ROHINI
COLLEGE OF ENGINEERING & TECHNOLOGY
Approved by AICTE and Affiliated to Anna University, (An ISO Certified Institution)

DEPARTMENT OF CIVIL ENGINEERING

CE8601 / Design of Steel Structural Elements

(VI – SEMESTER)

Course material

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SYLLABUS

CE8601 DESIGN OF STEEL STRUCTURAL ELEMENTS L T P C 3 2 0 4

OBJECTIVE:

- To introduce the students to limit state design of structural steel members subjected to compressive, tensile and bending loads, including connections. Design of structural systems such as roof trusses, gantry girders as per provisions of current code (IS 800 - 2007) of practice for working stress and Limit state Method.

UNIT I INTRODUCTION AND ALLOWABLE STRESS DESIGN 9+6

Structural steel types – Mechanical Properties of structural steel- Indian structural steel products- Steps involved in the Deign Process -Steel Structural systems and their Elements- -Type of Loads on Structures and Load combinations- Code of practices, Loading standards and Specifications - Concept of Allowable Stress Method, and Limit State Design Methods for Steel structures-Relative advantages and Limitations-Strengths and Serviceability Limit states.

Allowable stresses as per IS 800 section 11 -Concepts of Allowable stress design for bending and Shear –Check for Elastic deflection-Calculation of moment carrying capacity –Design of Laterally supported Solid Hot Rolled section beams-Allowable stress deign of Angle Tension and Compression Members and estimation of axial load carrying capacity.

UNIT II CONNECTIONS IN STEEL STRUCTURES 9+6

Type of Fasteners- Bolts Pins and welds- Types of simple bolted and welded connections Relative advantages and Limitations-Modes of failure-the concept of Shear lag-efficiency of joints- Axially loaded bolted connections for Plates and Angle Members using bearing type bolts –Prying forces and Hanger connection– Design of Slip critical connections with High strength Friction Grip bolts.- Design of joints for combined shear and Tension- Eccentrically Loaded Bolted Bracket Connections- Welds-symbols and specifications- Effective area of welds-Fillet and but Welded connections-Axially Loaded connections for Plate and angle truss members and Eccentrically Loaded bracket connections.

UNIT III TENSION MEMBERS 9+6

Tension Members - Types of Tension members and sections –Behaviour of Tension Members- modes of failure-Slenderness ratio- Net area – Net effective sections for Plates ,Angles and Tee in tension –Concepts of Shear Lag- Design of plate and angle tension members-design of built up tension Members-Connections in tension members – Use of lug angles – Design of tension splice.

UNIT IV COMPRESSION MEMBERS 9+6

Types of compression members and sections–Behaviour and types of failures-Short and slender columns- Current code provisions for compression members- Effective Length, Slenderness ratio –Column formula and column curves- Design of single section and compound Angles-Axially Loaded solid section Columns- Design of Built up Laced and Battened type columns – Design of column bases – Plate and Gusseted bases for Axially loaded colums- Splices for colums.

UNIT V DESIGN OF FLEXURAL MEMBERS 9+6

Types of steel Beam sections- Behaviour of Beams in flexure- Codal Provisions – Classification of cross sections- Flexural Strength and Lateral stability of Beams –Shear Strength-Web Buckling, Crippling and defection of Beams- Design of laterally supported Beams- Design of solid rolled section Beams- Design of Plated beams with cover plates - Design Strength of Laterally unsupported Beams – Design of laterally unsupported rolled section Beams- Purlin in Roof Trusses-Design of Channel and I section Purlins.

TEXT BOOK: (T)

T1. Gambhir. M.L., "Fundamentals of Structural Steel Design", McGraw Hill Education India
Pvt. Ltd., 2013

T2. Shiyekar. M.R., "Limit State Design in Structural Steel", Prentice Hall of India Pvt. Ltd,
Learning Pvt. Ltd., 2nd Edition, 2013.

T3. Subramanian.N, "Design of Steel Structures", Oxford University Press, New Delhi, 2013.

REFERENCES: (R)

R1. Narayanan.R.et.al. "Teaching Resource on Structural Steel Design", INSDAG, Ministry of
Steel Publications, 2002

R2. Duggal. S.K, "Limit State Design of Steel Structures", Tata McGraw Hill Publishing
Company, 2005

R3. Bhavikatti.S.S, "Design of Steel Structures" By Limit State Method as per IS:800–2007, IK
International Publishing House Pvt. Ltd., 2009

R4. Shah.V.L. and Veena Gore, "Limit State Design of Steel Structures", IS 800–2007
Structures Publications, 2009.

R5. IS800 :2007, General Construction In Steel - Code of Practice, (Third Revision), Bureau of
Indian Standards, New Delhi, 2007

ONLINE RESOURCES:

O1) www.nptel.ac.in
O2) https://nptel.ac.in/courses/105105162/

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 = 12×10^{-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

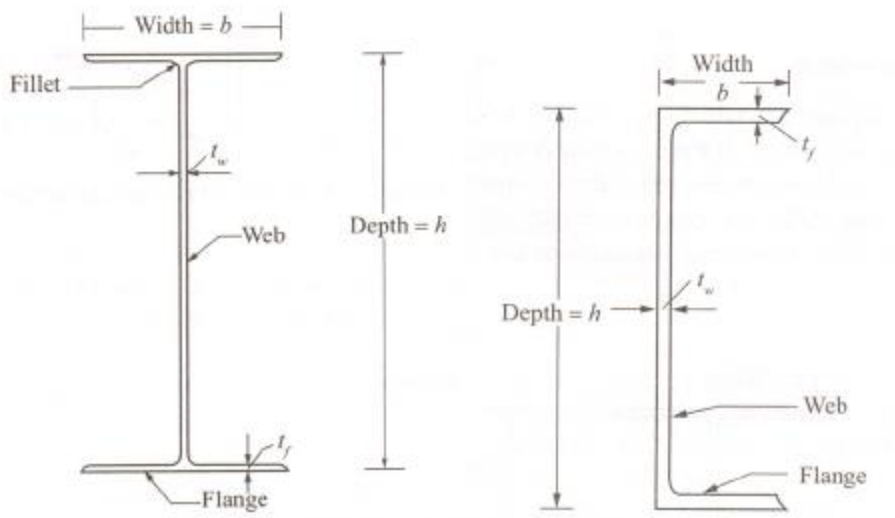
OTHER SERVICEABILITY LIMITS:-

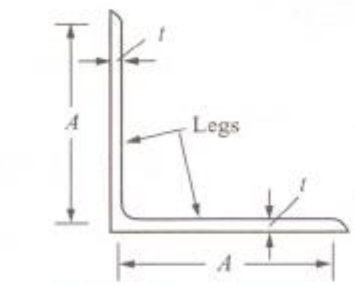
- Vibration Limit
- The flows which are subjected to vibration (supporting machineries) or to be checked for vibration under dynamic loads annex C IS 800-2007 gives the set of guide lines to take care of vibration limits.
- During concentration the following factors affects the durability of steel structure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

FIRE RESISTANCE:-

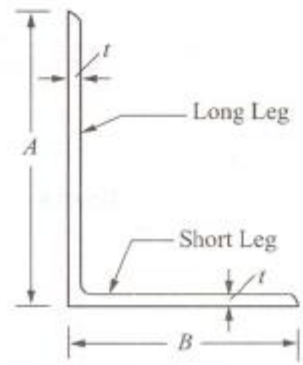
- Fire resistance level [FRL] specified in terms of limit depending upon the purpose for which the structure is used and the time taken to evacuate in case of fire.
- Section 16 of IS 800-2007 deals with fire resistance.
- In addition to the above the stability of structure to be checks due to over turning 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:

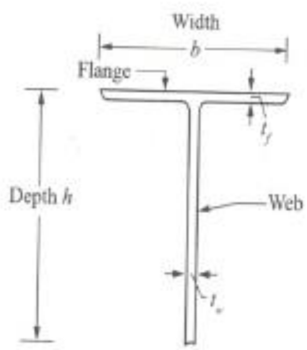




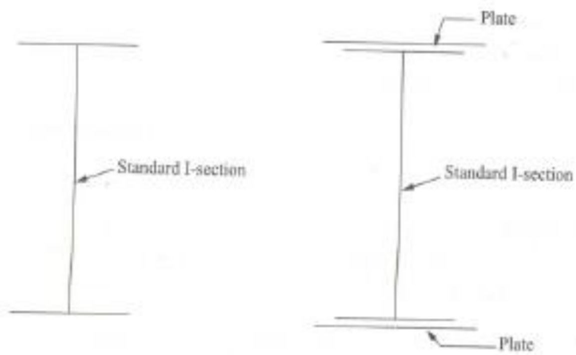
(a) Rolled steel equal angle.



(b) Rolled steel unequal angle.



Rolled steel T-section.

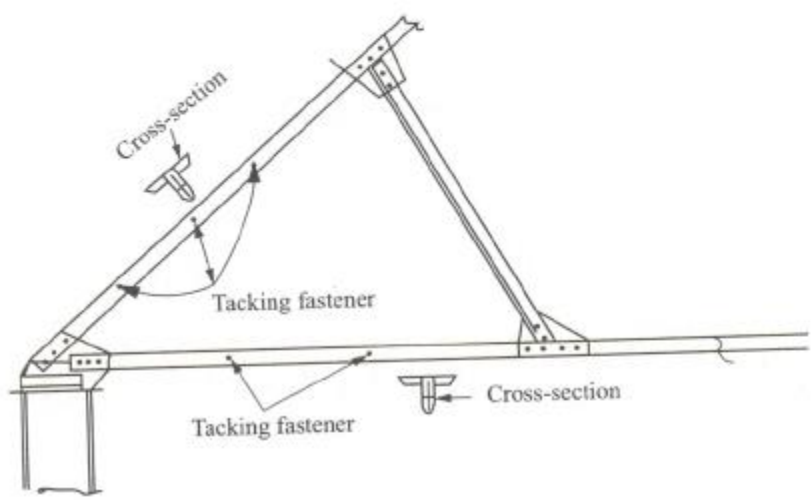


Standard section and built up section.

<u>DESIGN OF CONNECTIONS:-</u> [Section-10 IS 800-2007]

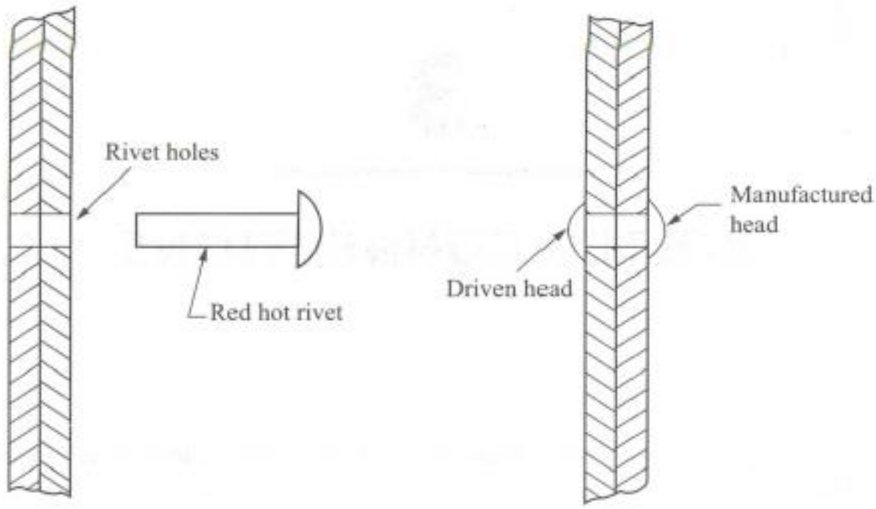
The possible connections in steel designs are

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



1. Riveted Connections:-

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



Riveting.

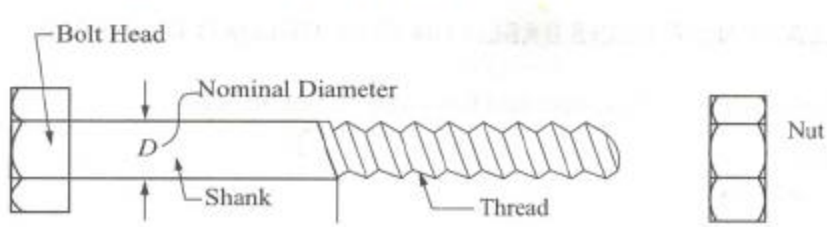
DISADVANTAGES:-

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) Block Bolts:- [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.

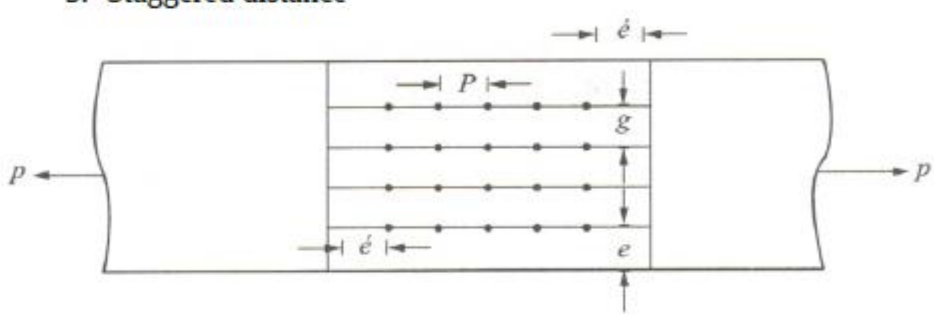
Generally the dia of bolt hole is 1.5mm to 2m larger than the nominal dia of shank.

b) Finished Bolts:- [Turned Bolts]

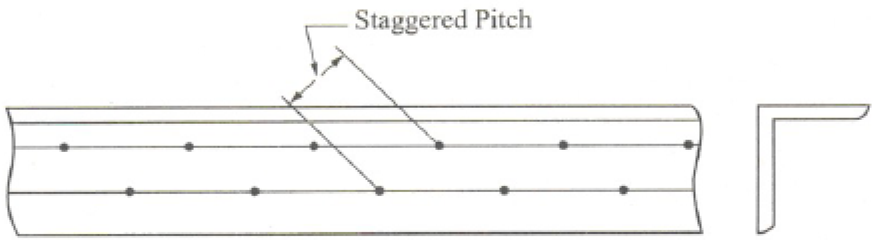
- These bolts are made from M.S.steel formed from hexagonal rods which are finished by turning to a circular shape within the bolts hole.
- The actual dimension of the bolt holes are kept 1.2 to 1.3mm larger than the nominal dia. Where the blot hole is kept 1.5mm larger than 'd'
- Here aligning the bolt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

TERMINOLOGY:-

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.

IS 800-2007 SPECIFICATIONS:- [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

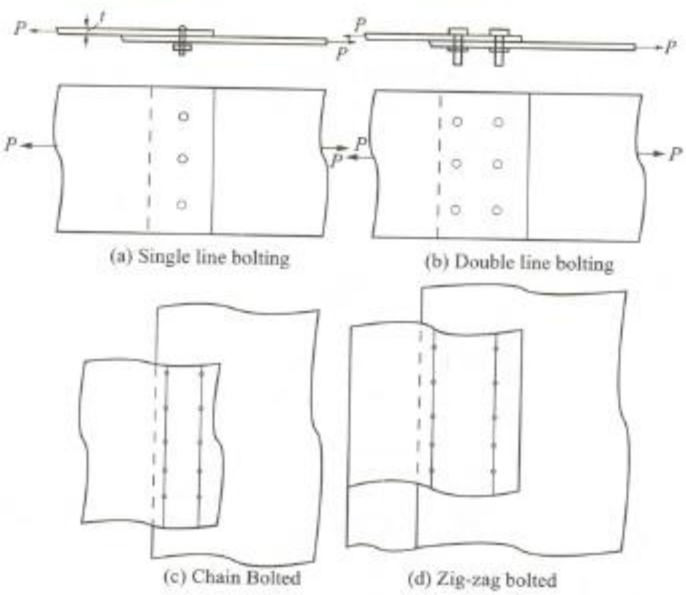
   (i) 1.7 times the hole dia in case of hand flame edges.

   (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

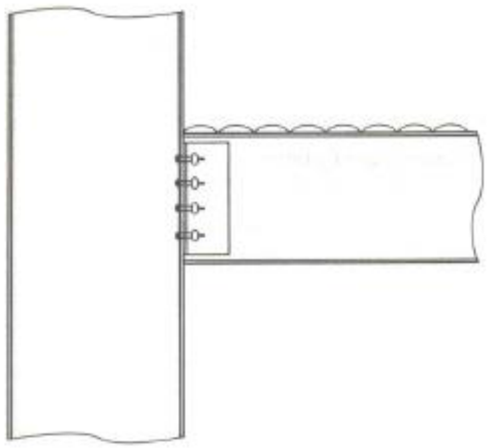


Types of lap joints.

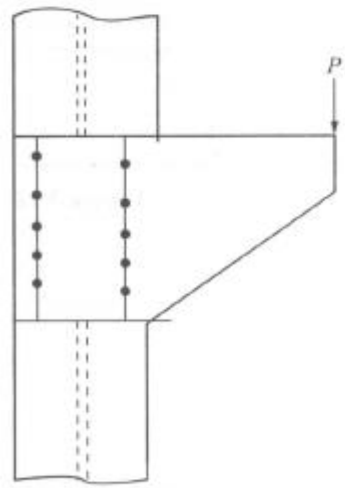
Lap joint is established by overlapping one plate to the other. Butt joint is made by placing the two plates to butt [edges facing each other] and connection

Internal forces on bolts:-

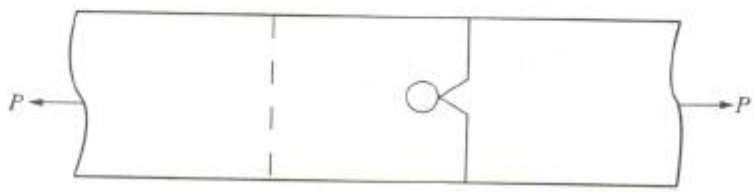
1. Single shear
2. Double shear
3. Pure tension
4. Pure moment
5. Shear & moment in the plane of connection
6. Shear & tension



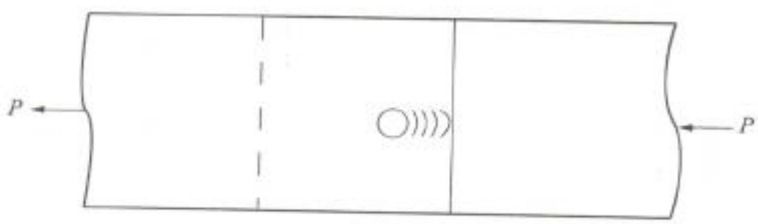
(e) Bolts subject to shear and tension



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



Bursting or shearing of plates.



Crushing of plates.

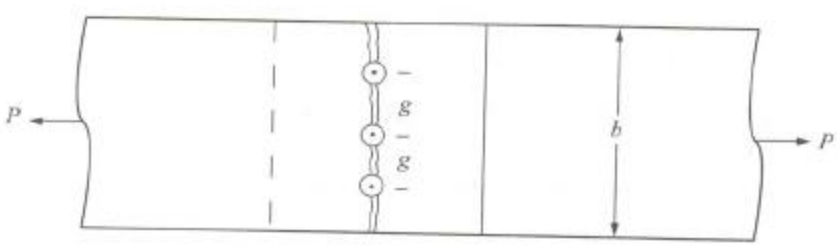
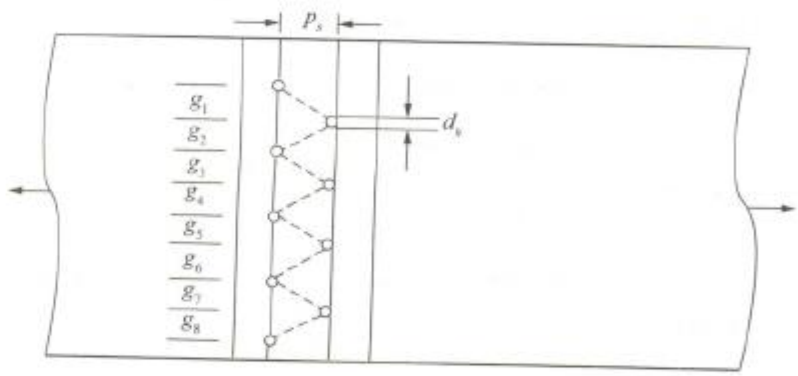


Figure 3.14 Rupture of plate.



