element in excess of the semi-compact section limit.

- Generally steel sections carrying axial compression fail by flexural buckling.
- The buckling strength of the compression members are affected by residual stresses, accidental eccentricities & slenderness ratio.
- To account for these factors the strength of members is subjected to axial compression defined by the above buckling classes 1,2,3&4 [Plastic, Compact, Semi-Compact & slender] given in table 10 IS 800-2007.

Table 6.1 Buckling class of cross-sections
[Refer Table 10 in IS 800]

Cross-Section (1)	Limits (2)	Buckling About Axis (3)	Buckling Class (4)
Rolled I-Sections 	$h/b_f > 1.2$: $t_f \leq 40$ mm	z-z	*a*
		y-y	*b*
	40 mm $< t_f \leq 100$ mm	z-z	*b*
		y-y	*c*
	$h/b_f > 1.2$: $t_f \leq 100$ mm	z-z	*b*
		y-y	*c*
	$t_f > 100$ mm	z-z	*d*
		y-y	*d*
Welded I-Section 	$t_f \leq 40$ mm	z-z	*b*
		y-y	*c*
	$t_f > 40$ mm	z-z	*c*
		y-y	*d*
Hollow Section 	Hot rolled	Any	*a*
	Cold formed	Any	*b*
Welded Box Section 	Generally (except as below)	Any	*b*
	Thick welds and $b/t_f < 30$	z-z	*c*
	$h/t_w < 30$	y-y	*c*
Channel, Angle, *T* and Solid Sections 		Any	*c*
Built-up Member 		Any	*c*

DESIGN COMPRESSIVE STRENGTH:- [cls 7.1 IS 800-2007]



Where,

P_d = Design compressive strength of column.

P_u = External compression (or) design load.

P_d = Aexfcd

A_e = Eff. Area

f_{cd} = design compressive stress

$$f_{cd} = \frac{f_y / \gamma_{mo}}{\varphi + \left[\varphi^2 - \lambda^2\right]^{0.5}} = \frac{x f_y}{\gamma_{mo}} \leq \frac{f_y}{\gamma_{mo}}$$

Where,

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

λ = non-dimensional eff. Slenderness ratio.

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

f_{cc} = Ruler buckling stress

$$= \frac{\pi^2 E}{\left(KL/r\right)^2}$$

Where,

KL/r = eff. Slender ratio (or) eff. length, KL to appropriate radius of gyration.

α = Imperfection factor given in table 7

X = Stress reduction factor [see table-8]

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

λ_{mo} = Partial safety factor for material strength.

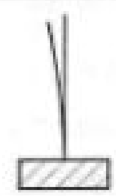
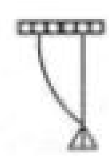
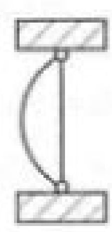
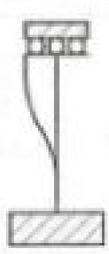
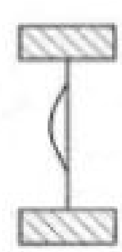
KL = Depends on support condition given in table – 11

The only variable in finding the permissible comp. stress (fcd) is slenderness ratio (L/r) for the given section coming under any of the buckling class a,b,c&d.

∴ Based on the slenderness ratio, design compressive stress can be taken from table 9, 9a, 9b, 9c (or) 9d IS 800-2007.

- The buckling class for various section are given in Table-10 IS 800-2007 and slenderness ratio is based on eff. length given in table-11; IS 800-2007.

Table 6.2 Effective length of prismatic compression members
[Refer Table 11 in IS 800]

Boundary Conditions				Schematic Representation	Effective Length
At One End		At the Other End			
Translation (1)	Rotation (2)	Translation (3)	Rotation (4)	(5)	(6)
Restrained	Restrained	Free	Free		2.0L
Free	Restrained	Free	Restrained		
Restrained	Free	Restrained	Free		1.0L
Restrained	Restrained	Free	Restrained		1.2L
Restrained	Restrained	Restrained	Free		0.8L
Restrained	Restrained	Restrained	Restrained		0.65L

1. Determine the design axial load capacity of the column ISHB 300@ 577 N/m if the length of the column is 3m and both ends are pined.

Given:-

Section => ISHB 300@577 N/m.

L => 3m.

End condition => Both ends are pinned.

Sln:-

To find slenderness ration:-

$$\lambda = \frac{KL}{r}$$

Where,

K = 1.0 [from table-11 IS 800-2007]

$$\gamma_{xx} = 129.5\,mm$$

$$\gamma_{yy} = 54.1\,mm$$

[from steel table]

$$\therefore r_{min} = 54.1\,mm$$

$$= \frac{1 \times 3000}{54.1}$$

$$\lambda = 55.45$$

To find design comp. stress:- [cls 7.1.2.1 IS 800-2007]

$$P_d = A_e f_{cd}$$

Where,

$$f_{cd} = \frac{f_y / \gamma_{mo}}{\varphi + [\varphi^2 - \lambda^2]^{0.5}} = \frac{x f_y}{\gamma_{mo}} \le \frac{f_y}{\gamma_{mo}}$$

$$\varphi = 0.5[1 + \alpha[\lambda - 0.2] + \lambda^2]$$

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

Buckling Class:- [Table-10 IS 800-2007]

Rolled steel I-section

$$\frac{h}{bf} = \frac{300}{250} = 1.2$$

tf = 10.6 < 100

About z-z axis –b

About y-y axis –c

∴ The section need to be check for buckling

Class-C

$\alpha = 0.49$ [from table-7 IS 800-2007]

$$f_{cc} = \frac{\pi^2 E}{\left(KL/r\right)^2} = \frac{\pi^2 \times 2 \times 10^5}{(55.44)^2}$$

$$f_{cc} = 641.98\ \text{N/mm}^2$$

$$\lambda=\sqrt{\frac{f_y}{f_{cc}}} = \sqrt{\frac{250}{641.98}}$$

$$\lambda = 0.624$$

$$\therefore \varphi=0.5\left[1+0.49(0.624-0.2)+(0.624)^2\right]$$

$$\varphi = 0.79$$

$\therefore$ Design compressive stress $f_{cd}=\dfrac{250/1.1}{0.79+\left[(0.79)^2-(0.624)^2\right]^{0.5}}$

$$f_{cd} = 178.33 \text{ N/mm}^2$$

$$\therefore P_d=7485\times178.33$$

$$P_d = 1334.7 \text{ KN}$$

50	183
60	168

Also referring table 9c IS 800-2007 [for buckling class-c] and λ = 55.45

$$f_{cd} = 174.8 \text{ N/mm}^2$$

$$P_d = 7485\text{x}174.8$$

$$P_d = 1308.5 \text{ KN}$$

DESIGN OF COMPRESSION MEMBERS:-

Step:1 => Assume the design comp. stress of the member [Generally for rolled steel sections assume f_{cd} = 135 N/mm², for angle section assume
f_{cd} = 90 N/mm² for builtup sections carrying
large loads assume f_{cd} = 200N/mm²

Step:2 => Reqd eff. Sectional area, $A=\dfrac{P_d}{f_{cd}}$

Step:3 => Select the section for the eff. Area and calculate. r_{min} [least of $r_{xx} \wedge r_{yy}$]

Step:4 => From the end co-ordinations, [decide the type of connection] determine the eff. Length.

Step:5 => Find the slenderness ratio and hence the design comp. stress f_{cd}

Step:6 => Find the actual load carrying capacity of the compression member.

Step:7 => If the calculated value of differs considering from the design load [P], revise the section.

1. Design a single angle strut connected to a gusset plate to carry a factored load of 180KN. Length of the strut is b/w c/c of intersection is 3m and the support condition is one end fixed & other end hinge with K=0.85

Given:-

Factored load, P = 180KN
L = 3m
K = 0.85

Sln:-

To find f_{cd}:-

Assume a design comp. stress f_{cd} = 90 N/mm²

To find A reqd:-

Reqd Area $A = \frac{P_d}{f_{cd}}$

$= \frac{180 \times 10^3}{90}$

A = 2000mm^2

Try ISA 90x90x12mm

Properties of ISA 90x90x12mm:-

$A = 2019 mm^2$

$\gamma_{xx} = \gamma_{yy} = 27.1 mm$

$\gamma_{uu} = 34.1 mm, \gamma_{vv} = 17.4 mm$

Buckling Class:-

Angle come under buckling class-c

$\frac{KL}{r} = \frac{0.85 \times 3000}{17.4}$

$\frac{KL}{r} = 146.55$

Refer Table 9c, IS 800-2007

140	66.2
150	59.2

f_{cd}= 61.615 N/mm^2

∴ Strength of strut = 2019 x 61.615

P_d = 232.7N/mm^2

P_d = 124.4 KN < 180KN

Revise the section:- Try ISA 130x130x8mm

$A = 2022 mm^2$

$I_{xx} = I_{yy} = 40.3$

$I_{uu} = 51.0 mm, I_{vv} = 25.5 mm$

$\frac{KL}{r} = \frac{0.85 \times 3000}{25.5}$

$\frac{KL}{r} = 100$

f_{cd} = 107 N/mm^2 [from table 9c IS 800-2007]

∴ Strength of strut = 2022x107

= 216.35 KN > 180KN

2. Design the above member when both ends are hinged.

Given:-

P = 180KN

L = 3m

Sln:-

To find A_{cd}:-

Assume a design comp. stress f_{cd} = 90N/mm^2

To find Areqd:-

$$A = \frac{P_d}{f_{cd}}$$

$$= \frac{180 \times 10^3}{90}$$

$A = 2000mm^2$

Try ISA 130x130x8mm

$A = 2022mm^2$

$r_{min} = 25.5mm$

$$\frac{KL}{r} = \frac{1 \times 3000}{25.5}$$

$$\frac{KL}{r} = 117.65$$

110	94.6
120	83.7

[from table 9© IS 800-2007]

$f_{cd} = 86.26\ N/mm^2$

∴ Strength of section = 2022 x 86.26

= 174.4KN < 180KN

Hence unsafe

∴ Revise the section with r_{min} more than 25.5mm

Try ISA 150x150x10mm

$A = 2903mm^2$

$r_{min} = 29.3\ mm$

$$\frac{KL}{r} = \frac{1 \times 3000}{29.3}$$

$$\frac{KL}{r} = 102.39$$

100	107
110	94.6

From table 9© $f_{cd} = 104\ N/mm^2$

∴ Strength of section = 2903 x 104

= 301.9 KN > 180KN

Hence safe

Effective length based on connection:-

* Generally eff. Length is computed based on table-11 IS 800-2007.
* Based on connectivity, welded joints are considered to be rigid.
* For welded joints case equal to K=0.65 to 0.7

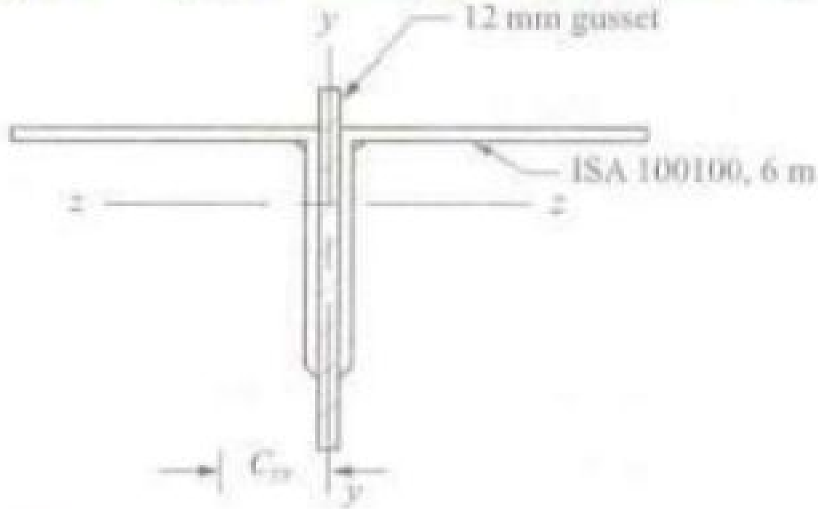
For Bolted Connection:-

a) When single bolts are provided on both sides.
b) When double bolts are provided.

K = 0.85

1. In a truss a strut which is IM long consists of 2 angles ISA 100x100x6mm. Find the design strength of the member if the angles are connected on both sides of a 12mm gusset plate using.

(i) One bolt (ii) Two bolts (iii) A rigid jt by welding



Given:-

$L = 3m$

2 ISA 100x100x6mm

Tks of gusset plate = 12mm

Sln:-

Section Properties of ISA 100x100x6mm:-

$A=1167mm^2$ $\qquad$ $I_{yy}= 111.3 \times 10^4 mm^4$

$r_{yy} = r_{zz} = 30.9mm$ $\qquad$ $I_{zz}= 111.3 \times 10^4 mm^4$

$C_{zz} = C_{yy} = 26.7$ mm

The local axis along the C/S is y-y & z-z as shown in fig.

r_{min} is the least of r_{yy} & r_{zz} of the composite section including 2 angles and a portion of gusset plate of size 100x12mm.

r_{zz} of the composite section is the same as r_{zz} of a single angle section.

Since the z-z axis is same for both the composite section & single angle section.

$$\therefore\ r_{zz} \text{ of composite section} = \sqrt{\frac{I_{zz}}{A}}$$

$$= \sqrt{\frac{111.3\times10^4}{1167}}$$

$$r_{zz} = 30.9mm$$

$$r_{yy} \text{ of composite section} = \sqrt{\frac{I_{yy}}{A}}$$

Where,

I_{yy} = M.O.I of composite section

I_{yy} = 2 [I_{yy} of one angle section +A(t/2 +cy)

$$= 2[111.3\times10^4+1167\left(\frac{12}{2}+26.7\right)^2$$

I_{yy} = 4.72 x $10^6 mm^4$

$$\therefore r_{yy} = \sqrt{\frac{4.72\times10^6}{2(1167)}}$$

$$r_{yy} = 44.97\text{mm}$$

$$\therefore r_{min} = 30.9 \text{ mm}$$

(i) One Bolt :- [Bolt ends hinged]

$$\therefore K = 1$$

$$\frac{KL}{r} = \frac{1\times3000}{30.9}$$

$$\frac{KL}{r} = 97.09$$

90	121	[from table 9c IS800-2007]
100	107	

The member belongs to buckling class-c since it is a angle section.

