UNIT-I

LIMIT STATE DESIGN

INTRODUCTION TO L.S.D OF STEEL STRUCTURES:-


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 STEEL STRUCTURES:-

- 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, sulphur, 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 = 7860kg/m³
   - Modulus of elasticity, E = 2 x 10⁵ N/mm²
   - Poissons ratio, µ = 0.3
   - Co-eff of thermal expansion α = 12 x 10⁻⁶ /°c
   - Modulus of rigidity, G = 0.76 x 10⁵ N/mm²

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/mm²
       Ultimate stress 410 N/mm²
The mechanical properties of all the grades are 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 \(- 2 \text{ KN/m}^2\)
   - For Commercial Buildings \(- 3 \text{ KN/m}^2\)

3. Wind Load [I.S 875 Part-III]
4. Snow Load [I.S 875 Part-IV]
5. Seismic Load (or) Earthquake Load [I.S 1893-2002]
6. Accidental Loads
7. Errection Loads
8. Crane Loads
These all are the loads acting on steel structures.

**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. Uncertainty in calculation of strength.
   I.S 800 recommends 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
4. Protective measure
5. Easy of maintenance

Section 15 of IS 800-2007 details with durability.

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

The possible connections in steel designs are
1. Riveted connections
2. Bolted Connections
3. Welded Connections
1. Riveted Connections:-

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

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 \( \rightarrow \) above Fe415 steel

   ![Bolt Diagram]

   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.
- This connection is used in machine parts subjected to dynamic loading.
- Since the connection is more tight and much better bearing contact this establish

2) Friction Type Bolts:- [HSFG – High Strength Friction Grib bolts]
- These are made from high strength steel rod, where the surface of a shank is kept unfinished and are tightened to a proof load using calculated wrenches.
- Nuts are prevented by using clamping devices.
- The shearing load is first resisted by the frictional force b/w the member and the head.
- It can be used for dynamic moving loads and HSFG Bolts replaces rivets.
- The nominal dia available are 16,20,24,30 & 36mm.

Advantages & Disadvantages of bolted connection Advantages of HSFG bolts over block bolts [R.B]

**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

![Diagram of pitch, gauge distance and edge distance.](image)
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 shall 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 $\Sigma$ where
         $\Sigma = \sqrt{250/fy}=\Sigma$
         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

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
made by providing a single cover or double cover plate on either side connected to the main plates.

**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
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

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

Where,

\( \gamma_{ml} \) 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) \[ A_n = (b - n_d) t \]

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

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_st’ 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 \( V_{dsp} = \frac{V_{nsb}}{Y_{mb}} \)

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

Where,
- \( f_u \) => 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

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

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 \( \beta_{ij} \) is given by

\[
\beta_{ij} = 1.075 - 0.005 \frac{l_j}{d}
\]

Limit of 0.75 \( \leq \beta_{ij} \leq 1.0 \)

(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.

\[
\beta_{ig} = \frac{8d}{3d + l_g}
\]

(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 \( \beta_{pk} \) is given by

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

(v) Thus the capacity of bolt in shear is

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

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

The design bearing strength of the bolt is

\[
V_{dbp} = \frac{V_{nbp}}{Y_{mb}}
\]
Where,

\[ V_{nbp} = 2.5kb \text{ dt } fu \]
\[ V_{nbp} \Rightarrow \text{Nominal bearing strength of bolt} \]
\[ K_b \Rightarrow \text{Smaller value of least of} \]
\[ \frac{e}{3d_o}, \frac{p}{3d_o} - 0.25, \frac{f_{ub}}{f_u}, 1.0 \]
\[ E \Rightarrow \text{End distance of the bolt} \]
\[ P \Rightarrow \text{Pitch distance} \]
\[ d_o \Rightarrow \text{Dia of bolt hole} \]
\[ d \Rightarrow \text{Nominal dia of bolt} \]
\[ t \Rightarrow \text{Sumation of tks of connecting plates experiencing bearing streis in same direction} \]
\[ f_{ub} \Rightarrow \text{Ultimate tensile stress of the bolt} \]
\[ fu \Rightarrow \text{Ultimate tensile stress of plate} \]

**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

\[ f_y = 250 \frac{N}{mm^2} \wedge f_u = 410 \frac{N}{mm^2} \]

Design strength of solid plate

(i) In yielding \[ = \frac{250}{1.1} = 227.27 \frac{N}{mm^2} \]

(ii) In rupture \[ = \frac{0.9 \times 410}{1.25} = 295.2 \frac{N}{mm^2} \]
∴ 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:-

\[ t = 20\text{mm} \]

**Bolt:** M20

Grade 4.6 => \( fu = 400\text{N/mm}^2 \)

\( fy = 250\text{N/mm}^2 \)

**Plate:**

Fe 410 [E250]

\( Fu = 410\text{ N/mm}^2 \)

\( Fy = 250\text{ N/mm}^2 \) [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 \( T_{an} = \frac{0.9 Anfu}{\gamma_{ml}} \)

\[ A_n = (b - nd_o) \times t \]

\[ ps = 0 \quad \therefore \text{Bolts are on a straight line} \]

\[ = (180 - 3\times22) 20 \quad \therefore d_o = 20+2=22 \]

\( An = 2280\text{mm}^2 \)

\( \gamma_{ml} = 1.25 \) [from table 5- I.S 800-2007]

\[ d_o = \text{Dia of bolt hole} = 20+2=22\text{mm} \]
\[ T_{dn} = 0.9 \times 2280 \times 410 \]
\[ \frac{1.25}{1.25} \]
\[ T_{dn} = 673.056 \text{ KN} \]

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

(i) **Strength of bolt in shear** \( v_{db} = \frac{v_{nsb}}{\gamma_{mb}} \)

\[ v_{nsb} = \frac{f_u}{\sqrt{3}} \left[ N_n A_{nb} + N_s A_{sb} \right] \]

\( N_n = \text{No. of shear planes @ the thread} = 1 \)
\( N_s = \text{No. of shear planes @ shank} [N_s = 0 \text{ for lap } N_s = 1 \text{ for D.C.B.J}] \)

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

\[ = 0.78 \times \frac{\pi \times 2^2}{4} \]

\[ A_{nb} = 245 \text{mm}^2 \]

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

\[ V_{nsb} = 339.481 \text{ KN} \]

\[ \therefore V_{dsb} = \frac{339.48}{1.25} \]

\[ V_{dsb} = 271.58 \text{ KN} \]

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

Take

\[ \beta_{ij} = \beta_{ig} = \beta_{pk} = 1 \]

\[ V_{dbp} = \frac{V_{nbp}}{\gamma_{mb}} \]

\[ V_{dbp} = 2.5 \text{ kb dt fu} \]

\( K_b = \text{least of } e/3d_o, p/3d_o-0.25, f_{ub}, 1.0 \]

\( \text{Fu} \)

\( E = \text{end distance [centre of the extreme end bolt to the edge Ir to direction of load.} \)

\[ k_b \Rightarrow \frac{30}{3 \times 22}, \frac{60}{3 \times 22} - 0.25 \]

\[ k_b \Rightarrow 0.45, 0.659, 0.976, 1 \]

Take \( K_b \text{ value of whichever less} \)

\[ V_{nbp} = 2.5 \times 0.45 \times 20 \times 20 \times 410 \]

\[ V_{nbp} = 186.3 \text{ KN} \]

\[ V_{dbp} = \frac{186.3}{1.25} \]
\[ V_{dBp} = 149.04 \text{ KN} \]
\[ \therefore \text{ Design strength of bolt} = 6 \times 149.04 \]
\[ V_{dBp} \text{ bolt} = 894.24 \text{ KN} \]
\[ \text{Design strength of the joint} = 271.58 \text{ KN} \]
\[ \text{Design strength of joint is the least of strength of joint} = 673.06 \text{ KN, 271.58 KN} \]
\& 894.24 \text{ KN} \]

**Strength of Solid Plate:-**

\[ \text{Strength of Solid Plate} = \frac{f_y \times A_g}{\gamma_{ml}} \]
[yielding sides the strength of solid plate]
\[ = \frac{250}{1.1} \times 180 \times 20 \]

\[ \text{Strength of solid plate} = 818.18 \text{ KN} \]
\[ \therefore \text{ Efficiency of joint} \eta = \frac{271.58}{818.18} \times 100 \]

\[ \eta = 33.19 \]

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]
\( F_u = 410 \text{ N/mm}^2 \)
\( F_y = 250 \text{ N/mm}^2 \)

**Bolt:-**
M20, Grade 4.6
\( \phi \) of bolt = 20 mm
\( f_{ub} = 400 \text{ N/mm}^2 \)
\( f_{yb} = 240 \text{ N/mm}^2 \)

**Sln:-**
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:-**

\[ T_{dn} = \frac{0.9 Anfu}{\gamma_{ml}} \]

\[ A_n = [b - nd_{ij}] t \]

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

\[ A_n = 2280 \text{mm}^2 \]

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

\[ d_o = 20 + 2 = 22 \]

\[ = \frac{2280 \times 0.9 \times 410}{1.25} \]

\[ T_{dn} = 673.056 \text{KN} \]

(2) **Strength of bolts:-**

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

\[ V_{dbp} = \frac{V_{nbp}}{\gamma_{nb}} \]

\[ V_{nbp} = 2.5 kb \cdot dt \cdot fu \]

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

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

\[ K_b = 0.45, 0.659, 0.976, 1 \]

Take \( K_b \) value of whichever is less

\[ \therefore K_b = 0.45 \]

\[ V_{nbp} = 2.5 \times 0.45 \times 20 \times 20 \times 410 \]

\[ V_{nbp} = 186.3 \text{KN} \]

\[ V_{dbp} = \frac{186.3}{1.25} \]

\[ V_{dbp} = 149.04 \text{KN} \]

\[ \therefore \text{Strength of bolt in bearing} = 6 \times 149.04 \]

\[ V_{dbp} = 894.24 \text{KN} \]

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

\[ V_{dsp} = \frac{V_{nsb}}{\gamma_{nb}} \]

\[ V_{nsb} = \frac{fu}{\sqrt{3}} \left[ N_n A_{nb} + N_k A_{sb} \right] \]
∵ Double cover butt jt provided each bolts resists shear along two planes, the section at the root & another section at the shank.

∴ $n_n=n_s=1$ for each bolts

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

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

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

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

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

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

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

$V_{nsb} = 774.8 \text{KN}$

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

Reduction factors $\beta_i = \beta_g = \beta_{pk} = 1$

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

Strength of the solid plate:-

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

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

Strength of the solid plate = 818.18KN

$$\eta = \frac{619.84}{818.18} \times 100 \ [\text{Tk of thinner plate is the least of sum of cover plate 20(or) 24mm}]$$

$$\eta = 75.76\%$$

(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:-
\( f_{ub} = 400 \text{ N/mm}^2 \)
\( f_{yb} = 240 \text{ N/mm}^2 \)

Plate:- \( f_u = 410 \text{ N/mm}^2 \), \( f_y = 250 \text{ N/mm}^2 \)

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

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

\[
T_{dn} = \frac{0.9 \, A_{nfu}}{Y_{ml}}
\]
\[
A_n = [b - n_d o]t
\]

Provide 18mm dia of bolt hole.
\[
= [50 - 1 \times 18] \times 14
\]
\[
A_n = 448 \text{ mm}^2
\]
\[
T_{dn} = \frac{0.9 \times 448 \times 410}{1.25} [Y_{ml} \rightarrow \text{table IS 800-2007}]
\]
\[
T_{dn} = 132.25 \text{ KN}
\]

**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]**

\[
V_{dsp} = \frac{V_{nsb}}{Y_{mb}}
\]
\[
V_{nsb} = \frac{f_u}{\sqrt{3}} [n_n A_{nb} + n_s A_{sb}]
\]
For lap joint $n_a = 1$
$n_s = 0$

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

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

Assume dia of bolt 16mm for IS

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

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

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

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

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

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

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

$V_{nbp} = 2.5k_b \cdot d \cdot f_u$

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

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

$K_b = 1, 0.676, 0.975, 1.0$

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

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

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

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

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

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

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

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

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

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

132.25 KN

248.32 KN

∴ Design strength of joint/50mm width = 57.94 KN
Check for hoop tension:-

The action of force

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

Where,

\[ P \Rightarrow \text{Internal Pressure}, \quad D \Rightarrow \text{Dia of Shell} \]
\[ T \Rightarrow \text{Tks of the shell} \]

\[ T = \frac{12 \times 1000}{2 \times 14} = 428.57 \text{ N/mm}^2 \]

\[ \therefore \text{For 50mm width hoop tension} = 428.57 \times 50 = 21.43 \text{ KN} \]

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

\[ = 32.14 \text{ KN} < 57.94 \text{ km} \]

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.

![Diagram showing zig-zag bolts and dimensions]

Given Data:-

Bolt : Grade 4.6

\[ f_{ub} = 400 \text{ N/mm}^2 \]
\[ f_{yb} = 240 \text{ N/mm}^2 \]

Plate:- \( f_u = 410 \text{ N/mm}^2 \)
\[ f_y = 250 \text{ N/mm}^2 \]

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 -1
\[ An_1 = (b - n_d_0) t \]
\[ = [50 – 1 \times 18] 14 \]
\[ An_1 = 448 \text{ mm}^2 \]

(ii) along section 2-2
\[ An_2 = [b - n_d_0 + \sum \left( \frac{|P_s|^2}{A_g} \right)] t \]

Where
- \( P_s \Rightarrow 40 \text{ mm} \)
- \( g \Rightarrow 25 \text{ mm} \)
- \( A_{n_2} = \left[ 50 – 2 \times 18 + \frac{2 \times 40^2}{4 \times 25} \right] 14 \)
- \( A_{n_2} = 644 \text{ mm}^2 \)

The least area decides the failure of plate.

\[ \therefore \text{ Section 1-1 is weaker} \]

**Design strength of plate @ the joint:-**
\[ T = \frac{0.9 \times Anfu}{Y_m} \]
\[ = \frac{0.9 \times 448 \times 410}{1.25} \]
\[ T = 132.25 \text{ KN} \]

**Design strength of bolt:-**
(i) Strength of bolt in shear:- [50mm width]
\[ V_{dsp} = \frac{V_{nsb}}{Y_{mb}} \]
\[ V_{nsb} = \frac{f_u}{\sqrt{3}} \left( n_n A_{nb} + n_s A_{sb} \right) \]
- \( n_n = 1, \ n_s = 0 \)
- \( A_{nb} = \frac{0.78 \times \pi d^2}{4} \]
- \( = \frac{0.78 \times \pi \times 16^2}{4} \)
- \( A_{nb} = 156.8 \text{ mm}^2 \)
- \( V_{nsb} = \frac{400}{\sqrt{3}} \left[ 2 \times 1 \times 156.8 \right] \)
- \( V_{nsb} = 72.42 \text{ KN} \)
- \( V_{dsp} = \frac{72.42}{1.25} \)
- \( V_{dsp} = 57.94 \text{ KN} \)
(ii) **Strength of bolt in bearing:**-[50mm width]

\[
V_{dbp} = \frac{V_{nbp}}{V_{mb}}
\]

\[
V_{nbp} = 2.5 \text{ kb. dt. } fu
\]

\[
K_b = \frac{e}{3d_o}, \frac{p}{3d_o} - 0.25, \frac{f_{ub}}{fu}, 1.0
\]

Assuming sufficient edge distance is available

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

Take \( K_b \) value is the least of 0.676

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

\[
V_{nbp} = 155.21 \text{ KN}
\]

\[
V_{dbp} = \frac{155.21}{1.25}
\]

\[
V_{dbp} = 124.17 \text{ KN}
\]

For 2 bolts \( V_{dbp} = 2 \times 124.17 \)

\[
V_{dbp} = 248.3 \text{ KN}
\]

\[.\] Design strength of bearing for 50mm width = 248.23 KN

**Check Hoop Tension:**-

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

\[
\frac{1000 \times 12}{2 \times 14}
\]

\[= 428.57 \text{ N/mm}^2\]

For 50mm width of hoop tension = 428.57 x 50

\[= 21.43 \text{ KN}\]

The actual hoop tension acting on shell = 21.43 KN

Applying Partial safety factor for load as 1.5 factored design load

\[
= 21.43 \times 1.5
\]

\[= 32.14 \text{ KN} < 57.94\]

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  \( \varphi = 20 \text{ mm} \), \( f_{ub} = 400 \text{ N/mm}^2 \)
  Grade 4.6

Sln:-
(i) **LAP JOINT**:
  1. Strength of bolt in shear: [cls 10.3.3 IS800-2007]

\[
V_{dsp} = \frac{V_{nsb}}{\gamma_{mb}}
\]
\[
V_{nsb} = \frac{f_{u}}{\sqrt{3}} \left[ n_n A_{nb} + n_s A_{sb} \right]
\]