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

Where,

'γ_{ml}' 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) $A_n = \left[b - nd_o + \sum_{i=t} \frac{P_{st}^2}{4g}\right]t$

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.

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

(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

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

(v) Thus the capacity of bolt in shear is

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

(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

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

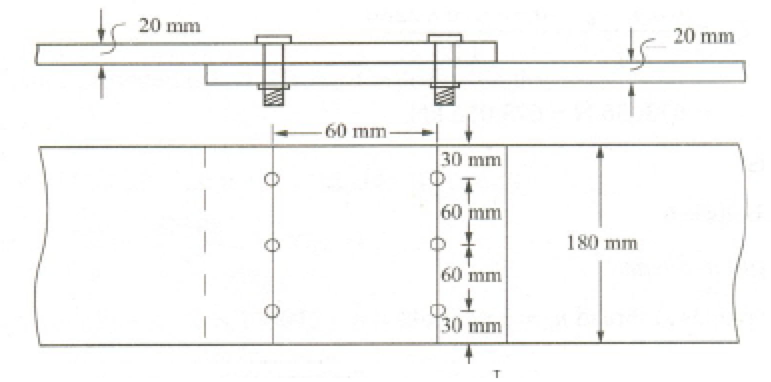
Design strength of solid plate

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

$A_n = (b - nd_o)t$

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

$$V_{nsb} = \frac{f_u}{\sqrt{3}}[N_n A_{nb} + N_s A_{sb}]$$

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]

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

A_{nb} = 245mm^2

$V_{nsb} = \frac{400}{\sqrt{3}}[1 \times 245 \times 6]$

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

V_{dsb} = 271.58 KN

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

Take

$\beta_{ij} = \beta_{ig} = \beta_{pk} = 1$

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

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 [∴ $K_b = 0.45$]

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

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

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

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

∴ 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

Strength of Solid Plate:-

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

[yielding sides the strength of solid plate]

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

Strength of solid plate = 818.18 KN

∴ 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.

Given Data:-

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

Plate:-

Fe410 [250]

Fu = 410 N/mm^2

Fy = 250 N/mm^2

Bolt:-

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) Strength of plate @ the joint:-

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

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

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

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

$\gamma_{ml} = 1.25$ [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) Strength of bolts:-

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

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

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

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

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

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

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

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

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

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

$\therefore$ 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 = $\frac{f_y A_g}{\gamma_{ml}}$

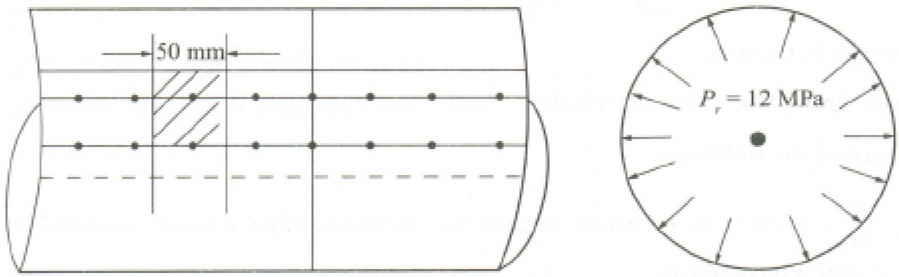
$= \frac{250}{1.1} \times 180 \times 20$ [Tks of thinner plate is the least of sum of cover plate 20(or) 24mm]

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 – 1x 18] x 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) Strength of bolt in bearing: [cls 10.3.4 IS 800-2007]

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

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

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

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

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

[$\therefore$ 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 $V_{dbp} = 2\times 124.16$

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

$\therefore$ Design strength of bearing for 50mm width = 248.32 KN

$\therefore$ Design strength of the joint per 50mm width is the least of 57.94 KN

132.25 KN

248.32 KN

$\therefore$ Design strength of jt/50mm width = 57.94 KN

Check for hoop tension:-

The action of force

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

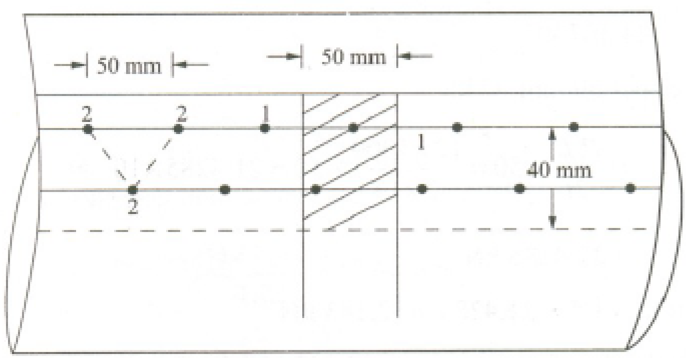
∴ For 50mm width hoop tension = 428.57 x 50 = 21.43 KN

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) Net area resisting the tearing force along section -

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

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

$$An_1 = 448\ mm^2$$

(ii) along section 2-2

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

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

Design strength of plate @ the joint:-

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

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

T = 132.25 KN

Design strength of bolt:-

(i) Strength of bolt in shear:- [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

$$K_b = \frac{54}{3\times18},\frac{50}{3\times18}-0.25,\frac{400}{410},1.0$$

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

Take K_b value is the least of 0.676

$$V_{nbp}=2.5\times0.676\times16\times14\times410$$

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

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

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

For 2 bolts $V_{dbp}=2\times124.17$

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

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

Check Hoop Tension:-

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

$= \frac{1000\times12}{2\times14}$

$= 428.57$ 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

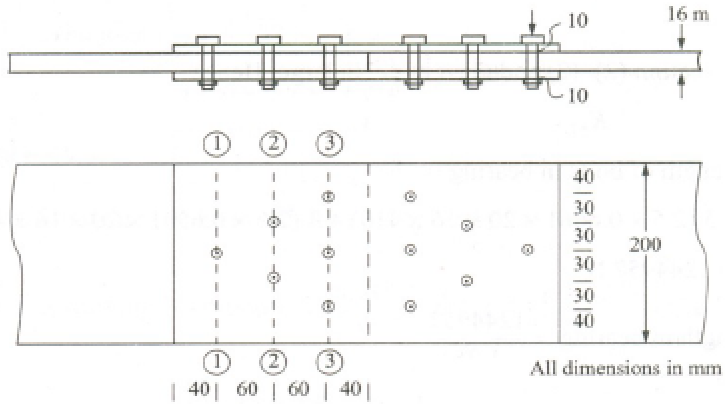
Applying Partial safety factor for load as 1.5 factored design load

= 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]

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

Take

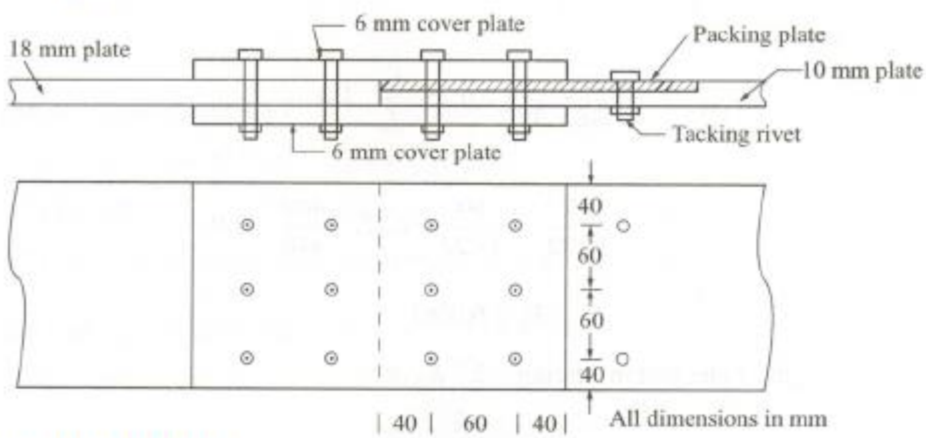
$K_b = 1$ ['t' is least of 16mm (2x10mm)]

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

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

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

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.

<u>Bolt Value:-</u>

(i) <u>Strength of bolt in single shear:-</u> [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

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

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

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

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

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

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

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

Design strength of bolt in single shear = 28.97 KN

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

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

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

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

p = 2.5d = 40mm

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

$$= \frac{30}{3\times18}, \frac{40}{3\times18} - 0.25, 1, 1$$

= 0.556, 0.491, 1,1

Take K_b = 0.491

$$V_{nbp} = 2.5\times0.49\times16\times12\times400$$

V_{nbp} = 94.08 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 = $\frac{160}{28.97}$

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

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:-

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

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

= [120 – 2 x 18]12

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

$$= \frac{0.9\times1008\times410}{1.25}$$

$$\boxed{T = 297.56\, KN > 160\, KN}$$

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

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

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

$$A_{sb}=\frac{\pi\times16^2}{4}$$

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

$$V_{nsb}=\frac{400}{\sqrt{3}}[1\times156.83+1\times201]$$

= 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]

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

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

$$K_b = \frac{30}{3\times18},\frac{40}{3\times18}-0.25,1,1$$

= 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 = $\frac{160}{66.109}$

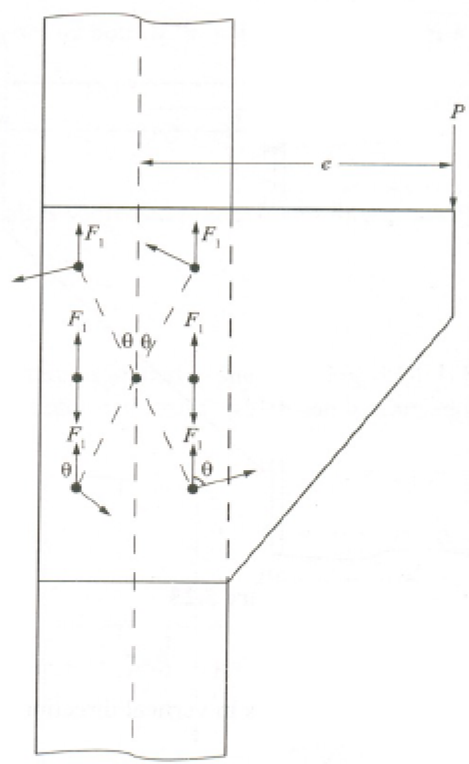
= 2.4 ~ 4Nos.

Strength of Plate @ the joint:-

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

∴ The plate is safe against tearing.

ECCENTRIC CONNECTION:-



(i) LINE OF ACTION OF ECCENTRIC LOAD IS IN PLANE OF THE GROUP OF BOLTS.
(ii) LINE OF ACTION OF ECCENTRIC LOAD IS IN PLANE PERPENDICULAR TO THE PLANE OF BOLTS.

(i) LINE OF ACTION OF ECCENTRIC LOAD IS IN PLANE OF THE GROUP OF BOLTS.

The equivalent load acting on the connections are

(i) Axial Load [P]
(ii) Moment due to eccentricity [M=Pe]

A direct shear force of F1 =P/N is transferred on each bolt.

The force due to the moment depends upon the radial distance from the C.G of the bolts which will act a right angles to the radial lines.

The force is directly proportional to the radial distance (or) $F_2 \alpha r$, $F_2 = K_r$

Where,

k=> Constant of Proportionality

r => Radial distance

The summation of force into ¿ lr distance is equat to the total moment acting on section. $\left[\sum F_2 r = P_{xe}\right]$

∴ Equating the relation $F_2 = K$

$$\sum kr^2 = pxe$$

$$K = \frac{pe}{\sum r^2}$$

$$F_2 = k \cdot r$$

$$F_2 = \frac{per}{\sum r^2}$$

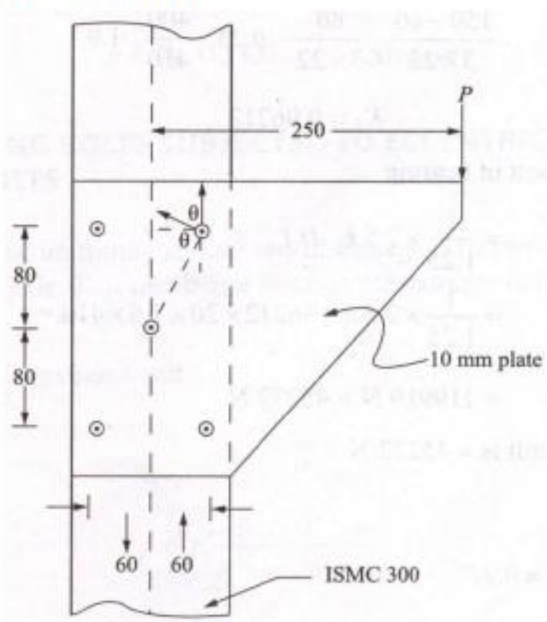
The resultant force for F1 & F2 given as

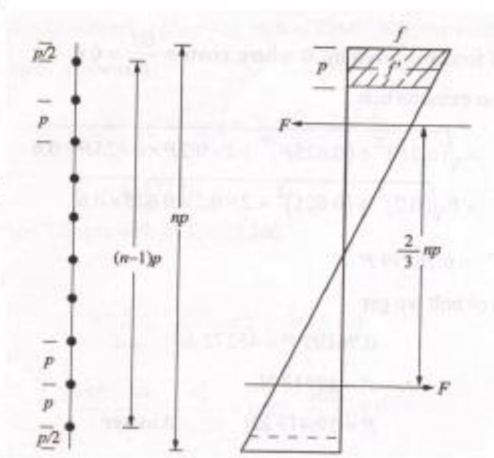
$$F = \sqrt{F_1{}^2 + F_2{}^2 + 2F_1 F_2 \cos\theta}$$

Where,

$\theta \Rightarrow$ The angle b/w F1 & F2

1. A bracket bolted to a vertical column is loaded as shown in the fig. If M20 bolts of grade 4.6 are used. Determine the maxi. Value of factored load 'P' which can be carried safely.





Given:-

The bracket is connect to the web of the channel section.

M20 bolt => Grade 4.6

d = 20mm

d_o= 22mm

fub = 400N/mm^2

Sln:-

The resultant force in each bolts $F=\sqrt{F_1{}^2+F_2{}^2+2F_1F_2\cos\theta}$

$F_1=\frac{p}{n}[DirectShear]$

$F_2=\frac{per}{\sum r^2}[forceduetomoment]$

For rolled steel section ultimate stress, fu = 410 N/mm^2

fy = 250 N/mm^2

The jt b/w the bracket plate & the web of ISMC 300 is a lap jt. Which is in single shear.

BOLT VALUE:-

1. Strength of bolt in single 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

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

= 245.04 mm^2

$$=\frac{400}{\sqrt{3}}[1\times 245.04]$$

$$V_{ncb}=56.589\,KN$$

$$V_{dsp}=\frac{56.589}{1.25}$$

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

∴ Design strength of bolt value = 45.7 KN

Resultant force on each bolts [extreme bolt] =

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

Direct Shear $F_1=\frac{P}{5}$

$F_1=0.2P$

Force due to moment $F_2=\frac{per}{\sum r^2}$

r = 100mm

$$= \frac{P \times 250 \times 100}{4 \times 100^2}$$

$$F_2 = 0.625\text{ P}$$

$$= \sqrt{(0.2P)^2 + (0.625P)^2 + 2 \times 0.2P \times 0.625P\,0.6}$$

$$F = 0.762\text{ P}$$

Equating the bolt value to the strength of the bolt. 45.27 = 0.762P

P = 59.4 KN

∴ The maxi design load allowable on the bracket = 59.41 KN

Design of bearing bolts subjected to eccentric loading.

$$n = \sqrt{\frac{6M}{Vp}}$$

Where,

N => No.of bolts

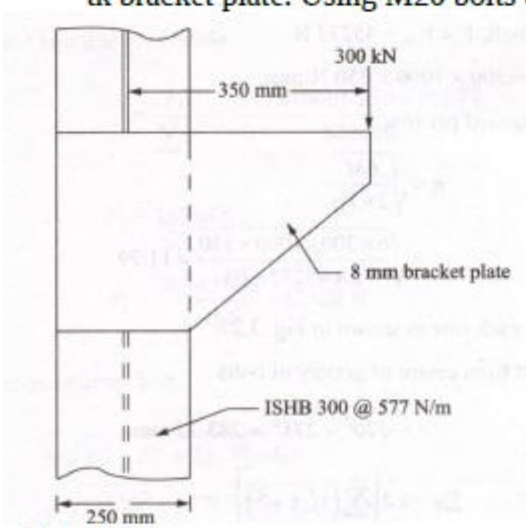
M => Moment

V => Bolt value

P => Pitch

The above expression is for single dine of bolts. If two vert.lines are adopted, '2V' value is adopted $\sqrt{\frac{6M}{2\,Vp}}$

1. A bracket is bolted to the flange of the column as shown in fig. using 8mm tk bracket plate. Using M20 bolts of grade 4.6 design the connection.



Given Data :-

φ of bolt = 20mm

Grade 4.6,

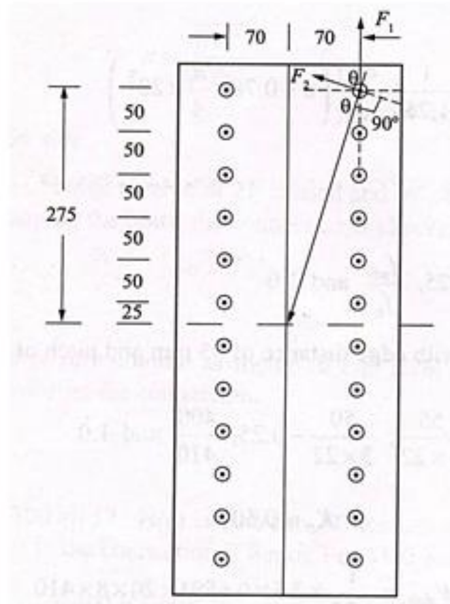
Fub = 400 N/mm^2

Rolled steel section Bracket is having fu is 410 N/mm^2

Fu = 410 N/mm^2

Sln:-

Assuming two vert.lines of bolts, n = $\sqrt{\frac{6M}{2Vp}}$



BOLT VALUE:-

(i) Strength of bolt in single shear:- [cls 10.3.3 IS 800-200

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

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

$n_n=1$, $n_s=0$

$$A_{nb}=\frac{0.78\times\pi\times20^2}{4}$$

$= 245\ mm^2$

$$=\frac{400}{\sqrt{3}}[245.0x1]$$

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

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

$$\boxed{V_{dsb}=45.26\,KN}$$

(ii) Strength of bolt in bearing:- [cls 10.3.4 IS 800-200

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

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

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

Assume the pitch 'p' is 2.5d=2.5 x 20 = 50mm

∴ Providing pitch of 50mm and edge distance of 50mm

$$=\frac{50}{3\times22},\frac{50}{3\times22}-0.25,\frac{400}{410},1$$

= 0.757, 0.5076, 0.976, 1

$K_b = 0.5076$

$$V_{nbp}=2.5\times0.5076\times20\times8\times410$$

V_{nbp} = 83.25 KN

$$=\frac{83.25}{1.25}$$

$$\boxed{V_{dbp}=66.59\,KN}$$

∴ Design strength of bolt value = 45.26 KN

∴ No. of bolts, n = $\sqrt{\frac{6M}{2Vp}}$

$$=\sqrt{\frac{6\times300\times0.35}{2\times45.27\times0.05}}$$

n = 11.79 ¿ 12Nos.