$\therefore$ Refer table-9 (C)

$f_{cd} = 111.07$ N/mm^2

Design strength of section = f_{cd} x A

= 111.07 x 2 x 1167

= 259.24 KN

(ii) Two Bolts:-

$$\therefore K = 0.85$$

$$\frac{KL}{r} = \frac{0.85\times3000}{30.9}$$

$$\frac{KL}{r} = 82.52$$

80	136	[from table 9c IS 800-2007]
90	121	

$f_{cd} = 132.22$ N/mm^2

$\therefore$ Design strength of section = f_{cd} x A

= 132.22 x 2 x 1167

= 308.6 KN

(iii) A rigid joint by welding:-

$$\therefore K = 0.7$$

$$\frac{KL}{r} = \frac{0.7\times3000}{30.9}$$

$$\frac{KL}{r} = 67.96$$

60	168	[from table 9c IS 800-2007]
70	152	

$f_{cd} = 155.26$ N/mm^2

$\therefore$ Design strength of section = f_{cd} x A

= 155.26 x 2 x 1167

= 362.38 KN

2. Determine the load carrying capacity of a column section as shown in fig. The actual length of the column is 4.5m. One end of the column is assumed as fixed and the other end hinged. The grade of steel [E250]

Given:-

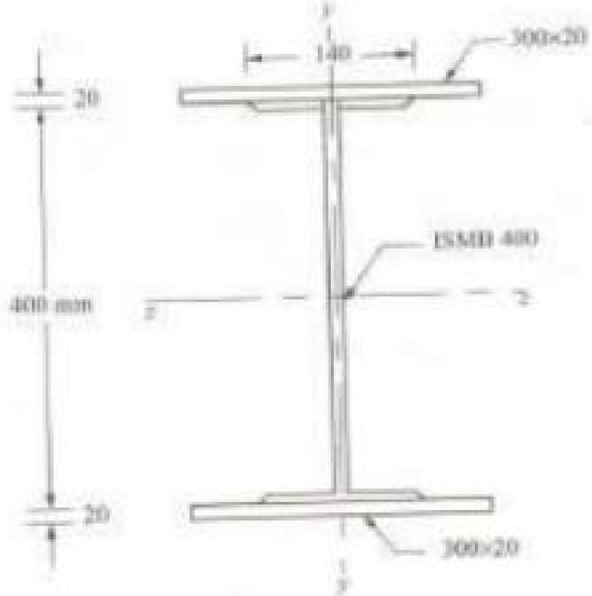
L = 4.5m

Support condition = One end fixed & other end hing

$\therefore$ K = 0.8

Sln:-

The design stress 'f_{cd}' of the composite section depends on $\frac{KL}{r_{min}}$ ratio and the buckling class.



Properties of ISMB 400:-

h = 400mm, bf = 140mm, tf = 16mm

tw = 8.9mm, r_{yy} = 28.2mm, r_{zz} = 161.5mm

$I_{zz} = 20458.4 \times 10^4 mm^4$, $I_{yy} = 622.1 \times 10^4 mm^4$

r_{min} is least of r_{zz} (or) r_{yy}

where, $r=\sqrt{\frac{I}{A}}$

I_{zz} of Composite section:-

$$I_{zz}=20458.4\times10^4+\frac{300\times20^3}{12}+\left[300\times20\times(430-220)^2\right]+\frac{300\times20^3}{12}+\left[300\times20\times(220-10)^2\right]$$

$$I_{zz}=734.18\times10^6 mm^4$$

I_{yy} of Composite section:-

$$I_{yy}=622.1\times10^4+\frac{20\times300^3}{12}+\left[20\times300\times(150-150)^2\right]+\frac{20\times300^3}{12}+\left[20\times300\times(150-150)^2\right]$$

$$I_{yy}=96.221\times10^6 mm^4$$

$$\therefore r_{zz}=\sqrt{\frac{I_{zz}}{A}}$$

A = 7846 + (2x300x20)

A = 19846mm²

734.18×10^6

94.152×10^6

19846

$\therefore r_{zz}=\sqrt{¿\frac{¿}{¿}¿}$

$$r_{zz} = 192.34mm$$

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

$$= \sqrt{\frac{96.221\times10^6}{19846}}$$

$$r_{yy} = 69.63mm$$

$$\therefore\ r_{min} = 69.63mm$$

To find slenderness ratio:-

$$\frac{KL}{r_{min}} = \frac{0.8\times4500}{69.63}$$

$$\frac{KL}{r_{min}} = 51.7$$

The buckling class of the built up section based on table-10 IS 800-2007. Tks of flange is 16+20 = 36mm < 40mm

$\therefore$ Along buckling about z-z axis is buckling class 'B' and buckling about y-y axis, therefore I_{yy} is less than I_{zz}

50	183
60	168

From table 9 (C) IS 800-2007

f_{cd} = 180.45 N/mm^2

$\therefore$ Load carrying capacity of the section = 180.45 x 19846
= 3581.2 KN

$$\text{Safe working load} = \frac{3581.2}{1.5}$$

= 2387.5 KN

3. Design a column 4m long to carrying a factor load of 6000KN column is effectively held at both ends and restrain in direction at one end. Design the column using beam section ISHB 450 @ 907 N/m

Given:-

L = 4m

Factor Load = 6000KN

One end fixed and other end hinged

$\therefore$ K = 0.8

Sln:-

The given section ISHB is checked for the axial load carrying capacity

$\therefore$ $P_d = A \times f_{cd}$

Properties of ISHB450 @ 907 N/m:-

A = 11789mm^2

I_{xx} = 40349.9x10^{4}mm^4, I_{yy} = 3045x10^{4}mm^4

Assuming f_{cd} = 200N/mm^2

$$\therefore Areqd = \frac{6000\times10^3}{200}$$

= 30000mm^2

∴ Area deficit = 30000-11789

= 18211mm^2

Selecting 20mm tk plate @ top 2 bottom flange portion.

2(20xb) = 18211

b = 455.275mm ¿ 500mm

∴ Assume the size of plate @ as 500x20mm @ top and bottom.

I_{zz} of composite section:-

$$I_{zz}=40349.9\times10^4+\frac{500\times20^3}{12}+\left[500\times20\times(480-245)^2\right]+\frac{500\times20^3}{12}+\left[500\times20(245-10)^2\right]$$

$$I_{zz}=1508.66\times10^6 mm^4$$

I_{yy} of composite section:-

$$I_{yy}=3045\times10^4+\frac{20\times500^3}{12}+\left[20\times500\times(250-250)^2\right]+\frac{20\times500^3}{12}+\left[20\times500\times(250-250)^2\right]$$

$$I_{yy}=447.12\times10^6 mm^4$$

Check for over hang:-

The over hang length is limited to 'lbt' over hang length = 500-250

= 250mm < 16(20) = 320mm

$$\therefore r_{zz}=\sqrt{\frac{I_{zz}}{A}}$$

A = 11789+(2x500x20)

A = 31789mm^2

$$r_{zz}=\sqrt{\frac{1508.66\times10^6}{31789}}$$

r_{zz} = 217.85 mm

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

$$=\sqrt{\frac{447.12\times10^6}{31789}}$$

r_{yy} = 118.60mm

$$\therefore r_{min}=118.60 mm$$

To find slenderness ratio:-

$$\frac{KL}{r_{min}}=\frac{0.8\times4000}{118.60}$$

$$\frac{KL}{r_{min}}=26.98$$

20	224
30	211

From table 9 © IS800-2007

f_{cd} = 214.926 N/mm^2

∴ Design load carrying capacity of the section = $f_{cd} \times A$

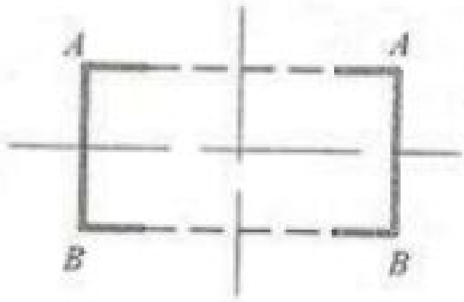
$= 214.926 \times 31789$

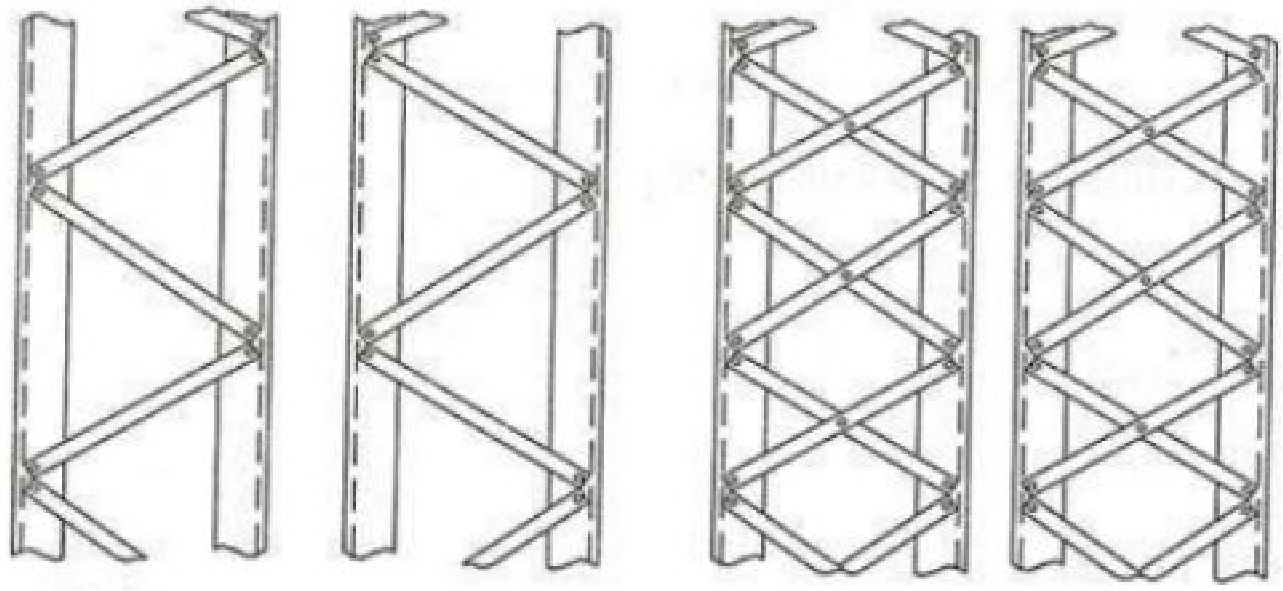
$P_d = 6832.3$ KN > 6000KN

Hence the assume section is safe.

Laced & Battened Columns:-

[cls 7.6 IS 800-2007] [cls 7.7 IS 800-2007]





Lacing on Face A | Lacing on Face B — Preferred Lacing Arrangement

(A) Single Laced System

Lacing on Face A | Lacing on Face B — Preferred Lacing Arrangement

(B) Double Laced System

- Lacings and battens are provided to establish a built up section. [generally using channels and angles]
- They do not increase the area of the section, but increase the mini. Radius of gyration [achieve by placing the members away from principle axis]
- The commonly used lateral systems are lacings or latticings battering.

Design of Laced Columns:-°

The general guide lines reqd are

1. The latticing system shall be uniform throughout.
2. In single lacing system, the direction of lattices on the opposite face should be the shadow of the other and not mutually opposite.
3. In bolted construction, the mini width of lacing bars shall be 3 times the nominal dia of bolts.
4. Tks of flat lacing bars shall not be less than 1/140 th of its eff. Length for single lacing & $1/16^{th}$ of eff. Length for double lacings.
5. Lacing bars shall be inclined at 40° to 70° to the axis of the built up members.

UNIT 3 BEAMS.

Design of laterally supported beam

Procedure:

1. A trial s/n is selected & assumed that it is going to be plastic section.
2. check for class it belongs.
3. check for bending strength.
4. check for shear strength.
5. check for deflection.

Bending strength of laterally supported beam:

a) If $V \leq 0.6 V_d$

Design bending strength, M_d.

$$M_d = \beta_b Z_p f_y \cdot 1/\gamma_{m0} \leq 1.2 Z_e f_y \cdot 1/\gamma_{m0} \text{ for SSB}$$

$$\leq 1.5 Z_e f_y \cdot 1/\gamma_{m0} \text{ for cantilever}$$

Where,

$\beta_b = 1.0$ for plastic & compact sections.

$= \frac{Z_e}{Z_p}$ for semi compact sections.

Z_p, Z_e = Plastic & elastic section moduli of c/s.

b) If $V > 0.6 V_d$

$$M_d = M_{dv}$$

M_{dv} - Design bending strength under high shear.

plastic / compact section:

$$M_{dv} = M_d - \beta \cdot (M_d - M_{fd}) \leq 1.27 Z_e f_y / \gamma_{mo}$$

where,

$$\beta = \left(\frac{2V}{V_d} - 1\right)^2$$

M_d - plastic design moment of whole s/n.

V - Factored applied SF.

V_d - Design shear strength.

M_{fd} - plastic design strength of area of c/s exclude shear area, Consider partial safety factor = γ_{mo}

Semi compact section:

$$M_{dv} = \frac{Z_e \cdot f_y}{\gamma_{mo}}$$

Shear strength of Laterally supported beam.

$$V_d = \frac{A_v f_{yw}}{\sqrt{3}} \cdot 1/\gamma_{mo}$$

Where,

A_v - Shear area

f_{yw} - yield strength of web.

Shear area:

a) I and channel sections.

i) Major axis bending:

Hot rolled - ht_w

Welded - dt_w

ii) Minor axis bending:

Hot rolled / welded - $2bt_f$.

b) Rectangular hollow sections of uniform tk:

i) Load ||el to depth, $h \Rightarrow \frac{Ah}{b+h}$

ii) Load ||el to width, $b \Rightarrow \frac{Ab}{b+h}$.

iii) Circular hollow tubes of uniform tk $\Rightarrow \frac{2A}{\pi}$

iv) plates and solid bars - A

where,

A - C/s area.

b - overall breadth of tubular S/n, breadth of I-S/n flanges.

d - clear depth of web b/w flanges.

h - overall depth of S/n

t_f - tk of flange

t_w - tk of web.