\( n_n = 1 \), \( n_s = 0 \)
\( A_{nb} = \frac{0.78 \times 20^2 \times \pi}{4} \)
\( A_{nb} = 245 \text{ mm}^2 \)
\( V_{nsb} = \frac{400}{\sqrt{3}} \times 1 \times 245 \)
\( V_{nsb} = 56.58 \text{ KN} \)
\( V_{dsp} = \frac{56.58}{1.25} \)
\( V_{dsp} = 45.26 \text{ KN} \)

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

\[
V_{dsp} = \frac{V_{nsb}}{\gamma_{mb}}
\]
\[ V_{nbp} = 2.5k_b dt \cdot fu \]

\[ K_b = \frac{e}{3d_o} \cdot \frac{p}{3d_o} - 0.25 \cdot \frac{fub}{fu} \cdot 1 \]

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]

\[ V_{dsp} = \frac{V_{nsb}}{Y_{mb}} \]

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

\[ A_{nb} = 0.78 \times \pi d^2 \frac{4}{4} \]
\[ = \frac{0.78 \times \pi \times 20^2}{4} \]
\[ A_{nb} = 245 \text{ mm}^2 \]

\[ A_{nsb} = \frac{\pi d^2}{4} \]
\[ = \frac{\pi \times 20^2}{4} \]
\[ A_{nsb} = 314.1 \text{ mm}^2 \]
\[ n_n = 1, n_s = 1 \]

\[ V_{nsb} = \frac{400}{\sqrt{3}} [1 \times 245 + 1 \times 314.1] \]
\[ V_{nsb} = 129.1 \text{ KN} \]

\[ V_{dsp} = \frac{129.1}{1.25} \]
\[ V_{dsp} = 103.28 \text{ KN} \]

2. Strength of bolt in bearing: [cls 10.3.4 IS 800-2007]
\[ V_{dsp} = \frac{V_{nsb}}{Y_{mb}} \]
\[ V_{nbp} = 2.5k_f dt \cdot f_u \]
\[ K_b = \frac{e}{3d_o}, \frac{p}{3d_o} - 0.25, \frac{f_{ub}}{f_u}, 1 \]

Take
\[ K_b = 1 \quad \text{[‘t’ is least of 16mm (2x10mm)]} \]
\[ \downarrow \]
\[ V_{nbp} = 2.5 \times 1 \times 20 \times 16 \times 410 \]
\[ V_{nbp} = 328 \text{ KN} \]
\[ V_{dbp} = \frac{328}{1.25} \]
\[ V_{dbp} = 262.4 \text{ KN} \]

Design strength of bolt in bearing = 262.4 KN
\[ \therefore \text{ Design strength of bolt} = 103.28 \text{ KN} \]

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

(ii) BUTT JOINT:-
1. Strength of the bolt in shear: [cls 10.3.3 IS 800-2007]

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

Design strength of bolt in shear = 103.28 KN

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

\[
V_{dsp} = \frac{V_{nsb}}{\gamma_{mb}} \\
V_{nbp} = 2.5 K_b \cdot dt \cdot f_u \\
K_b = \frac{e}{3d_o} \cdot \frac{P}{3d_o} - 0.25 \cdot \frac{f_{ub}}{f_u}, 1
\]

Take
\[
K_b = 1 \quad \text{[‘t’ is least of 16mm (2x6mm)]}
\]
\[
\downarrow
\]
\[
V_{nbp} = 2.5 \times 1 \times 20 \times 12 \times 410 \\
V_{nbp} = 246 \text{ KN}
\]
\[
V_{dpb} = \frac{246}{1.25} \\
V_{dpb} = 196.8 \text{ KN}
\]

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] \( f_u = 400 \text{ N/mm}^2 \)

No. of bolts required \( A = \frac{P}{BoltValue} \)
Where,
Bolt value is the least of strength of bolt in single shear & bearing.

Bolt Value:-

(i) **Strength of bolt in single shear:-** [cls 10.3.3 IS 800-2007]
\[
V_{nsb} = \frac{f_u}{\sqrt{3}} \left[ n_n A_{nb} + n_s A_{sb} \right]
\]
\( n_n = 1, n_s = 0 \)
\( A_{nb} = \frac{0.78 \times \pi \times 16^2}{4} \)
\( A_{nb} = 156.83 \text{ mm}^2 \)
\( V_{nsb} = 36.218 \text{ KN} \)
\[
V_{dsp} = \frac{V_{nsb}}{\gamma_{mb}} \]
\( = \frac{36.218}{1.25} \left[ \text{from table - 5} \right] \)
\( V_{dsp} = 28.97 \text{ KN} \)

Design strength of bolt in single shear = 28.97 KN

(ii) **Strength of bolt in bearing:-** [cls 10.3.4 IS 800-2007]
\[ V_{dsp} = \frac{V_{nsb}}{Y_{mb}} \]
\[ V_{nbp} = 2.5 k_b dt . f u \]

Assume the edge distance \( e = 1.5d, 27 \text{ mm}, 30\text{mm} \)
\( p = 2.5d = 40\text{mm} \)
\[ K_b = \frac{e}{3d_o} \cdot \frac{p}{3d_o} - 0.25, \frac{f_{ub}}{f_{u}}, 1 \]
\[ = \frac{30}{3 \times 18} \cdot \frac{40}{3 \times 18} - 0.25, 1,1 \]
\[ = 0.556, 0.491, 1,1 \]

Take \( K_b = 0.491 \)
\[ V_{nbp} = 2.5 \times 0.49 \times 16 \times 12 \times 400 \]
\[ V_{nbp} = 94.08 \text{ KN} \]
\[ V_{dbp} = \frac{94.08}{1.25} \]
\[ V_{dbp} = 75.26 \text{ KN} \]

Design strength of bolt in bearing = 75.26 KN
\( \therefore \) Design strength of bolt value = 28.97 KN
\( \therefore \) No.of bolts required \( n = \frac{160}{28.97} \)
\[ n = 5.5 \sim 6 \text{ Nos.} \]

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:**

\[ T = \frac{0.9 A_n f_u}{Y_{ml}} \]
\[ A_n = [b - nd_o] t \]
\[ = [120 - 2 \times 18]12 \]
\[ A_n = 1008 \text{ mm}^2 \]
\[ = \frac{0.9 \times 1008 \times 410}{1.25} \]
\[ T = 297.56 \text{ KN} > 160 \text{ KN} \]

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]

\[
V_{dsp} = \frac{V_{nsb}}{Y_{mb}}
\]

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

\( n_n = 1, \ n_s = 0 \)

\( A_{nb} = 0.78 \times \pi \times 16^2 \)

\( A_{nb} = 156.83 \ mm^2 \)

\( A_{sb} = \pi \times 16^2 \)

\( A_{sb} = 201 \ mm^2 \)

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

\( = 82.637 \ KN \)

\[
V_{dsp} = \frac{82.637}{1.25}
\]

\( V_{dsp} = 66.109 \ KN \)

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

\[
V_{dbp} = \frac{V_{nbp}}{Y_{mb}}
\]

\( V_{nbp} = 2.5 \ kb.dt.fu \)

\[
K_b = \frac{30}{3 \times 18} \cdot \frac{40}{3 \times 18} - 0.25, 1, 1
\]

\( = 0.556, 0.491, 1, 1 \)

Take \( K_b = 0.491 \)
\[ V_{nbp} = 2.5 \times 0.491 \times 16 \times 12 \times 401 \]
\[ V_{nbp} = 96.63 \text{ KN} \]
\[ V_{dbp} = \frac{96.63}{1.25} \]
\[ V_{dbp} = 77.304 \text{ KN} \]

Design strength of bolt value = 66.109 KN

No. of bolts, \( n = \frac{160}{66.109} \)
\[ = 2.4 \approx 4 \text{ Nos.} \]

Strength of Plate @ the joint:

\[ T_{dn} = \frac{0.9 A_n f_u}{Y_{ml}} \]
\[ A_n = [b - nd_o] t \]
\[ = [120 - 3 \times 18] 12 \]
\[ A_n = 924 \text{ mm}^2 \]
\[ = \frac{0.9 \times 924 \times 410}{1.25} \]
\[ T_{dn} = 272.76 \text{ KN} \sim 160 \text{ KN} \]

\[ \therefore \text{ The plate is safe against tearing.} \]

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 \( F_1 = P/N \) 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) \( F_2 = K_r \)

Where,

\( k \Rightarrow \) Constant of Proportionality

\( r \Rightarrow \) Radial distance

The summation of force into \( \sum kr^2 \) distance is equal to the total moment acting on section. \( \sum F_2 r = Pxe \)

\( \therefore \) Equating the relation \( F_2 = K \neq \sum kr^2 = pxe \)

\[ K = \frac{pe}{\sum r^2} \]

\[ F_2 = k \cdot r \]

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

The resultant force for F1 & F2 given as

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

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

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

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

For rolled steel section ultimate stress, $f_u = 410$ N/mm^2 
$\gamma = 250$ 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]

$$V_{dsp} = \frac{V_{nsb}}{Y_{mb}}$$
\[ V_{\text{snb}} = \frac{f_u}{\sqrt{3}} [n_n A_{\text{mb}} + n_s A_{\text{nsb}}] \]

\[ n_n = 1, \quad n_s = 0 \]

\[ A_{\text{mb}} = \frac{0.78 \times \pi \times 20^2}{4} \]

\[ = 245.04 \text{ mm}^2 \]

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

\[ V_{\text{ncb}} = 56.589 \text{ KN} \]

\[ V_{\text{dsp}} = \frac{56.589}{1.25} \]

\[ V_{\text{dsp}} = 45.27 \text{ KN} \]

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

\[ V_{\text{dbp}} = \frac{V_{\text{nbp}}}{V_{\text{mb}}} \]

\[ V_{\text{nbp}} = 2.5k_b dt \cdot f_u \]

\[ K_b = \frac{e}{3d_o}, \quad \frac{p}{3d_o} - 0.25, \quad \frac{fub}{f_u}, 1 \]

\[ = \frac{90}{3 \times 22} - 0.25 \times \frac{80}{410}, 1 \]

\[ = 1.36, 0.96, 0.975, 1 \]

\[ V_{\text{nbpb}} = 2.5 \times 0.96 \times 7.6 \times 400 \times 20 \]

\[ V_{\text{nbpb}} = 145.92 \text{ KN} \]

\[ V_{\text{dbp}} = \frac{145.92}{1.25} \]

\[ V_{\text{dbp}} = 116.74 \text{ KN} \]

\[ \therefore \text{ Design strength of bolt value } = 45.7 \text{ KN} \]

Resultant force on each bolts [extreme bolt] =

\[ F = |F_1^2 + F_2^2 + 2F_1 F_2 \cos \theta| \]

Direct Shear \[ F_1 = \frac{P}{5} \]

\[ F_1 = 0.2P \]

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

\[ r = 100 \text{ mm} \]
\[
F_2 = 0.625P
\]

\[
F = 0.762P
\]

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.

\[
n = \sqrt[2]{\frac{6M}{V_P}}
\]

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 \(\sqrt[2]{\frac{6M}{2V_P}}\)

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 :-
- \(\varphi\) of bolt = 20mm
- Grade 4.6,
  - \(F_{ub} = 400\) N/mm\(^2\)
Rolled steel section Bracket is having $fu$ is 410 N/mm$^2$

$$Fu = 410 \text{ N/mm}^2$$

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 800-2007]

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

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

$n_n = 1$, $n_s = 0$

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

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

$$V_{dsb} = \frac{56.58}{1.25} = 45.26 \text{ KN}$$

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

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

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

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

Assume the pitch ‘$p$’ is $2.5d = 2.5 \times 20 = 50$mm

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

$$= \frac{50}{3 \times 22} \times \frac{50}{3 \times 22} - 0.25, \frac{400}{410}, 1$$

$$= 0.757, 0.5076, 0.976, 1$$

$$K_b = 0.5076$$

$$V_{nbp} = 2.5 \times 0.5076 \times 20 \times 8 \times 410$$

$$V_{nbp} = 83.25 \text{ KN}$$
\[ V_{dbp} = \frac{83.25}{1.25} = 66.59 \text{ KN} \]

\[ \therefore \text{Design strength of bolt value} = 45.26 \text{ KN} \]

\[ \therefore \text{No. of bolts, } n = \sqrt{\frac{6M}{2V_p}} \]

\[ = \sqrt{\frac{6 \times 300 \times 0.35}{2 \times 45.27 \times 0.05}} \]

\[ n = 11.79 \approx 12 \text{ Nos.} \]

\[ n = 12 \text{ Nos.} \]

\[ \therefore \text{Provide 12 bolts on each side.} \]

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

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

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

\[ = \frac{300}{24} \]

\[ F_1 = 12.5 \text{ KN} \]

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

\[ = \frac{300 \times 0.35 \times 0.285}{\sum r^2} \]

\[ \therefore r = 275^2 + 75 \]

\[ r = 285 \text{ mm} \]

\[ \sum r^2 = \sum x_1^2 + y_1^2 \]

\[ \sum r^2 = \sum (x_1^2 + y_1^2) \]

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

\[ \sum r^2 = 85 \times 10^4 \text{ mm}^2 \]

\[ = \frac{300 \times 350 \times 285}{85 \times 10^4} \]

\[ F_2 = 35.12 \text{ KN} \]

\[ = 12.5^2 + 35.21^2 + 2 \times 12.5 \times 35.21 \times 0.26 \]

\[ F = 40.31 \text{ KN} < 45.26 \text{ KN} \]

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

\[ N = \sqrt{\frac{6M}{2V_p}} \]

- Check for the interaction 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
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/mm²

Sln:-
Here the plane of load is \( \perp \) to the plane of bolt. Therefore as per clause 10.3.6 the following interaction formula need to be satisfied.

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

**Direct Shear:**

\[
V_{sb} = \frac{P}{n} \quad n = \sqrt{\frac{6M}{2Vp}}
\]

**BOLT VALUE:**

1. *Strength of bolt in single shear:*

\[
V_{dsb} = \frac{V_{nsb}}{y_{mb}} \quad V_{nsb} = \frac{f_u}{\sqrt{3}} \left[ n_n A_{nb} + n_s A_{sb} \right]
\]

\( n_n = 1, \quad n_s = 0 \)

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

\[
A_{nb} = \frac{0.78 \times \pi \times 24^2}{4} = 356.82 \text{ mm}^2
\]

\[
V_{nsb} = \frac{400}{\sqrt{3}} [1 \times 352.86] = 81.484
\]

\[
V_{dsb} = 81.484 \div 1.25 = 65.19 \text{ KN}
\]

2. *Strength of bolt in bearing:*

\[
V_{dbp} = \frac{V_{nbp}}{y_{mb}} \quad V_{nbp} = 2.5k_b dt \cdot f_u
\]

\[
K_b = \frac{e}{3d_o} - \frac{P}{3d_o} - 0.25 \cdot \frac{f_{ub}}{f_u}, 1
\]

Assume the edge distance \( e = 1.5 \ d_o = 40.5 \text{ mm} \sim 50 \text{ mm} \)

\( P = 2.5 \ d = 60 \text{ mm} \sim 70 \text{ mm} \)
Take $K_b = 0.614$

$$V_{nbp} = 2.5 \times 0.614 \times 24 \times 410 \times 9$$

$$= 135.94 \text{ KN}$$

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

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

∴ Design strength of bolt value = 65.19 KN

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

$$= \sqrt{\frac{6 \times 600 \times 0.3}{2 \times 65.19 \times 0.07}}$$

$n = 10.8$; 11 Nos.

$n = 11$ Nos.

Provide 11 bolts on each side.