$\boxed{n = 12\ \text{Nos.}}$

∴ Provide 12 bolts on each side.

Resultant force on the extreme bolt should be less than the bolt value

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

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

$$=\frac{300}{24}$$

$$F_1=12.5\,KN$$

$$F_2=\frac{per}{\sum r^2}$$

$$=\frac{300\times0.35\times0.285}{\sum r^2}$$

$$\because r=\sqrt{275^2+75}$$

r = 285mm

$$\sum r^2=\sum x_1^2+y_1^2$$

$$\sum r^2=\sum (x_1^2+y_1^2)$$

$$= \quad 4\left[(75^2+25^2)+(75+75^2)+(75^2+125^2)+(75^2+175^2)+(75^2+225^2)+(75^2+275^2)\right]$$

$$\sum r^2=85\times10^4\,mm^2$$

$$=\frac{300\times350\times285}{85\times10^4}$$

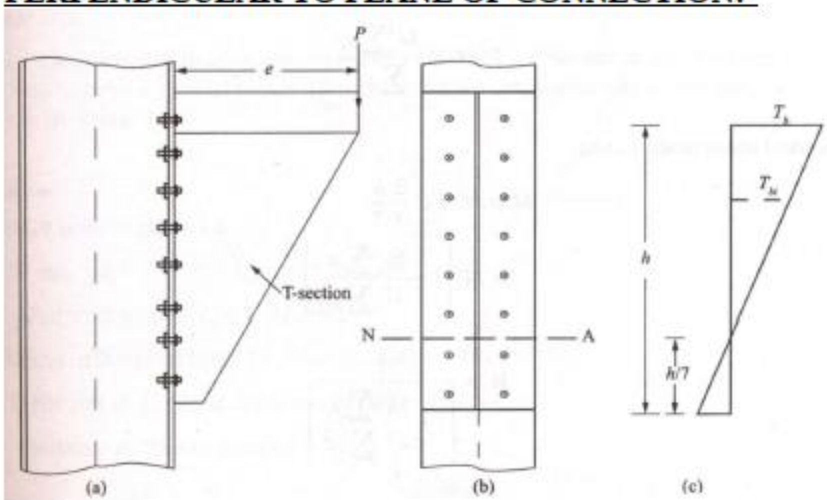
$$F_2=35.12\,KN$$

$$=\sqrt{12.5^2+35.21^2+2\times12.5\times35.21\times0.26}$$

F = 40.31KN < 45.26 KN

Hence the connection is safe. Provide 24Nos of M20 bolts along 2lines of bolts.

(ii) **ECCENTRIC CONNECTION WHEN PLANE OF LOAD IS PERPENDICULAR TO PLANE OF CONNECTION:-**



DESIGN PROCEDURE:-

- Assume the dia of bolt.
- Adopt a pitch of 2.5d for the bolts.
- Provide atleast two vertical rows of bolts, where the no. of bolts in each line

$$N = \sqrt{\frac{6M}{2Vp}}$$

- Check for the intraction relation b/w direct shear & tensile force developed on the extreme bolts as per cls 10.3.6 [IS 800-2007]

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

Where, T_{db} => Design tension capacity [cls 10.3.5]
V_{sb} => Factored shear force acting on the bolt
V_{db} => Design shear capacity [bolt value]
T_b => Factored tensile force acting on the bolt

$$T_b = \frac{M^1 yi}{\sum yi^2}$$

$$M^1 = \frac{M}{1 + \frac{2h}{2l}\left(\frac{\sum yi}{\sum yi^2}\right)}$$

$$T_{db} = \frac{T_{nb}}{\gamma_{mb}}$$

$$T_{nb} = 0.9 fubAn$$

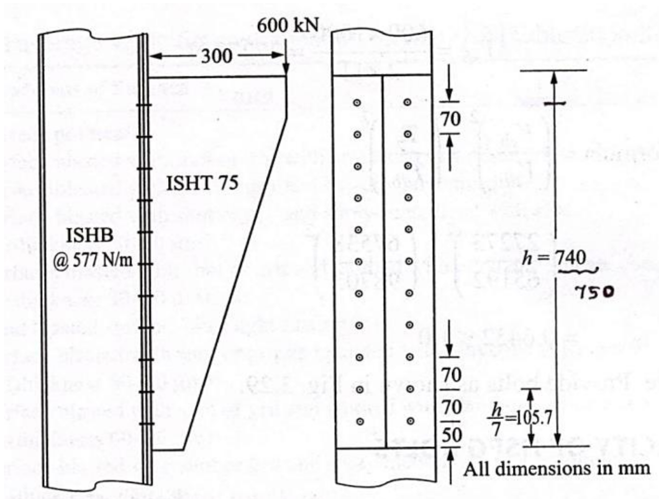
$$T_{db} = \frac{0.9Anfub}{\gamma_{mb}} < \frac{fubAsb}{\gamma_{mo}}$$

1. Design the connection of a bracket section ISHT 75 attached to the flange of ISHB300 @ 577 N/m should carry a verti factored load of 600KN at an eccentricity of 300mm. Use M24 bolt of grade 4.6

Given Data :-

ISHB @ 577N/m stanchion
ISHT 75 bracket
Load P = 600KN
E = 300mm
Bolt => M24 ϕ=24
Grade 4.6 fu=400 N/mm^2

Sln:-



Here the plane of load is ¿ lr to the plane of bolt. Therefore as per clause 10.3.6 the following intraction formula need to be satisfied.

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

<u>Direct Shear:-</u>

$$V_{sb}=\frac{P}{n}$$

$$n=\sqrt{\frac{6M}{2\,Vp}}$$

<u>BOLT VALUE:-</u>

1.<u>Strength of bolt in single shear:-</u>

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

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

$n_n=1,\ n_s=0$

Assume ultimate strength of rolled steel section is 410 N/mm^2

$$A_{nb}=\frac{0.78\times\pi\times 24^2}{4}$$

$$=356.82\text{mm}^2$$

$$=\frac{400}{\sqrt{3}}[1\times 352.86]$$

$$V_{dsb}=\frac{81.484}{1.25}$$

$$V_{dsb}=65.19\text{KN}$$

2. <u>Strength of bolt in bearing:-</u>

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

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

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

Assume the edge distance e = 1.5 d$_o$ = 40.5mm

$\simeq$ 50mm

P = 2.5 d = 60mm

$\simeq$ 70mm

$$= \frac{50}{3\times27}, \frac{70}{3\times27} - 0.25, \frac{400}{410}, 1$$

$$K_b = 0.617, 0.6140, 0.9756, 1$$

Take $K_b = 0.614$

$$V_{nbp} = 2.5\times0.614\times24\times410\times9$$

$$= 135.94 \text{ KN}$$

$$V_{dbp} = \frac{135.94}{1.25}$$

$$V_{dbp} = 108.752\text{KN}$$

$\therefore$ Design strength of bolt value = 65.19KN

No. of bolts, n = $\sqrt{\frac{6M}{2Vp}}$

$$= \sqrt{\frac{6\times600\times0.3}{2\times65.19\times0.07}}$$

$$n = 10.8\ ;\ 11\text{Nos.}$$

$$n = 11\text{Nos.}$$

Provide 11 bolts on each side.

$$\therefore V_{sb} = \frac{p}{n}$$

$$= \frac{600}{11\times2}$$

$$V_{sb} = 27.27\text{KN}$$

Since N.A is @ 107.14mm it lies b/w the Ist bolt & the IInd bolt.

The 1st bolt from the bottom is in tension and the other bolts are in compress

Tensile force in extreme bolt due to B.M

$$T_b = \frac{M^1 yi}{\sum yi^2}$$

$$M^1 = \frac{M}{1+\frac{2h}{2l}\left(\frac{\sum yi}{\sum yi^2}\right)}$$

Where, yi= The distance of there bolts in compression from the N.A

Bolt.No	2	3	4	5	6	7	8	9	10	11
$\sum$yi (mm)	12.85	82.85	152.85	222.85	292.85	362.85	432.85	502.85	572.85	642.85

$$\sum yi = 3278.5mm$$
$$\sum yi^2 = 1.479\times10^6 mm^2$$
$$Yi = 642.85mm$$
$$M^1 = \frac{M}{1+\frac{2h}{2l}\left(\frac{\sum yi}{\sum yi^2}\right)}$$
$$= \frac{300\times600\times10^3}{1+\frac{2\times750}{21}\left(\frac{3278.5}{1.479\times10^6}\right)}$$
$$M^1 = 155.4KN.m$$
$$T_b = \frac{M^1 yi}{\sum yi^2}$$
$$= \frac{155.4\times642.85\times10^6}{1.479\times10^6}$$
$$T_b = 67.54KN$$

Design

Design Tension capacity of bolt:- [cls 10.3.5]

$$T_{db} = \frac{T_{nb}}{\gamma_{mb}}$$
$$= \frac{0.9A_n fub}{\gamma_{mb}} \le \frac{fybAsb}{\gamma_{mb}}$$
$$A_n = \frac{0.78\times\pi\times24^2}{4} = 352.86mm^2$$
$$A_{sb} = \frac{\pi\times24^2}{4} = 452.39mm^2$$
$$T_{db} = \frac{0.9\times352.86\times400}{1.25} \le \frac{240\times452.3}{1.1}$$
$$= 101 < 98.70KN$$

Design tension capacity of bolt T_{dn} = 98.70 KN

$$\therefore \left(\frac{V_{sb}}{V_{dsb}}\right)^2 + \left[\frac{T_b}{T_{db}}\right]^2 \le 1.0$$
$$\left(\frac{27.27}{65.19}\right)^2 + \left(\frac{67.54}{98.7}\right)^2 \le 1.0$$
$$0.64 < 1$$

HSFG Bolts:- [cls 10.4 IS 800-2007]

- HSFG bolts are made of high tensile steel material [ultimate stress about 800N/mm^2] which are free tension then provided with nuts.

- The nuts are also clause using calibrated wrenches.
- Here the resistance to S.F id mainly by friction.
- Two types of HSFG bolts are
  (i) Parallel Shank
  (ii) Waisted Shank
- Parallel shanks are design for no slip at serviceability load.
- Hence the slip at higher loads and slip into bearing at ultimate load.
- These bolts are checked for bearing strength at ultimate load.

For no slip even at need not be check for bearing strength.

Shear Capacity of HSFG bolts:- [cls 10.4.3 IS800-2007]

ne = 1 for lap jt

ne = 2 for bult jt

$V_{nsf} = \mu_f . n_e k_n . F_o$

μ_f = Co-eff. Of friction (slip factor) as specified in table 20[μ_f =0.55]

n_e = No. of eff. Interfaces offering frictional resistance to slip.

k_n = 10 for fasteners in clearance holes 0.85 for fasteners in oversized & short shotted holes 0.7 for fasteners in long slotted holes.

γ_{mf} = 1.10 (if slip resistance is designed at service load)

F_o = Mini bolt tension @ threads at installation & may be taken as $A_{nb}.f_o$

A_{nb} = net area of the bolt@threads 0.78 $\pi d^2 /4$

f_o = Proof stress 0.7 f_{ub}

$$\boxed{V_{dsf} = \frac{V_{nsf}}{\gamma_{mf}}}$$

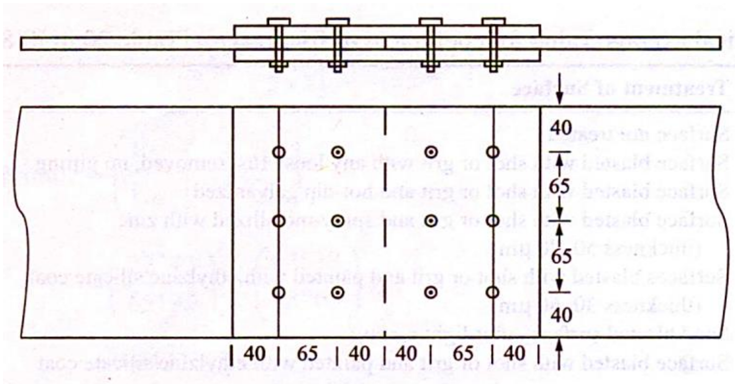
1. Determine the shear capacity of the bolts used in connecting two plates as shown in fig.

(i) If slip resistance is designated @ service load.

(ii) If slip resistance is designated @ ultimate load.

Given:-

(i) HSFG bolts of grade 8.8 is used.
(ii) Co-eff of friction μ =0.3
(iii) The clearance holes fasteners are used.



Given:-

Bolt => grade 8.8

f_{ub}= 800 N/mm^2

fy = 640 N/mm^2

Sln:-

Slip resistance for the bolts (V_{nsf})

$= \mu_f . n_e . k_n . f_o$

= 0.3 x 2 x 1 x 137.2

= 82.32 KN

Where,

μ =0.3

n_e = 2 [for D.C.B.J]

k_n = 1

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

d =65

d = 26 mm

1.5 d = 40

d = 26.67mm

Provide dia of bolt as 20mm. Dia of bolt hole is 22mm

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

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

$$f_o=f_o\times A_{nb}$$

= 560 x 245

f_o = 137.2 KN

$$\therefore V_{nsf}=0.3\times 2\times 1\times 137.2=82.32\,KN$$

(i) <u>When If slip resistance is designated @ service load:-</u>

Design strength of 1 bolt = $\frac{82.33}{1.1}$

= 74.85 KN

Design Strength of 6 bolts = 6 x 74.85

= 449.1

(ii) If Slip resistance is designated @ Ultimate load:-

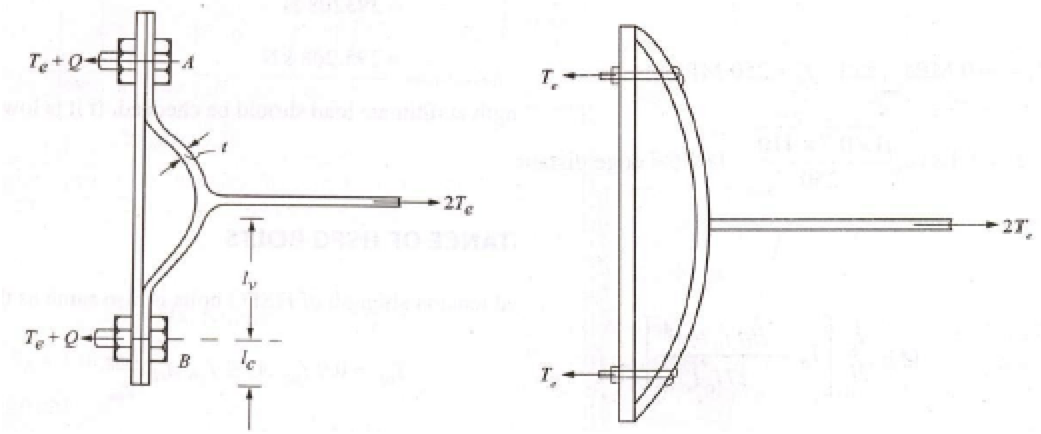
Design Strength of 1 bolt = $\frac{82.33}{1.25}$

= 65.87 KN

Design strength of 6 bolts = 6 x 65.87

= 395.22 KN

PRYING FORCE:- [cls 10.4.7 IS 800-2007]

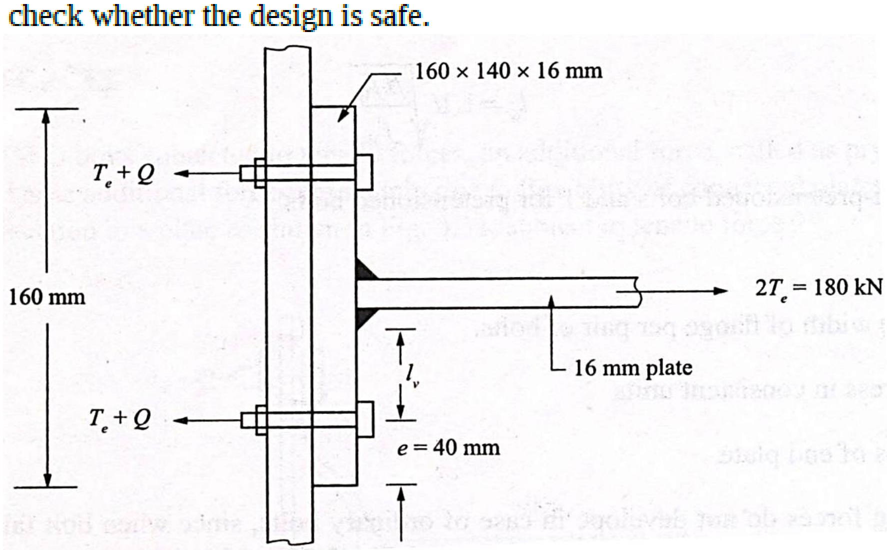


$$Q = \frac{l_v}{2l_e}\left[T_e - \frac{\beta\eta f_o b_e t^4}{27 l_e l_v^2}\right][Q\, f_o = 0.7 f_{ub}]$$

When,

$$l_e = 1.1t\sqrt{\frac{\beta f_o}{fy}} < e[Q\, f_o = 0.7 f_u]$$

1. The jt shown in fig. as to carry a factored load of 180KN. End plate used size 160x140x16mm. The bolts used are M20 HSFG bolts of grade 8.8 check whether the design is safe.



Given:-

M20 HSFG Grade 8.8

f_{ub}= 800 N/mm^2

Plate:-

f_u= 400 N/mm^2

27 = 180 KN

Assuming 8mm size weld a edge distance are of 40mm

$$l_v = \frac{160}{2} - 8 - 8 - 40$$

$$l_v = 24mm$$

l_e least of 1.1t $\sqrt{\frac{\beta f_o}{fy}}$ (or)40mm

For β = 1 for pretensioned bolt

$f_o = 0.7fu$ [∵ fu for plate assumed as 410 N/mm^2 fy=250 N/mm^2]

= 0.7 x 410

f_o= 287 N/mm^2

$$\because l_e = 1.1 \times 16\sqrt{\frac{1 \times 287}{250}}$$

= 18.86mm (or) 40mm

$$\boxed{l_e = 18.86mm}$$

$$\text{Prying force } Q = \frac{l_v}{2l_e}\left[T_e - \frac{\beta\eta f_o b_{et}{}^4}{27 l_e l_v{}^2}\right]$$

When,

$$\beta = 1, \eta = 1.5, T_e = 90KN, f_o = 0.7 \times 800 = 560N/mm^2$$

$$b_e = 140mm, t = 16mm, l_e = 18.86, l_v = 24mm$$

$$\therefore Q = \frac{24}{2 \times 18.86}\left[90 - \frac{1 \times 1.5 \times 560 \times 140 \times (16)^4}{27 \times 18.86 \times (24)^2}\right]$$

$$\boxed{Q = 40.55\ KN}$$

Total force on bolt = T + a

= 90+40.55

= 130.5 KN

$$\text{Tension capacity of bolt} = \frac{0.9 f_{ub} A_n}{\gamma_m}$$

$$= \frac{0.9 \times 800 \times A_n}{\gamma_m}$$

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

= 245 mm^2

$$= \frac{0.9 \times 800 \times 245}{1.25}$$

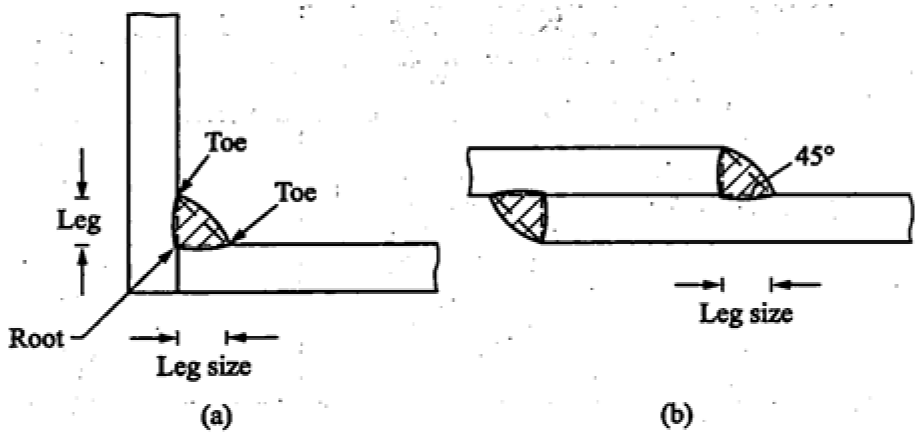
$$T_e = 141.12KN$$

Hence the design is safe.