EX1:

A roof of a hall measuring 8×12m consists of 100mm tk Rc slab supported on I-beams spaced 3m apart. The finishing load may be taken as 1.5 kN/m² & LL = 1.5 kN/m². Design the steel beam.

Soln:

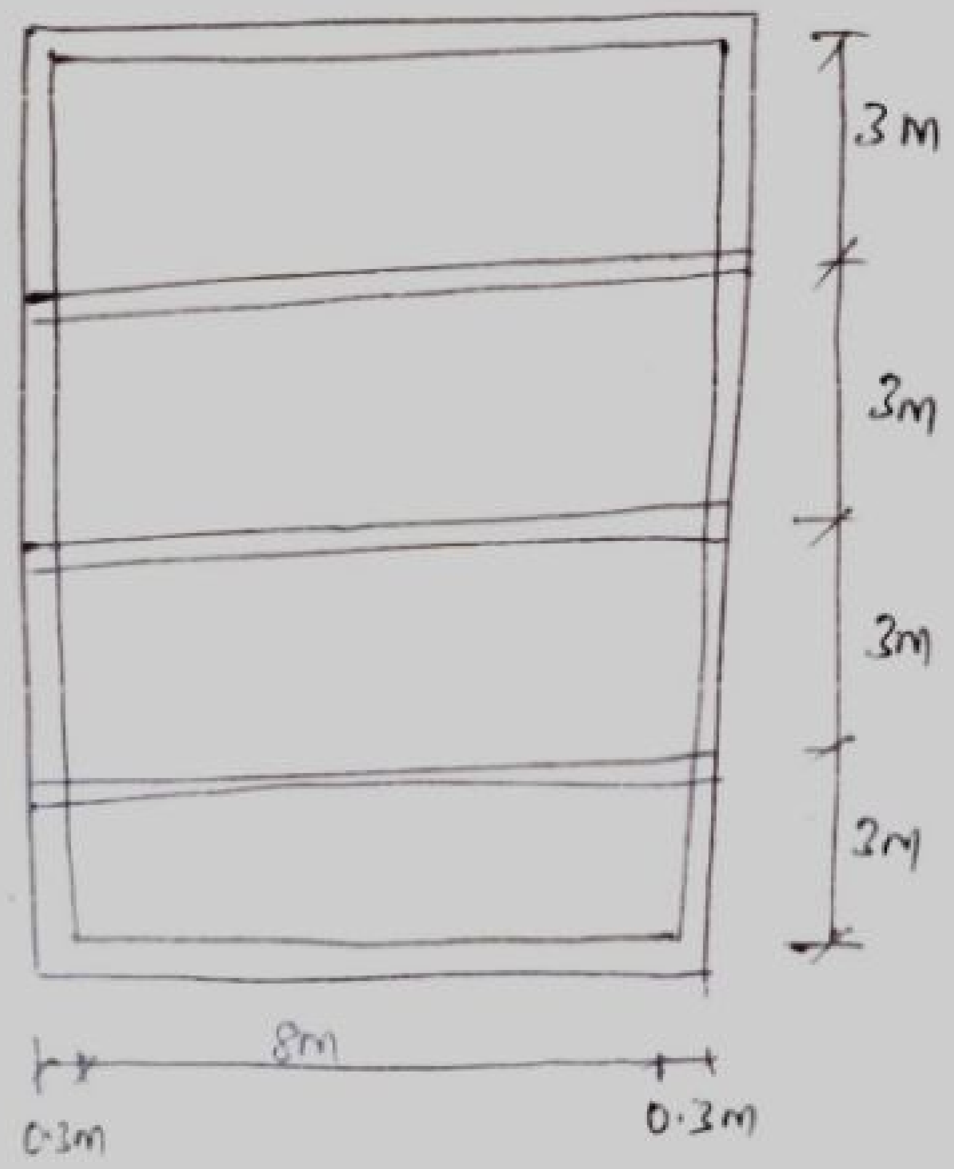
Each beam has a clear span of 3m width of slab.

∴ Weight of Rc slab = $0.1 \times 1 \times 3 \times 25 = 7.5$ kN/m

Finishing load = 1.5×3 = 4.5 kN/m

Self weight (assumed) = 0.8 kN/m.

∴ Total dead load = 12.8 kN/m



Live Load = 1×3×1.5 = 4.5 kN/m.

Total Load = 17.3 kN/m

Total factored Load = 1.5 × 17.3

= 25.95 kN/m.

Effective span of simply supported beam

= centre to centre distance of supports.

Assume width of support = 0.3 m

Effective span = 8 + 0.3 = 8.3 m

∴ Design moment, $M = \frac{wL^2}{8}$

$= \frac{25.95 \times 8.3^2}{8}$

$= 223.46$ kNm

Design shear force, $V = \frac{wL}{2} = \frac{25.95 \times 8.3}{2}$

$= 107.61$ kN.

plastic section modulus req = $\frac{M}{f_y} \times \gamma_{mo}$

$Z_p = \frac{223.46 \times 10^6 \times 1.1}{250}$

$= 983224 \text{ mm}^3$.

Try ISMB 400 which has $Z = 1175.2 \times 10^3 mm^3$.

The properties of the section as follows:

Depth of section, $h = 400mm$

Width of flange, $b = 140mm$

Tk of flange, $t_f = 16mm$

tk of web, $t_w = 8.9mm$

Sectional area, $A = 7846mm^2$.

$r_1 = 14mm$

Depth of web, $d = h - 2(t_f + r_1)$

$= 400 - 2(16+14)$

$= 340mm$.

Moment of Inertia about Z-Z axis,

$$I_{zz} = 20458.4 \times 10^4 mm^4$$

Elastic section modulus $Z_e = 1020 \times 10^3 mm^3$.

Section classification

$$\epsilon = \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{250}}$$

$$= 1.0$$

$$b/t_f = \frac{140}{16} = 8.75 < 9.4\epsilon$$

$$\frac{d}{t_w} = \frac{340}{8.9} = 38.2 < 84\epsilon$$

Hence the section is classified as plastic section.

Check for assumed self weight:

Wt of section = 0.604 kN/m.

Assumed weight = 0.800 kN/m.

Difference is not much. Hence the design is continued with moments and shears calculated as earlier.

Check for shear strength:

Design shear, V = 107.61 kN.

Design shear strength of the section,

$$V_d = \frac{f_y}{\sqrt{3}} \times \frac{1}{1.1} \times \text{shear area}.$$

$$= \frac{f_y}{\sqrt{3}} \times \frac{1}{1.1} \times h \times t_w$$

$$= \frac{250}{\sqrt{3}} \times \frac{1}{1.1} \times 400 \times 8.9$$

$$= 467.128\,kN > 107.61\,kN.$$

Hence the section is adquate.

$0.6\, V_d = 0.6 \times 467.128$

$= 280.277 > 107.61 KN$

Check for design moment capacity:

$\frac{d}{t_w} = 38.2$ which is less than 67ϵ, since $\epsilon = 1$

Hence, $M_d = \beta_b \cdot Z_p \cdot \frac{f_y}{\gamma_{mo}}$

$\beta_b = 1.0$ since it is Plastic section.

$\therefore M_d = 1.0 \times 1175.2 \times 10^3 \times \frac{250}{1.1}$

$= 267.091 KNm > 223.46 KNm$.

Hence adquate.

Check for deflection:

Total working Load $= 12.8 + 4.5 = 17.3 KN/m$.

$= 17.3 N/mm$.

Maximum deflection, $\delta = 5/384 \cdot \frac{wL^4}{EI}$.

$= \frac{5}{384} \cdot \frac{17.3 \times (8300)^4}{2 \times 10^5 \times 20458.4 \times 10^4}$

$= 26.127 mm$.

Permissible deflection for a beam in building $= \frac{l_e}{300}$

$$= \frac{8300}{300} = 27.67 mm.$$

Hence the deflection is within the permissible limit

∴ provide ISMB 400.

Ex 2:

Design a simply supported beam of effective span 1.5m carrying a factored concentrated load of 360kN at midspan.

Soln:

Max moment occurs @ midspan is given by,

$$M = \frac{WL}{4} = \frac{360 \times 1.5}{4} = 135 kNm.$$

$$\therefore Z_p \text{ req} \Rightarrow f_y \cdot \frac{Z_p}{\gamma_{mo}} = M$$

$$\therefore Z_p = \frac{135 \times 10^6}{250} \times 1.1$$

$$= 594 \times 10^3 mm^3.$$

Select section as ISMB 300 which has $Z_p = 651.7 \times 10^3 mm^3$

properties of ISMB 300 are:

Overall depth, $h = 300mm$.

Width of flange, $b = 140mm$

Thickness of flange, $t_f = 12.4mm$.

Depth of web, $d = h - 2(t_f + r_1)$

$= 300 - 2(12.4 + 14)$

$= 247.2mm$.

Thickness of web, $t_w = 7.5mm$.

$I_{zz} = 8603 \times 10^4 \ mm^4$.

$Z_e = 573.6 \times 10^3 \ mm^3$.

$Z_p = 651.7 \times 10^3 \ mm^3$.

Self weight of beam $= 0.452 kN/m$.

$\therefore$ Factored weight $= 1.5 \times 0.452 \ kN/m$.

$= 0.678 \ kN/m$.

Additional factored moment due to self wt

$= 1.5 \times 0.452 \times 1.5^2 / 8$

$= 0.190 kNm$.

Total factored moment, $M = 135 + 0.190$

$= 135.190 kNm$

Factored shearforce due to self wt $= 0.678 \times \frac{1.5}{2}$

$= 0.508 kN$

∴ Total factored SF on section $= \frac{360}{2} + 0.508$

$= 180.508 kN$

Section classification:

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

$$b/t_f = \frac{140}{12.4} = 11.29 < 15.7\epsilon$$

$$d/t_w = \frac{247.2}{7.5} = 32.96 < 84\epsilon$$

It is classified as a semi compact section.

Shear capacity of the section:

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

$$= \frac{250}{\sqrt{3}} \times \frac{1}{1.1} \times 300 \times 7.5$$

$$= 295.235 kN$$

Section is adequate to resist shear.

$$\therefore 0.6 V_d = 0.6 \times 295.235$$

$$= 177.145 kN$$

$$\therefore V > 0.6 V_d$$

Moment capacity of the section:

Since $V > 0.6V_d$ and Section belongs to class 3 category:

$$M_d = M_{dv} = \frac{Z_e f_y}{\gamma_{m0}} = \frac{573.6 \times 10^3 \times 250}{1.1}$$

$$= 130.36 \text{ kNm} < M.$$

Hence the section is not safe. Revise the section.

Try ISMB350.

Sectional properties:

Overall depth, $h = 350$mm.

Width of flange, $b = 140$mm.

Thickness of flange, $t_f = 14.2$mm.

Tk of web, $t_w = 8.1$mm.

Depth of web, $d = 350 - 2(14.2 + 140)$

$= 293.6$mm.

$Z_e = 778.9 \times 10^3 \text{ mm}^3$; $Z_p = 889.6 \times 10^3 \text{ mm}^3$.

$I_{zz} = 13630.3 \times 10^4 \text{ mm}^4$.

Section Classification:

$\epsilon = 1$

$b/t_f = \frac{140}{14.2} = 9.86$ b/w 9.4ϵ and 10.4ϵ.

$d/t_w = \frac{293.6}{8.1} = 36.24$ b/w 33.5ϵ and 42ϵ.

∴ It belongs to class 3 (Semi Compact) section

Shear capacity of the section:

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

$$= \frac{250}{\sqrt{3}} \times \frac{1}{1.1} \times 350 \times 8.1$$

$$= 371.99 \text{ kN} > 180.508 \text{ kN}$$

Section is adquate to resist shear.

$$0.6 V_d = 0.6 \times 371.99$$

$$= 223.198 \text{ kN} > 180.508 \text{ kN}.$$

Moment capacity of the section:

since $d/t_w < 67\epsilon$ and $V < 0.6 V_d$.

$$\therefore M_d = \beta_b \cdot Z_p \cdot f_y \cdot 1/\gamma_{mo}$$

$$\beta_b = \frac{Z_e}{Z_p} = \frac{778.9 \times 10^3}{889.6 \times 10^3}$$

$$M_d = \frac{778.9 \times 10^3}{889.6 \times 10^3} \times 889.6 \times 10^3 \times 250 \times 1/1.1$$

$$= 194.725 \text{ kNm} > 135.19 \text{ kNm}.$$

∴ The section is adquate to resist moment.

Max. deflection corresponds to working load,

$$\delta = WL^3/48EI = \frac{360 \times 10^3 \times 1500^3}{48 \times 2 \times 10^5 \times 13630.3 \times 10^4}$$

$$= 0.928 \text{ mm} \nless (1500/300)$$

Hence Section is adquate.

Hence use ISMB 350 as beam.

Web buckling strength:

Certain portion of beam at supports acts as Column to transfer the load from beam to the support. Hence under this compressive force the web may buckle. The Load dispersion angle may be taken as 45°. Hence there is need to check for web buckling.

However rolled sections are provided with suitable tk of web so that web buckling is avoided. In case of built up sections it is necessary to check for buckling of web and provide web stiffeners.

Effective web buckling strength is found based on the c/s of web:

$= (b_1 + n_1)\, t_w.$

b_1 = width of stiff bearing on flange.

$n_1 = \frac{1}{2} h$.

h = depth of section.

$\therefore F_{cdw} = (b_1 + n_1)\, t_w \cdot f_c$.

F_{cdw} - web buckling strength.

f_c - allowable comp stress corresponds to assumed web column.

Effective Length = 0.7d of web column.

$$r_y = \sqrt{I_y/A} \text{ of web}$$

$$= \sqrt{\frac{\frac{1}{12}(b_1+n_1)t^3}{(b_1+n_1)t_w}}$$

$$= \frac{t}{2\sqrt{3}}$$

$$\therefore \lambda = \frac{L_E}{r_y} = 0.7d\,\frac{2\sqrt{3}}{t} \simeq 2.5\,d/t_w$$

The buckling stress f_c.

$F_{cdw} = (b_1+n_1)\,t_w f_c$ may be found.

<u>Web crippling</u>:

Near the support web of beam may cripple due to lack of bearing capacity. The crippling occurs at the root of radius.