∴ $V_{sb} = \frac{p}{n}$

$$= \frac{600}{11 \times 2}$$

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

Since N.A is @ 107.14 mm it lies b/w the 1st bolt & the 2nd 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

$$T_{b} = \frac{M_i y_i}{\sum y_i^2}$$

$$M_i = \frac{M}{1 + \frac{2h}{2l} \left( \sum y_i \frac{f}{2} \right)}$$

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

<table>
<thead>
<tr>
<th>Bolt.No</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
<th>10</th>
<th>11</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sum y_i$ (mm)</td>
<td>12.85</td>
<td>82.85</td>
<td>152.85</td>
<td>222.85</td>
<td>292.85</td>
<td>362.85</td>
<td>432.85</td>
<td>502.85</td>
<td>572.85</td>
<td>642.85</td>
</tr>
</tbody>
</table>
\[
\sum y_i = 3278.5 \text{mm} \\
\sum y_i^2 = 1.479 \times 10^6 \text{mm}^2 \\
Y_i = 642.85 \text{mm} \\
M^1 = \frac{M}{1 + \frac{2h}{2l} \left( \frac{\sum y_i^2}{\sum y_i^2} \right)} \\
= \frac{300 \times 600 \times 10^3}{1 + \frac{2 \times 750}{21} \left( \frac{3278.5}{1.479 \times 10^6} \right)} \\
M^1 = 155.4 \text{KN.m} \\
T_b = \frac{M^1 y_i}{\sum y_i^2} \\
= \frac{155.4 \times 642.85 \times 10^6}{1.479 \times 10^6} \\
= 67.54 \text{KN} \\
\]

**Design Tension capacity of bolt:- [cls 10.3.5]**

\[
T_{db} = \frac{T_{nb}}{\gamma_{mb}} \\
= \frac{0.9 A_n f_{ub}}{\gamma_{mb}} \leq \frac{f_{yb} A_{sb}}{\gamma_{mb}} \\
A_n = \frac{0.78 \times \pi \times 24^2}{4} = 352.86 \text{mm}^2 \\
A_{sb} = \frac{\pi \times 24^2}{4} = 452.39 \text{mm}^2 \\
T_{db} = \frac{0.9 \times 352.86 \times 400}{1.25} \leq \frac{240 \times 452.3}{1.1} \\
= 101 < 98.70 \text{KN} \\
\]

**Design tension capacity of bolt** \(T_{dn} = 98.70 \text{ KN} \)

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

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

- HSFG bolts are made of high tensile steel material [ultimate stress about 800N/mm²] 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 bolt jt
\[ V_{nsf} = \mu_t, n_e, k_n, F_o \]
\[ \mu_t = \text{Co-eff. Of friction (slip factor) as specified in table} \]
\[ 20[\mu_t = 0.55] \]
\[ n_e = \text{No. of eff. Interfaces offering frictional resistance to slip.} \]
\[ k_n = 10 \text{ for fasteners in clearance holes} \]
\[ 0.85 \text{ for fasteners in oversized \& short slotted holes} \]
\[ 0.7 \text{ for fasteners in long slotted holes.} \]
\[ Y_{mf} = 1.10 \text{ (if slip resistance is designed at service load)} \]
\[ F_o = \text{Mini bolt tension @ threads at installation \& may be taken as} \]
\[ A_{nb}, f_o \]
\[ A_{nb} = \text{net area of the bolt@threads} \]
\[ 0.78 \pi d^2 / 4 \]
\[ f_o = \text{Proof stress} \]
\[ 0.7 f_{ub} \]

\[ V_{dsf} = \frac{V_{nsf}}{Y_{mf}} \]

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 \[ \mu = 0.3 \]
   (iii) The clearance holes fasteners are used.
Given:-
Bolt => grade 8.8
\( f_{ub} = 800 \text{ N/mm}^2 \)
\( f_y = 640 \text{ N/mm}^2 \)

Sln:-
Slip resistance for the bolts (\( V_{nsf} \))
\[
V_{nsf} = \mu f \cdot n \cdot k \cdot f_o
\]
\[
= 0.3 \times 2 \times 1 \times 137.2
\]
\[
= 82.32 \text{ KN}
\]
Where,
\( \mu = 0.3 \)
\( n = 2 \) [for D.C.B.J]
\( k = 1 \)
\[
A_{nb} = \frac{0.78 \pi d^2}{4}
\]
\( d = 65 \)
\( d = 26 \text{ mm} \)
\( 1.5 \times d = 40 \)
\( d = 26.67 \text{ mm} \)
Provide dia of bolt as 20mm. Dia of bolt hole is 22mm
\[
A_{nb} = \frac{0.78 \pi \times 20^2}{4}
\]
\( A_{nb} = 245 \text{ mm}^2 \)
\[
f_o = f_o \times A_{nb}
\]
\[
= 560 \times 245
\]
\( f_o = 137.2 \text{ KN} \)
\[
\therefore V_{nsf} = 0.3 \times 2 \times 1 \times 137.2 = 82.32 \text{ KN}
\]
(i) When If slip resistance is designated @ service load:-
Design strength of 1 bolt = \( \frac{82.33}{1.1} \)

49
\[
\text{Design Strength of 6 bolts} = 6 \times 74.85 = 449.1 \text{ KN}
\]

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

\[
\text{Design Strength of 1 bolt} = \frac{82.33}{1.25} = 65.87 \text{ KN}
\]

\[
\text{Design strength of 6 bolts} = 6 \times 65.87 = 395.22 \text{ KN}
\]

**PYING FORCE:** [cls 10.4.7 IS 800-2007]

\[
Q = \frac{l_y}{2l_e} \left[ T_e - \frac{\beta \eta f_o b_x}{27l_e l_y^2} \right]^4 \]

When,

\[
l_e = 1.1t \sqrt{\frac{\beta f_o}{f_y}} < e \left[ Q f_o = 0.7 f_u \right]
\]

1. The joint shown in fig. as to carry a factored load of 180 KN. End plate used size 160x140x16 mm. The bolts used are M20 HSFG bolts of grade 8.8 check whether the design is safe.
Given:-

M20 HSFG Grade 8.8

\( f_{ub} = 800 \text{ N/mm}^2 \)

Plate:-

\( f_u = 400 \text{ N/mm}^2 \)

27 = 180 KN

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

\[ l_v = \frac{160}{2} - 8 - 8 - 40 \]

\[ l_v = 24\text{mm} \]

\( l_e \) least of 1.1t \( \frac{\beta f_u}{\sqrt{f_y}} \) (or) 40mm

For \( \beta = 1 \) for pretensioned bolt

\( f_e = 0.7f_u \) [\( Q \) \( f_u \) for plate assumed as 410 N/mm\(^2 \) \( f_y = 250 \text{ N/mm}^2 \)]

\[ f_o = 287 \text{ N/mm}^2 \]

\[ Q l_e = 1.1 \times 16 \sqrt{\frac{1 \times 287}{250}} \]

\[ = 18.86\text{mm} \text{ (or) 40mm} \]

\[ l_e = 18.86\text{mm} \]

Prying force

\[ Q = \frac{l_v}{2l_e} \left[ T_e - \frac{B f_o b_{et}^4}{27 l_e l_v^2} \right] \]

When,

\( \beta = 1, \eta = 1.5, T_e = 90\text{KN}, f_o = 0.7 \times 800 = 560\text{N/mm}^2 \)
\[ Q = \frac{24}{2 \times 18.86} \left[ 90 - \frac{1 \times 1.5 \times 560 \times 140 \times (16)^4}{27 \times 18.86 \times (24)^2} \right] \]

\[ Q = 40.55 \text{ KN} \]

Total force on bolt = \( T + a \)
\[ = 90 + 40.55 \]
\[ = 130.5 \text{ KN} \]

Tension capacity of bolt = \( \frac{0.9 f_{ub} A_n}{\gamma_m} \)
\[ = \frac{0.9 \times 800 \times A_n}{\gamma_m} \]
\[ A_n = \frac{0.78 \times \pi \times 20^2}{4} \]
\[ = 245 \text{ mm}^2 \]
\[ = \frac{1.25}{0.9 \times 800 \times 245} \]
\[ T_e = 141.12 \text{ KN} \]

Hence the design is safe.

**NOTE:-**
Prying force ‘Q’ need to be added when the bolts are subjected to pure tension as well as combined shear & tension. For combined shear & tension the intraction relation is given in cls 10.4.6 need to be satisfied,
\[
\left[ \frac{V_{df}}{V_{df}} \right]^2 + \left[ \frac{T_{df}}{T_{df}} \right]^2 \leq 1.0
\]

**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.
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](image-url) 

(iii) **Slot & Plug weld**: 

*Note: Similarly there can be double U, double J and double bevel butt joints.*
1. Butt weld:-

- The size of weld is specified by effective throat tks.
- In case of complete penetration butt weld, it is taken as tks of the thinner part jt.
Double U & Double V Double J type butt welds are regarded as complete penetration butt welds.

For incomplete penetration butt welds, is taken as 5/8t

The eff. Length of butt weld is taken as length of full size weld.

The mini. Length of butt weld shall be 4 times the size of weld.

2. Filled weld:-
   (a) Size of fillet weld:-
      - The size of normal fillet weld is taken as mini weld leg size
      - For deep penetration weld with penetration not less than 2.4mm, the size of weld is mini. leg size + 2.4mm
   (b) Mini size of weld is 3mm. for plates of tks 10 to 20mm, min size is 5mm, for 20 to 32mm plates min size is 6mm & greater than 32mm plates min size is 8mm
   (c) Eff. Threat tks:-
      - It shall not be less than 3mm and shall not exceed 0.7t [upto 90°] where, t=tks of thinner plate.
   (d) Eff. Length:-
      - The eff. Length of the weld is the length of weld for which the specified size and throat tks exist.
      - The welding length provided is equal to the eff. Length t twice the size of weld
         \[ L = \text{eff} + 2s \]
      - Eff. Length should not be less than 4 times the size of weld.
   (e) The min. lap should be 4 times the tks of thinner part jt (or) 40mm whichever is more.

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

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

2. For fillet, slot and plug weld:-
   The design strength is based throat area [strength of weld =\( f_{wd} \times \text{lwx t} \)]
   The design strength is given by \( f_{wd} = f_{wn} / \gamma_{mw} \).
Where,
\[ f_{wn} = \frac{fu}{\sqrt{3}} \]

\[ fu \Rightarrow \text{smaller of the ultimate stress the weld (or) of the parent metal.} \]

**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

\[ \beta_{w} = 1.2 - \frac{0.2lj}{150tt} \leq 1.0 \]

\[ lj \Rightarrow \text{length of the jt in the direction of force transfer} \]

\[ tt \Rightarrow \text{throat size of the weld} \]

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:-**

\[ L_e = 200 \text{mm} \]

Grade of plate Fe=410

\[ Fu = 410 \text{ N/mm}^2 \]

Weld:- Shop welded

**Sln:-**

(i) **Double ‘V’ butt joint:-**

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

\[ = \text{fwd x lw x t} \]
\[ f_{wd} = \frac{f_{wn}}{\gamma_{mw}} \]
\[ f_{wn} = \frac{fu}{\sqrt{3}} \]

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

\[ f_{wn} = 236.71 \text{N/mm}^2 \]
\[ f_{wd} = \frac{236.71}{1.25} \]
\[ f_{wd} = 189.368 \text{N/mm}^2 \]

Strength of weld = \(189.368 \times 200 \times 16\)
\[ = 605.977 \text{KN} \]

(ii) Single ‘V’ butt joint:-

\[ t = \frac{5}{8} \text{s} \text{ [Incomplete Penetration]} \]
\[ = \frac{5}{8} \times 16 \]
\[ t = 10 \text{mm} \]

Strength of weld = \(\frac{fu/\sqrt{3} \times l \times w \times t}{\gamma_{mw}}\)
\[ = 189.368 \times 200 \times 10 \]

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.

![Diagram of plates connected by weld](image)

Given:-

TKS of Plate = 12mm
Grade of Plate Fe410

\[ fu = 410 \text{ N/mm}^2 \]
\[ fy = 250 \text{ N/mm}^2 \]

Sln:-
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 = \( t_p - 1.5 = 12 - 1.5 = 10.5 \)mm
Assume the size of weld = 10mm
\[
t = 0.7s \\
= 0.7 \times 10 \\
t = 7mm
\]

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

Where, \( \gamma_o = 1.1 \)

Strength of smaller plate (yielding criteria) = \( \frac{250 \times 1200}{1.1} \)
\[= 272.72 \text{ KN} \]

Strength of weld = \( \frac{fu/\sqrt{3}}{\gamma_{mw}} \times lw \times t \)
\[272.72 \times 10^3 = \frac{410/\sqrt{3}}{1.25} \times lw \times 7 \]

\[lw = 205.7 \text{ mm} \approx 205 \text{ mm} \]

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, \( t = 0.7 \times S \) [ ∵ Angle of fusion = 90°]
= 0.7 x 6
\( t = 4.2 \text{mm} \)
Strength of weld = Design stress of weld x Eff. Area
\[ = \frac{f_u}{\sqrt{3}} \times lw \times t \]
\[ 225 \times 10^3 = \frac{410}{\sqrt{3}} \times lw \times 4.2 \]
\( \therefore lw = 282.89 \text{mm} \approx 283 \text{mm} \)

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 \( L_1 = \) length of weld @ top
\( L_2 = \) length of weld @ bottom
∴ For the C.G of the weld to lie at 31mm from the outstanding leg
\( \therefore L_1 \times 31 = L_2 (100 - 31) \)
\( L_1 = 2.23L_2 \)
\( L_1 \times L_2 = 283 \)
\( 2.23L_2 + L_2 = 283 \)
\( 3.23L_2 = 283 \)
L₂ = 87.62mm
L₁ = 195.39mm ≈ 200mm

Provide 200mm length of weld @ the top and 90mm length of weld @ the bottom
∴ The min. overlap 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. **Strength of the solid plate:**
\[
\begin{align*}
&= \frac{f_y A_g}{Y_o} \\
&= \frac{250 \times 200 \times 10}{1.1} \\
&= 454.5 \text{ KN}
\end{align*}
\]

Mini. Size of weld = 3mm
Maxi. Size of weld = 10 - 1.5 = 8.5mm
Assume size of weld as 8mm 78.5mm

Strength of the weld = Design stress of weld x Eff. Area
\[
= \frac{f_u \sqrt{3}}{Y_{mw}} \times l_w \times t
\]
454.5 \times 10^3 = \frac{410/\sqrt{3}}{1.25} \times lw \times 0.7 \times 8
\]
\[lw = 428.6 \text{mm}
\]
Total length available for weld \( l = 200+200 \)
\[l = 400 \text{mm}
\]
Eff. Length available for weld, \( lew = l-2.5 \times 2 \)
\[= 400-2\times8\times2 \]
\[lew = 368 \text{mm}
\]
To find strength of weld for 368mm:-
\[\therefore \quad \text{End filled weld is provided for left} = 368 \text{mm}
\]
\[\therefore \quad \text{Design strength of weld for end fillet}
\]
\[= \frac{fu/\sqrt{3}}{Y_{nw}} \times lw \times t
\]
\[= \frac{410/\sqrt{3}}{1.25} \times 368 \times 5.6
\]
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,
\[\text{Area of plug weld reqd} = \frac{\text{AdditionalStrengthreqd}}{\text{DesignStressofweld}} \times \frac{64.3 \times 10^3}{410/\sqrt{3}} \times \frac{1.25}{339.5 \text{mm}^2}
\]
\[\therefore \quad \text{Provide one side of 10mm with 2 rectangular plug welds.}
\]
\[\text{le of oneside} = 10 \text{mm}
\]
\[\therefore \quad \text{other side} = 2(10)x = 339.54
\]
\[\therefore \quad x = 16.97 \text{mm}
\]

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:-
\[\text{Tks of the plate} = 12 \text{mm}
\]
\[\text{Tie member} = \text{IS MC 250(2Nos)}
\]
Sln:-

ISM 250 – Properties:-
A = 3867mm²
\( t_f = 14.1\text{mm} \)
\( t_w = 7.1\text{mm} \)

Strength of solid Plate:-

Strength of solid plate [channel] = \( \frac{f_y A g}{\gamma_o} \)
\[ = \frac{250 \times 3867}{1.1} \]
\[ = 878.86\text{KN} \]

∴ Strength of weld read = 878.86KN

Mini size of weld = 3mm [from table-21]
Maxi size of weld = 7.1-1.5=5.6mm
∴ Provide size of weld \( S = 4\text{mm} \)
∴ Throat tks, \( t = 0.7 \times S \)
\[ = 0.7 \times 4 \]
\[ t = 2.8\text{mm} \]

Strength of weld
\[ = \frac{f_u \sqrt{3}}{Y_{mw}} \times lw \times t \]
\[ = \frac{410 \sqrt{3}}{1.25} \times lw \times 2.8 \]
\[ lw = 1657.48\text{mm} \]

The available length along sides & end = 400+250+400 = 1050mm
[Since overlap is limited to 400mm]
[Either plug weld or slot weld can be provided]
Assuming 2 nos of 30mm wide to be provided along the end of the channel at equal spacing.
∴ Reqd length of slot = \( \frac{1657-1050+2 \times 4}{4} \)
Length reqd. for the slot = 153.75mm
∴ 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, \( M = P \times e \)

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 \( \phi \) to the C.G of the weld group and the radius vector,

\[
q_2 = \frac{P \times e \times \gamma_{\text{max}}}{I_{zz}}
\]

Resultant stress \( q = \sqrt{q_1^2 + q_2^2 + 2q_1 q_2 \cos \theta} \)

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.

\[ q_1 = \frac{P}{A} \]

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

Resultant stress \[ q = \sqrt{q_1^2 + q_2^2 + 2q_1q_2 \cos \theta} \]

Weld Group:-

t = threat tks

\[ t = 0.7S = 0.7 \times 6 = 4.2m \]

\[ y = \frac{320}{2} = 160 \text{mm} \]

\[ x = \frac{a_1x_1 + a_2x_2 + a_3x_3}{a_1 + a_2 + a_3} \]