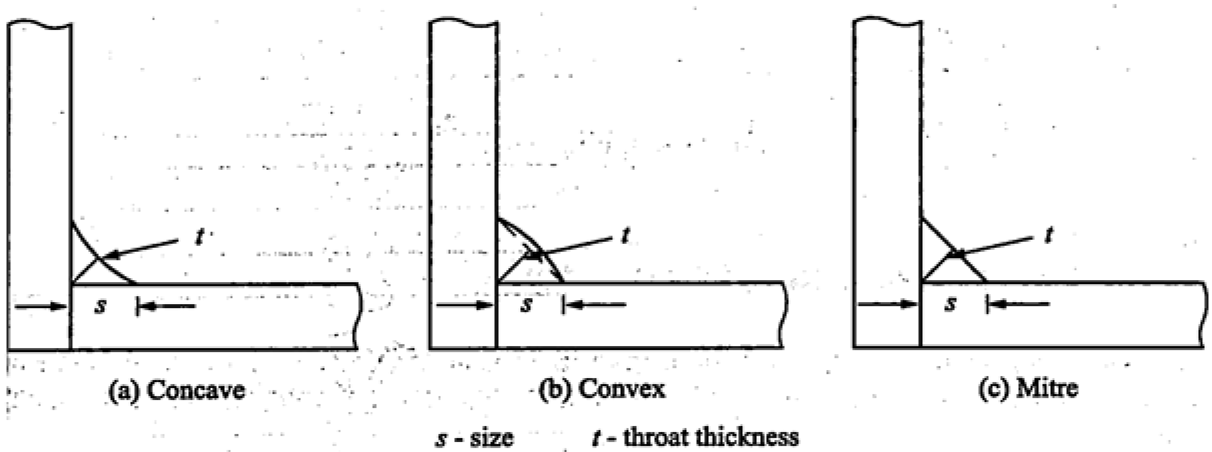
WELDED CONNECTIONS:- [IS 10.5, IS 800-2007]

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:-

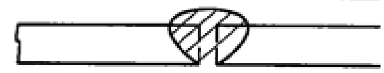
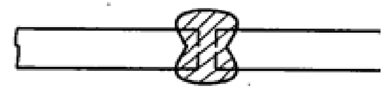
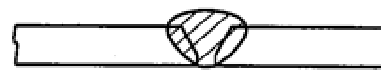
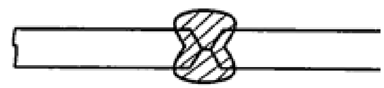
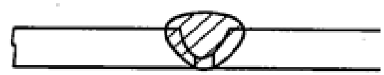
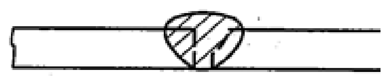
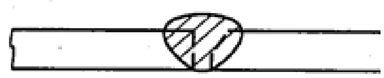
i) Lap weld
ii) Butt weld
iii) Slot weld
iv) Plug weld

(i) Lap weld:-

(ii) Butt weld:-

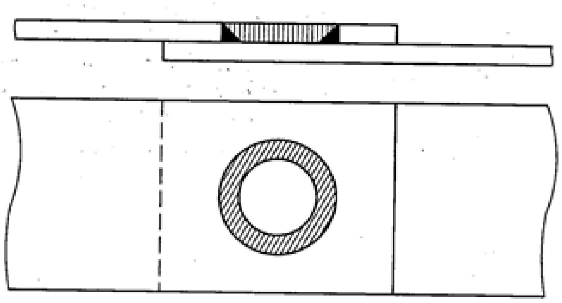
1. Single square butt weld
2. Double square butt weld
3. Single 'V' butt weld
4. Double 'V' butt weld
5. Single 'U' butt weld
6. Double 'U' butt weld
7. 'J' Butt weld

Types of butt weld

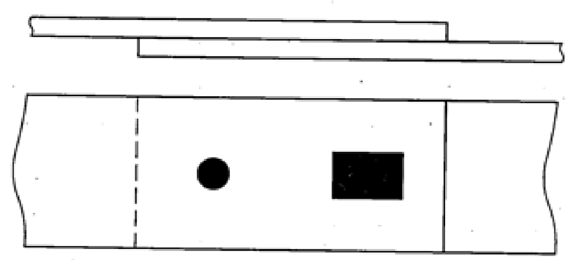
Sl. No.	Type of Butt Weld	Sketch
(a)	Square butt weld, on one side	
(b)	Square butt weld, both sides	
(c)	Single V butt joint	
(d)	Double V-butt joint	
(e)	Single U butt joint	
(f)	Single J-butt joint	
(g)	Single bevel butt joint	

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

(iii) Slot & Plug weld:-

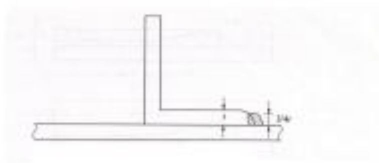


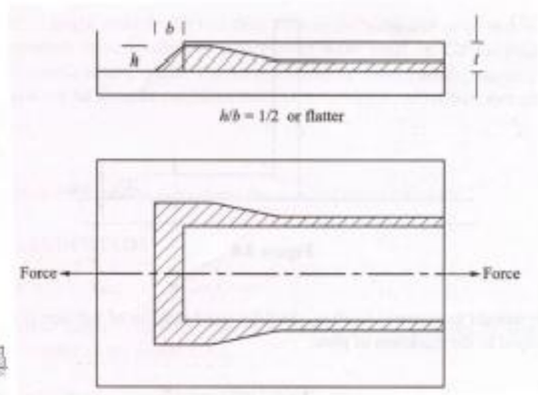
Slot weld.

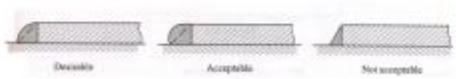


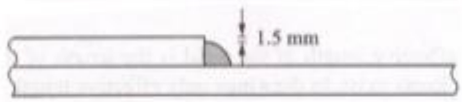
Plug welds.

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









1. <u>Butt weld:-</u>

- 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. <u>Filled weld:-</u>

(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^o] 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

$$L = leff + 2s$$

- 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

$$\beta_{1w} = 1.2 - \frac{0.2lj}{150tt} \le 1.0$$

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:-

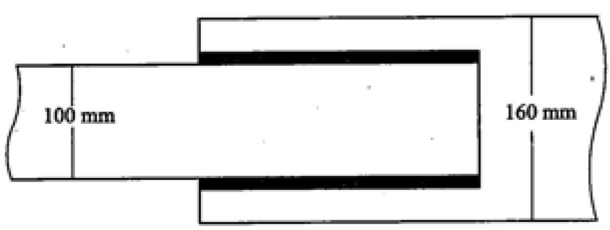
t = 5/8 s [Incomplete Penetration]
= 5/8 x 16
t = 10mm

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

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

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

$\boxed{lw = 205.7\ mm \simeq 205\ mm}$

Provide an over lap of 105mm.

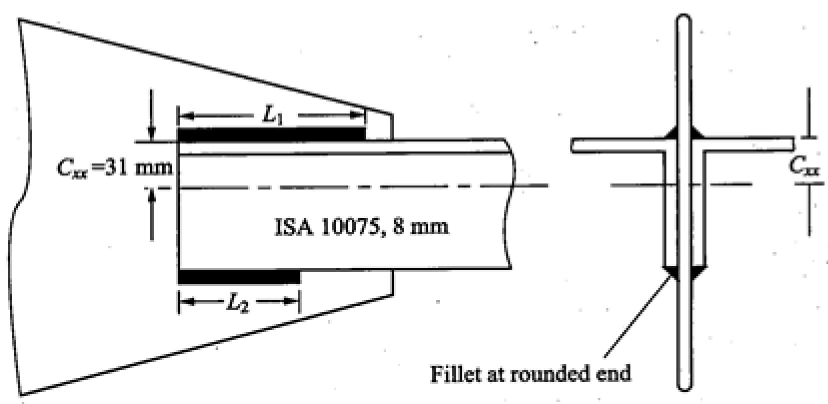
3. A tie member of a roof truss consist of 2Nos of ISA 100x75x8mm. The angles are connected to either side of a 10mm tk guset plate and the member is subjected to a working pull of 300KN. Design the welded connection. Assume the connections are made in the shop.

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.

To find C.G of weld:-

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

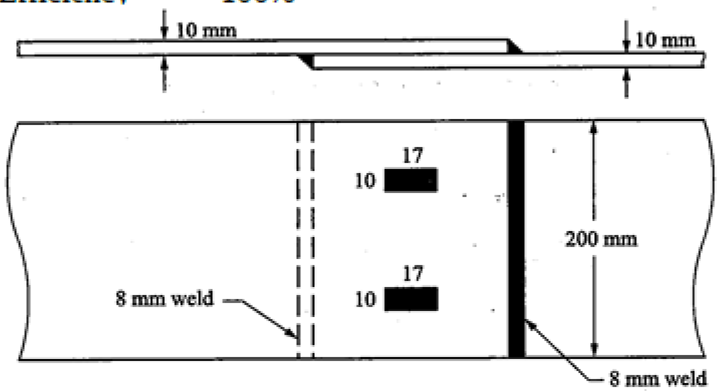
NOTE:-

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>

$$= \frac{fyAg}{\gamma_o}$$

$$= \frac{250 \times 200 \times 10}{1.1}$$

$$= 454.5 \text{ KN}$$

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

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

$$454.5 \times 10^3 = \frac{410/\sqrt{3}}{1.25} \times lw \times 0.7 \times 8$$

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

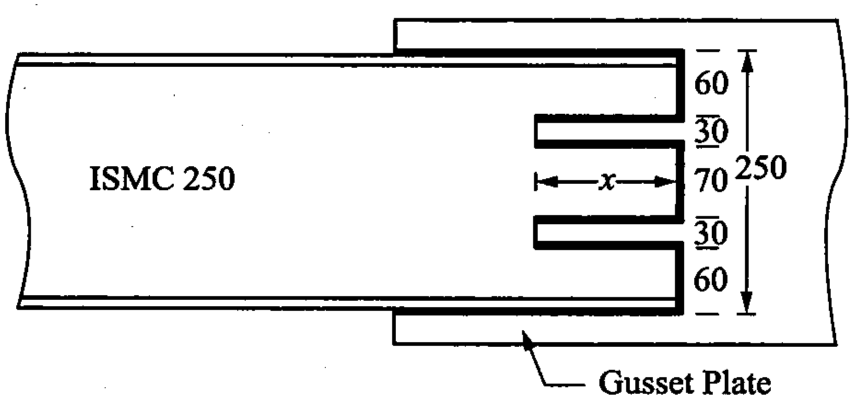
∴ x = 16.97mm

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:-

$A = 3867mm^2$

$t_f = 14.1mm$

$t_w = 7.1mm$

Strength of solid Plate:-

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

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

$= 878.86KN$

$\therefore$ Strength of weld read $= 878.86KN$

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

Maxi size of weld = 7.1-1.5=5.6mm

$\therefore$ Provide size of weld S = 4mm

$\therefore$ 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.

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

= 153.75mm

$\therefore$ Length reqd. for the slot = 153.75mm

$\therefore$ Provide 2 slots of length 154mm

ECCENTRIC CONNECTION:-

Plane of Moment is the same:-

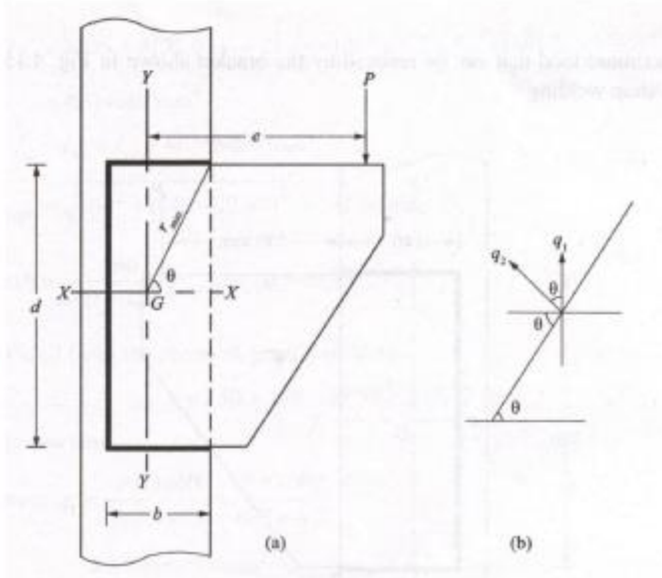
The eccentric load 'P' is equivalent to

(i) A direct axial load acting along the C.G of the group of weld.

(ii) A twisting Moment, M=P x e

Assuming uniform size of weld with throat tks, 't' and length of weld provided as shown in the fig with sides b&d, the direct shear stress,

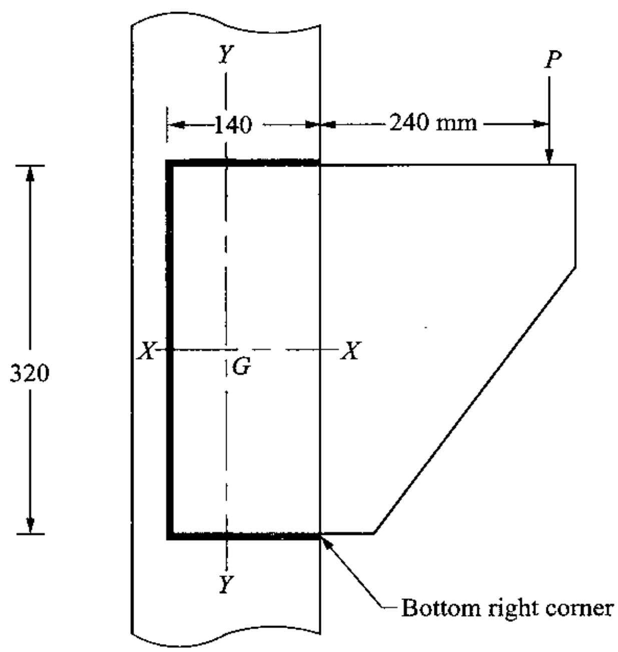
$$q_1 = \frac{P}{(2b+d)t}$$



The stress due to twisting moment acting ¿ lr to the C.G of the weld group and the radius vector,

$$q_2 = \frac{p \times e \times y_{max}}{I_{zz}}, \quad \text{Resultant stress } q = \sqrt{q_{1}{}^2 \times q_{2}{}^2 + 2q_1 q_2 \cos\theta}$$

1. Determine the max load that can be resisted by a bracket shown in fig. Fillet weld of size 6mm is provided as shop welding.



Given:-

Size of weld = 6mm

Depth = 300mm

Sln:-

Here the weld group is provided such that the plane of welding is parallel to the plane of moment.

$\therefore$ Direct shear stress, $q_1 = \frac{P}{A}$

Shear due to moment $q_2 = \frac{P \times e \times y_{max}}{I_{zz}}$

Resultant stress q = $\sqrt{q_{1^2} \times q_{2^2} + 2q_1 q_2 \cos\theta}$

<u>Weld Group:-</u>

t = threat tks

t = 0.7S

= 0.7(6) = 4.2m

$$\bar{y} = \frac{320}{2} = 160\,mm$$

$$\bar{x} = \frac{a_1 x_1 + a_2 x_2 + a_3 x_3}{a_1 + a_2 + a_3}$$

$$= \frac{140 \times 4.2 \times 70 + 3116 \times 4.2 \times 2.1 + 4.2 \times 140 \times 70}{588 + 1308.72 + 588}$$

$\bar{x}$ = 34.24mm

$$I_{xx} + I_{yy} = I_{zz}$$

$$I_{xx} = \frac{140 \times 4.2^3}{12} + 140 \times 4.2 \times (317.9 - 160)^2$$
$$+ \frac{311.6 \times 4.2}{12} + 4.2 \times 311.6(160 - 160)^2$$
$$+ \frac{140 \times 4.2^3}{12} + 140 \times 4.2(160 - 2.1)^2$$

= 14661121.44+10589132.71+14661121.44

I_{xx} = 39911375.59mm^4

$$I_{yy} = \left[\frac{4.2 \times (140)^3}{12} + 4.2 \times 140(70 - 34.2)^2\right] \times 2$$
$$+ \frac{311.6 \times (4.2)^3}{12} + 311.6 \times 4.2(34.2 - 2.1)^2$$

$$I_{yy} = 4.78 \times 10^6\,mm^4$$

$$\therefore I_{zz} = 44.69 \times 10^6\,mm^4$$

Direct shear stress $q_1 = \frac{P}{A}$

A = Total area of the weld group

$= \frac{P}{[140+320+140]\times 4.2}$

$q_1 = 3.968\times 10^{-4}\,PKN$

q_1 = 0.3968 PN

$q_2 = \frac{p\times e\times \gamma_{max}}{I_{zz}}$

Where,

e = (140-34.24) + 240

= 345.76mm

$\gamma_{max} = \sqrt{(160)^2+(140-34.24)^2}$

γ_{max} = 191.79mm

$= \frac{P\times 345.76\times 191.79}{44.69\times 10^6}$

$= 1.483\times 10^{-3}\,PKN/mm^2$

$q_2 = 1.483\,PN/mm^2$

Resultant stress $q = \sqrt{[0.3968\,P]^2+(1.483\,P)^2+2\times 0.3968\times 1.483\,P\times \cos\theta}$

Where,

θ = The angle made by radial distance with the C.G

$\tan\theta = \frac{160}{140-34.24}$

$\tan\theta$ = 1.51285

θ = 56˚32'

$= P\sqrt{2.3567+0.649}$

q = 1.7336P -→ (1)

The max load that can be applied to resist the stress the weld can take is

$= \frac{410/\sqrt{3}}{1.29}$

$= 189.37\,N/mm^2 \rightarrow (2)$

Equating (1) & (2)

1.7336P = 189.37

∴ P = 109.24 NS

Eccentric Connection - Plane of weld group ¿ lr to the plane of moment:-

For eccentric connection with plane of weld group ¿ lr to the plane of moment 2 types of stresses are developed,

(i) Direct shear stress, $q=\frac{P}{A}$

Where, A =2bt

(ii) The bending stress @ the extreme end of weld

$$F=\frac{M}{Z}=\frac{p\times e}{\frac{2t\times b^2}{6}}$$

$$F=\frac{6Pe}{2tb^2}$$

The equivalent stress, $f_e=\sqrt{f^2+3q^2}$

For the weld to be safe the above equivalent stress is equated to the design stress of weld.

