CE8602 - Design of Steel Structural Elements

Unit - I - Introduction

Introduction:-

Design of building $\xrightarrow{\text{consists of two parts}}$ Functional Design (Planning)
in account of ventilation, lighting & aesthetic (appearence) view

Design of building → structural design
loads acting on the structure/building & the loads transferred safely in to the ground

Loads transfered to the ground → various materials like asbestos sheets, tiles bricks, cement concrete, RC, steel, aluminium etc.

Present day → RCC (or) steel.

In tall structure → composite construction of steel & concrete are commonly used.

Common steel structures:- are used for

* Roof trusses $\xrightarrow{\text{such as}}$ factories, cinema halls, auditoriums
* Industrial structures → crane girders, columns
* water tanks
* chimneys
* Transmission towers for microwave & electric power
* plate girder and truss bridges for railways & roads
* Roof trusses & columns to cover platforms in railway stations & bus stands.
* single layer or double layer domes for auditoriums, exhibition halls, indoor stadium etc.,

Advantages of steel structures:-

* High strength → large structures, size of steel structural element is small.
  ∴ construction space is small & improving aesthetic view
* High durability & assured good quality
* steel structure — strengthened at any later time (just additional section is welded)
* Transported to other sites quickly → using bolted connections easily dismantled)

* Material is reusable.

Dis-advantages of steel structure :-

* Maintenance cost high (it needs Painting to prevent corrosion)
* corrosion
* steel members are costly.
* Fire resisting property is very poor.

Properties of Structural Steel :-

clause 2.2.4
IS : 800 – 2007, P. NO: 12

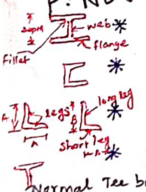
Properties of steel ~~stees~~.
→ Physical properties
- Unit mass of steel $\rho = 7850\ kg/m^3$
- Modulus of Elasticity $E = 2 \times 10^5\ N/mm^2$
- Poission's Ratio $\mu = 0.3$
- Modulus of rigidity $G = 0.769 \times 10^5\ N/mm^2$
- co-efficient of thermal expan $\alpha_t = 12 \times 10^{-6}/^\circ C$

→ Mechanical properties
IS : 800 – 2007, P. NO: 12
* Yield stress f_y
* Tensile or ultimat stress f_u
* Max. % elongation on a standard gauge length
* Notch toughness.

IS : 800 – 2007
P. NO : 13 & 14
Table : 1

Structural steel sections :-

P. NO : 19 IS : 800 – 2007



* Rolled steel I-section (Beam section) → ISJB (Junior), ISLB (Light), ISMB (medium wt), ISWB (wide flanged), ISHB (Heavy)
  (manufactured in India)
* Rolled steel channel sections – ISJC, ISLC, ISMC (medium wt), ISSC (special)
* Rolled steel Angle sections – equal angle ISA, ISA (un-equal Angle)
* Rolled steel Tee sections
  Normal Tee bars ISNT, ISHT (Heavy flanged), ISLT, ISLT (Special legged), ISJT – Junior Tee bars
  Standard Light Tee bar
* Rolled steel Bars
  Indian standard Round bar ISRO, ISSQ – Indian standard square bar
* Rolled steel Tubes → Light, Medium, heavy (40 mm Nominal Bore)
* Rolled steel Plates – ISPL Indian standard steel plates
  Thickness → 5, 6, 8, 10, 12, 14, 16, 18, 20, 22, 25, 28, 32, 36, 40, 45, 50, 56, 63, 71, 80 mm
  width - 160, 180, 200, 220, 250, 280, 320, 355, 400, 450, 500, 560, 630, 710, 800, 900, 1000, 1100, 1250, 1400, 1600, 1800, 2000, 2200, 2500 mm
* Rolled steel Flats
  width : 12, 16, 20, 25, 32, 40, 50, 63, 80, 100, 125, 160, 200, 250
  Thick : 5, 5.5, 6, 7, 8, 9, 10, 11, 12, 14, 16, 18, 20, 22, 25
  ISF → Flat
* Rolled steel sheets & strips
  width : 100, 110, 125, 140, 160, 180, 200, 220, 250, 280, 320, 255, 400, up to 1000
  Thick : 0.8, 0.9, 1, to 1.2, 1.4, 1.6, 1.8, 2, 2.2, 2.5, 2.8, 3.2,

Special considerations in steel Design :-

* Size & shape
* Buckling
* Minimum Thickness → cleaning & Painting — 6 mm
  not accessible for cleaning & Painting — 8mm
* connection Design. → Riveted, bolted & welded connections.

Limit state Design concepts :- IS: 800 - 2007, P. NO: 27

* It is the compressible method which will
* Take care of both strength & serviceability requirements of the structure.
* LSD method is to see that → the structure remains fit for use throughout its designed life by remaining with in the acceptable limit of saftey, strength & serviceability based on the risk involved.

IS: 800-2017
P. NO: 27

* The design is based on probable load and probable strength of materials.

Design Requirements :- IS: 800 - 2007 clause 5.1.2 ; P. NO: 27

* Steel structure designed and constructed should satisfy the requirements regarding
  * Stability
  * Strength
  * Serviceability
  * Requirements of deflection
  * Vibration
  * cracks
  * Due to fatigue
  * Corrosion & fire.
* The structure should meet the following requirements as mentioned in IS: 800 - 2007 clause 5.1.2 to 5.6.4.