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

\[ \bar{x} = 34.24 \text{mm} \]

\[ I_{xx} + I_{yy} = I_{zz} \]
\[ I_{xx} = \frac{140 \times 4.2^3}{12} + 140 \times 4.2 \times (317.9 - 160)^2 + 311.6 \times 4.2 \times 4.2 \times 311.6 (160 - 160)^2 + \frac{140 \times 4.2^3}{12} + 140 \times 4.2 (160 - 2.1)^2 = 14661121.44 + 10589132.71 + 14661121.44 \]

\[ I_{xx} = 39911375.59 \text{mm}^4 \]

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

\[ I_{yy} = 4.78 \times 10^6 \text{mm}^4 \]

\[ \therefore I_{zz} = 44.69 \times 10^6 \text{mm}^4 \]

Direct shear stress \[ q_1 = \frac{P}{A} \]

\[ A = \text{Total area of the weld group} \]

\[ P = 140 + 320 + 140 \times 4.2 \]

\[ q_1 = 3.968 \times 10^{-4} \text{ PKN} \]

\[ q_1 = 0.3968 \text{ PN} \]

\[ q_2 = \frac{P \times e \times y_{\max}}{I_{zz}} \]

Where,

\[ e = (140 - 34.24) + 240 = 345.76 \text{mm} \]

\[ y_{\max} = \sqrt{(160)^2 + (140 - 34.24)^2} \]

\[ y_{\max} = 191.79 \text{mm} \]

\[ \frac{P \times 345.76 \times 191.79}{44.69 \times 10^6} = 1.483 \times 10^{-3} \text{ PKN/mm}^2 \]

\[ q_2 = 1.483 \text{ PKN/mm}^2 \]

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

Where,

\[ \theta = \text{The angle made by radial distance with the C.G} \]

\[ \tan \theta = \frac{160}{140 - 34.24} \]

\[ \tan \theta = 1.51285 \]

\[ \theta = 56^\circ32' \]

\[ q = 1.7336P \rightarrow (1) \]
The max load that can be applied to resist the stress the weld can take is \( \frac{410}{\sqrt{3}} \)

\[ = 1.29 \]

\[ = 189.37 \text{ N/mm}^2 \rightarrow (2) \]

Equating (1) & (2)

\[ 1.7336P = 189.37 \]

\[ \therefore \ P = 109.24 \text{ NS} \]

**Eccentric Connection - Plane of weld group \( \perp \) lr to the plane of moment:**

For eccentric connection with plane of weld group \( \perp \) lr to the plane of moment 2 types of stresses are developed,

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

\[ \text{Where, } A = 2bt \]

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

\[ \frac{F}{Z} = \frac{p \times e}{2t \times b^2} \]

\[ F = \frac{6Pe}{2tb^2} \]

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 = \( \frac{fu \sqrt{3}}{Y_m} \)

1. Design a suitable fillet weld for an eccentrically loaded bracket plate. The working load \( P = 100\text{KN} \) and eccentricity, \( e = 150\text{mm} \). Tks of bracket plate is 12mm & the column used is ISHB300@618 \( \text{N/m} \) [Plane of weld group is \( \perp \) 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 \( b = 1.1 \sqrt{\frac{6M}{2tf_{wd}}} \)
DEPARTMENT OF CIVIL ENGINEERING
DESIGN OF STEEL STRUCTURES

(LIMIT STATE DESIGN)

(FOR VI – SEMESTER)

Course material

UNIT – II    DESIGN OF TENSION MEMBER

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Tension Members:

- Yielding of gross section
- Rupture of critical section
- Block shear @ end of connection

Generally tension members are known as tie member.

![Diagram of various shapes of tension members]

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]

\[ T_{dy} = \frac{A_g f_y}{\gamma_{mo}} \]

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,

\[ An = [ b - nd_n + \sum \frac{P_{si}}{u_{si}} ] t \]
a) For Angular section design strength of rupture:

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

Where,

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

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]

$$T_{db} = \frac{A_{vg}f_y}{\sqrt{3}Y_{mo}} + \frac{0.9A_mf_u}{Y_{ml}}$$

(or)

$$T_{db} = \frac{0.9A_{vm}f_u}{\sqrt{3}Y_{ml}} + \frac{A_{vg}f_y}{Y_{mo}}$$
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.

![Critical section diagram]

**Given:**

- Size of plate = 200mm x 12mm
- Dia of bolt = 16mm
- Grade Fe410 => \( f_u = 410 \text{ N/mm}^2 \)
  \( f_y = 250 \text{ N/mm}^2 \)

**Sln:**

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

\[ T_{dg} = \frac{A_g f_y}{\gamma_{mo}} \]

\( A_g = 130 \times 12 = 1560 \text{mm}^2 \)

\( = \frac{1560 \times 250}{1.1} \)

\( \Rightarrow (table 5) \)

\( T_{dg} = 354.5 \text{ KN} \)

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

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

\( A_n = [b - nd_n]t \)

\( n = 2 \)

\( b = 130 \text{mm} \)

\( = [130-2\times18]12 \)

\( A_n = 1128 \text{mm}^2 \)
3. Design strength due to block shear: [cls 6.4 IS 800-2007]

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

(or)

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

Where,

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

Section considered for \( A_{vg} \) is \((n'g) \times t\)

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

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

\( A_{vg} = 60 \times 12 = 720\text{mm}^2 \)

\( A_{vn} = [35+60-18] \times 12 = 924\text{mm}^2 \)

\( A_{tn} = [60-18] \times 12 = 504\text{mm}^2 \)

\[ T_{db_1} = \left( \frac{1140 \times 250}{\sqrt{3} \times 1.1} \right) + \left( \frac{0.9 \times 504 \times 410}{1.25} \right) \]

\[ T_{db_1} = 298.36 \text{KN} \]

(or)

\[ T_{db} = \left( \frac{0.9 \times 924 \times 410}{\sqrt{3} \times 1.25} \right) + \left( \frac{720 \times 250}{1.1} \right) \]

\[ T_{db_2} = 321.12 \text{KN} \]

∴ 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:-
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]

\[ T_{dg} = \frac{A_g f_y}{Y_{mo}} \]

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

\[ A_g = 864 \text{mm}^2 \]

\[ = \frac{864 \times 250}{1.1} \]

\[ T_{dg} = 196.36 \text{KN} \]

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

\[ T_{dn} = \frac{0.9 A_{nc} f_u}{Y_{ml}} + \beta A_{go} f_y \]

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

Where,

\[ A_{nc} = \text{Net area of the connected leg} = (90-6/2-18) \times 6 \]

\[ A_{nc} = 414 \text{mm}^2 \]

\[ A_g = \text{Gross area of the outstanding leg} = (60-6/2) \times 6 \]

\[ A_g = 342 \text{mm}^2 \]

\[ w = \text{outstanding leg width} = 60 \text{mm} \]

\[ t = \text{tks of angle} = 6 \text{mm} \]

\[ b_s = \text{shear lag width} = w + w_1 - t \]

Assume, \( w_1 = 90/2 = 45 \text{mm} \)

Provide \( w_1 = 50 \text{mm} \)

Assume, \( b_s = 60 + 50 - 6 \)
\[ b_s = 104 \text{mm} \]
\[ L_c = 90 \text{mm} \]

\[ = \left[ 1.4 - 0.076 \times \frac{60 \times 250 \times 104}{6 \times 410 \times 90} \right] \]
\[ = 0.864 \geq 1.44 \geq 0.7 \]

Which is true \[ 0.7 \leq 0.864 \leq 1.44 \]
\[ \therefore \beta = 0.864 \]
\[ T_{dn} = \frac{0.9 \times 414 \times 410 + 0.864 \times 342 \times 250}{1.25 + 1.1} \]

\[ T_{dn} = 189.369 \text{KN} \]

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

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

(or)

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

Where,

\[ A_{vg} = (30+4 \times 50)6 = 1380 \text{mm}^2 \]
\[ A_{vn} = [230-(4.5 \times 18)]6 = 894 \text{mm}^2 \]

Block shear failure takes place along line 1 to 3

‘\( A_{tg} \)’ is found along line 1-2

\[ A_{vg} = (30+4 \times 50)6 = 1380 \text{mm}^2 \]

\[ A_{tg} \] is taken along line 2-3

\[ A_{tn} = \left[ 40 - \frac{1}{2} \times 18 \right]6 \]
\[ A_{tn} = 186 \text{mm}^2 \]
\[ T_{db} = \frac{1380 \times 250}{\sqrt{3 \times 1.1}} + \frac{0.9 \times 186 \times 410}{1.25} \]
\[ T_{db} = 235.98 \text{KN} \]

(or)

\[ T_{db} = \frac{0.9 \times 894 \times 410}{\sqrt{3 \times 1.25}} + \frac{240 \times 250}{1.1} \]
\[ T_{db} = 206.9 \text{KN} \]

\[ \therefore \text{The least of strength of section in yielding, rupture and block shear is the design strength of the section.} \]

\[ \therefore \text{Design strength of the section = 189.369 KN} \]

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

Here the 60mm side is connected to gusset plate.
∴ Assume the line of bolts to be placed at a distance 60/2 = 30mm

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

\[
T_{dg} = \frac{A_g f_y}{Y_{mo}}
\]

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

\[
A_g = 864 \text{mm}^2
\]

\[
T_{dg} = \frac{864 \times 250}{1.1} = 196.36 \text{ KN}
\]

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

\[
T_{dn} = 0.9A_{nc} f_u + \frac{\beta A_{go} f_y}{Y_{ml} Y_{mo}}
\]

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

\[
\parallel
\]

Where, [cls 6.3.3 IS 800-2007]

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

\[
= 234 \text{mm}^2
\]

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

\[
= 522 \text{mm}^2
\]
\[ w => 90\text{mm} \]
\[ t => 6\text{mm} \]
\[ b_s = w + w_1 - t \]
\[ = 90 + 30 - 6 = 114\text{mm} \]
\[ \beta = 1.4 - 0.076 \left( \frac{90}{6} \right) - \frac{250}{410} \left( \frac{114}{60} \right) \leq \frac{410 \times 1.1}{250 \times 1.25} \geq 0.7 \]
\[ \beta = 0.079 \leq 1.44 \geq 0.7 \]
Max. limit for \( \beta \):
\[ \frac{f_u Y_{mo}}{f_y Y_{ml}} \]
\[ \therefore \text{Provide } \beta = 0.7 \]
\[ \therefore T_{dn} = 0.9 \times 234 \times 410 + 0.7 \times 522 \times 250 \]
\[ \frac{1.25}{1.1} \]
\[ T_{dn} = 152.12 \text{ KN} \]

3. Design strength of plate against block shear:- [cls 6.4 IS 800-2007]
\[ T_{db} = \frac{A_{vg} f_y}{\sqrt{3} Y_{mo}} + \frac{0.9 A_{tn} f_u}{Y_{ml}} \]
(or)
\[ T_{db} = \frac{0.9 A_{vn} f_u}{\sqrt{3} Y_{ml}} + \frac{A_{tg} f_y}{Y_{mo}} \]

Where,
\[ A_{vg} = \left[ 30 + (4 \times 50) \right] \times 6 \]
\[ A_g = 1380 \text{mm}^2 \]
\[ A_{tg} = 30 \times 6 = 180 \text{mm}^2 \]
\[ A_{tn} = \left[ 30 - \frac{18}{2} \right] \times 6 \]
\[ A_{tn} = 126 \text{mm}^2 \]
\[ A_{vn} = \left[ 230 - (4.5 \times 18) \right] \times 6 \]
\[ A_{vn} = 894 \text{mm}^2 \]
\[ T_{db} = \frac{1380 \times 250 + 0.9 \times 126 \times 410}{\sqrt{3} \times 1.1} \]
\[ 1.25 \]
\[ T_{db} = 218.27 \text{ KN} \]
\[ T_{db} = \frac{0.9 \times 894 \times 410 + 180 \times 250}{\sqrt{3} \times 1.25} \]
\[ 1.1 \]
\[ T_{db} = 193.27 \text{ KN} \]
\[ \therefore \text{ Design strength of the section} = 152.12 \text{ KN} \]

**DESIGN OF TENSION MEMBER:-**

**Design Procedure:-**

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

\[ A_g = \frac{1.1T_u}{f_y} \]

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

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

4. Find the strength of the assumed section considering
   (i) Strength of section in yielding of gross area
   (ii) Strength of section in rupture of critical section.
   (iii) Strength of section against block shear at the end of connection.

5. The strength of section obtained [Design strength of section] should be more than a factored tensile force ting on the section. If not, the section has to be revised and redesign the section.

6. The slenderness ratio has to be check for the tension member, as per table-3, IS 800-2007 [Pg.No:20]

\[ \lambda = \frac{l_{eff}}{Y_{min}} \]

Where,

\[ Y_{min} \Rightarrow \text{The least of } Y_{xx} \text{ & } Y_{yy} \text{ of the section. [from steel table]} \]

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:-**

\[ T_u = 225 \text{ KN} \]
\[ d = 20\text{mm} \]
\[ f_y = 400 \frac{N}{\text{mm}^2} \]

Grade 4.6 =>
\[ f_y = 250 \frac{N}{\text{mm}^2} \]
\[ d_o = 22 \text{ mm} \]

**Sln:-**

\[ n = \frac{T_u}{v} \]
Required area, \[ A_g = \frac{1.1 T_u}{f_y} \]
\[ = \frac{1.1 \times 225 \times 10^3}{250} \]
\[ A_g = 990 \text{ mm}^2 \]

to select ISA 100x75x8mm
\[ A_g = 1336 \text{ mm}^2 \] [from steel table]
\[ \gamma_{xx} = 31.4 \text{ mm} \quad \gamma_{yy} = 21.8 \text{ mm} \]
\[ \gamma_{\text{min}} = 21.8 \text{ mm} \]
\[ \lambda = 21.8 \] is connected to the gusset plate (assumed tks 10mm) by lap jt along the 100mm side.

**BOLT VALUE:-[M20]**

(i) Strength of bolt in single shear:- [cls 10.3.3 IS 800-2007]
\[ V_{dsp} = \frac{V_{nsp}}{Y_{mb}} \]
\[ V_{nsp} = \frac{f_u}{\sqrt{3}} (n_n A_{nb} + n_s A_{sb}) \]
\[ V_{dsp} = \frac{f_u}{\sqrt{3}} (n_n A_{nb} + n_s A_{sb}) \quad \n_s = 0 \quad f \to \text{ss} \]
\[ = \frac{400}{\sqrt{3}} \left[ 1 \times \pi \times 20^2 \times 0.78 \right] \]
\[ = 1.25 \text{ Table5 – IS 800-2007} \]
\[ V_{dsp} = 45.27 \text{ KN} \]

(ii) Strength of the bolt in bearing:- [cls 10.3.4 IS 800-2007]
\[ V_{dsp} = \frac{V_{nbp}}{Y_{mb}} = \frac{2.5 k_b d_t f_u}{Y_{mb}} \]
\[ k_b = \frac{e}{3 d_o}, \quad \frac{p}{3 d_o} - 0.25, \quad \frac{f_{ub}}{f_u}, 1 \]
Assume, \( e = 1.5 \) \( d_o = 1.5 \times 22 = 33 \text{ mm} \quad \& \quad 40 \text{ mm} \)
\[ p = 2.5 d = 2.5 \times 20 = 50 \text{ mm} \quad \& \quad 60 \text{ mm} \]
\[ k_b \Rightarrow \frac{40}{3 \times 22}, \frac{60}{3 \times 22} - 0.25, \frac{400}{410}, 1 \]
\[ = 0.606, 0.659, 0.975, 1 \]
\[ k_b = 0.606 \text{ (least value)} \]

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

\[ V_{dbp} = 79.5 \text{ KN} \]

\[ T_u = \frac{V}{V_{dbp}} = \frac{225}{45.27} = 4.97 \text{ Nos.} \]

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]

\[ T_{dg} = \frac{A_g f_y}{Y_{mo}} \]

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

\[ A_g = 1336 \text{ mm}^2 \]

\[ = 1336 \times 250 \]

\[ = 334 \text{ KN} \]

\[ T_{dg} = 303.636 \text{ KN} \]

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

\[ T_{dn} = \frac{0.9 A_{nc} f_u + \beta A_{go} f_y}{Y_{ml}} \]

Where,

\[ \beta = 1.4 - 0.076 \left( \frac{w}{t} \right) \left( \frac{f_y}{f_u} \right) \left( \frac{b_s}{L_c} \right) \leq f_u Y_{mo} \leq 0.7 \]

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

\[ A_{nc} = 592 \text{ mm}^2 \]

\[ A_{go} = \left[ 75 - \frac{8}{2} \right] - 8 \]

\[ A_{go} = 568 \text{ mm}^2 \]

\[ w = 75 \text{ mm} \]

\[ t = 50 \text{ mm} \]

\[ b_s = w + w_1 - t \]

\[ = 75 + 50 - 8 \]

\[ b_s = 117 \text{ mm} \]
\[ L_c = 100\text{mm} \]
\[ \beta = 1.4 - 0.076 \left( \frac{75}{8} \right) \left( \frac{250}{410} \right) \left( \frac{117}{100} \right) \leq \left( \frac{410 \times 1.1}{250 \times 1.25} \right) \geq 0.7 \]
\[ = 0.89 \leq 1.44 \geq 0.7 \]
\[ \beta = 0.07 \leq 0.89 \leq 1.44 \]
\[ \therefore \beta = 0.89 \]
\[ T_{dn} = \frac{0.9 \times 592 \times 410}{1.25} + \frac{0.89 \times 568 \times 250}{1.1} \]
\[ T_{dn} = 289.6 \text{KN} \]