Design stress of weld = $\frac{fu/\sqrt{3}}{\gamma_m}$

1. Design a suitable fillet weld for an eccentrically loaded bracket plate. The working load P=100KN and eccentricity, e = 150mm.Tks of bracket plate is 12mm & tha column used is ISHB300@618 N/m [Plane of weld group is ¿ lr to the plane of moment]

NOTE:-

To find the eff. depth of weld (b) considering only the moment case, the eff. depth is assumed as b = 1.1 $\sqrt{\frac{6M}{2tf_{wd}}}$

UNIT-II

Tension Members:-

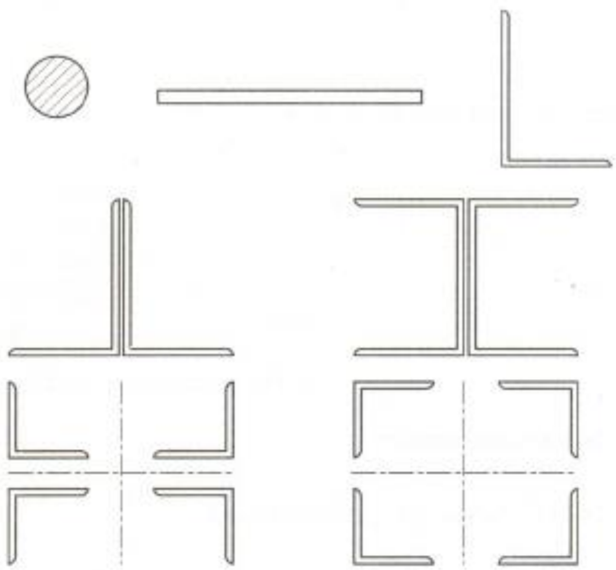
Tension members are design to satisfy the design strength of the member against

(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]

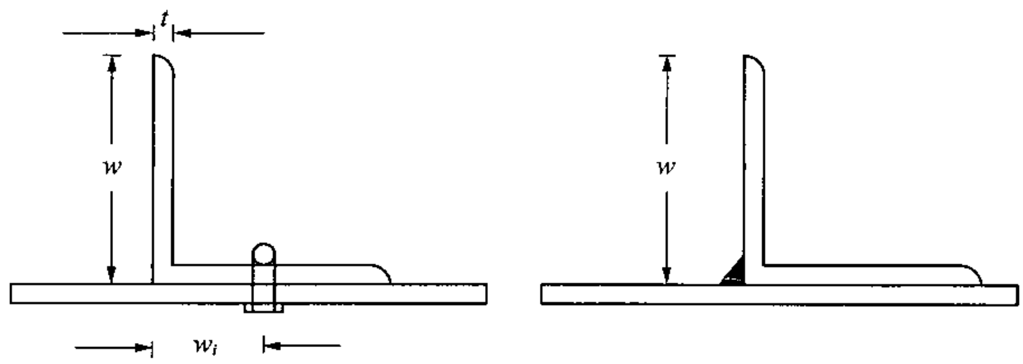
$$T_{dg} = \frac{A_g f_y}{\gamma_{mo}}$$

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,

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



$b_s = w + w_i - t$ $b_s = w$

a) For Angular section design strength of rupture:-

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

Where,

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

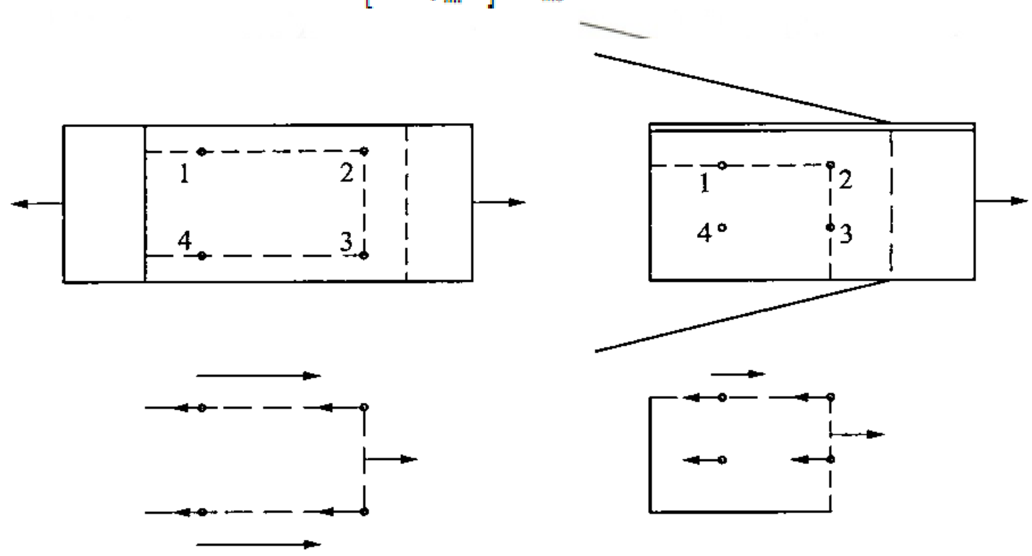
3. Design strength of member due to block shear failure @ the end connection:- [cls 6.4 IS 800-2007]

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)

b) Welded Connection:-

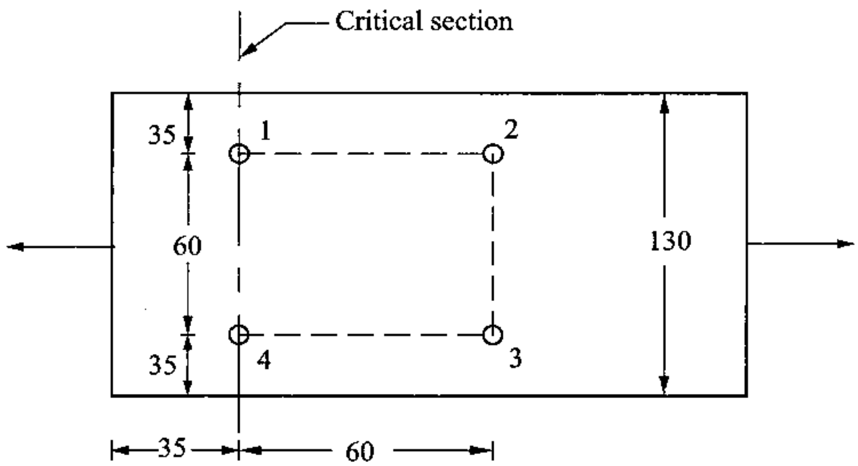
Appropriate length of member is considered around the end weld.

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 = 1560mm²

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

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 =1128mm²

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

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_{vg} 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)

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

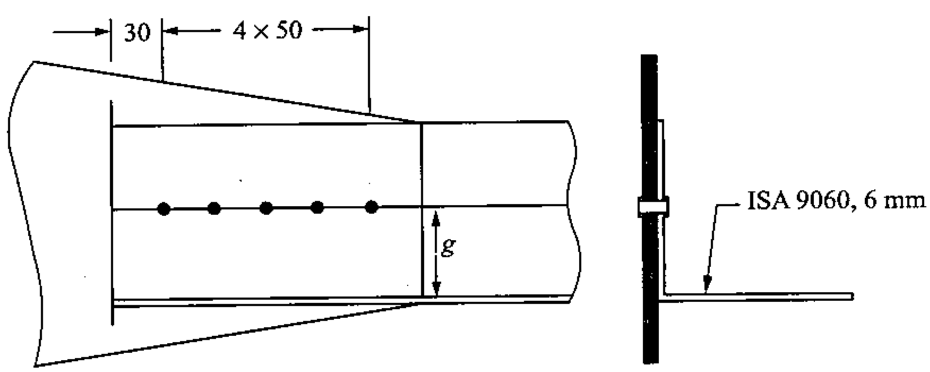
$T_{db_2} = 321.12 KN$

∴ 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]

$$\therefore\beta=0.864$$

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

$$\boxed{T_{dn} = 189.369\ \text{KN}}$$

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=1380mm^2

A_{vn}= [230-(4.5x18)]6=894mm^2

Block shear failure takes place along line 1 to 3

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

A_{vg} = (30+4x50)6=1380mm^2

A_{tg} is taken along line 2-3

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

A_{tn} = 186mm^2

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

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

(or)

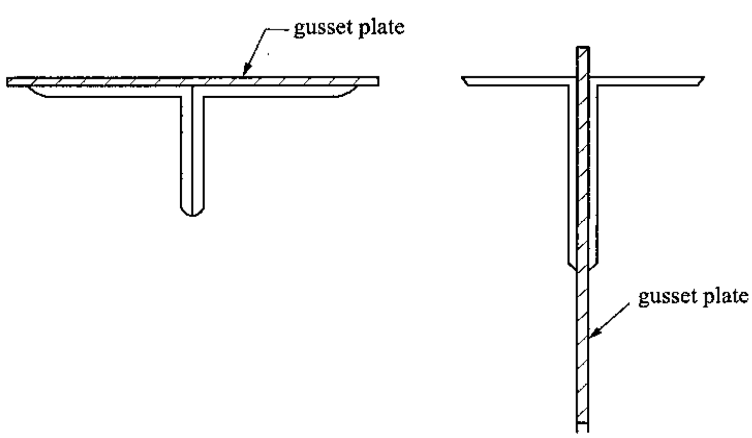
$$T_{db}=\frac{0.9\times 894\times 410}{\sqrt{3}\times 1.25}+\frac{240\times 250}{1.1}$$

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.



(a) (b)

Sln:-

Here the 60mm side is connected to gusset plate.

$\therefore$ 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]

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

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

$$\Downarrow$$

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 $\frac{f_u\gamma_{mo}}{f_y\gamma_{ml}}$

$\therefore$ 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]

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

(or)

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

Where,

$$A_{vg} = [30 + (4 \times 50)]6$$
$$A_g = 1380 mm^2$$
$$A_{tg} = 30 \times 6 = 180 mm^2$$
$$A_{tn} = \left[30 - \frac{18}{2}\right] \times 6$$
$$A_{tn} = 126 mm^2$$
$$A_{vn} = [230 - (4.5 \times 18)]6$$
$$A_{vn} = 894 mm^2$$

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

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

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

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

$\therefore$ 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.

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

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]

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

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:-

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

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

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

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

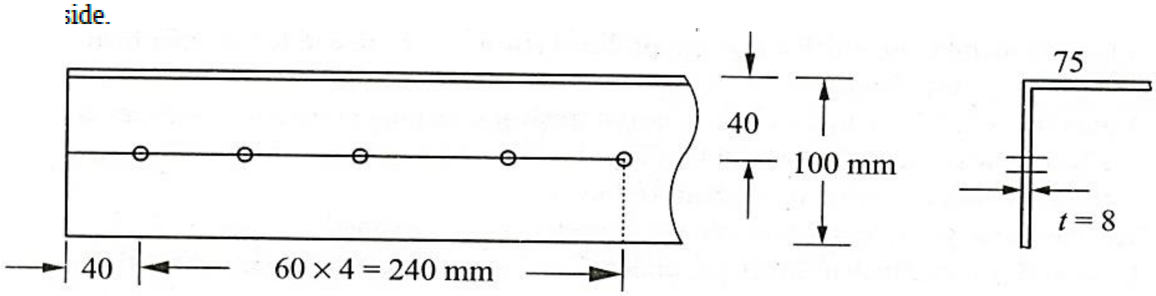
To select ISA 100x75x8mm

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

γ_{xx}=31.4mm γ_{yy} = 21.8mm

γ_{min} = 21.8mm

$\lambda = \overline{21.8}$ 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]

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

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

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

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

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]

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

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

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,

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

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

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

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

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

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

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

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

$$T_{db_1} = \left[\frac{2240 \times 250}{\sqrt{3} \times 1.1}\right] + \left[\frac{0.9 \times 312 \times 410}{1.25}\right]$$

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

$$T_{db} = \left[\frac{0.9 \times 1448 \times 410}{\sqrt{3} \times 1.25}\right] + \left[\frac{400 \times 250}{1.1}\right]$$

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

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

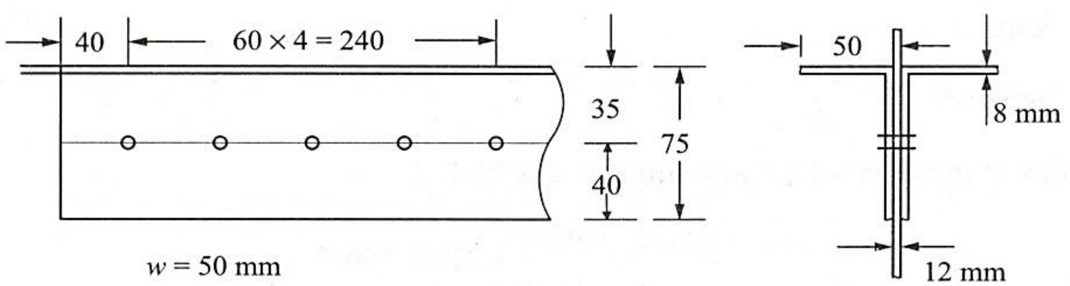
f_y= 250 N/mm²

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

$$\lambda max = 350 \, table 3$$

$$\lambda act < \lambda max$$

Sln:-



$w = 50$ mm

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

To find A_g:-

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

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

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

Area each angle reqd = 990/2 = 495mm²

$\therefore$ Select the section from steel table having area 30% more than 495mm^2

Try ISA 70x70x5mm

A_g=667mm^2 [from steel table]

$$\gamma_{xx}=21.5\,mm$$
$$\gamma_{yy}=21.5\,mm$$

Bolt Value:- [M20]

(i) Strength of bolt in double shear:-

Assuming gusset plate of tks = 10mm

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

$$V_{nsp}=\frac{f_u}{\sqrt{3}}\left[n_n A_{nb}+n_s A_{sb}\right]$$
$$n_n=n_s=1$$
$$A_{nb}=\frac{0.78\times\pi\times20^2}{4},\ A_{sb}=\frac{\pi\times20^2}{4}=314.16\,mm^2$$

$$V_{dsb}=\frac{\frac{400}{\sqrt{3}}\left[1\times245+1\times314.16\right]}{1.25}$$

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

(ii) Strength of bolt in bearing:-

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

Assume e=1.5 d$_o$= 33mm ¿ 40mm

p =2.5d = 50mm ¿ 60mm

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

= 0.606, 0.659, 0.975, 1

$\therefore$ Take k_b = 0.606 [least value]

$$V_{dbp}=\frac{2.5\times0.606\times20\times10\times410}{1.25}$$

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

$\therefore$ Design strength of bolt value = 99.38KN

$\therefore$ No. of bolts = 225/99.38

= 2.26 ¿ 3 Nos.

$\therefore$ 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

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

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

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

$$\therefore \beta=1.19$$

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

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

> 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)

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

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

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

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

For 2 angles

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

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

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]

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

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

$$n_n=n_s=1$$

$$A_{nb}=\frac{0.78\times\pi\times20^2}{4}=245\,mm^2$$

$$A_{sb}=\frac{\pi\times20^2}{4}=314.16\,mm^2$$

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

$$= [1-[0.0125\times10]]$$

$$\beta_{pk} = 0.875$$

$$V_{nsb}=\frac{400}{\sqrt{3}}[1\times245+1\times314.16]\times0.875$$

= 112.99 KN

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

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

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

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

= 0.606, 0.659, 0.975, 1

∴ Take k_b = 0.606 [least value]

$$V_{dbp}=2.5\times0.606\times20\times10\times410$$

= 124.23 KN

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

∴ Design strength of bolt value = 90.39 KN

∴ No. of bolts = $\frac{T_u}{v}$

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

$$n=5.5\simeq6\,Nos.$$

∴ 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]

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

$A_g=300\times10=3000\,mm^2$ [Tks of thinner plate]

$$= \frac{3000\times250}{1.1}$$

T_{dg} = 681.81 KN > 500KN

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 = 2340mm^2

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

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

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

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

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

= 670mm^2

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

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

= 1760mm^2

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

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

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

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

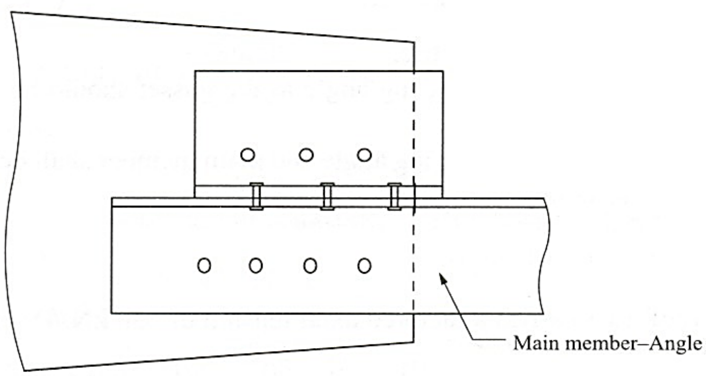
Hence the connection is safe.

LUG ANGLES:-

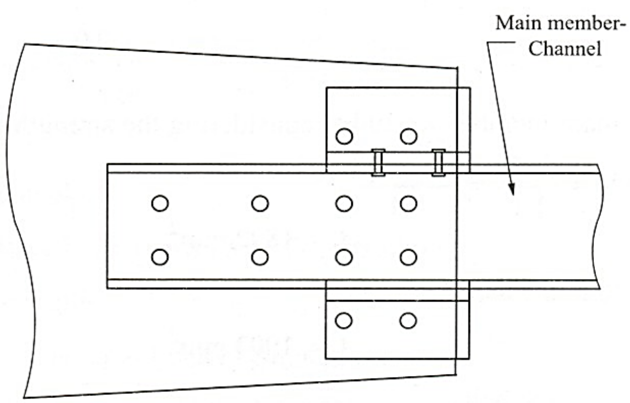
- The length of end connections of heavily loaded tension members may be reduced by using lug angles as shown in fig.
- There is savings in gusset plate but additional cost is incurred from the material of lug angles & the connections for the lug angles.
- The design of tension member with the use of 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]

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

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

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

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]

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

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

$$n_n = 1, n_s = 0$$

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

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

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

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

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

(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

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

K_b = 0.606, 0.66, 0.959, 1

∴ Take K_b = 0.606

$$V_{dbp} = \frac{2.5\times 0.606\times 20\times 12\times 410}{1.25}$$

$$\boxed{V_{dbp} = 119.26 \text{ KN}}$$

∴ Design strength of bolt value = 45.264 KN

∴ No. of bolts = $\frac{T_u}{v}$

$= \frac{430}{45.264}$

= 9.49 ¿ 10 Nos.