To find crippling strength of web.

$$F_w = (b_1+b_2)\,t_w \cdot \frac{f_{yw}}{\gamma_{m0}}$$

b_1 - Stiff bearing length.

f_{yw} - yield stress of web.

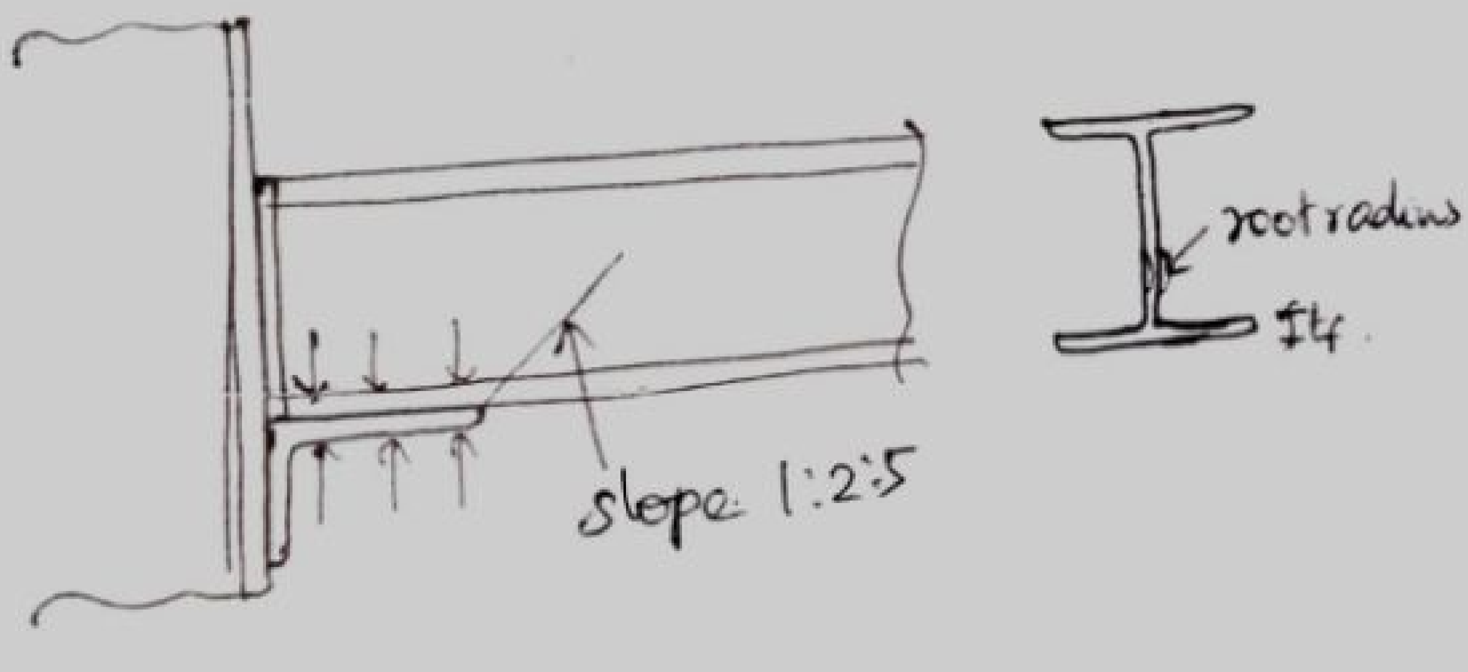
n_c - Length obtained by dispersion through flange to web junction @ slope 1:2.5 to plane of flange.

In design F_w > Load transferred by bearing.

But the care is taken in fixing the web tk of rolled steel sections to avoid such failure. Hence if rolled steel sections is selected as a beam section there is no need to check for this failure. However when built up sections are selected, the web should be checked for this local failure.

EX3:

Check the section selected in ex 3 for web buckling and web crippling if stiff bearing is over a length b_1 = 75mm.



Soln:

Section selected was ISMB 400.

End reaction = End shear = 107.61kN.

Stiff bearing @ ends = 75mm.

t_w = 8.9mm ; t_f = 16mm

root radius = 14mm.

Depth of section, h = 400mm.

$\therefore$ Depth of web $= 400 - 2(t_f + r_1)$

$= 400 - 2(16+14)$

$= 340mm.$

Check for web buckling:

$$\lambda = 2.5\, d/t_w = \frac{2.5 \times 340}{8.9} = 95.5$$

Hence from table 9c of IS 800-2007,

$$f_c = 121 - \frac{5.5}{10}(121 - 107) = 113.3 N/mm^2$$

$$n_1 = \frac{400}{2} = 200mm.$$

$\therefore$ Web buckling resistance of the s/n,

$$F_{cdw} = (b_1 + n_1)\, t_w\, f_c.$$

$= (75+200) \times 8.9 \times 113.3$

$= 277.30 kN > 107.61 kN.$

Hence the s/n is safe against web buckling.

Check for web crippling:

Flange tk $= 16$ mm

root radius $= 14mm$

$\therefore n_2 = 2.5(16+14) = 75mm.$

Strength of web against web crippling.

$F_w = (b_1 + n_2)\, t_w\, f_{yw} \cdot \frac{1}{\gamma_{m0}}$

$= (75+75) \times 8.9 \times 250 \times \frac{1}{1.1}$

$= 303.409\,kN$

$>$ Load transferred by bearing (107.61 kN)

Hence safe.

Ex 6

Determine udl carrying capacity of the welded plate girder, when it is used as a cantilever beam of 4m effective span and check it for shear, defln, web buckling and web-crippling. Assume stiff bearing length as 100mm.

Soln:

Section moduli

$I_{zz} = \frac{1}{12}\left[200 \times 832^3 - 184 \times 800^3\right]$.

$= 1748.173 \times 10^6\,mm^4$.

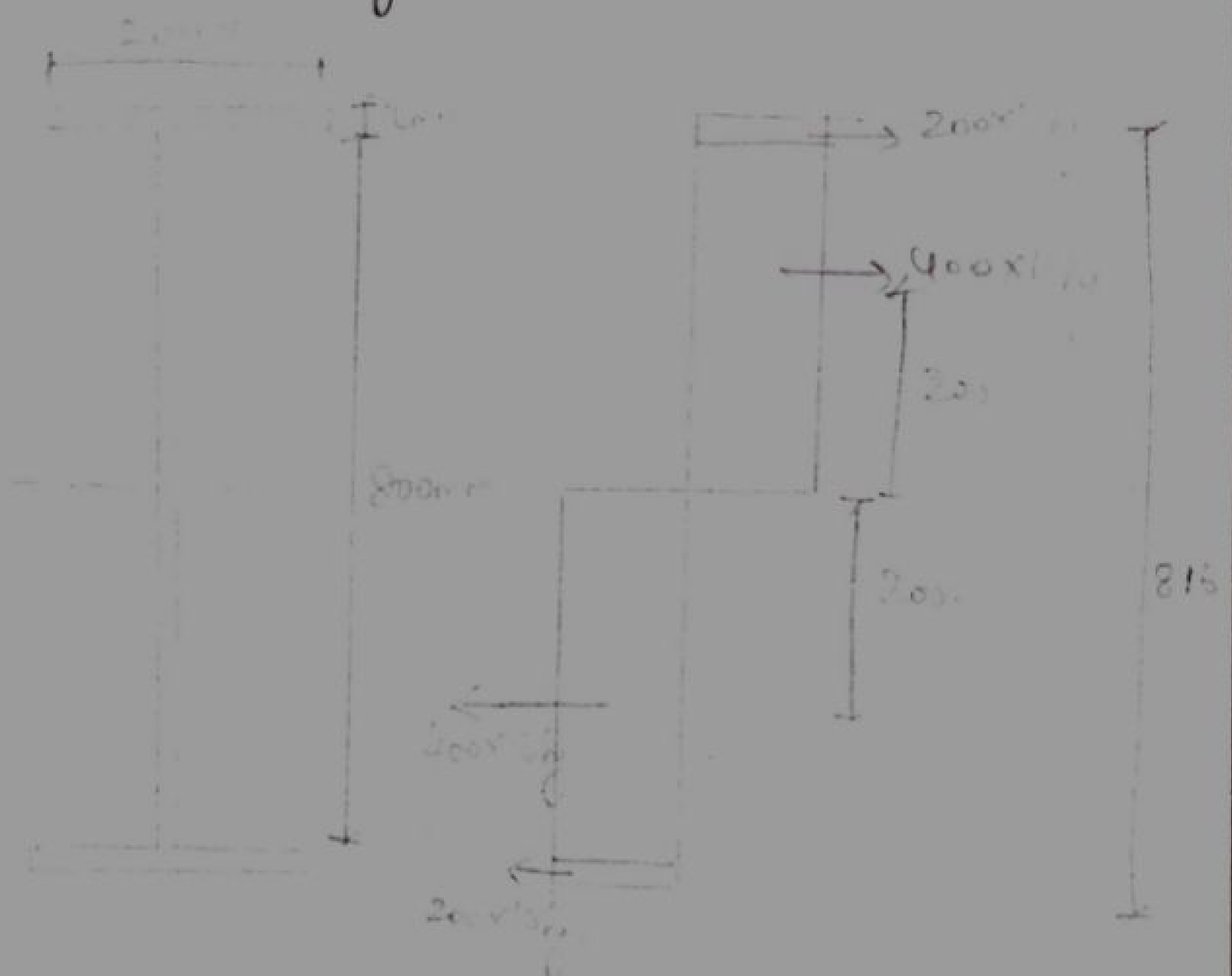
$\therefore Z_e = \frac{I_{zz}}{y_{max}} = \frac{1748.173 \times 10^6}{\left(\frac{832}{2}\right)}$

$= 4202.338 \times 10^3\,mm^3$.

plastic NA is at mid depth since stress is f_y in top half and f_y (tensile) in bottom half.

$$M_p = (200\times16\times816 + 400\times16\times400)f_y$$

$$\therefore Z_p = M_p/f_y = 5171.199\times10^3\ mm^3.$$



Section classification:

$$\epsilon = 1$$

$$b/t_f = \frac{200}{16} = 12.5 \ ; \text{ between } 9.4t \text{ and } 13.6t$$

$$d/t_w = \frac{800}{16} = 50 < 84\epsilon.$$

It belongs to semi compact class of section.

Assume $V < 0.6V_d$.

$$\therefore M_d = \beta_b \cdot Z_p \cdot f_y \cdot 1/\gamma_{m0} = \frac{Z_e}{Z_p} \cdot Z_p f_y \cdot 1/\gamma_{m0}$$

$$= Z_e \cdot f_y \cdot 1/\gamma_{m0}$$

$$= \frac{4202.338 \times 250}{1.1} = 955.0768 \text{ kNm} \quad -(1)$$

Let factored udl be w kN/m length.

$$M = \frac{wL^2}{2} = w \times 4^2/2 = 8w \quad -(2)$$

Equate (1) & (2),

$$8w = 955.0768$$

$$w = 119.385 \text{ kN/m}$$

$$\therefore \text{shear, } V = wL = 119.385 \times 4$$

$$= 477.538 \text{ kN}.$$

Check for shear:

$$V_d = f_y/\sqrt{3} \cdot \frac{1}{\gamma_{m0}} \cdot h \cdot t.$$

$$= \frac{250}{\sqrt{3}} \times 1/1.1 \times 832 \times 16$$

$$= 1746.747 \text{ kN} > V$$

Hence safe.

$$0.6 V_d = 286.523 \text{ kN} > V$$

Hence calculated M_d is correct.

Check for deflection:

for cantilever beam,

$$\delta = \frac{wL^4}{8EI_{zz}}$$

w = working Load = $\frac{119.38 \times 1000}{1.5 \times 100}$ N/mm.

= 79.587 N/mm.

$$\therefore \delta = \frac{79.587 \times (4000)^4}{8 \times 2 \times 10^5 \times 1748.173 \times 10^6}$$

$= 7.28\,mm < \frac{4000}{300}$.

Hence safe.

Check for web buckling:

Slenderness ratio, $\lambda = 2.5\, h/t_w$

$= 2.5 \times \frac{816}{16}$

$= 127.5$

From table 9c,

$f_c = 83.7 - \frac{7.5}{10}(83.7 - 74.3)$

$= 76.65\ N/mm^2$.

$\therefore F_{cdw} - (b_1 + n_1)\, t_w\, f_c$

$= (100 + 416) \times 16 \times 76.65$

$= 632.82\ kN > V$

∴ web is safe.

Check for web crippling

$$F_w = (b_1 + 2.5 t_f) f_y \frac{1}{\gamma_{m0}} \cdot t_w$$

$$= (100 + 2.5 \times 16) \times 250 \times \frac{1}{1.1} \times 16$$

$$= 509.09 \text{ kN} > V$$

Hence safe.

Design of built up section:

- When moment is to be resisted heavy. rolled s/ns should not be sufficient.

-

EX 5

Design a simply supported beam of 10m effective span carrying a total factored load of 60kN/m. The depth of beam should not exceed 500mm. The compression flange of the beam is laterally supported by floor construction. Assume stiff end bearing is 75mm.

Soln:

$L = 10m = 10,000mm$

$w = 60 kN/m$

$$\text{Max BM, } M = \frac{wL^2}{8} = \frac{60 \times 10^2}{8}$$

$$= 750 \text{ kNm} = 750 \times 10^6 \text{ Nmm}$$

$$\therefore Z_p \text{ req} = \frac{M \gamma_{mo}}{f_y} = \frac{750 \times 10^6 \times 1.1}{250}$$

$$= 3300 \times 10^3 mm^3.$$

Since depth is restricted to 500mm, select ISMB450 and suitable plate over flanges.

Z_p of ISMB 450 = 1553.4×10^3

Z_p to be provided by cover plates

$$= (3300 - 1553.4) \times 10^3$$

$$= 1746.6 \times 10^3 mm^3 \quad \text{—} (3)$$

If the distance b/w the plates is 'd',

plastic moment resisted = $A_p \cdot f_y \cdot d$

Additional Z_p provided by cover plates,

$$Z_p \text{ of plates} = \frac{A_p \cdot f_y \cdot d}{f_y} \cdot \frac{1}{\gamma_{mo}}$$

$$= \frac{A_p d}{1.1} \quad \text{—} (4)$$

Equate (3) & (4),

$$\therefore \frac{A_p d}{1.1} = 1746.6 \times 10^3$$

Take,

$$d = 450 + t = 450mm$$

We get,

$$A_p = \frac{1746.6 \times 10^3 \times 1.1}{450} = 4269.5 mm^2$$

provide 220 × 20mm plates on each side.