> 225 KN

3. Design strength of plate against block shear:- [cls 6.4 IS 800-2007]
\[ T_{db} = \frac{A_{vg} f_v}{\sqrt{3} \gamma_{mo}} + \frac{0.9 A_{tn} f_u}{Y_{ml}} \]
(or)
\[ T_{db} = \frac{0.9 A_{vn} f_u}{\sqrt{3} Y_{ml}} + \frac{A_{tg} f_y}{Y_{mo}} \]

Where,
\[ A_{vg} = \left[ 40 + (4 \times 60) \right] \times 8 \]
\[ A_{vg} = 2240 \text{mm}^2 \]
\[ A_{tg} = 50 \times 8 \]
\[ A_{tg} = 400 \text{mm}^2 \]
\[ A_{vn} = \left[ 280 - (4.5 \times 22) \right] \times 8 \]
\[ A_{vn} = 1448 \text{mm}^2 \]
\[ A_{tn} = \left[ 50 - \frac{22}{2} \right] \times 8 \]
\[ A_{tn} = 312 \text{mm}^2 \]
\[ T_{db_1} = \left[ \frac{2240 \times 250}{\sqrt{3} \times 1.1} \right] + \left[ \frac{0.9 \times 312 \times 410}{1.25} \right] \]
\[ T_{db_1} = 386.026 \text{KN} \]
\[ T_{db} = \left[ \frac{0.9 \times 1448 \times 410}{\sqrt{3} \times 1.25} \right] + \left[ \frac{400 \times 250}{1.1} \right] \]
\[ T_{db_2} = 337.697 \text{KN} \]

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.
\[ \therefore \text{The assume section ISA 100x75x8mm is safe.} \]
2. Solve the above problem using angle section on opposite sides of gusset plate

Given:-

\[ T_u = 225 \text{KN} \]
\[ d = 20 \text{mm} \]
\[ d_e = 22 \text{mm} \]

Grade 4.6 => \( f_u = 400 \text{N/mm}^2 \)
\( f_y = 250 \text{N/mm}^2 \)

Sln:-

\[ A_g = \frac{1.1 T_u}{f_y} = \frac{1.1 \times 225 \times 10^3}{250} \]
\[ A_g = 990 \text{mm}^2 \]

Area each angle reqd = \( 990/2 = 495 \text{mm}^2 \)

\[ \therefore \text{Select the section from steel table having area 30\% more than 495mm}^2 \]

Try ISA 70x70x5mm

\[ A_g = 667 \text{mm}^2 \text{ [from steel table]} \]

\[ \gamma_{xx} = 21.5 \text{mm} \]
\[ \gamma_{yy} = 21.5 \text{mm} \]

Bolt Value:- [M20]

(i) Strength of bolt in double shear:-

Assuming gusset plate of tks = 10mm

\[ V_{dsb} = \frac{V_{nsp}}{Y_{mb}} \]

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

\[ n_n = n_s = 1 \]

\[ A_{nb} = \frac{0.78 \times \pi \times 20^2}{4}, A_{sb} = \frac{\pi \times 20^2}{4} = 314.16 \text{mm}^2 \]
\[ V_{dsb} = \frac{400}{\sqrt{3}} \left[ 1 \times 245 + 1 \times 314.16 \right] \]

\[ V_{dsb} = 103.3 \text{ KN} \]

(ii) Strength of bolt in bearing:-

\[ V_{dbp} = \frac{V_{nbp}}{Y_{mb}} \]

\[ = \frac{2.5k_b d_f f_u}{Y_{mb}} \]

Assume \( e = 1.5 \) \( d_e = 33 \text{ mm} \) & 40mm

\( p = 2.5d = 50 \text{ mm} \) & 60mm

\[ k_b = \frac{40}{3 \times 22}, \frac{60}{3 \times 22} \]

\[ = 0.606, 0.659, 0.975, 1 \]

\[ \therefore \text{ Take } k_b = 0.606 \text{ [least value]} \]

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

\[ V_{dbp} = 99.38 \text{ KN} \]

\[ \therefore \text{ Design strength of bolt value } = 99.38 \text{ KN} \]

\[ \therefore \text{ No. of bolts } = \frac{225}{99.38} \]

\[ = 2.26 \rightarrow 3 \text{ Nos.} \]

\[ \therefore \text{ Provide 3 nos. of 20mm bolts for pitch 60mm & tks edge distance 40mm.} \]

Check for strength of section:-

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

\[ T_{dg} = \frac{A_g f_y}{Y_{mo}} \]

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

\[ A_g = 1334 \text{ mm}^2 \]

\[ = \frac{1334 \times 250}{1.1} \]

\[ T_{dg} = 303.18 \text{ KN} > 225 \text{ KN} \]

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

\[ T_{dn} = \frac{0.9 A_{nc} f_u + \beta A_{go} f_y}{Y_{ml} Y_{mo}} \]

Where,
\[ \beta = 1.4 - 0.076 \left( \frac{w}{t} \right) \left( \frac{f_s}{f_u} \right) \left( \frac{b_s}{f_y} \right) \leq f_u Y_{mo} \geq 0.7 \]

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

\[ A_{nc} = 456 \text{ mm}^2 \]

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

\[ A_{go} = 675 \text{ mm}^2 \]

\[ w = 70 \text{ mm} \]

\[ w_i = 35 \text{ mm} \quad \text{&} \quad 40 \text{ mm} \]

\[ b_s = w + w_i - t \]

\[ = 70 + 40 - 10 \]

\[ = 100 \]

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

\[ = 1.89 \leq 1.44 \geq 0.7 \]

\[ \beta = 0.7 \leq 1.19 \leq 1.44 \]

\[ : \beta = 1.19 \]

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

\[ T_{dn} = 316.87 \text{ KN} \]

\[ > 225 \text{ KN} \]

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

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

(or)

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

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

\[ A_{vg} = \left[ \left[ 40 + 2 \times 60 \right] \right] \times 5 \]

\[ = 1600 \text{ mm}^2 \]

\[ A_{vn} = \left[ \left[ 40 + 2 \times 60 \right] - 2.5 \times 22 \right] \times 5 \]

\[ = 1050 \text{ mm}^2 \]

\[ A_{tg} = 30 \times 5 \times 2 = 300 \text{ mm}^2 \]

\[ A_{tn} = \left[ \left[ 30 - 0.5 \times 22 \right] \right] \times 5 \]

\[ = 190 \text{ mm}^2 \]

\[ T_{db_1} = \frac{1600 \times 250}{\sqrt[3]{1.1}} + \frac{0.9 \times 190 \times 410}{1.25} \]
\[ T_{db_1} = 266.03 \text{ KN} \]

For 2 angles
\[ T_{db_2} = \frac{0.9 \times 1050 \times 410}{\sqrt{3} \times 1.25} + \frac{300 \times 250}{1.1} \]

\[ T_{db_2} = 247.137 \text{ KN} \]

For 2 angles
\[ T_{db} = 247.137 \text{ KN} \text{ [least value of these two]} \]

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
   - \( d = 20 \text{mm} \)
   - \( d_o = 22 \text{mm} \)
   - 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]
   \[ V_{dsb} = \frac{V_{nsp}}{Y_{mb}} \]
   \[ V_{nsp} = \frac{f_u}{\sqrt{3}} \left[ n_n A_{nb} + n_s A_{sb} \right] \]
   \( n_n = n_s = 1 \)
   \( A_{nb} = \frac{0.78 \times \pi \times 20^2}{4} = 245 \text{ mm}^2 \)
   \( A_{sb} = \frac{\pi \times 20^2}{4} = 314.16 \text{ mm}^2 \)
   \( \beta_{pk} = \left[ 1 - 0.0125 \text{ tpk} \right] \)
\[
\begin{align*}
\beta_{pk} &= 0.875 \\
V_{nsb} &= \frac{400}{\sqrt{3}} \left[ 1 \times 245 + 1 \times 314.16 \right] \times 0.875 \\
&= 112.99 \text{ KN} \\
V_{dsb} &= \frac{112.99}{1.25} \\
&= 90.392 \text{ KN}
\end{align*}
\]

2. Strength of bolt in bearing:- [cls 10.3.4 IS 800-2007]
\[
V_{dbp} = \frac{V_{nbp}}{Y_{mb}} \\
= 2.5k_b \cdot d \cdot f_u
\]
Assume \(e=1.5 \text{ d}_o = 33\text{ mm} \) \( \theta = 40\text{ mm} \)
\(p = 2.5 \text{ d} = 50\text{ mm} \) \(\phi = 60\text{ mm} \)
\[
k_b = \frac{40}{3 \times 22, 3 \times 22} - 0.25, \frac{400}{410}, 1
\]
\[
= 0.606, 0.659, 0.975, 1
\]
\[
\therefore \text{ Take } k_b = 0.606 \text{ [least value]}
\]
\[
V_{dbp} = 2.5 \times 0.606 \times 20 \times 10 \times 410
\]
\[
= 124.23 \text{ KN}
\]
\[
V_{dbp} = 99.38\text{ KN}
\]
\[
\therefore \text{ Design strength of bolt value } = 90.39 \text{ KN}
\]
\[
\therefore \text{ No. of bolts } = \frac{T_u}{V}
\]
\[
= \frac{500}{90.39}
\]
\[
n = 5.5 \approx 6\text{ Nos.}
\]
\[
\therefore \text{ Provide 6 nos. of 20mm bolts on each side}
\]
Providing the 6 bolts on each side of the connecting plate, it can be arranged 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]
\[
T_{dg} = \frac{A_g f_y}{Y_{mo}}
\]
\[
A_g = 300 \times 10 = 3000 \text{ mm}^2 \text{ [Tks of thinner plate]}
\]
\[
= \frac{3000 \times 250}{1.1}
\]
\[
T_{dg} = 681.81 \text{ KN } > 500\text{KN}
\]
2. Strength of the plate against rupture:- [cls 6.3.1 IS 800-2007]

\[ T_{dn} = \frac{0.9A_n f_u}{Y_{ml}} \]

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

\[ = [300-3\times(22)]\times 10 \]

\[ A_n = 2340\text{mm}^2 \]

\[ = 0.9 \times 2340 \times 410 \]

\[ = 1.25 \]

\[ T_{dn} = 690.77 \text{KN} > 500 \text{KN} \]

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

\[ T_{db} = \frac{A_{vg} f_y + 0.9A_{vn} f_u}{\sqrt{3Y_{mo}}} \frac{Y_{ml}}{Y_{mo}} \]

(or)

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

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

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

\[ A_{vn} = |40 + 60 - 1.5\times 22| \times 10 \]

\[ = 670 \text{mm}^2 \]

\[ A_{tg} = |2\times 110| \times 10 = 2200 \text{mm}^2 \]

\[ A_{tn} = |2|110| - 2\times 22| \times 10 \]

\[ = 1760 \text{mm}^2 \]

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

\[ T_{db_1} = 650.76 \text{KN} \]

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

\[ T_{db_2} = 614.190 \text{KN} > 500 \text{KN} \]

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.

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:-**

\[ T_u = 430\text{KN} \]

**Sln:-**

(i) No lug angle is provided:-

Assume, \( d = 20\text{mm} \)

\[ d_o = 22\text{mm} \]

Tks of gusset plate = 12mm

**BOLT VALUE:- [M20]**

\[ A_g = \frac{1.1T_u}{f_y} \]

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

\[ A_g = 1892\text{mm}^2 \]

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

Select ISA 110x110x12mm

\[ A_g = 2502\text{mm}^2 \]

\[ \gamma_{xx} = \gamma_{yy} = 33.4\text{ mm} \]

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

\[ V_{dsb} = \frac{V_{nsp}}{Y_{mb}} \]

\[ V_{nsp} = \frac{f_u}{\sqrt{3}} \left[ n_nA_{nb} + n_sA_{sb} \right] \]

\[ n_n = 1, n_s = 0 \]

\[ A_{nb} = 245\text{mm}^2 \]

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

\[ V_{nsb} = 56.58\text{ KN} \]

\[ V_{dsb} = \frac{56.58}{1.25} \]

\[ V_{dsb} = 45.264\text{ KN} \]

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

\[ V_{dgb} = \frac{V_{nbp}}{Y_{mb}} \]
\[ V_{nbp} = 2.5k_b d_t f_u \]

Assume \( e = 40 \text{mm} \)

\[ P = 60 \text{mm} \]

\[ K_b = \frac{40}{3 \times 22} \times \frac{60}{3 \times 22} - 0.25, 0.40, 1 \]

\[ K_b = 0.606, 0.66, 0.959, 1 \]

\[ \therefore \text{Take } K_b = 0.606 \]

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

\[ V_{dbp} = 119.26 \text{ KN} \]

\[ \therefore \text{Design strength of bolt value = 45.264 KN} \]

\[ \therefore \text{No. of bolts} = \frac{T_u}{v} = \frac{430}{45.264} = 9.49 \approx 10 \text{ Nos.} \]

\[ \therefore \text{Provide 10 nos of 20mm dia to bolts edge distance 40mm & pitch of 60mm.} \]

Check for strength of section:

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

\[ T_{dg} = \frac{A_g f_y}{\gamma_{mo}} \]

\[ A_g = \left[ \left( 110 - \frac{12}{2} \right) + \left( 110 - \frac{12}{2} \right) \right] \times 12 \]

\[ A_g = 2496\text{mm}^2 \]

\[ A_g = \frac{2496 \times 250}{1.1} \]

\[ T_{dg} = 567.27 \text{ KN} > 430 \text{ KN} \]

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

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

Where,

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

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

\[ = 1248 \text{mm}^2 \]

\[ A = \left[ 110 - 22 - \frac{12}{2} \right] \times 12 \]

\[ = 984\text{mm}^2 \]
w = 110 mm
w₁= 60mm
bₖ = w+w₁-t
    = 110+60-12
    = 158mm
Lᵣ = 580 mm

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

\[ = 1.28 \leq 1.44 \geq 0.7 \]

∴ \[ \beta = 1.28 \]

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

\[ T_{dn} = 653.53 \text{ KN} \]

\[ > 430 \text{ KN} \]

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

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

(or)

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

Where,

\[ A_{vg} = \left[ 40 + \left( 9 \times 60 \right) \right] 12 \]
\[ = 6960 \text{ mm}^2 \]

\[ A_{tg} = \left[ 50 \times 12 \right] \]
\[ = 600 \text{ mm}^2 \]

\[ A_{vn} = \left[ 580 - \left( 9.5 \times 22 \right) \right] 12 \]
\[ = 4452 \text{ mm}^2 \]

\[ A_{tn} = \left[ 50 - \frac{22}{2} \right] 12 \]
\[ = 468 \text{ mm}^2 \]

\[ T_{db,1} = \frac{6960 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 468 \times 410}{1.25} \]

\[ T_{db,1} = 997.5 \text{ KN} > 430 \text{ KN} \]

\[ T_{db,2} = \frac{0.9 \times 4452 \times 410}{\sqrt{3} \times 1.25} + \frac{600 \times 250}{1.1} \]

\[ T_{db,2} = 895.13 \text{ KN} > 430 \text{ KN} \]

(ii) Lug Angle is Provided:-

* When lug angle is provided the member of bolts reqd for establishing the connection reduces thereby reducing the overall length of overlap.
The connection b/w main member gusset plate is designed for \( \frac{T_u}{2} \).

The lug angle is designed for a force of \( \frac{T_u}{2} \) [Increased by 30%].

The connection b/w the main member lug angle is designed for 40% of \( \frac{T_u}{2} \) and connection b/w angle & gusset plate designed for 20% of \( \frac{T_u}{2} \).