∴ 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]

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

$$A_g = \left[\left(110 - {}^{12}\!/_{2}\right) + \left(110 - {}^{12}\!/_{2}\right)\right]\times 12$$

A_g = 2496mm²

$$= \frac{2496\times 250}{1.1}$$

T_{dg} = 567.27 KN > 430 KN

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

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

Where,

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

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

$$= 1248\, mm^2$$

$$A = \left[110 - 22 - {}^{12}\!/_{2}\right]\times 12$$

= 984mm²

w = 110 mm

w_1= 60mm

$b_s = w+w_1-t$

$= 110+60-12$

$= 158mm$

L_c = 580 mm

$$\beta=1.4-0.076\left(\frac{110}{12}\right)\left(\frac{250}{410}\right)\left(\frac{158}{580}\right)\leq\left(\frac{410\times1.1}{250\times1.25}\right)\geq0.7$$

$$=1.28\leq1.44\geq0.7$$

$$\therefore \beta=1.28$$

$$T_{dn}=\frac{0.9\times984\times410}{1.25}+\frac{1.28\times1248\times250}{1.1}$$

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

> 430 KN

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

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

(or)

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

Where,

$$A_{vg}=[40+(9\times60)]12$$

$= 6960mm^2$

$$A_{tg}=[50\times12]$$

$= 600mm^2$

$$A_{vn}=[580-(9.5\times22)]12$$

$= 4452mm^2$

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

$= 468mm^2$

$$T_{db_1}=\frac{6960\times250}{\sqrt{3}\times1.1}+\frac{0.9\times468\times410}{1.25}$$

T_{db_1} =997.5 KN > 430KN

$$T_{db_2}=\frac{0.9\times4452\times410}{\sqrt{3}\times1.25}+\frac{600\times250}{1.1}$$

T_{db_2} = 895.13 KN > 430KN

(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:-

$$A_g=\frac{1.\frac{1\times T_u}{2}\times 1.3}{f_y} = \frac{1.1\times 215\times 1.3\times 10^3}{250} = 1230\text{mm}^2$$

Try ISA 80x80x12mm

A_g =1781mm²

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

Connection b/w gusset plate & lug angle:-

$$\text{No. of bolts} = \frac{1.\frac{2\times T_u}{2}}{v}$$

$$= \frac{1.2\times 215}{45.26}$$

n = 5.7 ¿ 6 Nos

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

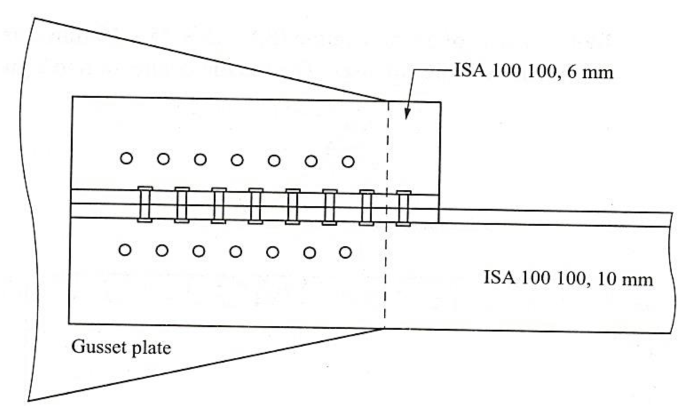
Connection b/w lug angle & main membe:-

$$n=\frac{1.\frac{4\times T_u}{2}}{v}$$

$$= \frac{1.4\times 215}{45.26}$$

n = 6.65 ¿ 7 Nos

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



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]:-

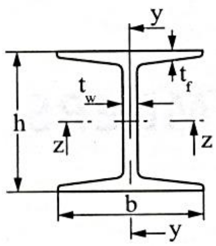
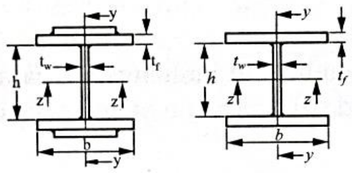
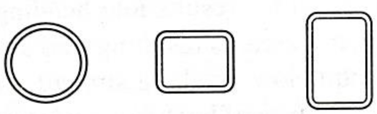
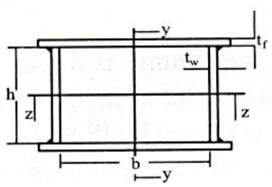
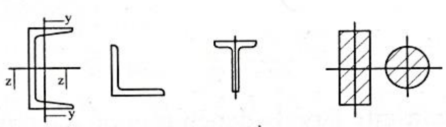
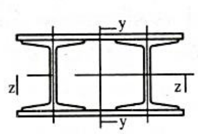
C/S in which the extreme fiber in compression can reach yield stress, but cannot develop the plastic moment of resistance, due to local buckling. The width to tks of plate element shall be less than that specified under class-3 (Semi-Compact) but greater than that specified under class-2 in table-21.

d) Class 4 [Slender]:-

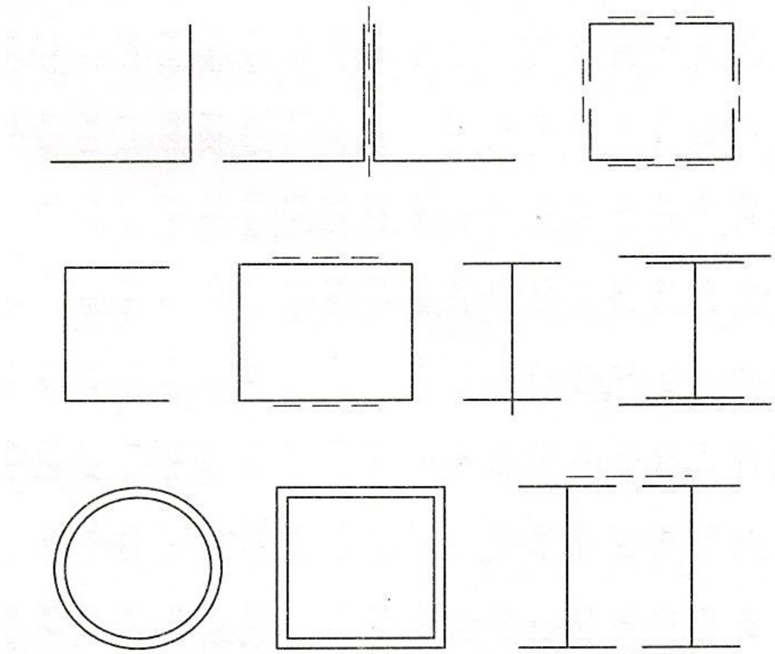
C/S in which the elements buckle locally even before reaching yield stress. The width to tks ratio of plate elements shall be greater than that specified under class-3 in table 21. In such cases, the eff. Sections for design shall be calculated either by following the provisions of IS 801 to account 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 	$h/b_f > 1.2$: $t_f \le 40$ mm	z-z	a
		y-y	b
	$40 \text{ mm} < t_f \le 100$ mm	z-z	b
		y-y	c
	$h/b_f > 1.2$: $t_f \le 100$ mm	z-z	b
		y-y	c
	$t_f > 100$ mm	z-z	d
		y-y	d
Welded I-Section 	$t_f \le 40$ 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

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

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

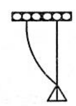
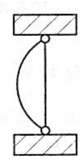
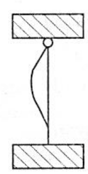
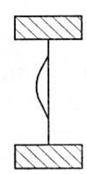
The only variable in finding the permissible comp. stress (fcd) is slenderness ratio (L/r) for the given section coming under any of the buckling class a,b,c&d.

∴ 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]

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

$$\gamma_{yy} = 54.1\,mm \quad \text{[from steel table]}$$

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

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

$$\lambda = 55.45$$

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

$$P_d = A_e f_{ed}$$

Where,

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

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

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

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

$\therefore$ 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 N/mm^2

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

λ = 0.624

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

φ = 0.79

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

f_{cd} = 178.33 N/mm²

$\therefore P_d=7485\times178.33$

P_d = 1334.7 KN

50	183
60	168

Also referring table 9c IS 800-2007 [for buckling class-c] and λ = 55.45

f_{cd} = 174.8 N/mm²

P_d = 7485x174.8

P_d =1308.5 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, $A=\frac{P_d}{f_{cd}}$

Step:3 => Select the section for the eff. Area and calculate. r_{min} [least of $\gamma_{xx} \wedge \gamma_{yy}$]

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:-

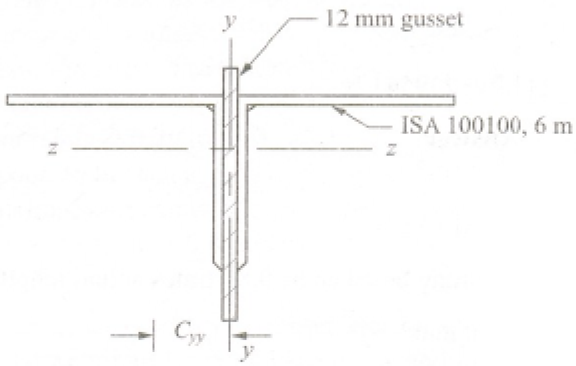
- Generally eff. Length is computed based on table-11 IS 800-2007.
- Based on connectivity, welded joints are considered to be rigid.
- For welded joints case equal to K=0.65 to 0.7

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:-

$A=1167mm^2$ $\quad$ $I_{yy}= 111.3 \times 10^4 mm^4$

$r_{yy} = r_{zz} = 30.9mm$ $\quad$ $I_{zz}= 111.3 \times 10^4 mm^4$

$C_{xx} = C_{yy} = 26.7$ 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 $10^6 mm^4$

$$\therefore r_{yy} = \sqrt{\frac{4.72 \times 10^6}{2(1167)}}$$

r_{yy} = 44.97mm

$\therefore r_{min}$ = 30.9 mm

(i) One Bolt :- [Bolt ends hinged]

$\therefore$ K = 1

$$\frac{KL}{r} = \frac{1 \times 3000}{30.9}$$

$$\frac{KL}{r} = 97.09$$

90	121
100	107

[from table 9c IS800-2007]

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

$$\frac{KL}{r} = \frac{0.85 \times 3000}{30.9}$$

$$\frac{KL}{r} = 82.52$$

80	136
90	121

[from table 9c IS 800-2007]

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

$$\frac{KL}{r} = \frac{0.7 \times 3000}{30.9}$$

$$\frac{KL}{r} = 67.96$$

60	168
70	152

[from table 9c IS 800-2007]

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:-

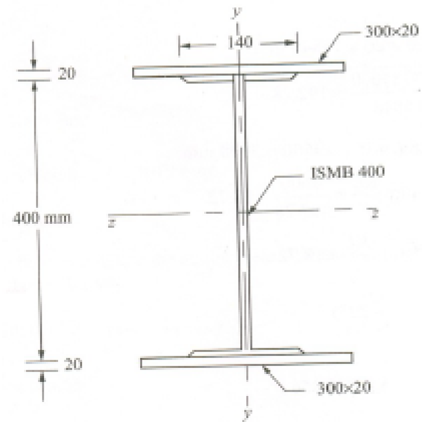
L = 4.5m

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_{xx} = 161.5mm

I_{zz} = 20458.4 x 10^4mm^4, I_{yy} = 622.1 x 10^4mm^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 = 19846mm^2

734.18×10^6

94.152×10^6

19846

$\therefore r_{zz}=\sqrt{¿\frac{¿}{¿}¿}$

$$r_{zz} = 192.34\text{mm}$$

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

$$= \sqrt{\frac{96.221\times10^6}{19846}}$$

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

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 x 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

$\therefore$ Area deficit = 30000-11789
= 18211mm²

Selecting 20mm tk plate @ top 2 bottom flange portion.

2(20xb) = 18211

b = 455.275mm ¿ 500mm

$\therefore$ 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 = 31789mm²

$$r_{zz}=\sqrt{\frac{1508.66\times10^6}{31789}}$$

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/mm²

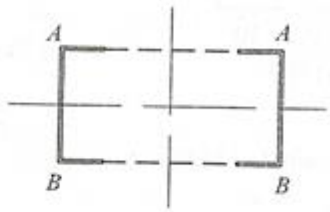
∴ Design load carrying capacity of the section = $f_{cd} \times A$

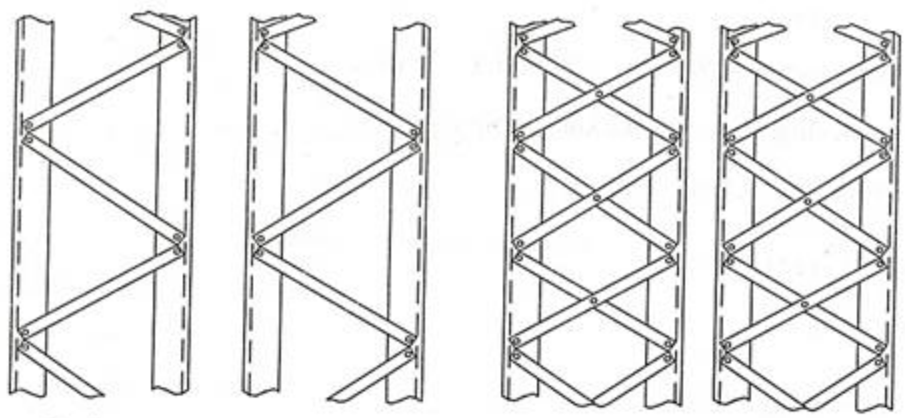
$$= 214.926 \times 31789$$

$$P_d = 6832.3\ KN > 6000KN$$

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 | Lacing on Face A | Lacing on Face B

Preferred Lacing Arrangement | Preferred Lacing Arrangement

(A) Single Laced System | (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.

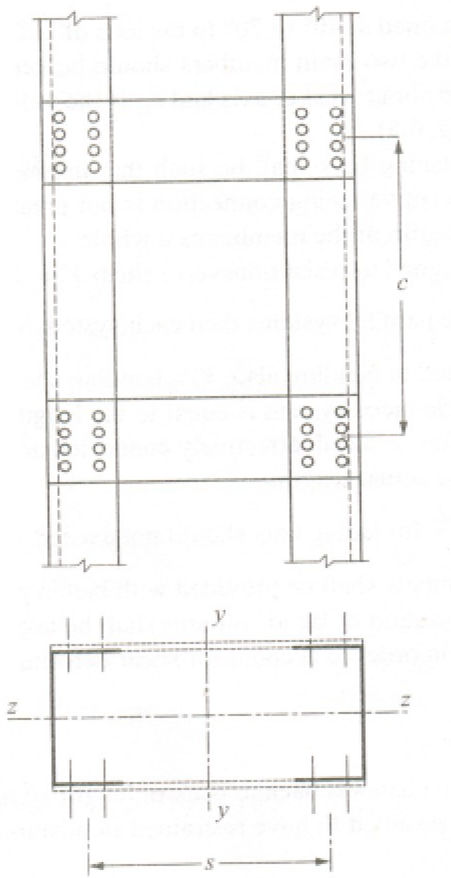
Design of Laced Columns:-°

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 & 1/16th of eff. Length for double lacings.
5. Lacing bars shall be inclined at 40° to 70° to the axis of the built up members.

6. The distance b/w the two main member should be kept, such that
   $r_{yy} > r_{zz}$ where,
   r_{yy} = Radius of gyration about the weaker axis.
   r_{zz} = Radius of gyration of stronger axis [major axis] of the individual members.
7. Maxi. Spacing of lacing bars shall be such that, the maxi. Slenderness ratio of the main member b/w consecutive lacing connections is not greater than 50 (or) 0.7 times of the unfavourable slenderness ratio of the member as a hole.
8. The lacing shall be design to resist a transverse shear, 'V_t = 2.5% P' [Axial load of column] If there are two transverse parallel systems then each system has to resist a shear force of '$V_t/2$'
9. If the column is subjected to bending also the shear due to bending moment has to be added with 'V_t'
10. The eff. Length of a single laced system is equal to the length b/w the inner faster ness. For welded joints and double lacing system, Effectively connected at the intersection, eff. Length is taken as 0.7 times the actual length.
11. The slenderness ratio KL/r for lacing shall not exceed 145. [$\because$ $\lambda_{max}=145$]
12. The eff. Slenderness ratio of laced columns shall be taken as 1.05 times the actual maxi. Slenderness ratio in order to account for shear deformation effects.

Design of Batten column:-



1. Similar to lacings, battens are design for transverse force V_t = 2.5% P
2. The batten plates should be symmetrical & spaced uniformly throughout. The eff. Slenderness ratio is 1.1 times the maxi. Actual slenderness ratio of the column to account for shear deformation.
3. Spacing shall be such that slenderness ratio of the column in any part is not greater than 50 and not greater than 0.7 times the slenderness ratio of the member as a hole about z-z axis.
4. The design shear and moment for the batten plates is given by the following relations.

$$V_b = \frac{V_c C}{N_s}$$

$$M = \frac{V_t C}{2N}$$

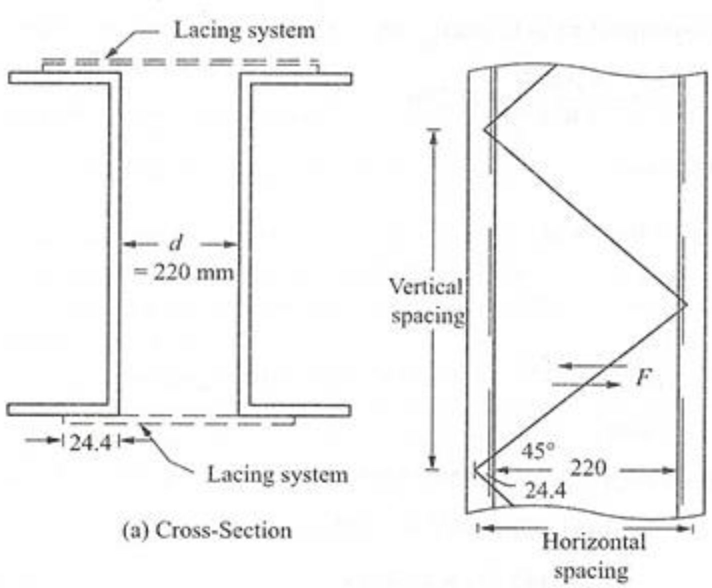
Where,

C = c/c distance along longitudinal direction.

N = No. of batten plates.

1. Design a laced column with 2 channels back to back of length 10m to carry an axial

factored load of 1400KN. The column may be assume to have restrain in position but not in direction at both ends. [Hinged ends]



(a) Cross-Section

(b) Elevation

Given:-

$P = 1400KN$, $L = 10m$, $K = 1$

Condition Both ends are hinged.

Sln:-

Assume f_{cd} as $135 N/mm^2$

To find A_{reqd}:-

$$A_{reqd} = \frac{P}{f_{cd}}$$

$$= \frac{1400 \times 10^3}{135}$$

$A_{reqd} = 10370.37mm^2$

$\therefore$ Area of each channel reqd $= \frac{10370.37}{2}$

$= 5185.2mm^2$

Try 2-ISMC 350 @ 421 N/m

$A = 5366mm^2$

$W = 421N/m$; $I_{zz} = 10008.0 \times 10^4 mm^4$; $I_{yy} = 430.6 \times 10^4 mm^4$

$r_{zz} = 136.6mm$; $r_{yy} = 28.3mm$; $c_{yy} = 24.4mm$

The lacing system is provided such that $r_{yy} > r_{zz}$. This is achieve by providing sufficient spacing b/w the two channels.

$$\therefore r_{min} = r_{zz}$$

r_{zz} of combined section = r_{zz} of individual channel section.

$\therefore$ r_{zz} of combined section = 136.6mm

Slenderness ratio:-

$$\frac{KL}{r_{min}} = \frac{1 \times 10000}{136.6}$$

$$\frac{KL}{r_{min}} = 73.206$$

For laced columns the maxi. Slenderness ratio can be increased by 5%

$$\therefore \frac{KL}{r_{min}} = 73.206 \times 1.05$$

$$= 76.86$$

70	152
80	136

From table 9 © IS800-2007

From table 10 the builtup section comes under the buckling class 'C'

$f_{cd} = 141.024 N/mm^2$

$\therefore$ Load carrying capacity of column, $P_d = f_{cd} \times A$

$= 141.024 \times 5366 \times 2$

$P_d = 1513.46$ KN > 1400KN

$\therefore$ Assumed section 2ISMC 350 is sufficient.

Design of Lateral system:- [Lacing System]

The clear distance b/w the two channels is arrived based on the condition r_{yy} ¿ r_{zz}

$I_{yy} = I_{zz}$

I_{zz} of composite section is twice the I_{zz} of an individual channel section.

$$I_{zz}(comp) = 2I_{zz}(individual)$$

$$= 2 \times 10008 \times 10^4$$

$$I_{zz} = 2.0016 \times 10^8 mm^4$$

I_{yy}:-

I_{yy} of composite section is found for the 2 channels from the centroidal Axis

$$I_{yy(comp)} = 2\left[I_{yy_{(self)}} + Ah^2\right]$$

(one channel)

$$= 2\left[430.6\times10^4 + 5366\times\left(\frac{d}{2}+24.4\right)^2\right]$$

$I_{yy} = I_{zz}$

$$2\left[430.6\times10^4 + 5366\left(\frac{d}{2}+24.4\right)^2\right] = 2.0016\times10^8$$

$$2\left[430.6\times10^4 + 5366\left(\frac{d^2}{4}+595.36+24.4d\right)\right] = 2.0016\times10^8$$

$$2\left[430.6\times10^4 + 1341.5d^2 + 3194701.76 + 130930.4d\right] = 2.0016\times10^8$$

$$2\left[1341.5d^2 + 130.930\times10^3 d + 750.07\times10^4\right] = 2.0016\times10^8$$

$$1341.5d^2 + 130.93\times10^3 d + 750.07\times10^4 = 100.08\times10^6$$

$$1341.5d^2 + 130.93\times10^3 d = 92.579\times10^6$$

d = 218

$\therefore$ d = 220mm

Assume the lacings to be provided at 45° to the horizontal.