Check for shear:

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

$$= \frac{250}{\sqrt{3}} \times \frac{1}{1.1} \times 450 \times 9.4$$

$$= 559.044 kN$$

$$0.6 V_d = 333.026 kN.$$

$$V = 60 \times 10/2 = 300 kN.$$

Hence the Section is safe in shear.

Section classification:

Internal element of compression flange:

$$\frac{b}{t_f} = \frac{150}{17.4} = 8.6 < 29.3\epsilon \text{ and}$$

$$\frac{d}{t_w} = \frac{450 - 2(17.4 + 15)}{9.4} = 40.9 < 84\epsilon.$$

Hence plastic section,

$$\therefore M_d = 1.0 \times \frac{Z_p \cdot f_y}{\gamma_{mo}}$$

$$= (Z_p \text{ of I-s/n} + Z_p \text{ of plates}) \cdot f_y/\gamma_{mo}$$

$$= \left[1553.4 \times 10^3 + 220 \times 20\,(450+20)\right] \times \frac{250}{1.1}$$

$$= 828.045 \text{ kNm} > M$$

Hence safe.

Check for deflection:

$$\text{Working load} = \frac{60}{1.5} \text{ kN/m} = 40 \text{ kN/m}$$

$$= 40 \text{ N/mm}$$

$$\therefore \delta = \frac{5wL^4}{384EI} = \frac{5 \times 40 \times (10000)^4}{384 \times 2 \times 10^5 \times I_{zz}}$$

$I_{zz} = I_{zz}$ of ISMB 400 + I_{zz} due to plates.

$$= 30390.8 \times 10^4 + 2A_p(d/2)^2$$

$$= 30390.8 \times 10^4 + 2 \times 20 \times 220\,(225+10)^2$$

$$= 789.88 \times 10^6 \text{ mm}^4$$

$$\therefore \delta = \frac{5 \times 40 \times (10000)^4}{384 \times 2 \times 10^5 \times 789.884 \times 10^6}$$

$$= 32.97 mm.$$

From table 6 in IS 800 - 2007,

$$\text{permissible defln, } L/240 = \frac{10000}{240}$$

$$= 41.67 mm$$

Hence safe.

Check for web buckling:

$h = 450mm$

$$\therefore \lambda = 2.5\, h/t_w = 2.5 \times \frac{450}{9.4}$$

$$= 119.68$$

from table 9c in IS 800 - 2007,

$$f_{cd} = 94.6 - \frac{9.68}{10}(94.6 - 83.7)$$

$$= 84.13 N/mm^2.$$

$$\therefore F_{cdw} = (b_1 + n_1)\, t_w \cdot f_{cd}$$

$$= \left(75 + 450/2\right) \times 9.4 \times 84.13$$

$$= 237.246 \times 10^3 N < 300kN.$$

Hence needs increased effective bearing length b

provided b = 175mm,

$$F_{cdw} = \left(175 + \frac{450}{2}\right) \times 9.4 \times 84.13$$

$$= 316.94 \text{ kN} > 300 \text{ kN}$$

Hence safe.

Check for web crippling:

$$F_w = (b_1 + 2.5t_f) \cdot f_y \cdot \frac{1}{\gamma_{m0}} \cdot t_w$$

$$= [175 + 2.5 \times (17.4 + 20)] \times 250 \times \frac{1}{1.1} \times 9.4$$

$$= 573.61 \text{ kN} > 300 \text{ kN}$$

Hence safe.

Design of connection b/w flange plates and flange:

Bolts/welds joining the plates and flange are to be designed for horizontal shear at that level.

Shear stress @ level of plates & flanges

$$= \frac{F}{bI_{zz}} (a\bar{y})$$

$$= \frac{300 \times 10^3}{150 \times 789.884 \times 10^6} (220 \times 20 \times 225)$$

$$= 2.506 \text{ N/mm}^2.$$

If bearing type bolts are used.

Strength in single shear $= \frac{f_u}{\sqrt{3}} \times 0.78 \times \pi/4\, d^2 \times 1/1.25$

Use 16mm bolts,

Strength in single shear $= \frac{400}{\sqrt{3}} \times 0.78 \times \pi/4 \times 16^2 \times 1/1.25$

$= 28.974\ kN$.

Strength in bearing is more, if minimum edge distances are provided. There are 2 bolts in a pitch distance.

Hence SF per pitch length = Shear strength of 2 bolts

$p \times (150 \times 2.506) = 2 \times 28974$

$p = 154.16mm$.

provide 20mm ϕ bolts @ 150mm c/c.

The distance b/w the centres of any 2 adjacent fasteners in a line lying in direction of stress shall not exceed 16t or 200mm whichever is less. in tension members.

12t or 200mm whichever is less in compression members.

Design strength of laterally unsupported beams:

- Beams are normally used so as to bend abt major zz axis rather than bend about minor yy axis, since they have higher value of I about that axis. In such cases when compression flange is not supported it has a tendency to bend in lateral direction with twisting.

- Bending of compression flange with twisting reduces the load carrying capacity of S/n.

- Restraint against torsional rotation at supports may be provided by

a) being built into walls

b) web/flange cleats

c) bearing stiffeners acting in conjuction with bearing of beams.

d) Lateral end frames/external supports providing lateral restraint to compression flanges @ ends.

- The member may be treated as laterally supported in the following cases:

a) Bending is abt minor axis

b) S/n is hollow/solid bar

c) Non dimensionalised slenderness ratio $\lambda_{LT} < 0$.

$$\lambda_{LT} = \sqrt{f_y / f_{crb}}$$

Take imperfection factors

$\alpha_{LT} = 0.21$ for rolled steel S/n's

$\alpha_{LT} = 0.4$ for welded steel S/n's

The design bending strength of laterally unsupported beam as governed by torsional buckling.

$$M_d = \beta . Z_p . f_{bd} .$$

$\beta = 1.0$ for plastic and compact section.

$= \frac{Z_e}{Z_p}$ for semi compact section.

Effective Length for lateral torsional buckling:

Mentioned in Table 16 in IS 800-2007 for different support conditions.

Ex 6:

An ISMB 500 section is used as a beam over span of 6m, with simply supported at ends. Determine the maximum factored udl load that beam can carry if the ends are restrained against torsion but compression flange is laterally unsupported.

Soln:

For ISMB 500,

Over all depth, $h = 500mm$.

Width of flange, $b = 180mm$.

Tk of flange, $t_f = 17.2mm$

tk of web, $t_w = 10.2mm$.

$r_{yy} = 35.2mm$.

Effective length for torsional buckling $= 6m$

$$\therefore \frac{KL}{r} = \frac{6\times1000}{35.2} = 170.45$$

$$h/t = \frac{500}{17.2} = 29.06.$$

From table 14 in IS800,

To find f_{crb} values:

h/t →	25	29.6	30
KL/r ↓			
170	136.7	. X	121.3
170.45	. . .	O	
180	127.1	Y	112.2.

To get value for $h/t_f = 29.6$ & $KL/r = 170.45$ it needs double linear interpolation.

First get values @ x & y corresponds to $h/t = 29.6$

To get values @ x $\left(\frac{KL}{r} = 170;\ h/t_f = 29.6\right)$

$$f_{crb} = 136.7 - \frac{4.6}{5}(136.7 - 121.3)$$

$$= 124.16\ N/mm^2$$

To get value @ y $\left(\frac{KL}{r} = 180;\ h/t_f = 29.6\right)$

$$f_{crb} = 127.1 - \frac{4.6}{5}(127.1 - 112.2)$$

$$= 114.57\ N/mm^2$$

∴ The value of f_{crb} @ $h/t_f = 29.6$ and $\frac{KL}{r} = 170.45$

To get value @ O.

$$f_{crb} = 124.16 - \frac{0.45}{10}(124.16 - 114.57)$$

$$= 123.5\ N/mm^2.$$

Refer to table 13a in IS 800-2007,

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

we can find $f_{bd} = 77.3\ N/mm^2$ for $f_{crb} = 100$

and $f_{bd} = 106.8\ N/mm^2$ for $f_{crb} = 150$

∴ For $f_{crb} = 123.5$,

$$f_{bd} = 77.3 + \frac{23.5}{50}(106.8 - 77.3)$$

$= 91.165 N/mm^2$.

Section classification:

$$\epsilon = \sqrt{\frac{250}{250}} = 1.0$$

$$b/t_f = \frac{180}{17.2} = 10.46 < 10.5\epsilon$$

$$d = h - 2(t_f + R)$$

$$= 500 - 2(17.2 + 17) = 431.6mm$$

$$\therefore d/t_w = \frac{431.6}{10.6} \leq 105t$$

Hence it belongs to class 2 category.

$$\therefore M_d = \beta_b \cdot Z_p \cdot f_{bd}.$$

$$\beta_b = 1 \;;\; Z_p = 2047.7 \times 10^3 \, mm^3 \;;\; f_{bd} = 91.165 N/mm^2$$

$$\therefore M_d = 1 \times 2047.7 \times 10^3 \times 91.165$$

$$= 189.14 kNm$$

$$M_d = wL^2/8$$

$$189.14 = w \times 6^2/8$$

$$\therefore w = 42.03 kN/m.$$

Self weight $= 86.9 kg/m$.

$= 86.9 \times 9.81 = 852 N/m$.

$= 0.852 kN/m$.

∴ Factored selfwt = 1.5 × 0.852 = 1.278 kN/m

∴ Super imposed udl that beam can carry

= 42.03 − 1.278 = 40.752 kN/m.

check for shear, defln, web buckling & web crippling to ensure that the above load can be carried safely.

Design of Laterally un supported beams:

Assume fbd to find Zp of S/n. After selecting trial S/n, calculations are made to find M of S/n. If S/n selected is not adquate, larger section is to be tried. for selected S/ns all checks are to be applied.

Design of purlins:

- Member ~~resist~~ which rest b/w roof trusses & supports roof sheeting.
- I S/n's, channels, angles, Z S/n's are commonly used.
- It should be spaced on main rafter of trusses
- Most spaced @ 1.35 to 1.4m when AC sheets of length 1.6m.
- Component of DL/LL = W cosθ / W sinθ.
- WL is always normal to rafter.
- SS purlins, $BM = WL^2/8$
- Continuous purlins, $BM = WL^2/10$.

Design of plate girder:

procedure for design of plate girder

1. Assume Self weight is equal to $W/200$

where,

W – total factored Load

determine factored shear force and moment for the design of plate girder.

2. Decide whether to use or not to use transverse stiffeners, and assume the value of k. ie., d/t_w

Determine the economical depth as,

$$d = \left[\frac{Mk}{f_y}\right]^{1/3}$$

Select available plate around this depth.

3. Determine the area of flange required to resist moment. Proportion it so that b_f/t satisfy requirements of plastic/ compact / semi-compact section.

4. Check the moment capacity of the plate girder

5. Find shear resistance of the web using either simple post-critical method / by tension field method

6. Design the weld connecting flange plate & web plate

7. Design the end bearing stiffeners.

8. Design the connection of stiffener.

9. Design Load carrying stiffeners if required.

10. Design the intermediate stiffeners if required.

Ex 1:

Design a welded plate girder of span 24m to carry an super imposed load of 35kN/m. Avoid use of bearing and intermediate stiffeners. Use Fe415 (E250) steel.

Soln:

1. Moment and shear force:

Span = 24m.

Super Imposed Load = 35kN/m.

$\therefore$ Factored Load = 35×1.5 = 52.5kN/m.

$$\text{Self weight} = \frac{52.5 \times 24}{200}$$

$$= 6.3\,kN/m.$$

$\therefore$ Total factored Load = 52.5 + 6.3

= 58.8kN/m.

Maximum moment, $M = \frac{wL^2}{8}$

$= \frac{58.8 \times 24^2}{8}$

$= 4233.6 \text{ kNm}$

Max shear force = End reaction

$M = \frac{wL}{2} = \frac{58.8 \times 24}{2}$

$= 705.6 \text{ kN}$

2. Depth of web plate:

If stiffeners are to be avoided,

$k = d/t_w \leq 67$

∴ Economical depth $d = \sqrt[3]{\frac{Mk}{f_y}}$

$= \left[\frac{4233.6 \times 10^6 \times 67}{250}\right]^{1/3}$

$= 1042.99 \text{ mm}$

Use 1000mm depth plates

$t_w = \frac{1000}{67} \geq 14.92$

Select $t_w = 16\text{mm}$

Hence, Use web plate as 1000 X 16mm.

3. Selection of flange:

Neglect the moment capacity of web, area of flange required is,

$$\frac{A_f \cdot f_y \cdot d}{1.1} \geq M$$

$$\frac{A_f \cdot 250 \times 1000}{1.1} \geq 4233.6 \times 10^6$$

$$\therefore A_f = 18628 \text{ mm}^2.$$

To keep the flange in semi compact category,

$$b/t_f \leq 13.6$$

Assume, $t_f = b/12$

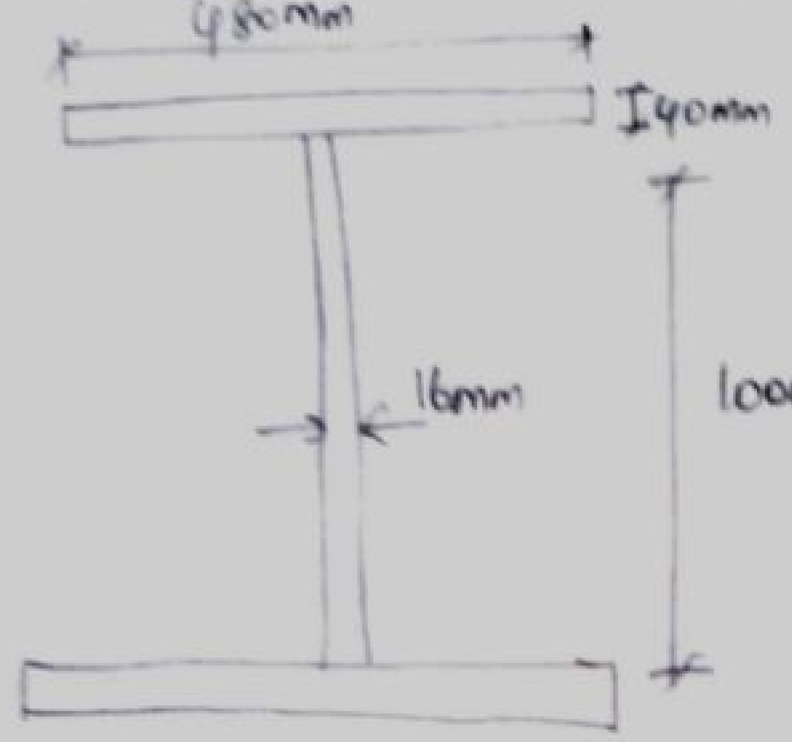
$$\therefore A_f = 12 t_f \cdot t_f = 18268$$

$$\therefore t_f = 39.33 \text{mm}.$$

Select 40mm plates.