Connection for Main Member:

\[
\begin{align*}
n &= \frac{1}{2} \times \frac{T_u}{v} \\
&= \frac{215}{45.26} \\
&= 4.75 \quad \therefore \quad \text{Provide 5 nos of 20mm bolts.}
\end{align*}
\]

Lug Angles:

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

Try ISA 80x80x12mm

\( A_g = 1781 \text{mm}^2 \)

\( \gamma_{xx} = \gamma_{yy} = 23.9 \text{mm} \)

Connection b/w gusset plate & lug angle:

\[
\begin{align*}
\text{No. of bolts} &= \frac{1}{2} \times \frac{2 \times T_u}{v} \\
&= \frac{1.2 \times 215}{45.26} \\
&= 5.7 \quad \therefore \quad \text{Provide 6 nos of bolts b/w gusset plate and lug angle.}
\end{align*}
\]

Connection b/w lug angle & main membe:

\[
\begin{align*}
n &= \frac{1}{2} \times \frac{4 \times T_u}{v} \\
&= \frac{1.4 \times 215}{45.26} \\
&= 6.65 \quad \therefore \quad \text{Provide 7 nos of bolts b/w lug angle and main member.}
\end{align*}
\]
DEPARTMENT OF CIVIL ENGINEERING
DESIGN OF STEEL STRUCTURES

(LIMIT STATE DESIGN)

(FOR VI – SEMESTER)

Course material

UNIT III - DESIGN OF COMPRESSION MEMBER

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MADURAI - 20
UNIT-III
DESIGN OF COMPRESSION 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]

<table>
<thead>
<tr>
<th>Cross-Section</th>
<th>Limits</th>
<th>Buckling About Axis</th>
<th>Buckling Class</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>z-z</td>
<td>a</td>
</tr>
<tr>
<td>Rolled I-Sections</td>
<td></td>
<td>y-y</td>
<td>b</td>
</tr>
<tr>
<td></td>
<td>(h/b_t &gt; 1.2):</td>
<td>z-z</td>
<td>b</td>
</tr>
<tr>
<td></td>
<td>( t_r \leq 40) mm</td>
<td>y-y</td>
<td>b</td>
</tr>
<tr>
<td></td>
<td>(40) mm &lt; ( t_r \leq 100) mm</td>
<td>z-z</td>
<td>c</td>
</tr>
<tr>
<td></td>
<td></td>
<td>y-y</td>
<td>c</td>
</tr>
<tr>
<td></td>
<td>(h/b_t &gt; 1.2):</td>
<td>z-z</td>
<td>b</td>
</tr>
<tr>
<td></td>
<td>( t_r \leq 100) mm</td>
<td>y-y</td>
<td>c</td>
</tr>
<tr>
<td></td>
<td>( t_r &gt; 100) mm</td>
<td>z-z</td>
<td>d</td>
</tr>
<tr>
<td></td>
<td></td>
<td>y-y</td>
<td>d</td>
</tr>
<tr>
<td>Welded I-Section</td>
<td>(t_r \leq 40) mm</td>
<td>z-z</td>
<td>b</td>
</tr>
<tr>
<td></td>
<td></td>
<td>y-y</td>
<td>c</td>
</tr>
<tr>
<td></td>
<td>(t_r &gt; 40) mm</td>
<td>z-z</td>
<td>c</td>
</tr>
<tr>
<td></td>
<td></td>
<td>y-y</td>
<td>d</td>
</tr>
<tr>
<td>Hollow Section</td>
<td>Hot rolled</td>
<td>Any</td>
<td>a</td>
</tr>
<tr>
<td></td>
<td>Cold formed</td>
<td>Any</td>
<td>b</td>
</tr>
<tr>
<td>Welded Box Section</td>
<td>Generally</td>
<td>Any</td>
<td>b</td>
</tr>
<tr>
<td></td>
<td>(except as below)</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Thick welds and</td>
<td>z-z</td>
<td>c</td>
</tr>
<tr>
<td></td>
<td>(b/t_r &lt; 30)</td>
<td>y-y</td>
<td>c</td>
</tr>
<tr>
<td></td>
<td>(h/t_w &lt; 30)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Channel, Angle, T and Solid Sections</td>
<td></td>
<td>Any</td>
<td>c</td>
</tr>
<tr>
<td>Built-up Member</td>
<td></td>
<td>Any</td>
<td>c</td>
</tr>
</tbody>
</table>
DESIGN COMpressive STRENGTH:- [cls 7.1 IS 800-2007]

\[ P_d > P_u \]

Where,
- \( P_d \) = Design compressive strength of column.
- \( P_u \) = External compression (or) design load.

\[ P_d = A_{exfcd} \]

\[ A_{ex} = \text{Eff. Area} \]

\[ f_{ed} = \text{design compressive stress} \]

\[ f_{ed} = \frac{f_y}{\varphi + \left[ \varphi^2 - \lambda^2 \right]^{0.5}} = \frac{x f_y}{Y_{mo}} \leq \frac{f_y}{Y_{mo}} \]

Where,
- \( \varphi = 0.5 \left[ 1 + \alpha \left[ \lambda - 0.2 \right] + \lambda^2 \right] \)
- \( \lambda \) = non-dimensional eff. Slenderness ratio.

\[ f_{cc} = \text{Ruler buckling stress} \]

\[ f_{cc} = \frac{\pi^2 E}{KL/r} \]

Where,
- \( KL/r \) = eff. Slender ratio (or) eff. length, KL to appropriate radius of gyration.
- \( \alpha \) = Imperfection factor given in table 7.
\[ X = \text{Stress reduction factor [see table-8]} \]
\[ = \frac{1}{1 + \left( \phi + \left| \phi^2 - \lambda^2 \right|^{0.5} \right)} \]

\[ \lambda_{mo} = \text{Partial safety factor for material strength.} \]

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>
<thead>
<tr>
<th>Boundary Conditions</th>
<th>Schematic Representation</th>
<th>Effective Length</th>
</tr>
</thead>
<tbody>
<tr>
<td>At One End</td>
<td>At the Other End</td>
<td></td>
</tr>
<tr>
<td>Translation (1)</td>
<td>Rotation (2)</td>
<td>(5)</td>
</tr>
<tr>
<td>Restrained</td>
<td>Restrained</td>
<td>2.0L</td>
</tr>
<tr>
<td>Restrained</td>
<td>Free</td>
<td>Free</td>
</tr>
<tr>
<td>Restrained</td>
<td>Free</td>
<td>1.0L</td>
</tr>
<tr>
<td>Restrained</td>
<td>Restrained</td>
<td>1.2L</td>
</tr>
<tr>
<td>Restrained</td>
<td>Free</td>
<td>Free</td>
</tr>
<tr>
<td>Restrained</td>
<td>Restrained</td>
<td>0.8L</td>
</tr>
<tr>
<td>Restrained</td>
<td>Restrained</td>
<td>0.65L</td>
</tr>
</tbody>
</table>
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 pinned.

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,
\[ K = 1.0 \quad \text{[from table-11 IS 800-2007]} \]
\[ y_{xx} = 129.5 \text{ mm} \quad \text{[from steel table]} \]
\[ y_{yy} = 54.1 \text{ mm} \]
\[ \therefore r_{min} = 54.1 \text{ mm} \]
\[ = \frac{1 \times 3000}{54.1} \]
\[ \lambda = 55.45 \]

To find design comp. stress:- [cls 7.1.2.1 IS 800-2007]
\[ P_d = A_e f_{ed} \]

Where,
\[ f_{ed} = \frac{f_y}{y_{mo}} = \frac{x f_y}{y_{mo}} \leq f_y \]
\[ \varphi = 0.5 \left[ 1 + \alpha \left( \lambda - 0.2 \right) + \lambda^2 \right] \]
\[ = \sqrt{\frac{f_y}{f_{cc}}} = \sqrt{\frac{f_y (KL/r)^2}{\pi^2 E}} \]

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
\[ \therefore \text{The section need to be check for buckling} \]

Class-C
\[ \alpha = 0.49 \quad \text{[from table-7 IS 800-2007]} \]
\[ f_{cc} = \frac{\pi^2 E}{(KL/r)^2} = \frac{\pi^2 \times 2 \times 10^5}{(55.44)^2} \]
\[ f_{cc} = 641.98 \text{ N/mm}^2 \]
\[ \lambda = \sqrt{\frac{f_y}{f_{cc}}} = \sqrt{\frac{250}{641.98}} \]

\[ \lambda = 0.624 \]

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

\[ \varphi = 0.79 \]

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

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

\[ \therefore P_d = 7485 \times 178.33 \]

\[ P_d = 1334.7 \text{ KN} \]

Also referring table 9c IS 800-2007 [for buckling class-c] and \( \lambda = 55.45 \)

\[ f_{cd} = 174.8 \text{ N/mm}^2 \]

\[ P_d = 7485 \times 174.8 \]

\[ P_d = 1308.5 \text{ KN} \]

**DESIGN OF COMPRESSION MEMBERS:**

**Step:1 =>** Assume the design comp. stress of the member [Generally for rolled steel sections assume \( f_{cd} = 135 \text{ N/mm}^2 \), for angle section assume \( f_{cd} = 90 \text{ N/mm}^2 \) for builtup sections carrying large loads assume \( f_{cd} = 200 \text{N/mm}^2 \)]

**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, \( P = 180 \text{KN} \)

\( L = 3 \text{m} \)

\( K = 0.85 \)

**Sln:-**

To find \( f_{cd} \):

Assume a design comp. stress \( f_{cd} = 90 \text{ N/mm}^2 \)

To find \( A \) reqd:-
Reqd Area \[ A = \frac{P_d}{f_{cd}} \]
\[ = \frac{180 \times 10^3}{90} \]
\[ A = 2000 \text{mm}^2 \]
Try ISA 90x90x12mm

Properties of ISA 90x90x12mm:-
\[ A = 2019 \text{mm}^2 \]
\[ \gamma_{xx} = \gamma_{yy} = 27.1 \text{mm} \]
\[ \gamma_{uu} = 34.1 \text{mm}, \gamma_{vv} = 17.4 \text{mm} \]

Buckling Class:-
Angle come under buckling class-c
\[ KL = 0.85 \times 3000 \]
\[ r = 17.4 \]
\[ KL = 146.55 \]

Refer Table 9c, IS 800-2007

| \[ f_{cd} \] = 61.615 N/mm² |
|---|---|
| 140 | 66.2 |
| 150 | 59.2 |

∴ Strength of strut = 2019 x 61.615
\[ P_d = 232.7 \text{N/mm}^2 \]
\[ P_d = 124.4 \text{KN} < 180 \text{KN} \]

Revise the section:- Try ISA 130x130x8mm
\[ A = 2022 \text{mm}^2 \]
\[ I_{xx} = I_{yy} = 40.3 \]
\[ I_{uu} = 51.0 \text{mm}, I_{vv} = 25.5 \text{mm} \]
\[ KL = 0.85 \times 3000 \]
\[ r = 25.5 \]
\[ KL = 100 \]
\[ f_{cd} = 107 \text{N/mm}^2 \text{[from table 9c IS 800-2007]} \]
∴ Strength of strut = 2022 x 107
\[ = 216.35 \text{KN} > 180 \text{KN} \]

2. Design the above member when both ends are hinged.
Given:-
\[ P = 180 \text{KN} \]
\[ L = 3 \text{m} \]

Sln:-
To find \( A_{cd} \):
Assume a design comp. stress \( f_{cd} = 90 \text{N/mm}^2 \)
To find \( A_{\text{reqd}} \):-
\[ A = \frac{P_d}{f_{cd}} \]
\[ = \frac{180 \times 10^3}{90} \]
\[ A = 2000 \text{mm}^2 \]

Try ISA 130x130x8mm
\[ A = 2022 \text{mm}^2 \]
\[ r_{\text{min}} = 25.5 \text{mm} \]
\[ \frac{KL}{r} = \frac{1 \times 3000}{25.5} = 117.65 \]
\[ f_{cd} = 86.26 \text{ N/mm}^2 \]

\[ \therefore \text{Strength of section} = 2022 \times 86.26 = 174.4 \text{KN} < 180 \text{KN} \]
Hence unsafe

\[ \therefore \text{Revise the section with } r_{\text{min}} \text{ more than } 25.5 \text{mm} \]

Try ISA 150x150x10mm
\[ A = 2903 \text{mm}^2 \]
\[ r_{\text{min}} = 29.3 \text{ mm} \]
\[ \frac{KL}{r} = \frac{1 \times 3000}{29.3} = 102.39 \]
\[ f_{cd} = 104 \text{ N/mm}^2 \]

\[ \therefore \text{Strength of section} = 2903 \times 104 = 301.9 \text{ KN} > 180 \text{KN} \]
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

![Diagram of a truss with a strut and gusset plate](image)

Given:-
- L = 3m
- 2 ISA 100x100x6mm
- Tks of gusset plate = 12mm

Sln:-
Section Properties of ISA 100x100x6mm:
- \( A = 1167 \text{mm}^2 \)
- \( I_{yy} = 111.3 \times 10^4 \text{mm}^4 \)
- \( I_{zz} = 111.3 \times 10^4 \text{mm}^4 \)
- \( r_{yy} = r_{zz} = 30.9 \text{mm} \)
- \( C_{xx} = C_{yy} = 26.7 \text{mm} \)

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.

\[
\therefore r_{zz} \text{ of composite section } = \sqrt{\frac{I_{zz}}{A}} = \sqrt{\frac{111.3 \times 10^4}{1167}} = 30.9 \text{mm}
\]

\[
r_{yy} \text{ of composite section } = \sqrt{\frac{I_{yy}}{A}}
\]

Where,
- \( I_{yy} \) = M.O.I of composite section
- \( I_{yy} = 2 \left( I_{yy} \text{ of one angle section} + A(t/2 + cy)\right) \)
- \( I_{yy} = 4.72 \times 10^6 \text{mm}^4 \)
\[ r_{yy} = \sqrt{\frac{4.72 \times 10^6}{2 \times 1167}} \]
\[ r_{yy} = 44.97 \text{ mm} \]
\[ r_{min} = 30.9 \text{ mm} \]

(i) One Bolt: [Bolt ends hinged]
\[ K = 1 \]
\[ \frac{KL}{r} = \frac{1 \times 3000}{30.9} \]
\[ \frac{KL}{r} = 97.09 \]

\[ f_{cd} = 111.07 \text{ N/mm}^2 \]

Design strength of section = \( f_{cd} \times A \)
\[ = 111.07 \times 2 \times 1167 \]
\[ = 259.24 \text{ KN} \]

(ii) Two Bolts:
\[ K = 0.85 \]
\[ \frac{KL}{r} = \frac{0.85 \times 3000}{30.9} \]
\[ \frac{KL}{r} = 82.52 \]

\[ f_{cd} = 132.22 \text{ N/mm}^2 \]

Design strength of section = \( f_{cd} \times A \)
\[ = 132.22 \times 2 \times 1167 \]
\[ = 308.6 \text{ KN} \]

(iii) A rigid joint by welding:
\[ K = 0.7 \]
\[ \frac{KL}{r} = \frac{0.7 \times 3000}{30.9} \]
\[ \frac{KL}{r} = 67.96 \]

\[ f_{cd} = 155.26 \text{ N/mm}^2 \]

Design strength of section = \( f_{cd} \times A \)
\[ = 155.26 \times 2 \times 1167 \]
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 \( \frac{KL}{r_{\text{min}}} \) ratio and the buckling class.

Properties of ISMB 400:-
- \( h = 400\text{mm} \)
- \( b_f = 140\text{mm} \)
- \( t_f = 16\text{mm} \)
- \( t_w = 8.9\text{mm} \)
- \( r_{yy} = 28.2\text{mm} \)
- \( r_{xx} = 161.5\text{mm} \)
- \( I_{zz} = 20458.4 \times 10^4 \text{mm}^4 \)
- \( I_{yy} = 622.1 \times 10^4 \text{mm}^4 \)
- \( r_{\text{min}} \) is least of \( r_{zz} \) (or) \( r_{yy} \)

where, \( r = \sqrt{\frac{I}{A}} \)

\( I_{zz} \) of Composite section:

\[
I_{zz} = 20458.4 \times 10^4 + \frac{300 \times 20^3}{12} + \left[ 300 \times 20 \times (430 - 220)^2 \right] + \frac{300 \times 20^3}{12} + \left[ 300 \times 20 \times (220 - 10)^2 \right]
\]

\[ I_{zz} = 734.18 \times 10^6 \text{mm}^4 \]

\( I_{yy} \) of Composite section:

\[
I_{yy} = 622.1 \times 10^4 + \frac{20 \times 300^3}{12} + \left[ 20 \times 300 \times (150 - 150)^2 \right] + \frac{20 \times 300^3}{12} + \left[ 20 \times 300 \times (150 - 150)^2 \right]
\]

\[ I_{yy} = 96.221 \times 10^6 \text{mm}^4 \]

\[ \therefore r_{zz} = \sqrt{\frac{I_{zz}}{A}} \]

A = 7846 + (2 \times 300 \times 20)
A = 19846 \text{mm}^2

\[ 734.18 \times 10^6 \]
\[ 94.152 \times 10^6 \]
\[ 19846 \]

\[ \therefore r_{zz} = \sqrt{\frac{6}{G}} \]
\[ r_{zz} = 192.34\text{mm} \]

\[ r_{yy} = \sqrt{\frac{I_{yy}}{A}} = \sqrt{\frac{96.221 \times 10^6}{19846}} \]

\[ r_{yy} = 69.63\text{mm} \]

\[ \therefore r_{\text{min}} = 69.63\text{mm} \]

To find slenderness ratio:

\[ \frac{K L}{r_{\text{min}}} = \frac{0.8 \times 4500}{69.63} = 51.7 \]

The buckling class of the built up section based on table-10 IS 800-2007. Tks of flange is 16+20 = 36mm < 40mm

\[ \therefore \text{Along buckling about } z-z \text{ axis is buckling class ‘B’ and buckling about } y-y \text{ axis, therefore } I_{yy} \text{ is less than } I_{zz} \]

\[ \begin{array}{|c|c|} 
\hline
50 & 183 \\
60 & 168 \\
\hline
\end{array} \]

From table 9 ( C ) IS 800-2007

\[ f_{cd} = 180.45 \text{ N/mm}^2 \]

\[ \therefore \text{Load carrying capacity of the section} = 180.45 \times 19846 \]

\[ = 3581.2 \text{ KN} \]

Safe working load = \frac{3581.2}{1.5} = 2387.5 \text{ 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:

\[ L = 4\text{m} \]

Factor Load = 6000KN

One end fixed and other end hinged

\[ \therefore K = 0.8 \]

Sln:-

The given section ISHB is checked for the axial load carrying capacity

\[ \therefore P_d = A \times f_{cd} \]

Properties of ISHB450 @ 907 N/m:-

\[ A = 11789\text{mm}^2 \]

\[ I_{xx} = 40349.9 \times 10^4\text{mm}^4, I_{yy} = 3045 \times 10^4\text{mm}^4 \]

Assuming \( f_{cd} = 200\text{N/mm}^2 \)

\[ \therefore A_{reqd} = \frac{6000 \times 10^3}{200} = 30000\text{mm}^2 \]
Area deficit = 30000-11789 = 18211mm²

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

2(20xb) = 18211

b = 455.275mm & 500mm

∴ Assume the size of plate @ as 500x20mm @ top and bottom.