Horizontal Spacing = d+24.4+24.4

= 220+48.8

= 268.8mm

Hori spacing = 268.8mm

Vertical spacing = 2[horizontal spacing]

= 2x268.8

= 537.6mm

he limit for slenderness ratio for each channel b/w the lacings vertically is 50

$\therefore$ Slenderness ratio for vertical spacing

[Each Channel] $= \frac{KL}{r}$

$= \frac{1\times537.6}{28.3}$

= 18.99 < 50

ransverse shear to be resisted by each lacing system is 2.5% of axial load. [Clause .6.6.1 IS 800-2007]

Load = $\frac{2.5\times1400}{100}$

= 35KN

$\therefore$ Transverse shear to be resisted by each lacing bar is 17.5KN

$$L = \frac{268.8}{\cos 45^\circ}$$

L = 380.14mm

Mini tks of lacing bar = $\frac{L}{40}$

$= \frac{380.14}{40}$

= 9.5mm

∴ Provide 10mm tk flat plates for lacing bar. Assume dia of bolt as 20mm, width of lacing bar = 3xdia

= 3x20

b = 60mm

∴ The assumed lacing bar is 60x10mm

Connection for lacing Bar:- [20mm dia]

1. Strength of bolt is single shear:- [cls 10.3.3 IS 800-2007]

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

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

n_n = 1, n_s =0

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

V_{nsp} = 56.59 KN

V_{dsp} = 45.272 KN

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

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

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

$$e=1.5d_o=33\,mm\simeq 40\,mm$$

$$p=25.d=50\,mm\simeq 60\,mm$$

k_b = 0.606, 0.659, 0.975, 1

∴ k_b = 0.606

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

V_{dbp} = 99.384KN

∴ The strength of bolt value = 45.272KN

∴ No. of bolts = $\frac{17.5}{45.27}=0.39\,{}^{N}\!/_{N_o}$

∴ Provide one 20mm φ bolt on each side of connection.

Strength of lacing bar:- [60x10mm]

Slenderness ratio of lacing bar = $\frac{KL}{r}$

$$=\frac{1\times 380.14}{\gamma_{\min}}$$

$$r_{\min}=\sqrt{\frac{I_{zz}}{A}}\ \text{(or)}\ \sqrt{\frac{I_{yy}}{A}}$$

$$I_{zz} = \frac{60 \times 10^3}{12} = 5000\,mm^4$$

$$I_{yy} = \frac{60^3 \times 10}{12} = 180 \times 10^3\,mm^4$$

$$= \sqrt{\frac{5000}{600}}$$

$$r_{min} = 2.88$$

$$= \frac{1 \times 380.14}{2.88}$$

Slenderness ratio = 131.99 < 145 [cls 7.6.6.3 IS 800-2007]

130	74.3
140	66.2

From table 9© IS 800-2007

$f_{cd} = 72.68\ N/mm^2$

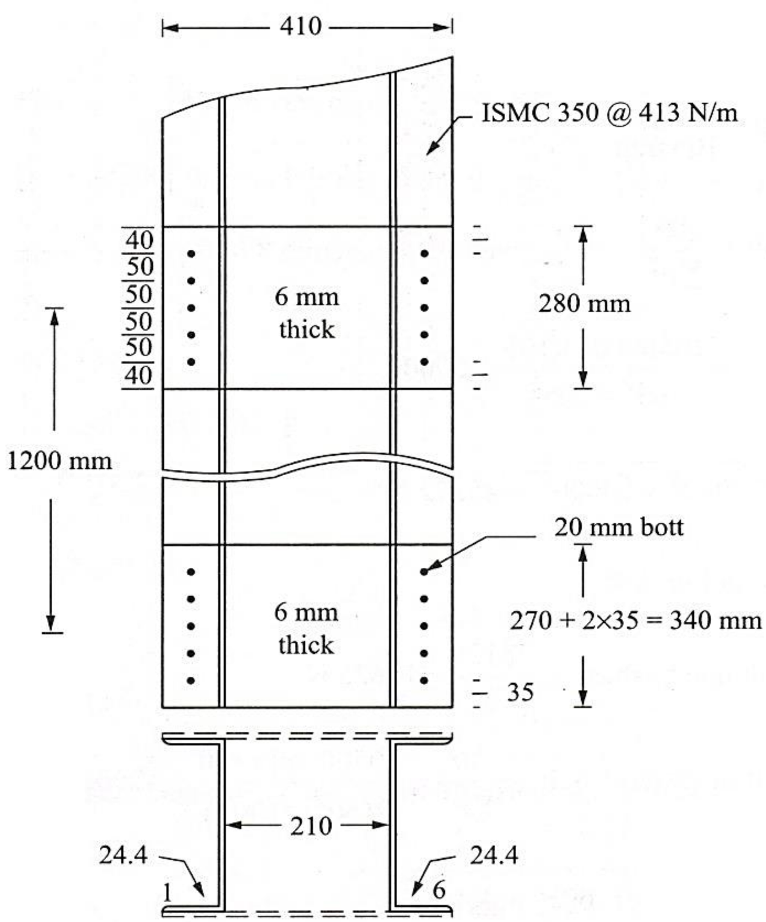
Load Carrying capacity of section = 72.68 x 60 x 10

$p_d = 43.61KN > 17.5KN$

Hence the lacing system is safe.

2. Design the above built up column using battens as lateral system. The sections selected are 2ISMC350@413N/m with clear spacing of 220mm.

[∴ The section is design as per the previous problem]



d = 220mm

Slenderness ratio = $\frac{KL}{r} = 73.21$

Slenderness ratio of battens is 1.1 times $\frac{KL}{r}$

= 73.21 x 1.1

= 80.53

C/C horizontal distance b/w the batten plate S = d+24.4+24.4

S = 268.8mm

If 'C' is the spacing of the battens. The value of 'C' is found based the relation

C/rmin<50 [The slenderness ratio of each channel b/w 2 battens plates is limited to 50]

C = 28.3 x 50

C = 141.5mm

$\therefore$ Assume C = 1200mm

Transverse shear $V_t = \frac{2.5 \times 1400}{100}$

V_t = 35KN

As per clause 7.7.2.1 IS 800-2007, shear to be resisted by Batten plate

$$V_b = \frac{V_t C}{NS}$$

$$M = \frac{V_t C}{2N}$$

Where,

N = 2

$\therefore V_b = \frac{35 \times 1200}{2 \times 268.8}$

V_b = 78.125KN

$\therefore M = \frac{35 \times 1200}{2 \times 2}$

= 105.00KN mm

M = 10.5KN.m

Width & tks of Batten Plate:-

The end batten plate should have width (depth) greater than S (268.8mm)

$\therefore$ Provide width of batten plate as 270mm width of intermediate batten plate should be greater than ¾ width of end batten plate.

$\therefore$ Provide width of intermediate batten plate = $\frac{3}{4} \times 270$

= 202.5mm

$\therefore$ Width as 210mm

Tks of batten plate should be greater than S/50

$t = \frac{268.8}{50}$

t = 5.376mm

$\therefore$ Provide the tks of 6mm

Check for stresses in Batten Plate:-

Shear stress $= \frac{V_b}{A}$

$= \frac{78.125\times10^3}{210\times6}$

$= 62$ N/mm^2

Permissible shear stress $= \frac{f_y}{1.1\sqrt{3}}$

$= 131.21$ N/mm^2

Shear Act stress < Permissible shear stress

Actual bending stress $\sigma_b = \frac{M}{Z}$

$= \frac{10.5\times10^6\times6}{td^2}$

$= \frac{10.5\times10^6\times6}{6\times210^2}$

$= 38.09$ N/mm^2

Permissible bending stress $= \frac{f_y}{1.1}$

$= 227.27$ N/mm^2

Act bending stress < Permissible bending stress

Hence the breadth of the section has to be increased.

Providing an edge of 35mm on both sides over all depth of section is 210+35+35 = 280mm

To find Actual bending stress:-

Actual bending stress $= \frac{10.5\times10^6\times6}{(280)^2\times6}$

$= 133.9$ N/mm^2

Shear stress $= 46.5$ N/mm^2

$\therefore$ Provide intermediate plate of size 280 x 6mm and end batten plate of size [270+70=340mm] 340 x 6mm

Connections for intermediate batten plate:-

- Bolts are placed along a vertical line on the batten plate.
- Force in the extreme bolt should be less than the bolt value for the connection to be safe.

Assume 20mm dia bolt

$\therefore$ Bolt value = 45.27 KN

The transverse shear acting on a connection = 78.125KN

$\therefore$ No. of bolts $= \frac{78.125}{45.27}$

$= 1.72$ ¿ 3 Nos.

Since moment also acts on the connection provide 3 Nos of bolts.
The force due to moment on extreme.

$$\text{Bolt } F_m = \frac{My}{\sum y^2}$$

$$= \frac{10.5\times10^6\times10^5}{(105^2+105^2)}$$

$$F_m = 50KN$$

$$F_s = \frac{F}{n} = \frac{78.125}{3}$$

$$= 26.04 \text{ KN}$$

$$\text{Resultant force} = \sqrt{F_m^2 + F_s^2}$$

$$= \sqrt{50^2+26.04^2}$$

$$= 56.57 \text{ KN} > 45.27 \text{ KN}$$

∴ 3 bolts are not sufficient we have to increase the no. of bolts.
∴ Assume 5 Nos of bolt along the vertical line.

$$\text{Force due to moment } F_m = \frac{My}{\sum y^2}$$

$$= \frac{10.5\times10^6\times10^5}{2(105^2)+2(105^2)}$$

$$= 25KN$$

$$F_s = \frac{78.125}{5}$$

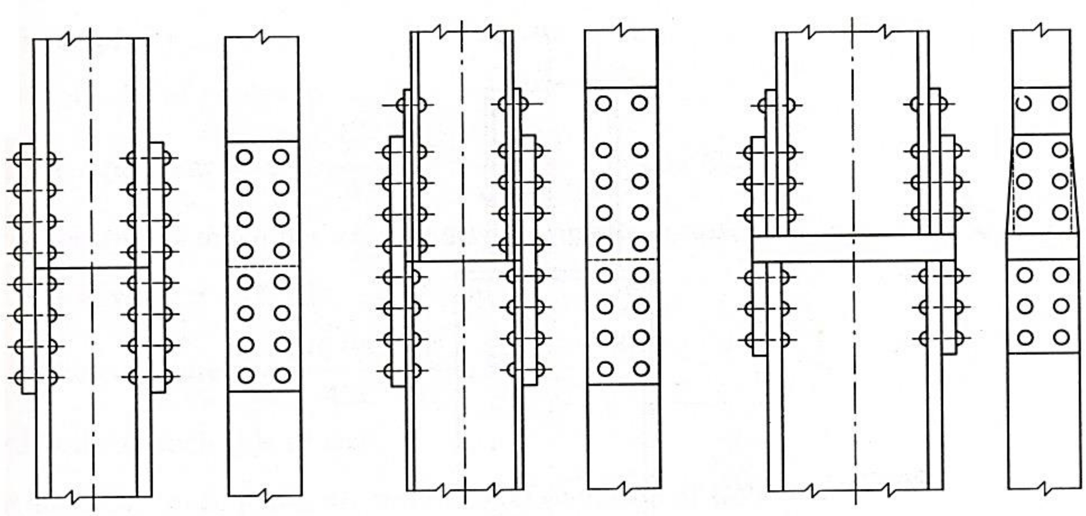
$$= 15.62 \text{ KN}$$

$$\text{Resultant Force} = \sqrt{25^2+15.62^2}$$

$$= 29.48 \text{ KN} < 45.27 \text{ KN}$$

Hence 5 Nos of 20mm dia bolts are provided in both sides.

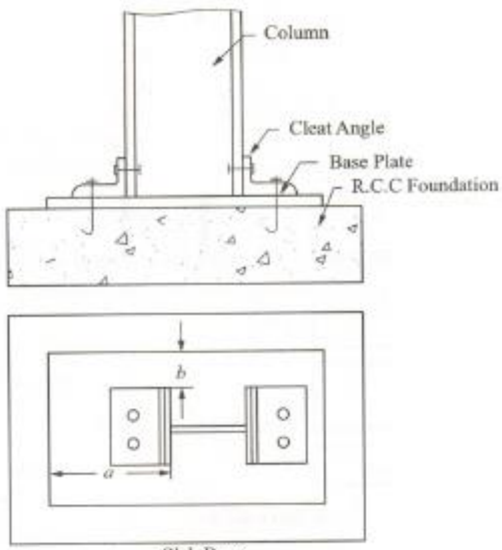
COLUMN SPLICE:-



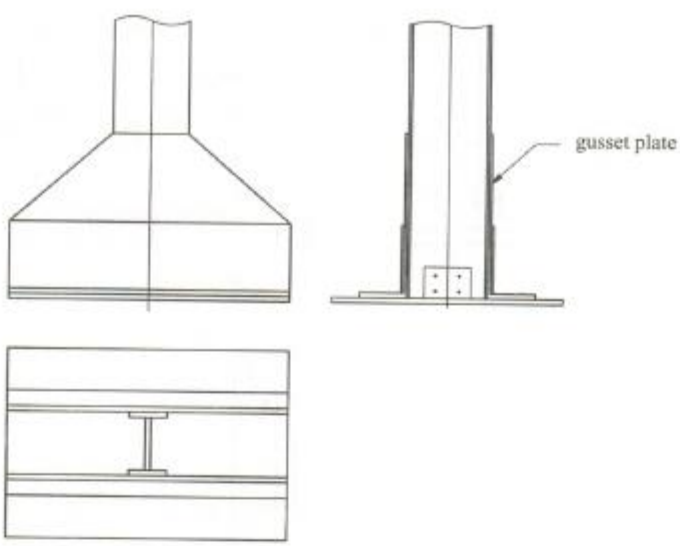
(a) (b) (c)

- When two pieces of the section are connected together to get the reqd length of column, is called a column splice.
- In a building the section of column may be change from storey to storey (for economy) and in cases when the length reqd exceeds standard size of the section available.

COLUMN BASES:-
(i) Slab Base, (ii) Gussetted Base, (iii) Grillage Foundation



Slab Base



(i) Slab Base:-

- It is used in columns carrying small loads. [Approximately upto 1000KN]
- The load is transferred to the base plate through bearing, with the help of cleat angles.

(ii) Gussetted Base:-

- Gussetted Base when the column carries heavy load [App. 1000-2000KN]
- The column is connected to the base plate using gusset plates and cleat angles.
- The load is transferred to the base party to bearing & party to gusset.

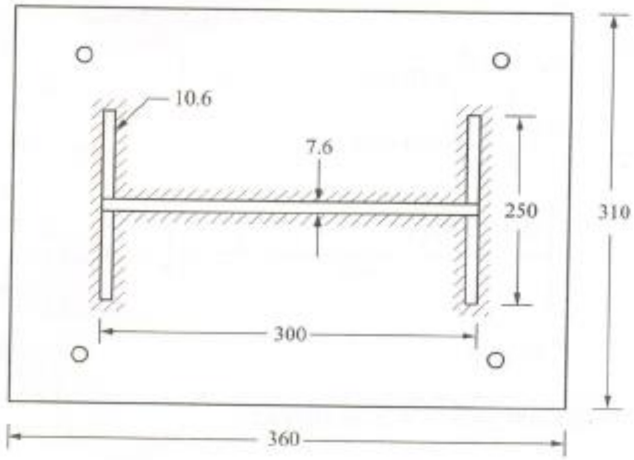
Design of slab base (or) simple base:-

1. The bearing strength of concrete is $0.45f_{ck}$
2. Area of base plate reqd is $\dfrac{P_u}{0.4f_{ck}}$

   Assume the size of plate such that the projections of base plate from the column on both sides (a&b) are kept more or less same.
3. Find the base intensity pressure
   $$w = \frac{P_u}{Areaofbaseplate}$$
4. Min thickness of base plate reqd. is
   $$t_s = \left[\frac{2.5w\left(a^2 - 0.3b^2\right)\gamma_{mo}}{f_y}\right]^{0.5} > t_f$$
5. Connection: If bolted connection is provided 2 cleat angles of size ISA 65x65x6mm are used which are connected with 20mm dia of bolts.

   If welded connection is used the size of weld is arrived based on the length of weld available alround the column.
6. The Base Plate is connected to the foundation concrete using 4 Nos of 20mm dia and 300mm long Anchor bolts.

1. Design a slab base for a column ISHB 300@577 N/m which is subjected to factored axial load of 1000KN use M20 concrete for the concrete pedestal.



Given:-

ISHB 300@577 N/m
P_u = 1000KN
f_{ck} = 20N/mm^2

Sln:-

1. Bearing stress in concrete $\sigma_{bc} = 0.45 f_{ck}$
= 0.45 x 20

σ_{bc} = 9 N/mm^2

2. Area of base plate reqd, $A = \dfrac{P_u}{0.45 f_{ck}}$

$= \dfrac{1000 \times 10^3}{9}$

A = 111.11 x 10^{3}mm^2

Assuming equal projection on both sides with a = b =30mm, size of base plate assumed is 310x360mm.

∴ Area Provided = 111.6x10^{3}mm^2

3. Pressure intensity @ base $w = \dfrac{P_u}{Areaofbaseplate}$

$= \dfrac{1000 \times 10^3}{111.6 \times 10^3}$

w = 8.96 N/mm^2 < 9N/mm^2

4. Mini. Tks of base plate $= \left[\dfrac{2.5 w \left(a^2 - 0.3 b^2 \right) \gamma m_o}{f_y} \right]^{0.5}$

$= \left[\dfrac{2.5 \times 8.96 \left(30^2 - 0.3 \times 30^2 \right) \times 1.1}{250} \right]^{0.5}$

$(t_s) 7.87\, mm < 10.6\, mm (t_f)$

∴ Provide tks of base plate as 12mm

5. Bolted Connection:-

Provide 2 cleat angle ISA 65x65x6mm connected using 20mm dia 'J' anchor bolts for a length of 300mm.

2) Design the above problem using welded connection

Sln:-

1. Bearing stress in concrete $\sigma_{bc} = 0.45 f_{ck}$

$= 0.45 \times 20$

$\sigma_{bc} = 9\ N/mm^2$

2. Area of base plate reqd, $A = \dfrac{P_u}{0.45 f_{ck}}$

$= \dfrac{1000 \times 10^3}{9}$

$A = 111.11 \times 10^3 mm^2$

Assuming equal projection on both sides (for economy) size of plate adopted is 310x360mm.

∴ Area Provided $= 111.6 \times 10^3 mm^2$

3. Pressure intensity @ base $w = \dfrac{P_u}{Actual area}$

$= \dfrac{1000 \times 10^3}{111.6 \times 10^3}$

$w = 8.96\ N/mm^2 < 9 N/mm^2$

4. Mini. Tks of base plate $= \left[\dfrac{2.5w\left(a^2 - 0.3b^2\right)\gamma m_o}{f_y}\right]^{0.5}$

$= \left[\dfrac{2.5 \times 8.96\left(30^2 - 0.3 \times 30^2\right) \times 1.1}{250}\right]^{0.5}$

Tks of flange = 10.6mm

∴ Provide tks of base plate as 12mm.

5. Welded Connection:-

Providing fillet weld alround the I-section, length available is

Length available = 4(250)-2(7.6)+2(300)-2(10)

1w = 1563.6mm

Design strength of weld:-

Providing a weld of grade 410 N/mm²(f_u)

$$\begin{array}{cc} \text{Stress} & \text{x Area} \\ \downarrow & \downarrow \end{array}$$

Design strength of weld $= \dfrac{f_u / \sqrt{3}}{\gamma m_w} \times (lw \times t)$

$= \dfrac{410 / \sqrt{3}}{1.25} \times (1563.6 \times 0.7)$

For the available length, the size of weld reqd is found.