Width of plate req = $\frac{18628}{40}$ = 465.7mm.

Hence use 480mm wide and 40mm th plates.



4. Check for moment capacity of the girder:

Since it is assumed that only flanges resist the moment and flange is a semi compact section.

$$M_d = \frac{Z_e \cdot f_y}{\gamma_{mo}}$$

$$I_{zz} = 2\left[\frac{1}{12} \times 480 \times 40^3 + 480 \times 40 \times \left[\frac{1000 + 2 \times 40}{2}\right]^2\right]$$

$$= 2 \times 5196.24 \times 10^6 \, mm^4$$

$$Z_e = \frac{I_{xx}}{y_{max}} = \frac{2 \times 51994.24 \times 10^6}{540} = 19.238 \times 10^6 \, mm^3$$

$$\therefore M_d = \frac{19.238 \times 10^6 \times 250}{1.1} = 4372.256 \times 10^6 \, Nmm$$

$$= 4372.256 \, kNm > m.$$

Hence section is adquate.

5. shear resistance of web:

$$V_d = \frac{V_n}{\gamma_{mo}} = \frac{A_v f_{yw}}{\gamma_{mo} \cdot \sqrt{3}} = \frac{d t_w \cdot f_{yw}}{\gamma_{mo} \cdot \sqrt{3}}$$

$$\therefore V_d = \frac{1000 \times 16 \times 250}{1.1 \times \sqrt{3}} = 2099 kN > 705.6 kN$$

Hence section is adquate.

No stiffeners are required.

6. check for end bearing:

Bearing strength of web,

$$F_w = (b_1 + n_2) t_w \cdot \frac{f_{yw}}{\gamma_{mo}}.$$

Assume, minimum stiffeners provided at support = 100mm.

Dispersion length $n_2 = 2.5 \times 40$
$= 100mm$.

$$F_w = (100+100) \times 16 \times \frac{250}{1.1} = 727 kN > 705.6 kN.$$

Hence adquate.
End stiffeners is not required.

But a minimum of 5mm is to be provided, since thickness of web is 16mm. Intermittent welds may be provided

$\therefore$ % of weld length = $\frac{0.92}{5} \times 100 = 18.4$

Use 40mm long welds with a gap of 160mm which satisfies that:

a) Minimum weld length = 40mm.

b) Maximum unwelded length = 12t

= 12 × 16mm

= 192mm

and also required % welding.

Final result:

Web = 1000 × 16mm

Flange = 480 × 40mm.

No stiffeners are required.

Weld: 5mm intermittent of length 40mm and a gap of 160mm.

7. Design of weld connecting web plate and flange:

Maximum shear force = 705.6kN.

Shear stress in flange at level of junction of web and flange,

$$q = \frac{F}{bI}(a\bar{y}).$$

$$= \frac{705.6\times10^3}{480\times2\times5601.28\times10^6}\left[480\times16\,(500+16/2)\right]$$

$$= 0.512\ N/mm^2.$$

∴ shear force per mm length in the junction

$= 0.512\times480 = 245.76N.$ — ①

If 's' is the size of shop weld, throat th is 0.7S.

provide weld on both sides of web strength / unit length.

$$= 2\times0.7S\times\frac{410}{\sqrt{3}}\times\frac{1}{1.25}$$

$= 265.1S$ — ②

Equate ① & ②,

$245.76 = 265.1S$

$\therefore S = 0.92mm.$

Ex2

Design the above plate girder using thin web and end stiffener, but avoid intermediate stiffeners.

Soln:

1. Maximum moment = 4236.6 kNm

Max shear force = 705.6 kN.

2. Depth of web:

If $d/t_w > 200$, intermediate transverse stiffeners are to be provided.

If $d/t_w \leq 67$, end as well as intermediate transverse stiffeners are not required but thick webs are required. It is around $k = d/t_w = 100$, that thin webs with only end stiffeners can be used.

Try $k = d/t_w = 100$.

$$\text{Economical depth} = \sqrt[3]{\frac{Mk}{f_y}}$$

$$= \left[\frac{4233.6 \times 10^6 \times 100}{250}\right]^{1/3}$$

$$= 1192\,mm.$$

provide 1200mm wide plates.

$$\therefore \frac{12000}{t_w} = 100$$

Use $t_w = 12mm$.

Use 1200× 12mm plates for web

3.Flange:

Assume only flanges resist moment, area of flange A_f req is given,

$$\frac{A_f \cdot f_y \, d}{1.1} \geq 4233.6 \times 10^6$$

$$\frac{A_f \times 250 \times 1200}{1.1} \geq 4233.6 \times 10^6$$

$$\therefore A_f = 15523 mm^2$$

To keep the flange in semi compact category,

$$b/t_f \leq 13.6$$

Assume $b = 13.6 t_f$.

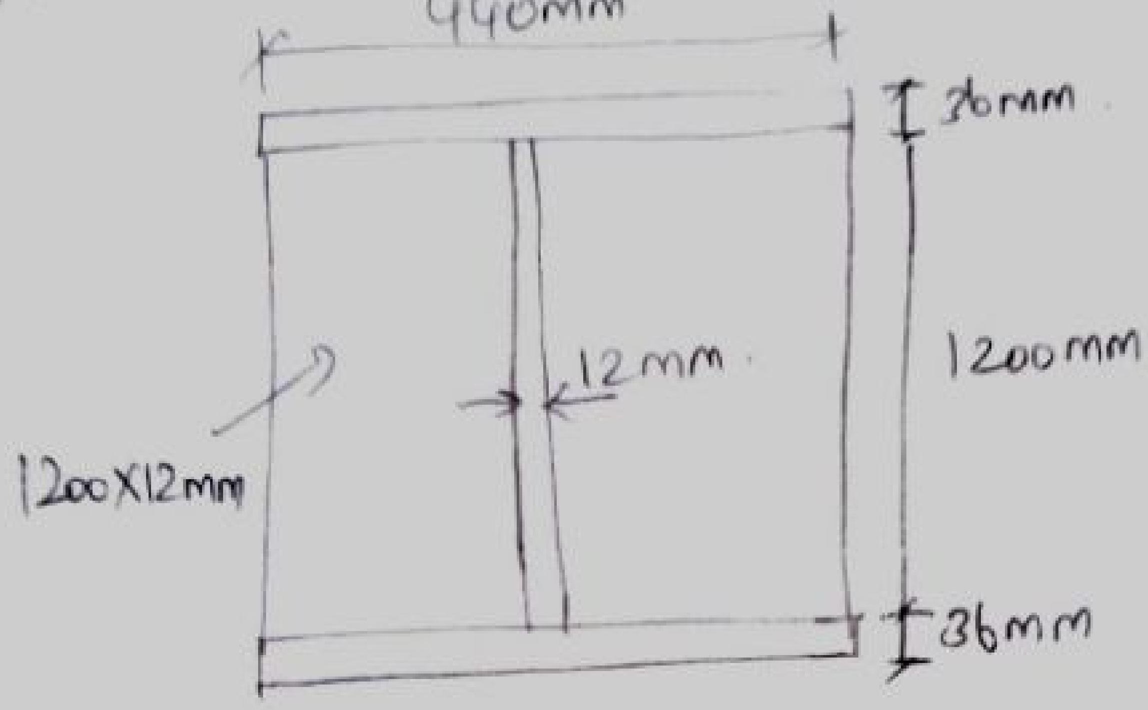
$$\therefore A_f = 13.6 \times t_f \cdot t_f = 15523$$

$$\therefore t_f = 33.8mm$$

Select 36mm t_f plates

$$b = \frac{15523}{36} = 131\,mm.$$

Use 440mm wide and 36mm tk plates.



4 Check for moment capacity:

Assume only flanges resist moment,

$$I_{zz} = 2\left[\frac{1}{12} \times 440 \times 36^3 + 440 \times 36 \times \left(600 + \frac{36}{2}\right)^2\right]$$

$$= 1.21028 \times 10^{10}$$

$$\therefore Z_e = \frac{I_{zz}}{y_{max}} = \frac{1.21028 \times 10^{10}}{636}$$

$$= 19.029 \times 10^6\,mm^3.$$

Since it is plastic section,

$$M_d = \frac{Z_e \cdot f_y}{\gamma_{mo}} = \frac{19.029 \times 10^6 \times 250}{1.1}$$

$$= 4324.9\,kNm > M$$

Hence adequate

5. Shear resistance of web:

Since only transverse stiffeners are to be provided only at support, $K_v = 5.35$

$$d/t_w > 67$$

Hence resistance to shear buckling should be verified.

Consider simple post critical method.

$$\tau_{cr} = \frac{\pi^2 E \cdot K_v}{12(1-\mu^2)(d/t_w)^2} = \frac{5.35 \times \pi^2 \times 2 \times 10^5}{12(1-0.3^2)\left(\frac{1200}{12}\right)^2}$$

$$= 96.7 \, N/mm^2.$$

$$\therefore \lambda_w = \sqrt{\frac{f_{yw}}{\sqrt{3} \cdot \tau_{cr}}} = \sqrt{\frac{250}{\sqrt{3} \times 96.7}} = 1.22$$

Since it is more than 1.2.

$$\tau_b = \frac{f_{yw}}{\sqrt{3} \cdot \lambda_w^2} = \frac{250}{\sqrt{3} \times 1.22^2} = 96.97 \, N/mm^2.$$

$$\therefore V_{cr} = d \times t_w \cdot \tau_b$$

$$= 1200 \times 12 \times 96.97$$

$$= 1396.44 \, kN.$$

$$\therefore V_d = \frac{V_n}{\gamma_{mo}} = \frac{V_{cr}}{\gamma_{mo}} = \frac{1396.44}{1.1}$$

$$= 1269.49\,kN > 705.6\,kN$$

Hence shear strength is adequate.

6. Local capacity of the web:

$$F_w = \frac{(b_1 + n_2)\, t_w \cdot f_{yw}}{\gamma_{mo}}$$

Take $b_1 = 0$

$$n_2 = 2 \times 2.5 \times 40 = 200\,mm$$

$$\therefore F_w = \frac{(0+200) \times 12 \times 250}{1.1} = 545.45\,kN$$

$$< 705.6\,kN$$

Hence end stiffeners should be provided.

7. Design of end stiffeners:

$$\text{Outstand of flange} = \frac{440-12}{2} = 214\,mm$$

Try a pair of 200 × 12 mm flats

$$\therefore 14 t_f = 14 \times 12 = 168\,mm.$$

∴ Core area of stiffener on each side = 168×12mm.

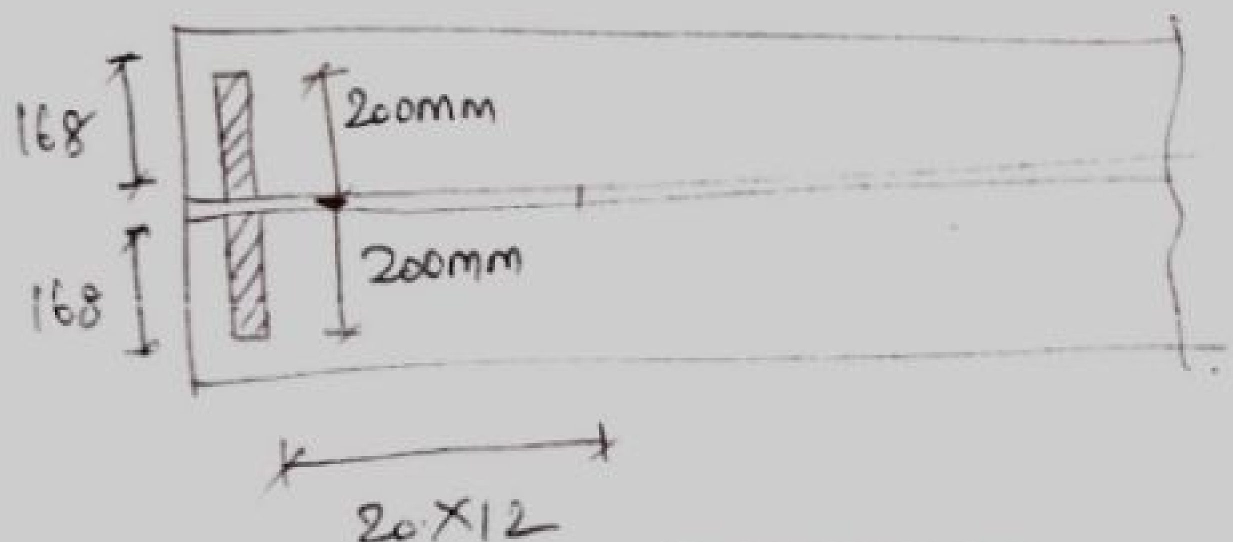
Fig shows the core area of stiffener along with effective area of web ($20t_w$) assume web area is available on only one side.

Area for buckling resistance $= (2\times168 + 20\times12)\times12^3$

$= 6912\ mm^2$

$I_x = 1/_{12} \times 12 (168 + 168 + 12)^3 + 1/_{12} (20\times12)\times12^3$

$= 42.175 \times 10^6 mm^4$.



$r = \sqrt{I/A} = \sqrt{\dfrac{42.175\times10^6}{6912}}$

$= 78.12mm$.

$\therefore \lambda = 0.7 \times \dfrac{1200}{78.12} = 10.75$

From table 9c in IS 800,

$f_{cd} = 226.5\ N/mm^2$.