$I_{zz}$ of composite section:

$I_{zz} = 40349.9 \times 10^4 + \frac{500 \times 20^3}{12} + \frac{500 \times 20 \times 480 - 245^2}{12} + \frac{500 \times 20 \times 480 - 245^2}{12} + \frac{500 \times 20 \times 245 - 10^2}{12} + \frac{500 \times 20 \times 245 - 10^2}{12} + \frac{500 \times 20 \times 245 - 10^2}{12} + \frac{500 \times 20 \times 245 - 10^2}{12}$

$I_{zz} = 1508.66 \times 10^6 mm^4$

$I_{yy}$ of composite section:

$I_{yy} = 3045 \times 10^4 + \frac{20 \times 500^3}{12} + \frac{20 \times 500 \times 250 - 250^2}{12} + \frac{20 \times 500^3}{12} + \frac{20 \times 500 \times 250 - 250^2}{12} + \frac{20 \times 500 \times 250 - 250^2}{12} + \frac{20 \times 500 \times 250 - 250^2}{12} + \frac{20 \times 500 \times 250 - 250^2}{12}$

$I_{yy} = 447.12 \times 10^6 mm^4$

Check for over hang:

The over hang length is limited to ‘lbt’ over hang length = 500-250

= 250mm < 16(20) = 320mm

∴ $r_{zz} = \sqrt{\frac{I_{zz}}{A}}$

A = 11789 + (2x500x20)

A = 31789mm²

$r_{zz} = \sqrt{\frac{1508.66 \times 10^6}{31789}}$

$r_{zz} = 217.85 mm$

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

$= \sqrt{\frac{447.12 \times 10^6}{31789}}$

$r_{yy} = 118.60 mm$

∴ $r_{min} = 118.60 mm$

To find slenderness ratio:

$\frac{KL}{r_{min}} = 0.8 \times 4000$

$\frac{KL}{r_{min}} = 26.98$

\[
\begin{array}{|c|c|}
\hline
20 & 224 \\
30 & 211 \\
\hline
\end{array}
\]

From table 9 © IS800-2007

$f_{cd} = 214.926 N/mm²$
\[ \text{Design load carrying capacity of the section} = f_{cd} \times A \\
= 214.926 \times 31789 \\
P_d = 6832.3 \text{ KN} > 6000\text{KN} \]

Hence the assume section is safe.

Laced & Battened Columns:-
[cls 7.6 IS 800-2007] [cls 7.7 IS 800-2007]

- 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,
\[ r_{yy} = \text{Radius of gyration about the weaker axis}. \]
\[ r_{zz} = \text{Radius of gyration of stronger axis} \text{ major axis} \text{ of the individual members}. \]

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, \( V_t = 2.5\% \text{ P} \) [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. [ ∴ \( \lambda_{\text{max}} = 145 \)]

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 \( V_t = 2.5\% \text{ P} \)
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,
\[ C = c/c \text{ distance along} \]
longitudinal direction.
\[ N = \text{No. of batten plates}. \]

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]

Given: -

\[ P = 1400\text{KN}, \quad L = 10\text{m}, \quad K = 1 \]

Condition: Both ends are hinged.

Sln: -

Assume \( f_{cd} \) as 135N/mm\(^2\)

To find \( A_{reqd} \):

\[ A_{reqd} = \frac{P}{f_{cd}} \]
\[ \frac{1400 \times 10^3}{135} = 10370.37 \text{mm}^2 \]

\[ A_{\text{reqd}} = 10370.37 \text{mm}^2 \]

\[ \therefore \text{Area of each channel reqd} = \frac{10370.37}{2} = 5185.2 \text{mm}^2 \]

Try 2-ISMC 350 @ 421 N/m

\[ A = 5366 \text{mm}^2 \]

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

\[ r_{zz} = 136.6 \text{mm}; \ r_{yy} = 28.3 \text{mm}; \ c_{yy} = 24.4 \text{mm} \]

The lacing system is provided such that \( r_{yy} \geq r_{zz} \). This is achieved by providing sufficient spacing b/w the two channels.

\[ \therefore r_{\text{min}} = r_{zz} \]

\[ r_{zz} \text{ of combined section} = r_{zz} \text{ of individual channel section.} \]

\[ \therefore r_{zz} \text{ of combined section} = 136.6 \text{mm} \]

Slenderness ratio:

\[ KL_{r_{\text{min}}} = \frac{1 \times 10000}{136.6} \]

\[ KL_{r_{\text{min}}} = 73.206 \]

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

\[ \therefore \frac{KL}{r_{\text{min}}} = 73.206 \times 1.05 \]

\[ = 76.86 \]

From table 9 © IS800-2007

| 70 | 152 |
| 80 | 136 |

From table 10 the built-up section comes under the buckling class ‘C’

\[ f_{cd} = 141.024 \text{N/mm}^2 \]

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

\[ = 141.024 \times 5366 \times 2 \]

\[ P_d = 1513.46 \text{ KN} > 1400 \text{KN} \]

\[ \therefore \text{Assumed section 2ISMC 350 is sufficient.} \]

Design of Lateral system:- [Lacing System]

The clear distance b/w the two channels is arrived based on the condition \( r_{yy} > r_{zz} \)

\[ I_{yy} = I_{zz} \]

\[ I_{zz} \text{ of composite section is twice the } I_{zz} \text{ of an individual channel section.} \]

\[ I_{zz}\text{[comp]} = 2I_{zz}\text{[individual]} \]

\[ = 2 \times 10008 \times 10^4 \]

\[ I_{zz} = 2.0016 \times 10^8 \text{mm}^4 \]

\[ I_{yy} \]

\[ I_{yy} \text{ of composite section is found for the 2 channels from the centroidal Axis} \]
\[ I_{yy/\text{comp}} = 2 \left[ I_{yy/\text{self}} + Ah^2 \right] \]

(one channel)

\[ = 2 \left[ 430.6 \times 10^4 + 5366 \times \left( \frac{d}{2} + 24.4 \right)^2 \right] \]

\[ I_{yy} = I_{zz} \]

\[ 2 \left[ 430.6 \times 10^4 + 5366 \left( \frac{d}{2} + 24.4 \right)^2 \right] = 2.0016 \times 10^8 \]

\[ 2 \left[ 430.6 \times 10^4 + 5366 \left( \frac{d^2}{4} + 595.36 + 24.4d \right) \right] = 2.0016 \times 10^8 \]

\[ 2 \left[ 430.6 \times 10^4 + 1341.5d^2 + 3194701.76 + 130930.4d \right] = 2.0016 \times 10^8 \]

\[ 2 \left[ 1341.5d^2 + 130.93 \times 10^3 d + 750.07 \times 10^4 \right] = 2.0016 \times 10^8 \]

\[ 1341.5d^2 + 130.93 \times 10^3 d + 750.07 \times 10^4 = 100.08 \times 10^6 \]

\[ d = 218 \]

\[ \therefore d = 220 \text{mm} \]

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

Horizontal Spacing = \( d + 24.4 + 24.4 \)

= 220 + 48.8

= 268.8 mm

Hori spacing = 268.8 mm

Vertical spacing = 2 [horizontal spacing]

= 2 \times 268.8

= 537.6 mm

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

\[ \therefore \text{Slenderness ratio for vertical spacing} = \frac{KL}{r} \]

\[ = \frac{1 \times 537.6}{28.3} \]

\[ = 18.99 < 50 \]

Transverse shear to be resisted by each lacing system is 2.5% of axial load. [Clause 7.6.6.1 IS 800-2007]

Load = \[ \frac{2.5 \times 1400}{100} \]

= 35 KN

\[ \therefore \text{Transverse shear to be resisted by each lacing bar is 17.5 KN} \]

\[ L = \frac{268.8}{\cos 45^\circ} \]

L = 380.14 mm

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]
   \[ V_{dsp} = \frac{V_{nsp}}{Y_{mb}} \]
   \[ V_{nsp} = \frac{f_u}{\sqrt{3}} [n_n A_{nb} + n_s A_{sb}] \]
   \[ n_n = 1, \quad n_s = 0 \]
   \[ = \frac{400}{\sqrt{3}} [1 \times 0.78 \times \pi \times 20^2] \]
   \[ V_{nsp} = 56.59 \text{ KN} \]
   \[ V_{dsp} = 45.272 \text{ KN} \]

2. Strength of bolt in bearing:- [cls 10.3.4 IS 800-2007]
   \[ V_{dbp} = \frac{V_{nbp}}{Y_{mb}} = \frac{2.5k_b d^3 f_u}{Y_{mb}} \]
   \[ k_b = \frac{e}{3d_o}, \quad p = 0.25, \quad \frac{f_{ub}}{f_u}, 1 \]
   \[ e = 1.5d_o = 33 \text{ mm} \approx 40 \text{ mm} \]
   \[ p = 25, \quad d = 50 \text{ mm} \approx 60 \text{ mm} \]
   \[ k_b = 0.606, 0.659, 0.975, 1 \]
∴ \[ k_b = 0.606 \]
   \[ = \frac{2.5 \times 0.606 \times 20 \times 10 \times 410}{1.25} \]
   \[ V_{dbp} = 99.384 \text{ KN} \]
∴ The strength of bolt value = 45.272KN
∴ No. of bolts = \[ \frac{17.5}{45.27} = 0.39 \quad N_o \]
∴ Provide one 20mm \( \phi \) bolt on each side of connection.

Strength of lacing bar:- [60x10mm]

\[ \text{Slenderness ratio of lacing bar} = \frac{KL}{r} \]
\[ = \frac{1 \times 380.14}{Y_{min}} \]
\[ r_{min} = \sqrt{\frac{I_{zz}}{A}} \quad (or) \quad \sqrt{\frac{I_{yy}}{A}} \]
\[ I_{zz} = \frac{60 \times 10^3}{12} = 5000 \text{ mm}^4 \]
\[ I_{yy} = \frac{60 \times 10}{12} = 180 \times 10^3 \text{ mm}^4 \]
\[ r_{\text{min}} = \frac{5000}{600} \]
\[ = 2.88 \]
\[ = \frac{1 \times 380.14}{2.88} \]

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

<table>
<thead>
<tr>
<th>130</th>
<th>74.3</th>
</tr>
</thead>
<tbody>
<tr>
<td>140</td>
<td>66.2</td>
</tr>
</tbody>
</table>

From table 9© IS 800-2007

\[ f_{cd} = 72.68 \text{ N/mm}^2 \]

Load Carrying capacity of section = 72.68 x 60 x 10

\[ p_d = 43.61 \text{ KN} > 17.5 \text{ KN} \]

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.

[ \because \text{ The section is design as per the previous problem}]

Sln:-
Slenderness ratio of battens is 1.1 times

\[
\frac{KL}{r} = 73.21 \\
\frac{KL}{r} = 73.21 \times 1.1 \\
= 80.53
\]

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

\( S = 268.8 \text{mm} \)

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

\[ C/r_{\text{min}} < 50 \]

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

\( C = 28.3 \times 50 \)

\( C = 141.5 \text{mm} \)

∴ Assume \( C = 1200 \text{mm} \)

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

\( V_t = 35 \text{KN} \)

As per clause 7.7.2.1 IS 800-2007, shear to be resisted by Batten plate
\[ V_b = \frac{V_t C}{NL} \]
\[ M = \frac{V_t C}{2N} \]

Where,

\[ N = 2 \]

\[ V_b = \frac{35 \times 1200}{2 \times 268.8} \]

\[ V_b = 78.125 \text{KN} \]

\[ M = \frac{35 \times 1200}{2 \times 2} \]

\[ = 105.00 \text{KN mm} \]

\[ M = 10.5 \text{KN.m} \]

**Width & tks of Batten Plate:**

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

\[ t = \frac{268.8}{50} \]

\[ t = 5.376 \text{mm} \]

\[ \therefore \text{Provide the tks of 6mm} \]

**Check for stresses in Batten Plate:**

Shear stress \[ = \frac{V_b}{A} \]

\[ = \frac{78.125 \times 10^3}{210 \times 6} \]

\[ = 62 \text{ N/mm}^2 \]

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

\[ = 131.21 \text{ N/mm}^2 \]

Shear Act stress < Permissible shear stress

Actual bending stress \[ \sigma_b = \frac{M}{Z} \]

\[ = \frac{10.5 \times 10^6 \times 6}{td^2} \]

\[ = \frac{10.5 \times 10^6 \times 6}{6 \times 210^2} \]
Permissible bending stress = \( \frac{f_y}{1.1} \) = 227.27 N/mm\(^2\)

Actual 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 = \( \frac{10.5 \times 10^6 \times 6}{280^2 \times 6} \) = 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 \( \approx \) 3 Nos.

Since moment also acts on the connection provide 3 Nos of bolts.

The force due to moment on extreme.

Bolt \( F_m = \frac{M_y}{\sum Y^2} \) = \( \frac{10.5 \times 10^6 \times 10^5}{105^2 + 105^2} \)

\( F_m = 50KN \)

\( F_s = \frac{F}{n} = \frac{78.125}{3} \)

\( = 26.04 KN \)

Resultant force = \( \sqrt{F_m^2 + F_s^2} \) = \( \sqrt{50^2 + 26.04^2} \) = 56.57 KN > 45.27 KN

\( \therefore \) 3 bolts are not sufficient we have to increase the no. of bolts.

Assume 5 Nos of bolt along the vertical line.