$1000 \times 10^3 = 189.37 \times 0.73 \times 1563.6$

$S = \dfrac{1000 \times 10^3}{189.37 \times 0.7 \times 1563.6}$

S = 4.82mm

Provide 6mm fillet weld alround the column provide 20mm dia 'J' anchor bolts at the 4 corners of the base plate with length 300mm.

Gusseted Base:-

- When the load on the column is higher gusset plates are provided along the flanges of the C/S.
- The load is transferred by bearing through the base plate and also partly through the gusset plate.

Design Procedure:-

1. Area of base plate, $A=\frac{P_u}{0.45 f_{ck}}$
2. Assume various members of gusset base
   (i) Tks of gusset plate assumed as 16mm
   (ii) Size of gusset angle is assume such that the vertical leg can accomadate 2 bolts in one vertical line. The other leg is assume such that 1 bolt can be provided.

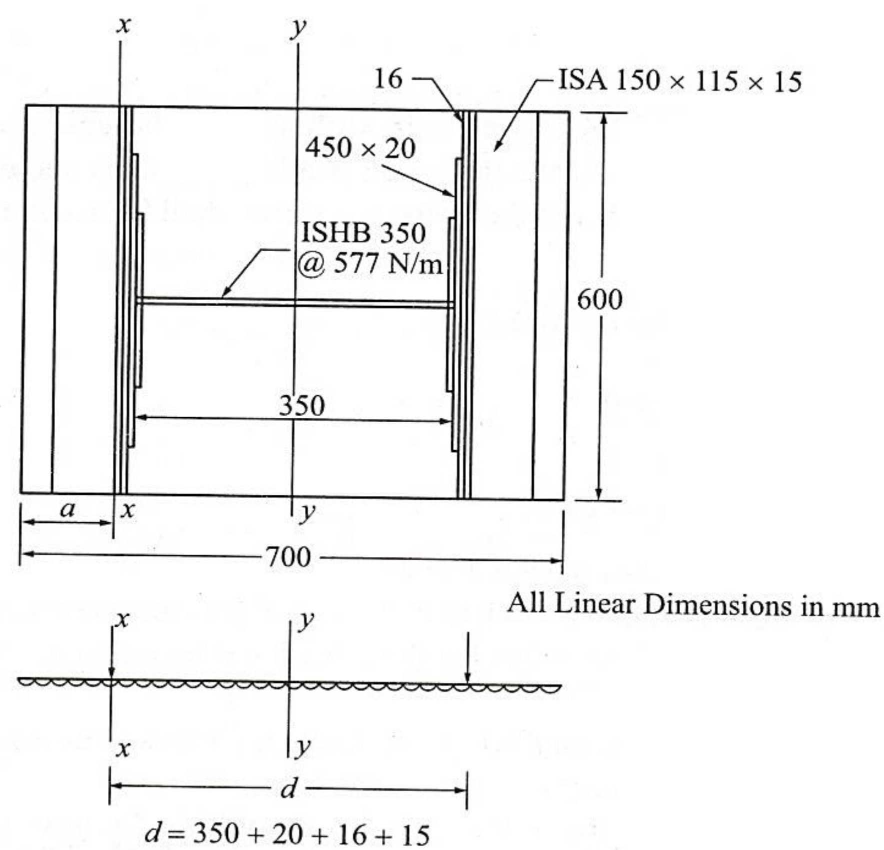
(iii) The tks of angle is kept approximately equal to the tks of gusset plate.

3. Width of gusseted base is kept suc that it will just project the outside the gusset angle and hence

$$\text{Length} = \frac{Areaofplate}{width}$$

4. The load is assumed to be transferred 50% by bearing and 50% by fasteners.
5. Tks of base plate is computed by flexural strength at the critical sections.

1. Design a gusseted base for a column ISHB 350@710N/m with 2 plates 450 x 20mm carrying a factored load of 3600KN. The column is to be supported on concrete pedestals to be built with M20 concrete.



2. Beam ISLB 500 at 750 N/m carries total factored ude of 300KN. It is supported on columns ISHB 300 at 630N/m at each end. The connection is made using M20 bolts of grade 4.6 and steel Fe410. Design the connection.

Given:-

ISLB 500

D = 500mm, bf=180mm, tf=14.1mm, tw=9.2mm

ISHB 300

D = 300mm, bf=250mm, tf=10.6mm

Sln:-

Try angle 100x100x8mm one on each side of beam.

a) Angle connecting beam web:-

The connecting bolts will be in double shear

Strength of bolts in double shear = $2\times\left[0.462 f_u\left(n_n A_{n6}\right)\right]$

$= 2\times0.462\times400\times245 / 1000$

= 90.55KN

Strength of bolts in bearing = $2d_{tpfu}$

= 2x20x9.2x410

= 147KN

Least bolt value = 90.55KN

No. of bolts = $\dfrac{\text{Re}\,action\,@\,eachend}{boltvalue}$

$= \dfrac{\frac{300}{2}}{90.55}$

Nos. of bolt = 1.7 says 2 Nos.

Provide 2 bolts @ 50mm pitch with edge distance of 40mm.

Mini. Length of angle reqd = 2x40+50

= 130mm

b) Angle connecting column flange:-

Connecting bolts will be in single shear and bearing on 8mm tks of angle

Strength of bolts in single shear = $0.462 f_u\left(n_n A_{nb}\right)$

= 0.462x400x245

= 45.3KN

Strength of bolts in bearing = $2\,dt_p f_u$

= 2x20x8x410

= 31KN

Least bolt value = 45.3KN

No. of bolts = $\dfrac{300/2}{45.3}$

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

Assume,

e = 1.5d$_o$ = 33mm ¿ 40mm

p = 2.5d = 50mm ¿ 60mm

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

= 0.606, 0.66, 0.959, 1

Take k_b = 0.606

$$= \frac{2.5\times0.606\times20\times8\times410}{1.25}$$

V_{dbp} = 79.5KN

∴ Design strength of bolt value = 45.27KN

∴ No. of bolts = $\frac{340}{45.27}$

= 7.51 ¿ 8Nos

∴ Provide 20mm φ bolt of 8 Nos.

Check for strength of section:-

(i) Design strength of section against yielding:- [cls 6.2 IS 800-2007]

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

$$A_g = \frac{\left(100 - \frac{8}{2}\right)\times250}{1.1}$$

T_{dg} = 349.04KN

(ii) Design 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,

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

w = 100

w$_1$ =50

b$_s$ = 100+50-8

b$_s$ = 142mm

L$_s$ = 460mm

$$\beta = 1.4 - 0.076\left(\frac{100}{8}\right)\left(\frac{250}{410}\right)\left(\frac{142}{460}\right) \le \frac{250\times1.1}{410\times1.25} \ge 0.7$$

= 1.22 ¿ 1.44 ¿ 0.7

∴ β = 1.22

$$A_{nc} = \left(100 - 22 - \frac{8}{2}\right)8 = 592\, mm^2$$

$$A_{go} = \left(100 - \frac{8}{2}\right)8 = 768\,mm^2$$

$$= \frac{0.9 \times 592 \times 410}{1.25} + \frac{1.22 \times 768 \times 250}{1.1}$$

T_{dn} = 387.7KN > 340KN

3) Design strength of plate against block shear:- [cls 6.3.4 IS800-2007]

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

(or)

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

Where,

$$A_{vg} = [40 + 7(60)]^8 \quad = 3680\text{mm}^2$$

$$A_{vn} = [460 - 7.5(22)]^8$$

$= 2360\text{mm}^2$

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

$$A_{tn} = \left(50 - \frac{22}{2}\right)^8$$

$= 312\text{mm}^2$

$$T_{db1} = \frac{3680 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 312 \times 410}{1.25}$$

= 574.97KN

$$T_{db2} = \frac{0.9 \times 2360 \times 410}{\sqrt{3} \times 1.25} + \frac{400 \times 250}{1.1}$$

= 493.13KN > 340KN

Design strength of section is against yielding rupture & block shear as greater than the external load of 340KN

∴ The assumed section ISx100x100x8mm is safe.

2. Explain different modes of failure of tension member.

1. Cross section yielding:-

Generally a tension member without bolt holes, can resist loads upto the ultimate load without failure. But such a member will deform in the longitudinal direction considerably nears 10% to 15% of its original length before fracture. At such a large deformation a structure become in serviceable.

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

2. Net section Rupture:-

A tension member is after connected to the main of other members by bolts or welds, when connected using bolts tension members have holes & hence reduced cross section being referred to the net area.

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

= 3.31 says 4Nos.

Provide 2 bolts on each side of flange check the tks of the angle:-

Factored shear resistance = $\dfrac{A_v \cdot f_{yw}}{\sqrt{3}\gamma_{mo}}$

$= 0.525\, A_v f_{yw}$

= 0.525x2x100x8x250

= 209.9KN > 150KN

Hence safe

ASSIGNMENT-II

1. Design a tension member to carry a factored load of 340KN use 20mm dia of black bolt and gusset plate of 8mm thickness.

Given:-

Factored load = 340KN

d = 20mm, d_o = 22mm

Thickness of gusset plate = 8m

Sln:-

No. of bolts = Tu/V

To find A_g:-

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

$$= \frac{1.1 \times 340 \times 10^3}{250}$$

A_g = 1496mm^2

Try ISA 100x100x8mm

A_g =1539mm^2

$$\gamma_{xx} = \gamma_{yy} = 30.7\,mm$$

BOLT VALUE:-

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

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

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

$n_n = 1$, $n_s = 0$

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

= 245mm^2

$$= \frac{400}{\sqrt{3}}\left[\frac{1 \times 245.04}{1.25}\right]$$

V_{dsb} = 45.27KN

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

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

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

Assume,

e = 1.5d₀ = 33mm ¿ 40mm

p = 2.5d = 50mm ¿ 60mm

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

= 0.606, 0.66, 0.959, 1

Take k_b = 0.606

$$= \frac{2.5\times0.606\times20\times8\times410}{1.25}$$

V_{dbp} = 79.5KN

∴ Design strength of bolt value = 45.27KN

∴ No. of bolts = $\frac{340}{45.27}$

= 7.51 ¿ 8Nos

∴ Provide 20mm φ bolt of 8 Nos.

Check for strength of section:-

(i) Design strength of section against yielding:- [cls 6.2 IS 800-2007]

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

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T_{dg} = 349.04KN

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

Where,

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

w = 100

w_1 = 50

b_s = 100+50-8

b_s = 142mm

L_s = 460mm

$$\beta = 1.4 - 0.076\left(\frac{100}{8}\right)\left(\frac{250}{410}\right)\left(\frac{142}{460}\right) \leq \frac{250\times1.1}{410\times1.25} \geq 0.7$$

= 1.22 ¿ 1.44 ¿ 0.7

∴ β = 1.22

$$A_{nc} = \left(100 - 22 - \frac{8}{2}\right)8 = 592\,mm^2$$

$$A_{go}=\left(100-\frac{8}{2}\right)8=768\,mm^2$$

$$=\frac{0.9\times592\times410}{1.25}+\frac{1.22\times768\times250}{1.1}$$

T_{dn} =387.7KN > 340KN

3) Design strength of plate against block shear:- [cls 6.3.4 IS800-2007]

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

(or)

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

Where,

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$$A_{tg}=50\times8=400\,mm^2$$

$$A_{tn}=\left(50-\frac{22}{2}\right)^8$$

$= 312\text{mm}^2$

$$T_{db1}=\frac{3680\times250}{\sqrt{3}\times1.1}+\frac{0.9\times312\times410}{1.25}$$

= 574.97KN

$$T_{db2}=\frac{0.9\times2360\times410}{\sqrt{3}\times1.25}+\frac{400\times250}{1.1}$$

= 493.13KN > 340KN

Design strength of section is against yielding rupture & block shear as greater than the external load of 340KN

∴ The assumed section ISx100x100x8mm is safe.

2. Explain different modes of failure of tension member.

1. Cross section yielding:-

Generally a tension member without bolt holes, can resist loads upto the ultimate load without failure. But such a member will deform in the longitudinal direction considerably nears 10% to 15% of its original length before fracture. At such a large deformation a structure become in serviceable.

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

2. Net section Rupture:-

A tension member is after connected to the main of other members by bolts or welds, when connected using bolts tension members have holes & hence reduced cross section being referred to the net area.

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

(iii) Block shear failure:-

Originally observed is bolted shear connection at sloped bea
Block shear is now reqd as potential failure of made the ends of axially loa
member also.

In this failure made the failure of member occurs along a pa
fussion on one plates & shear on ¿ lr plane along the fasterners as shown i

$$T_{db1} = \frac{A_{vg} f_y}{\sqrt{3}\gamma_{mo}} + \frac{0.9 f_u A_{vn}}{\gamma_{ml}}$$

(or)

$$T_{db2} = \frac{0.9 f_y A_{vn}}{\sqrt{3}\gamma_{ml}} + \frac{f_y A_{tg}}{\gamma_{mo}}$$

Where,

A_{vg}, A_{vn} = min gross area & net area in shear along section (
A_{tg}, A_{tn} = min gross area & net area from hole to toe of the a
(2-3)

Working stress method of steel design:-
Permissible stresses:-

1. Axial tension, $\sigma_{at} = 0.6 f_y$
2. Axial compression, $\sigma_{ac} \leq 0.6 f_y$ [depends upon L/R ratio]
3. Bending compression, $\sigma_{bc} = 0.66 f_y$
4. Per shear stress, $\tau_c = 0.45 f_y$ [generally taken as $0.4f_y$]

1. Procedure for finding permissible and compressive stress of steel sections:-
PROCEDURE:-

- Assume design stress of the member (generally rolled steel section assumed f_{cd} = 135N/mm^2) for angle section f_{cd} = 90N/mm^2, for builtup section f_{cd} = 200N/mm^2
- Required eff. Sectional area is $A = \frac{P_d}{f_{cd}}$
- Select a section for the eff. Area calculate γ_{min} (least of $\gamma_{xx} \wedge \gamma_{yy}$)
- From the end condition (decide the type of connection) determine eff. Length
- Find slenderness ratio and hence design stress f_{cd}
- Find actual load carrying capacity of compression member. $p_d = f_{cd} \times A_e$
- If the calculating value of p_d difference consider by from design load P, revise the section.

2. Design a double angle discontinues strut to carry factored axial load 170KN. The length of the strut b/w c/c of intersection is 3.85m, f_y =250N/mm^2
Given:-

L=3.85m
f_y = 250N/mm^2
P = 170KN

Sln:-

Assume f_{cd} = 90 N/mm^2

$$A_g=\frac{170\times10^3}{2\times90}$$

A_g = 944.4mm^2

For safe design increase 30%

$$A_g=944.4\times1.3$$

$$A_g=1227.2\,mm^2$$

$\therefore$ Try 2ISA 90x90x8mm

$$I_{xx}=I_{yy}=104.2\times10^4\,mm^4$$

$$\gamma_{xx}\,\gamma_{yy}=25.1\,mm$$

$$\gamma_{xx}=\gamma_{yy}=27.5\,mm$$

$$r_{uu}=34.7\,mm$$

$$r_{vv}=17.5\,mm$$

A* = 2x1379 = 2758mm^2

I_y *= 2[104.2x10^4+1379(25.1+10/2)^2

$$r_y*¿\sqrt{\frac{4.583\times10^6}{2758}}$$

$r_y*¿$ = 40.76mm
¿

K = 1

$$\therefore\frac{KL}{\gamma_{\min}}=\frac{1\times3850}{27.5}$$

$$\frac{KL}{\gamma_{\min}}=140$$

$\therefore\ f_{cd}=66.2\,{}^{N}\!/_{mm^2}$ [from table 9c]

$\therefore P_d=f_{cd}\times A$

= 66.2x2758

= 182.58KN > 170KN

$\therefore$ Hence the assumed section is safe.

$$\partial_{act}<\partial_{allow}$$

$\therefore$ Hence the section is safe in deflection

4. Check for web buckling:-

$$F_{cdw}=(b_1+n_1)t_w f_c$$

$$n_1=\frac{h}{2}=\frac{600}{2}=300\,mm$$

b_1 =100mm

$$\lambda w=\frac{2.5d}{tw}$$

$$= \frac{2.5 \times 523.4}{11.2}$$

λw = 116.83

110	94.6
120	83.7

[from table 9 © IS800-2007]

$\therefore$ f_c = 87.155 N/mm^2

$\therefore \hbar dw = (100+300) \times 11.2 \times 87.155$

$\hbar dw$ = 390.454KN > 225KN

Hence the section is safe against web buckling.

5. Check for web crippling:-

$$F_w = \frac{(b_1 + n_2) t_w f_{yw}}{\gamma_{mo}}$$

$$n_2 = 2.5 (t_f + t_1)$$

= 2.5 (21.3+17) = 82.5mm

$$F_w = \frac{(100 + 82.5) 11.2 \times 250}{1.1}$$

F_w = 464.545KN > 225KN

Hence the section is safe against web crippling.

3. An ISMB section of depth 500mm is used as a beam over as a span of 6m with s.s ends. Determine the maxi. Factored udl that the beam can carry if the ends are restrained against torsion, but compression flange is laterally unsupported.

Given:-

Span = 6m

Depth = 500mm

Section => ISMB 500

Sln:-

Section Properties of ISMB500:-

A = 11074mm^2, b_f = 180mm, t_f = 17.2mm, t_w = 10.2mm, I_{zz} = 45218.3x10^{3}mm^4, I_{yy} = 1369.8x10^{3}mm^4, r_1 = 17mm, z_{pz} = 2074.67x10^{3}mm^3, z_{ez} = 1808.7x10^{3}mm^3, r_{yy} = 35.2mm, r_{zz} = 202.1mm

$$d = h - 2 (t_f + r_1)$$

= 500-2(17.2+17)

d = 431.6mm

To find maxi.Moment & S.F:-

The maxi. Moment of the beam M = $\frac{wl^2}{8}$

The design moment capacity of the section $M_d = \beta_b Z_p f_{bd}$

Where,

f_{bd} => Taken from table-13 for f_{cr}, b given in table-14 [cls 8.2.2 IS800-2007]

f_{cr}, b :- [Table-14 IS 800-2007]

The critical stress f_{cr}, b is found based on slenderness ratio $\frac{KL}{r}$ and $\frac{h}{t_f}$ ratio

Here, K=1

$$\therefore \frac{KL}{\gamma_{\min}} => \frac{1\times 6000}{35.2} = 170.45$$

$$\frac{h}{t_f} => \frac{500}{17.2} = 29.07$$

$\frac{KL}{r}$	h/t_f		
	25	29.07	30
170	136.7	124.16	121.3
170.45	136.26	123.75	120.89
180	127.1	114.97	112.2

From table-14 [IS 800-2007]

$\therefore$ Critical stress $f_{cr}, b = 123.75\,N/mm^2$

f_{bd} :-

Refer [table-13 IS800-2007] α_{LT} =0.21 (for R.S section)

$\therefore$ Refer table – 13(a) IS 800-2007

150	106.8
100	77.3

From table-13(a)

$\therefore$ f_{bd} = 91.31 N/mm² (for f_{cr}, b = 123.75N/mm²

$\therefore$ The design bending strength of ISMB $M_d = \beta_b Z_p f_{bd}$

Buckling Class:- [Table-2 IS 800-2007]

$$\frac{b}{t_f} = \frac{0.5b_f}{t_f} = \frac{90}{17.2} = 5.2 \sum 9.4 \sum ¿$$

¿

$$\frac{d}{t_w} = \frac{431.6}{10.2} = 42.31 \sum 84 \sum ¿$$

¿

$\therefore$ The section comes under plastic

$\therefore$ $\beta_b = 1$ [cls 8.2.2 IS 800-2007]

$M_d = 1\times 2074.67\times 10^3 \times 91.31$

M_d =189.44KN.m

$M = \frac{wl^2}{8}$

M_d = 189.44KN.m

The safe udl the section can carry is found by equating M & M_d

M = M_d

$$189.44\text{x}10^6 = \frac{w\times 6000^2}{8}$$

w = 42.09KN/m