∴ Buckling resistance = 226.5 × 6912

= 1565.5 kN > 705.6 kN.

Hence the stiffener is safe.

Check for bearing capacity of stiffener:

$$F_{psd} = \frac{A_q \cdot f_{yq}}{0.8\,\gamma_{m0}}$$

A_q = Area of stiffener in contact with flange.

= 2 × 200 × 12

= 4800 mm^2.

$$\therefore F_{psd} = \frac{4800 \times 250}{0.8 \times 1.1}$$

= 1363.6 kN > 705.6 kN

Hence the stiffener is adequate.

Check for torsional restraint provided by stiffener:

As per clause 8.7.4,

$$I_s \geq 0.34\,\alpha_s\,D^3\,T_{cf}.$$

$$I_y = 2 \times \frac{1}{12} \times 36 \times 440^3 + \frac{1}{12} \times 1200 \times 12^3$$

$= 511.277 \times 10^6$ mm^4.

$A = 2 \times 36 \times 440 + 1200 \times 12$

$= 46080 \text{ mm}^2$

$\therefore r_y = \sqrt{\dfrac{511.277 \times 10^6}{46080}} = 105.3 \text{mm}$

$\therefore \lambda = \dfrac{24 \times 1000}{105.3} = 227.85$

$\therefore \alpha_s = \dfrac{30}{\lambda^2} = \dfrac{30}{227.85^2} = 5.779 \times 10^{-4}$

$I_s \geq 0.34 \alpha_s \cdot D^2 t_{cf}$

$\geq 0.34 \times 5.779 \times 10^{-4} \times (1272)^3 \times 36$

$\geq 14.558 \times 10^6$

Hence torsional restraint is sufficient.

8 Weld connecting web and flange:

Shear force, $V = 705.6 \text{kN} = 705.6 \times 10^3 \text{N}$.

$I_z = 1.21028 \times 10^{10} \text{mm}^4$.

$\therefore$ Shear stress $= \dfrac{V}{b I_z} \cdot (a\bar{y})$.

∴ Shearforce per unit length = 186.5 N/mm.

If size of weld is 'S' & if it is provided on both side of web strength of weld per unit length.

$= 2(0.7S) \times 1 \times \frac{f_u}{\sqrt{3}} \times \frac{1}{1.25}$

$= 2 \times 0.70 \times S \times \frac{410}{\sqrt{3}} \times \frac{1}{1.25}$

$= 265.1 \text{ N/mm}$.

$265.1S = 186.5$

$\therefore S = 0.7\text{mm}$

For 12mm plates, min. size of weld to be used = 5mm.

∴ percentage of weld length req $= \frac{0.7}{5} \times 100$

$= 14$.

Min. length of intermittent welds = 40mm.

Max. length of unwelded portion = 12×12 = 144mm.

∴ provide 40mm length with 5mm weld on both sides and give a gap of 140mm.

9 Weld connecting stiffener and web:

Shear carried by web directly:

$\lambda = 2.42\, d/t = 2.42 \times \frac{1200}{12} = 242.$

$n_1 = D/2 = \frac{1200 + 2 \times 36}{2}$

$= 636 mm$

From table 9c in IS 800;

$f_{cd} = 26.2 - 2/10\,(26.2 - 24.3)$

$= 25.82 N/mm^2.$

If b is assumed zero,

Area of web resisting shear = $b t_w = 632 \times 12$

∴ Load directly transferred by web

$= 632 \times 12 \times 25.82$

$= 196.7 kN$

∴ Shear to be transferred through weld.

= Reaction − shear directly transferred by web

$= 705.6 - 196.7 = 508.9 kN.$

Length of weld = $1200 - 2 \times 12$

$= 1176 mm$

Shear to be transferred = $\frac{508.9}{1176} = 0.433\ kN/mm.$

To this additional shear of

$$= \frac{t_w^2}{5 b_s} \text{ is to be added.}$$

$$= \frac{12 \times 12}{5 \times 200} = 0.144 \, kN/mm.$$

$\therefore$ Total design shear for weld $= 0.433 + 0.144$

$= 0.577 \, kN/mm$

$= 577 \, N/mm. \quad \text{--- (1)}$

If 's' is the fillet weld size and weld is provided on both sides, strength of weld

$$= 2 \times 0.7 s \times \frac{410}{\sqrt{3}} \times \frac{1}{1.5}$$

$$= 220.95 \, N/mm \quad \text{--- (2)}$$

Equate (1) & (2),

$$220.95 s = 577$$

$$\therefore s = 2.62 mm$$

provide 5mm size intermittent welds for a length of 55mm and a gap of 45mm. Welding on both sides.

<u>Result</u>:

Web: 1200 × 12mm.

Flange: 440 × 12mm

End stiffeners: 200 × 12mm.

Weld Connecting flange and web: 40mm with a gap of 140mm, size 5mm on both sides weld connecting stiffener and web: 55mm and a gap of 45mm. size 5mm on both sides.

EX3:

Design the plate girder given in ex 1 using intermediate stiffeners.

Soln:

1. Moment and shear.

Maximum moment = 4233.6kNm.

Max. shear force = 705.6kN.

2. Depth of web:

If stiffener spacing c is between d and 3d

where,

d = depth of web; then serviceability requirements

$K = d/t_w \leq 200$.

Take $K = d/t_w = 190$, we get economical depth as,

$$d = \left[\frac{MK}{f_y}\right]^{1/3}$$

$$= \left[\frac{4233.6 \times 10^6 \times 190}{250}\right]^{1/3} = 1476\text{mm}.$$

Use 1500mm plates.

$\therefore t_w = \frac{1500}{190} = 7.89mm$.

$\therefore$ Use 1500mm wide, 8mm tk plates.

provide stiffeners at every 2m interval ($3d \geq c \geq d$)

3. Flange:

Assume flange alone resists the moment,

$$\frac{A_f \cdot f_y \cdot d}{1.1} \geq M.$$

$$\frac{A_f \times 250 \times 1500}{1.1} \geq 4233.6 \times 10^6$$

$$\therefore A_f = 12418 mm^2.$$

To keep flange in semi plastic class,

$$b_f \leq 13.6\, t_f.$$

Take $b_f = 13.6 t_f$.

$$\therefore 13.6 t_f \cdot t_f \geq 12418$$

$$\therefore t_f \geq 30.2mm$$

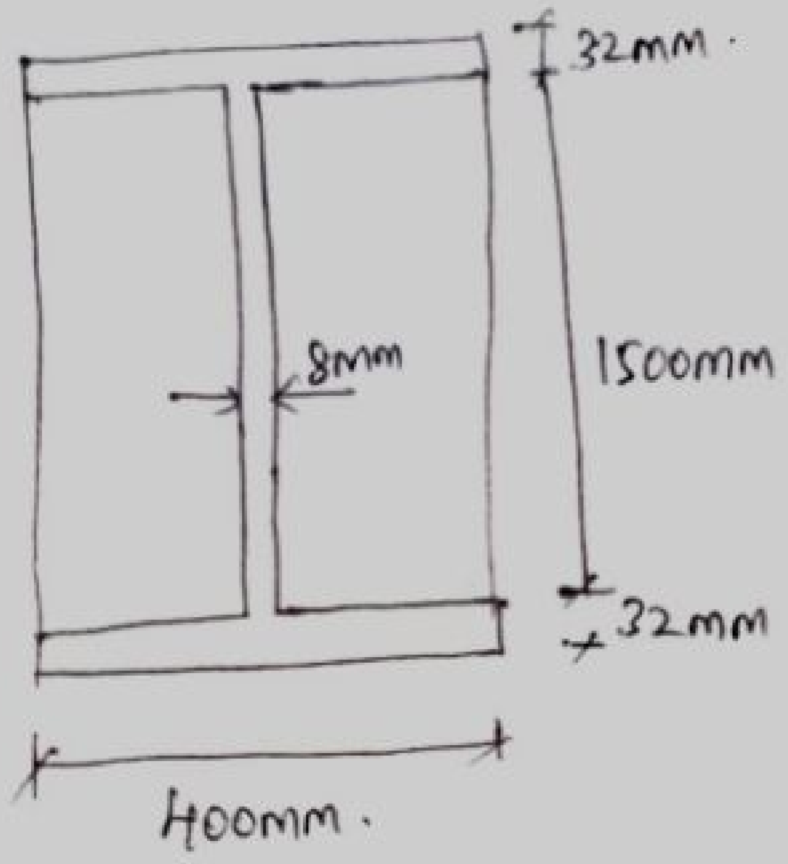
provide 32mm tk plates.

$$\therefore b_f = \frac{12418}{32} = 388mm.$$

Use 400mm wide and 32mm tk plates.

4. Check for shear buckling:

Use simple post critical method,



For $c/d \geq 1.0$

$$k_v = 5.35 + \frac{4}{(c/d)^2} = 5.35 + \frac{4}{\left(\frac{2000}{1000}\right)^2} = 7.6$$

$$\tau_{cr} = \frac{k_v \cdot \pi^2 E}{12(1-\mu^2)(d/t_w)^2}$$

$$= \frac{7.6 \times \pi^2 \times 2\times10^5}{12(1-0.3^2)\left(\frac{1500}{8}\right)^2} = 39.08$$

$$\lambda_w = \sqrt{\frac{f_{yw}}{\sqrt{3}\,\tau_{cr}}} = \sqrt{\frac{250}{\sqrt{3}\times 39.08}} = 1.92$$

Since, $\lambda_w > 1.2$,

$$\tau_b = \frac{f_{yw}}{(\sqrt{3}\,\lambda_w^2)} = \frac{250}{\sqrt{3}\times 1.92^2} = 39.15\ N/mm^2.$$

$\therefore V_n = V_{cr} = A_v \tau_b$

$= 1500 \times 8 \times 39.15$

$= 469.8 \text{ kN} < 705.6 \text{ kN}$

Hence intermediate stiffeners are to be used to improve buckling strength of the slender web and shear capacity of end panel should be checked.

5. Check for end panel:

Since it is going to be stiffened web panel, it should be checked as per clause 8.5.3 of IS800.

$$V_p = \frac{d t f_y}{\sqrt{3}} = \frac{1500 \times 8 \times 250}{\sqrt{3}}$$

$$V_p = 1732.05 \text{ kN}$$

$$H_q = 1.25 V_p \left(1 - \frac{V_{cr}}{V_p}\right)^{0.5}$$

$$= 1.25 \times 1732.05 \left(1 - \frac{469.8}{1732.05}\right)^{0.5}$$

$$= 1848.26 \text{ kN}$$

$$\therefore R_f = H_q/2 = 924.13 \text{ kN}$$

$$M_{tf} = \frac{H_q d}{10} = \frac{1848.26 \times 1500}{10}$$

$$= 277.239 \text{ kNm}$$

The end panel is to be checked as a beam spanning between the flanges to resist R_{tf} and M_{tf}.

Area resisting shear $= t_w \cdot d = 8 \times 1500$

$= 12000 \text{ mm}^2$

$$V_d = \frac{A_v \cdot f_{yw}}{\sqrt{3} \cdot \gamma_{mo}} = \frac{12000 \times 250}{\sqrt{3} \times 1.1}$$

$V_d = 1574.59 \text{ kN} > 705.6 \text{ kN}$

End panel can safely carry the shear due to the anchoring forces.

$$I = \frac{1}{12} \cdot t_w \cdot c^3 = \frac{1}{12} \times 8 \times 2000^3$$

$= 5333.3 \times 10^6 \text{ mm}^4$.

$$y_{max} = c/2 = \frac{2000}{2} = 1000 \text{ mm}.$$

$$M_q = \frac{I}{y_{max}} \times \frac{f_y}{\gamma_{mo}} = \frac{5333.3 \times 10^6}{1000} \times \frac{250}{1.1}$$

$= 1212.12 \text{ kNm} > M_{tf}$.

Hence the end panel can carry the bending moment due to anchor forces.

6. Design of end stiffeners:

Reaction at end $= 705.6 \text{ kN}$

Compressive force due to moment $M_{tf} = \frac{M_{tf}}{c}$

$$= \frac{277.239 \times 10^6}{2000} = 138.62 \times 10^3 \, N$$

$$= 138.62 \, kN.$$

$\therefore$ Total compression $= 705.6 + 138.62$

$= 844.22 \, kN. \quad \leftarrow (1)$

Strength of the stiffener clause (8.7.5.2):

$$F_{psd} = \frac{A_q \cdot f_{yq}}{0.8 \gamma_{mo}} = \frac{A_q \times 250}{0.8 \times 1.1} \quad - (2)$$

Equate (1) & (2),

$$\frac{A_q \times 250}{0.8 \times 1.1} = 844.22 \times 10^3$$

$$\therefore A_q = 2972 \, mm^2.$$

Check for out stand:

It should not be more than $20 t_q = 20 \times 10$

$= 200 mm.$

The requirement is satisfied.

Since it is more than $14 \times 10 mm$, the core section is based on width $14 \times 10 = 140 mm^2$.

$\therefore$ Core area of each stiffener $= 140 \times 10$

$= 1400 mm^2.$

Buckling check for stiffeners:

Considering stiffeners only,

$$I_s = \frac{1}{12} \times 10 \times (400-8)^3 - \frac{1}{12} \times 10 \times 8^3$$

$$= 50.196 \times 10^6 \text{ mm}^4.$$

Effective area $= 2 \times 140 \times 10 = 2800 \text{ mm}^2.$

$$r = \sqrt{I/A} = \sqrt{\frac{50.196 \times 10^6}{2800}}$$

$$= 133.89.$$

$$KL_c = 0.7d = 0.7 \times 1500$$

$$= 1050 \text{ mm}.$$

$$\lambda = \frac{KL_c}{r} = \frac{1050}{133.89} = 7.84.$$

From table 9c in IS 800,

$$f_{cd} = 227 \text{ N/mm}^2.$$

Assume $20 \times$ web tk on only one side,

effective area $= 2 \times 140 \times 10 + 20 \times 20 \times 8$

$= 6000 \text{ mm}^2$

∴ Buckling resistance of stiffener = 6000 × 227

= 1362 kN

> 844.22 kN

Hence stiffener is adquate.

Check stiffener for load bearing (clause 8.7.4).