Force due to moment \( F_m = \frac{M_y}{\sum Y^2} \)
\[ F_s = \frac{10.5 \times 10^6 \times 10^5}{2|105^2| + 2|105^2|} = 25\text{KN} \]

\[ F_s = \frac{78.125}{5} = 15.62\text{KN} \]

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:-

- 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
(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:-
- Gusseted 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 $\frac{P_u}{0.4f_{ck}}$

   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

   $$ w = \frac{P_u}{\text{Area of base plate}} $$

4. Min thickness of base plate reqd. is

   $$ t_s = \left[ \frac{2.5w(a^2 - 0.3b^2)y_{mo}}{f_y} \right]^{0.5} > t_f $$

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  $\sigma_{bc} = 0.45f_{ck}$
       
       $= 0.45 \times 20$
\[
\sigma_{bc} = 9 \text{ N/mm}^2
\]

2. Area of base plate reqd,
\[
A = \frac{P_u}{0.45f_{ck}} = \frac{1000 \times 10^3}{9}
\]
\[A = 111.11 \times 10^3 \text{mm}^2\]

Assuming equal projection on both sides with \(a = b = 30\text{mm}\), size of base plate assumed is 310x360mm.

\[\therefore \text{Area Provided} = 111.6 \times 10^3 \text{mm}^2\]

3. Pressure intensity @ base
\[
w = \frac{P_u}{\text{Actual area}} = \frac{1000 \times 10^3}{111.6 \times 10^3}
\]
\[w = 8.96 \text{ N/mm}^2 < 9 \text{N/mm}^2\]

4. Mini. Tks of base plate
\[
t = \left[ 2.5w \left( a^2 - 0.3b^2 \right) y m_o \right]^{0.5}
\]
\[= \left[ 2.5 \times 8.96 \left( 30^2 - 0.3 \times 30^2 \right) \times 1.1 \right]^{0.5}
\]
\[t_s = 7.87 \text{ mm} < 10.6 \text{ mm} (t_f)\]

\[\therefore \text{Provide tks of base plate as 12mm}\]

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
\[\sigma_{bc} = 0.45f_{ck}\]
\[= 0.45 \times 20\]
\[\sigma_{bc} = 9 \text{ N/mm}^2\]

2. Area of base plate reqd,
\[
A = \frac{P_u}{0.45f_{ck}} = \frac{1000 \times 10^3}{9}
\]
\[A = 111.11 \times 10^3 \text{mm}^2\]

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

\[\therefore \text{Area Provided} = 111.6 \times 10^3 \text{mm}^2\]

3. Pressure intensity @ base
\[
w = \frac{P_u}{\text{Actual area}} = \frac{1000 \times 10^3}{111.6 \times 10^3}
\]
4. Mini. Tks of base plate

\[ w = 8.96 \text{ N/mm}^2 < 9 \text{N/mm}^2 \]

\[ Tks = \left[ \frac{2.5w(a^2-0.3b^2)y_m \gamma_m}{f_y} \right]^{0.5} \]

\[ = \frac{2.5 \times 8.96(30^2-0.3 \times 30^2) \times 1.1}{250} \]

Tks of flange = 10.6mm

\[ \therefore \text{ Provide tks of base plate as 12mm.} \]

5. Welded Connection:-

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

\[ \text{Length available} = 4(250)-2(7.6)+2(300)-2(10) \]

\[ 1w = 1563.6 \text{mm} \]

Design strength of weld:

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

\[ \text{Stress x Area} \]

\[ \downarrow \quad \downarrow \]

\[ \frac{f_u}{\gamma m_w} \]

\[ = \frac{\sqrt{3}}{1.25} \times |l w \times t| \]

Design strength of weld =

\[ = \frac{410}{1.25} \times (1563.6 \times 0.7) \]

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

\[ 1000 \times 10^3 = 189.37 \times 0.73 \times 1563.6 \]

\[ S = \frac{1000 \times 10^3}{189.37 \times 0.7 \times 1563.6} \]

\[ S = 4.82 \text{mm} \]

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

Gusseted Base:-

\[ \checkmark \quad \text{When the load on the column is higher gusset plates are provided} \]

\[ \checkmark \quad \text{along the flanges of the C/S.} \]

\[ \checkmark \quad \text{The load is transferred by bearing through the base plate and also partly through the gusset plate.} \]

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 accommodate 2 bolts in one vertical line. The other leg is assume such that 1 bolt can be provided.
The tks of angle is kept approximately equal to the tks of gusset plate.
3. Width of gusseted base is kept such that it will just project the outside the gusset angle and hence
   \[ \text{Length} = \frac{\text{Area of plate}}{\text{width}} \]
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.

ASSIGNMENT-I

1. Draw the various possible forces in bolted connections:-
   a) Shear Plane on thread:-
   b) Two planes subject to shear:-
c) Bolts in Direct Tension: -  
d) Bolts resisting pure moment: -  
e) Bolts subject to shear and tension: -  

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 = \[ 2 \times \left( \frac{0.462 f_u n_A n_b}{1000} \right) \]  
\[ \begin{align*}  
&= 0.462 \times 400 \times 245 \\
&= 90.55KN  
\end{align*} \]  

Strength of bolts in bearing = \[ 2dtpf_u \]  
\[ \begin{align*}  
&= 2 \times 20 \times 9.2 \times 410 \\
&= 147KN  
\end{align*} \]  

Least bolt value = 90.55KN  
No. of bolts = \[ \frac{\text{Reaction @ each end}}{\text{bolt value}} \]  
\[ \begin{align*}  
&= \frac{300}{2} \\
&= 150  
\end{align*} \]  

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 = \[ 0.462 f_u n_A n_b \]  
\[ \begin{align*}  
&= 0.462 \times 400 \times 245 \\
&= 45.3KN  
\end{align*} \]  

Strength of bolts in bearing = \[ 2dtpf_u \]  
\[ \begin{align*}  
&= 2 \times 20 \times 8 \times 410 \\
&= 31KN  
\end{align*} \]  

Least bolt value = 45.3KN  
No. of bolts = \[ \frac{300}{2} \]  
\[ \frac{45.3}{2} \]
= 3.31 says 4Nos.
Provide 2 bolts on each side of flange check the tks of the angle:-

Factored shear resistance = \[
\frac{A_v f_{yw}}{\sqrt{3Y_{mo}}}
\]

= 0.525 \cdot f_{yw}
= 0.525 \times 2 \times 100 \times 8 \times 250
= 209.9\text{KN} > 150\text{KN}

Hence safe

ASSIGNMENT-II
1. Design a tension member to carry a factored load of 340\text{KN} use 20mm dia of black bolt and gusset plate of 8mm thickness.

Given:-
Factored load = 340\text{KN}
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} = 1496\text{mm}^2

Try ISA 100x100x8mm
A_{g} = 1539\text{mm}^2
\]
y_{xx} = y_{yy} = 30.7 \text{mm}

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

\[
V_{dsb} = \frac{V_{nsb}}{Y_{mb}}
\]

= \frac{f_u}{\sqrt{3}} \left[ \frac{n_n A_{nb} + n_s A_{sb}}{Y_{mb}} \right]
\]
n_n = 1, n_s = 0
\]

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

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

V_{dsb} = 45.27\text{KN}

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

\[
V_{dbp} = \frac{V_{nbp}}{Y_{mb}}
\]
\[ \frac{2.5k_b d_f f_u}{\gamma_{mb}} \]

Assume,
- \( e = 1.5d_o = 33\text{mm} \)
- \( d = 40\text{mm} \)
- \( p = 2.5d = 50\text{mm} \)
- \( d = 60\text{mm} \)

\[
k_b = \frac{40}{3\times22} = \frac{60}{3\times22} = -0.25, \quad \frac{400}{410}, 1
\]
\[= 0.606, 0.66, 0.959, 1\]

Take \( k_b = 0.606 \)

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

\( V_{dhp} = 79.5\text{KN} \)

∴ Design strength of bolt value = 45.27KN

∴ No. of bolts = \( \frac{340}{45.27} \)

= 7.51 \( \approx \) 8 Nos

∴ Provide 20mm \( \varphi \) bolt of 8 Nos.

Check for strength of section:-

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

\[
T_{dg} = \frac{A_g f_y}{\gamma_{mo}}
\]

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

\( T_{dg} = 349.04\text{KN} \)

(ii) Design strength of section against rupture:- [cls 6.3.3 IS 800-2007]

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

Where,

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

\[
w = 100
\]

\[
w = 50
\]

\[
b_s = 100 + 50 - 8
\]

\[
b_s = 142\text{mm}
\]

\[
l_s = 460\text{mm}
\]

\[
\beta = 1.4 - 0.076 \left( \frac{100}{8} \right) \left( \frac{250}{410} \right) \left( \frac{142}{460} \right) \leq 250 \times 1.1 \geq 0.7
\]

\[= 1.22 \approx 1.44 \approx 0.7
\]

∴ \( \beta = 1.22 \)

\[
A_{nc} = \left(100 - 22 - \frac{8}{2}\right) \approx 592\text{mm}^2
\]
\[ A_{go} = \left(100 - \frac{8}{2}\right) = 768 \text{ mm}^2 \]
\[ = \frac{0.9 \times 592 \times 410}{1.25} + 1.22 \times 768 \times 250 \]
\[ T_{dn} = 387.7 \text{KN} > 340 \text{KN} \]

3) Design strength of plate against block shear: - [cls 6.3.4 IS800-2007]
\[ T_{db} = \frac{A_{vg} f_u}{\sqrt{3 m_o Y_{ml}}} \]
\[ (or) \]
\[ T_{db} = \frac{0.9 A_{vn} f_u + A_{tg} f_y}{\sqrt{3 Y_{ml}}} \]

Where,
\[ A_{vg} = [40 + 7(60)]^8 = 3680 \text{mm}^2 \]
\[ A_{vn} = [460 - 7.5(22)]^8 = 2360 \text{mm}^2 \]
\[ A_{tg} = 50 \times 8 = 400 \text{ mm}^2 \]
\[ A_{tn} = \left(50 - \frac{22}{2}\right)^8 = 312 \text{mm}^2 \]
\[ T_{db1} = \frac{3680 \times 250}{\sqrt{3 \times 1.1 \times 1.25}} = 574.97 \text{KN} \]
\[ T_{db2} = \frac{0.9 \times 2360 \times 410 + 400 \times 250}{\sqrt{3 \times 1.25 \times 1.1}} = 493.13 \text{KN} > 340 \text{KN} \]

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.

\[ T_{dg} = \frac{f_y A_g}{Y_{mo}} \]

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.

\[ T_{dn} = \frac{0.9 f_y A_n}{Y_{ml}} \]
(iii) Block shear failure:-
Originally observed is bolted shear connection at sloped beams ends.
Block shear is now reqd as potential failure of made the ends of axially load tension member also.
In this failure made the failure of member occurs along a parts including fussion on one plates & shear on \( \dot{i} \) lr plane along the fasterners as shown in fig.

\[
T_{db1} = \frac{A_{vg} f_y}{\sqrt{3} Y_{mo}} + \frac{0.9f_y A_{vn}}{Y_{ml}}
\]

(or)

\[
T_{db2} = \frac{0.9f_y A_{vn}}{\sqrt{3} Y_{ml}} + \frac{f_y A_{tg}}{Y_{mo}}
\]

Where,
\( A_{vg}, A_{vn} \) = min gross area & net area in shear along section (1-2) & (4-3)
\( A_{tg}, A_{tn} \) = min gross area & net area from hole to toe of the angle section (2-3)

Working stress method of steel design:-
Permissible stresses:-
1. Axial tension, \( \sigma_{at} = 0.6f_y \)
2. Axial compression, \( \sigma_{ac} \leq 0.6f_y \) [depends upon L/R ratio]
3. Bending compression, \( \sigma_{bc} = 0.66f_y \)
4. Per shear stress, \( \tau_c = 0.45f_y \) [generally taken as 0.4f_y]

ASSIGNMENT-III
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 \( Y_{min} \) (least of \( Y_{xx} \cdot Y_{yy} \) )
   - 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\(^2\)

\[
A_g = \frac{170 \times 10^3}{2 \times 90} \\
A_g = 944.4mm^2
\]

For safe design increase 30%

\[
A_g = 944.4 \times 1.3 \\
A_g = 1227.2 \text{ mm}^2
\]

∴ Try 2ISA 90x90x8mm

\[
I_{xx} = I_{yy} = 104.2 \times 10^4 \text{ mm}^4 \\
\gamma_{xx} = \gamma_{yy} = 25.1 \text{ mm} \\
\gamma_{uu} = 34.7 \text{ mm} \\
\gamma_{vv} = 17.5 \text{ mm} \\
A^* = 2 \times 1379 = 2758 \text{ mm}^2 \\
I_y = 2 \left[ 104.2 \times 10^4 + 1379(25.1 + 10/2) \right] \\
\gamma_{uu} = \frac{4.583 \times 10^6}{2758} \\
\gamma_{vv} = 40.76 \text{ mm} \\
K = 1 \\
\therefore \frac{KL}{\gamma_{min}} = \frac{1 \times 3850}{27.5} \\
\frac{KL}{\gamma_{min}} = 140
\]

∴ \( f_{cd} = 66.2 \frac{N}{\text{mm}^2} \) [from table 9c]

\[
P_d = f_{cd} \times A \\
= 66.2 \times 2758 \\
= 182.58 \text{ KN} > 170 \text{ KN}
\]

∴ Hence the assumed section is safe.

\[
\delta_{act} < \delta_{allow}
\]

∴ Hence the section is safe in deflection

4. Check for web buckling:

\[
F_{cdw} = \left[ b_1 + n_1 \right] t_w f_c \\
n_1 = \frac{h}{2} = \frac{600}{2} = 300 \text{ mm} \\
b_1 = 100 \text{ mm} \\
\lambda_w = \frac{2.5d}{tw}
\]
\[
\lambda_w = \frac{2.5 \times 523.4}{11.2} = 116.83
\]

<table>
<thead>
<tr>
<th>(\text{Grade})</th>
<th>(f_{yw})</th>
</tr>
</thead>
<tbody>
<tr>
<td>110</td>
<td>94.6</td>
</tr>
<tr>
<td>120</td>
<td>83.7</td>
</tr>
</tbody>
</table>

[from table 9 © IS800-2007]

\[f_c = 87.155 \text{ N/mm}^2\]

\[hdw = (100 + 300) \times 11.2 \times 87.155\]

\[hdw = 390.454 \text{ KN} > 225 \text{ KN}\]

Hence the section is safe against web buckling.

5. Check for web crippling:

\[F_w = \frac{(b_1 + n_2)t_wf_{yw}}{Y_{mo}}\]

\[n_2 = 2.5(t_f + t_1) = 2.5(21.3 + 17) = 82.5 \text{ mm}\]

\[F_w = \frac{(100 + 82.5)11.2 \times 250}{1.1}\]

\[F_w = 464.545 \text{ KN} > 225 \text{ KN}\]

Hence the section is safe against web crippling.

3. An ISMB section of depth 500mm is used as a beam over 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:

- \(A = 11074 \text{ mm}^2\), \(b_f = 180 \text{ mm}\), \(t_f = 17.2 \text{ mm}\), \(t_w = 10.2 \text{ mm}\), \(I_{zz} = 45218.3 \times 10^3 \text{ mm}^4\), \(I_{yy} = 1369.8 \times 10^3 \text{ mm}^4\), \(r_1 = 17 \text{ mm}\), \(z_{yw} = 2074.67 \times 10^3 \text{ mm}^3\), \(z_{ez} = 1808.7 \times 10^3 \text{ mm}^3\), \(r_{yy} = 35.2 \text{ mm}\), \(r_{zz} = 202.1 \text{ mm}\)

\[d = h - 2(t_f + r_1) = 500 - 2(17.2 + 17) = 431.6 \text{ mm}\]

To find maxi. Moment & S.F:-

The maxi. Moment of the beam \(M = \frac{wl^2}{8}\)

The design moment capacity of the section \(M_d = \beta_d Z_p f_{bd}\)

Where,

\(f_{bd} => \text{ Taken from table-13 for } f_{cr}, b \text{ given in table-14 [cls 8.2.2 IS800-2007]}\)

\(f_{cr}, b => \text{ [Table-14 IS 800-2007]}\)
The critical stress \( f_{cr} \), \( b \) is found based on slenderness ratio \( \frac{KL}{r} \) and \( \frac{h}{t_f} \) ratio.

Here, \( K=1 \)

\[
\therefore \quad \frac{KL}{r} \geq \frac{1 \times 6000}{35.2} = 170.45
\]

\[
\frac{h}{t_f} \geq \frac{500}{17.2} = 29.07
\]

<table>
<thead>
<tr>
<th>( \frac{KL}{r} )</th>
<th>( \frac{h}{t_f} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>25</td>
<td>29.07</td>
</tr>
<tr>
<td>170</td>
<td>136.7</td>
</tr>
<tr>
<td>170.45</td>
<td>136.26</td>
</tr>
<tr>
<td>180</td>
<td>127.1</td>
</tr>
</tbody>
</table>

From table-14 [IS 800-2007]

\[
\therefore \quad \text{Critical stress } f_{cr}, b = 123.75 \text{N/mm}^2
\]

\( f_{bd} \):

Refer [table-13 IS800-2007] \( \alpha_{LT} = 0.21 \) (for R.S section)

\[
\therefore \quad \text{Refer table – 13(a) IS 800-2007}
\]

| 150 | 106.8 |
| 100 | 77.3 |

\[
\therefore \quad f_{bd} = 91.31 \text{N/mm}^2 \text{ (for } f_{cr}, b = 123.75 \text{N/mm}^2) \]

\[
\therefore \quad \text{The design bending strength of ISMB } M_d = \beta_b Z_p f_{bd}
\]

Buckling Class:- [Table-2 IS 800-2007]

\[
\frac{b}{t_f} = \frac{0.5b}{t_f} = \frac{90}{17.2} = 5.2 \sum 9.4 \sum \iota
\]

\[
\frac{d}{t_w} = \frac{431.6}{10.2} = 42.31 \sum 84 \sum \iota
\]

\[
\therefore \quad \text{The section comes under plastic}
\]

\[
\therefore \quad \beta_b = 1 \quad [\text{cls 8.2.2 IS 800-2007}]
\]

\[
M_d = 1 \times 2074.67 \times 10^3 \times 91.31
\]

\[
M_d = 189.44 \text{KN.m}
\]

\[
M = \frac{wl^2}{8}
\]

\[
M_d = 189.44 \text{KN.m}
\]

The safe udl the section can carry is found by equating \( M \) & \( M_d \)

\[
M = M_d
\]
\[189.44 \times 10^6 = \frac{w \times 6000^2}{8}\]
\[w = 42.09 \text{KN/m}\]