Take stiff bearing, $b_1 = 0$

$n_2 = 2.5\, t_f = 2.5 \times 32 = 80mm$.

Local capacity of web,

$$F_w = \frac{(b_1 + n_2)\, t_w \cdot f_{yw}}{\gamma_{mo}} = \frac{(0+80) \times 8 \times 250}{1.1}$$

= 145.454 kN.

∴ The stiffener is to be designed for a force

= 844.22 − 145.454

= 698.766 kN

$f_{cd} = 227 N/mm^2$

Area of stiffener alone = 2 × 200 × 10 = 4000 mm^2.

Bearing capacity of stiffener alone = $\frac{227 \times 4000}{1.1}$

= 825.454 kN > 698.77 kN

Hence stiffener is safe.

Use end stiffeners are 200 × 10 mm are adquate.

7. Design of intermediate stiffeners:

As SF goes on reducing towards mid span, the first stiffener from end is critical. Since first intermediate stiffener is at c = 2m from end,

Shear on this stiffener = R − 2w

= 705.6 − 2×58.8

= 588 kN.

In this case, c = 2000 mm

d = 1500 mm

$\therefore c/d = \frac{2000}{1500} = 1.33 < \sqrt{2}$.

Hence minimum, $I_s = \frac{1.5 d^3 t_w^3}{c^2}$

$= \frac{1.5 \times 1500^3 \times 8^3}{2000^2}$

= 648000 mm⁴.

Try intermediate stiffeners of size 120×10mm on each side

$I_s = \frac{1}{12} \times 10 \times (120+8+120)^3 - \frac{1}{12} \times 10 \times 8^3$

$= 12.71 \times 10^6 \text{ mm}^4 > I_s$ req

Hence adequate.

Check for buckling:

. Shear buckling resistance of web alone,

$$V_{cr} = 469.8\,kN$$

· shear strength of stiffeners alone req $= \dfrac{V - V_{cr}}{\gamma_{mo}}$

$$= \frac{588 - 469.8}{1.1} = 107.45\,kN.$$

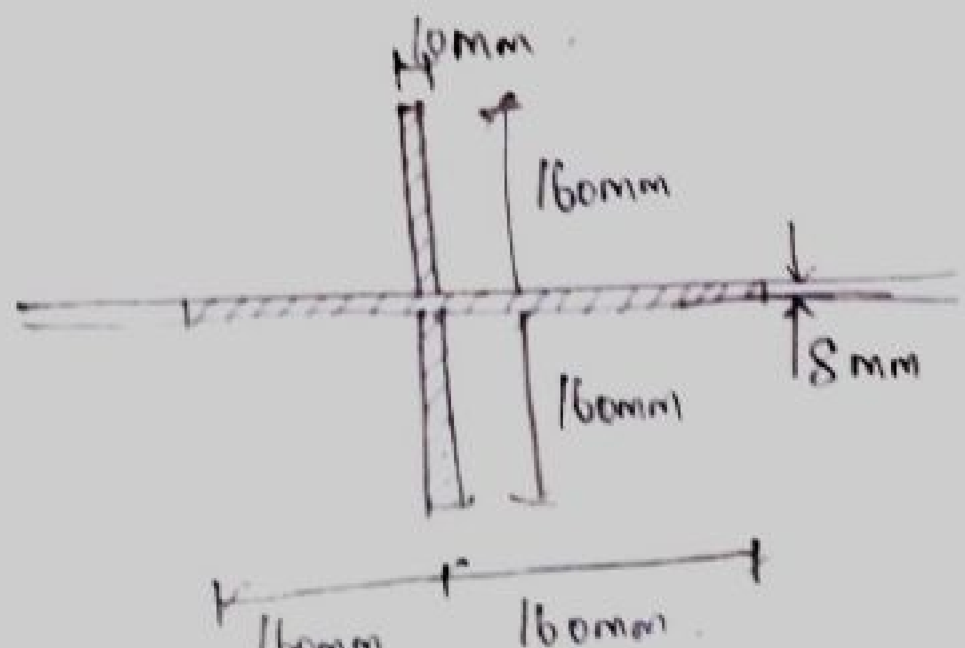
Buckling resistance of intermediate stiffener (cl: 8.7.15):

Consider $20 \times t_w = 20 \times 8 = 160\,mm$ width of web on both sides along with stiffener.

$$I_x = I_s + 2 \times \tfrac{1}{12} \times 160 \times 8^3$$

$$= 12.71 \times 10^6 + 13653$$

$$= 12.73 \times 10^6\,mm^4.$$



Area: $2 \times 120 \times 10 + 2 \times 160 \times 8 = 4960\,mm^2$

$$\cdot\ r = \sqrt{I/A} = \sqrt{\frac{12.73 \times 10^6}{4960}} = 50.66$$

$KL = 0.7 \times 1500 = 1050mm$

$\therefore \lambda = KL/r = \frac{1050}{50.66} = 20.73.$

From table 9c in IS 800,

$f_{cd} = 224 N/mm^2$

$\therefore$ Buckling resistance $= 224 \times 4960$

$= 1111 kN.$

$> 107.45 kN.$

Hence the stiffener is safe.

Design of welds are to be carried out similar to previous example.

Important Notes:

1. Economical depth of plate girder,

$$d = \left[\frac{MK}{f_y}\right]^{1/3} \quad \text{where } k = d/t_w.$$

If $K < 67$, it is thick web design. It does not require any stiffener.

If $K = 100$; it is economical & it may not require intermediate stiffener, but end stiffeners are required.

For larger value of k, intermediate stiffeners are req. In such cases, end panel should be checked for local failures by post buckling mtd / tension field method.

Reg. No. : [ ]

Question Paper Code : 50112

B.Arch. DEGREE EXAMINATIONS, APRIL/MAY 2024.

Sixth Semester

AR 3602 — STRUCTURAL DESIGN OF STEEL

(Regulations 2021)

Time : Three hours | Maximum : 100 marks

(Use of IS 800 and IS SP-6 is allowed)

Answer ALL questions.

PART A — (10 × 2 = 20 marks)

1. Write how standard sections are designated as per IS? What do the numbers in these designations indicate?

2. What are the disadvantages of using steel in construction?

3. With neat sketches show the possible failure modes of a bolted joint.

4. Show how the strength of a welded joint is determined.

5. How the sectional area of a tension member is modified when used with bolted connection?

6. What are splices and when they are used?

7. State the significance of effective length for compression members.

8. Sketch four typical built up sections used in heavy compression members.

9. What is meant by lateral support in beams?

10. Why symmetric sections are preferred for beams?

PART B — (5 × 16 = 80 marks)

11. (a) Listing out the required characteristics of structural steel, explain the significance of its ductility in design and construction.

Or

(b) What are the different types of structural steel? Define and explain all the mechanical properties to be tested and satisfied as per codal requirements.

12. (a) Using M16 bolts of grade 4.6, design the bolted connection for joining two steel plates of grade 410, width 800 mm and thickness 20 mm each by

(i) Lap Joint (8)

(ii) Double cover butt joint with 12 mm cover plates. Axial tensile force transferred by the plates is 300 kN. (8)

Or

(b) (i) Two plates of thickness 12 mm each are to be connected by groove weld by butting the plates against each other. The joint has to carry a tensile force of 350 kN. Assuming the effective length as 150 mm, find the size of the weld required both for single V-groove and Double V-groove joints. (10)

(ii) Find the size of the weld if it is provided as a fillet by rapping the plates one over the other. (6)

13. (a) Determine the tensile strength of a roof truss diagonal, 100 × 75 × 6 mm connected to a gusset plate by a 4 mm weld with an effective length of 200 mm on each side of the connected leg. Assume yield strength of steel as 250 MPa.

Or

(b) A tensile force of 350 kN is to be carried by a single unequal angle through a gusset plate using 20 mm black bolts. Design the section with the required number of bolts to transmit the force. Take yield strength of steel as 250 MPa.

14. (a) Design the load carrying capacity of a single discontinuous 50 × 50 × 5 mm angle, when used as a compression member connected to a gusset plate with two bolts. The centre to centre of end connections is 1.5 m. The grade of steel is E250.

Or

(b) Design a single angle section to carry a compressive force of 120 kN with centre to centre between end connections being 2 m. Grade of steel is E250.

15. (a) An ISMB 350 at 52.4 kg/m section is used as a simply supported beam with an effective span of 5 m. Find the design bending strength and design shear strength of the beam.

Or

(b) Design a simply supported I section of clear span of 8.5 m to carry a factored bending moment of 275 kNm and a factored shear force of 125 kN. Grade of steel is E250. Assume that adequate lateral support is provided to the compresstion flange.

Reg. No. : | | | | | | | | | | | | |

Question Paper Code : 90119

B.Arch. DEGREE EXAMINATIONS, APRIL/MAY 2025.

Sixth Semester

AR 3602 — STRUCTURAL DESIGN OF STEEL

(Regulations 2021)

Time : Three hours | Maximum : 100 marks

(Use of IS 800 and steel hand book is permitted)

Answer ALL questions.

PART A — (10 × 2 = 20 marks)

1. What are the properties of mild steel?

2. List any four standard steel section available in market.

3. State the advantages of bolt connection over welded connection.

4. Differentiate between a Lap joint and Butt Joint.

5. What are the various types of tension members?

6. What is net sectional area?

7. Define slenderness ratio of a compression member.

8. What is meant by built-up compression members?

9. What is a laterally supported beam?

10. How does a laterally supported beam fail?

PART B — (5 × 16 = 80 marks)

11. (a) Sketch the typical stress-strain curve of mild steel, indicating the three important regions and write a short note on each of them?

Or

(b) State the advantages and disadvantage of steel as a structural material.

12. (a) Two plates 10 mm and 18-mm thick are to be joined by a double cover butt joint. Assuming cover plates of 8-mm thickness, design the joint to transmit a factored load of 500 kN. Assume Fe 410 plate and grade 4.06 bolt.

Or

(b) Determine the size and length of the fillet weld for the lap joint to transmit a factored load of 120 kN shown in Fig. 1; assuming site welds, Fe 410 steel. Assume width of the plate as 75 mm.

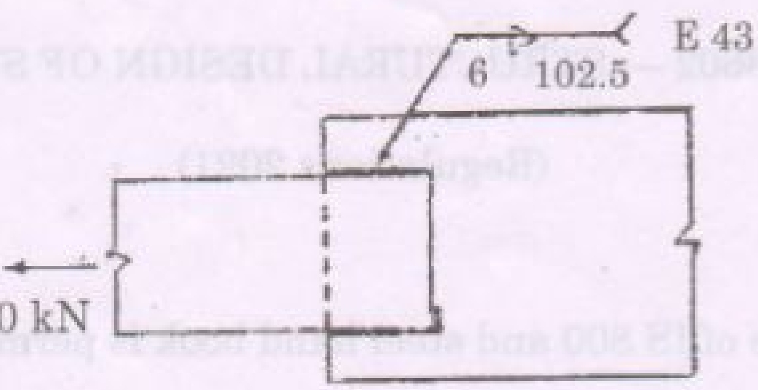


Fig. 1

13. (a) Design a tension splice to connect two plates of size $220\,\text{mm} \times 20\,\text{mm}$ and $200\,\text{mm} \times 10\,\text{mm}$, for a design load of 220 kN. Also sketch the details of the bolt joint. Assume 4.6 grade bolt.

Or

(b) Design a single angle to carry a tensile load of 500 kN. Assume that the length of the member is 3 m.

14. (a) Design a rolled steel beam section column to carry an axial load 1100 kN. The column is 4 m long and adequately restrained in position but not in direction at both ends.

Or

(b) A rolled steel beam section ISHB 350 @ 0.674 kN/m is used as a stanchion. If the unsupported length of the stanchion is 4 m, determine safe load carrying capacity of the section.

15. (a) A beam simply supported over an effective span of 7 m, carries an uniformly distributed load of 50 kN/m inclusive of its own weight. The depth of the beam is restricted to 450 mm design the beam, assuming that the compression flange of the beam is laterally supported by a floor construction. Take $f_y = 250\,\text{N/mm}^2$ and $E = 2 \times 10^5\,\text{N/mm}^2$. Assume width of the support is 230 mm.

Or

(b) ISMB 550 @ 1.037 kN/m has been used as simply supported beam over a span of 4 m. The ends of beam are restrained against torsion and lateral bending. Determine the safe UDL per metre, which the beam can carry.

Reg. No. : | | | | | | | | | | | | |

Question Paper Code : 40124

B.Arch. DEGREE EXAMINATIONS, NOVEMBER/DECEMBER 2024.

Sixth Semester

AR 3602 — STRUCTURAL DESIGN OF STEEL

(Regulations 2021)

Time : Three hours Maximum : 100 marks

Answer ALL questions.

PART A — ($10 \times 2 = 20$ marks)

1. Define the term tensile strength.
2. Mention two advantages and two disadvantages of using steel as a construction material.
3. State any two advantages and two disadvantages of bolted joints in teel construction.
4. Name any types of welded joints commonly used in structural applications.
5. State any two factors affecting the permissible stresses in tension members.
6. Define the term net sectional area for a tension member.
7. Define the term "slenderness ratio" with respect to compression members.
8. State the importance of the limit state method in the design of columns.
9. State the function of flexural member in structural engineering.
10. Define the term "moment of inertia" in relation to beam design.

PART B — ($5 \times 16 = 80$ marks)

11. (a) Discuss the properties of steel and compare the advantages and disadvantages of using steel versus concrete as the primary material for a high-rise building structure, in terms of strength, flexibility, time, construction and cost.

Or

(b) Discuss about the various steel section available in the market and their application in building construction with suitable sketches.

12. (a) Describe the various types of bolted and welded joints used for axially loaded members and the failure modes associated with bolted and welded joints under axial loading.

Or

(b) Explain in detail the key assumptions made in the design of bolted and welded joints for axially loaded members using the limit state method.

13. (a) Explain the factors affecting the net sectional area in the design of axially loaded tension members.

Or

(b) Illustrate the function and design considerations of a lug angle in tension members.

14. (a) Explain the behavior of different cross-sectional shapes used for compression members under axial loading.

Or

(b) Describe the design considerations for built-up sections in compression members using the limit state method.

15. (a) Explain the behavior of laterally supported beams under flexural loading.

Or

(b) Discuss the factors affecting the bending strength of a laterally supported beam.