P. NO: 27 to 32

Loads on structure :-

IS:800:2007
P.NO:15
clause 3.2.

* For the purpose of designing any element, member or a structure, the following loads (actions) and their effects shall be taken into account, where applicable with partial saftey factors and combinations.

* Dead loads IS:875 (Pt:1)
  Unit wt — Asbestos sheet = 0.13 kN/m^2 ; Brick masonry = 20 kN/m^3, cement plaster = 20 kN/m^3, Granite stone masonry 24 kN/m^3
  PCC : 24 kN/m^3, RCC = 25 kN/m^3 ; IS 875 Pt:2-1987 ; Steel – 78.5 kN/m^3
* Imposed loads → Live load, crane load, snow load, dust load, wave load, earth pressure load, etc — IS:875 (Pt:4), IS:875 (Pt:5)
* Wind loads IS:875 (Pt:3)
* Earthquake load
* Impact load
* Erection loads
* Temperature effects.

Load combination :- IS:800-2007; clause 3.5, P.NO:16

* D.L + I.L
* D.L + I.L + W.L (or) EQ. Load
* D.L + W.L (or) EQ. load
* D.L + erection load

Connections :-

* Riveted connection
* Bolted connection < Lap joint / Butt joint
* Welded connection

Riveted connections :-

* Riveting is a method of joining together pieces of metal by inserting ductile metal pins called rivets into holes of pieces to be connected and forming a heat at the end of the rivet to prevent each metal piece from comming out.

* Rivets of various shapes are made from mild steel.

* The Rivet head is generally round & called → a button head

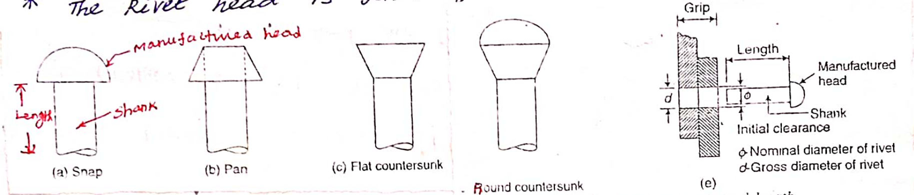


Fig. 4.1 *Rivet types, grip and length*

* The size of rivet is the diameter of the shank

* Rivet holes are → drilled or punched in to the plates to be riveted.

* The size of holes are 1.5mm larger than the size of rivets upto 25mm dia bolt.

* 2mm larger than the size of rivet more than 25mm dia bolt.

Riveting Process :-

* A rivet is heated uniformly to light red colour and then inserted in the hole.

* The head is kept pressed on the plate, while the projected shank is hammered to form another head.

* on cooling the rivet contracts and grips the plate tightly.

* Rivets may be driven by hydraulic or by pneumatic power.

classification based on method of driven :-

* Power-driven shop rivets $\xrightarrow{\text{are}}$ Power driven in the fabrication shop

* power-driven field rivets → power driven at site — their strength less than the shop rivets

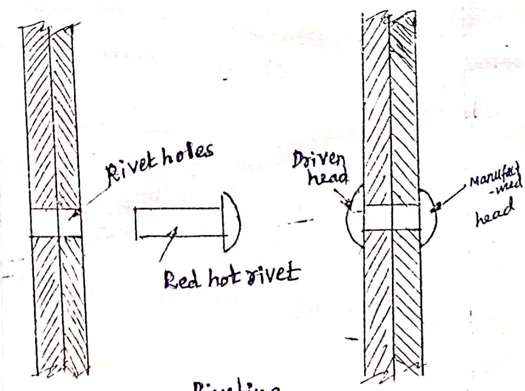
* Hand driven rivets $\xrightarrow{\text{have}}$ less strength than power driven rivets

* cold-driven rivets of diameter more than 10mm are not Permitted by specifications.

Dis-advantages of of Riveting :-

* It is associated with high level of noise pollution
* It needs heating the rivet to red hot
* Inspection of connection is a skilled work.
* Removing Poorly installed rivets is costly.
* Labour cost is high.

* Design procedure for riveted connections is same as that for bolted connections except that the effective diameter of rivets may be taken as rivet hole diameter instead of nominal diameter of rivet.

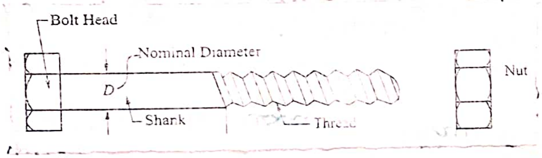


Riveting.

Bolted connections

* A bolt is a metal pin with a head formed at one end and shank threaded at the other in order to receive a nut.

* Bolts are used for joining together pieces of metals by inserting them through holes in the metal and tightening the nut at the threaded ends.



Bolt & nut

Truss, bracing & temporary connection required during erection
Light structure ...

Classifications of Bolts:

IS 1364 (Pt 1) specification.
yield strength = 240 N/mm²
ultimate strength = 400 N/mm²
= IS 3640

* Unfinished (Black) Bolts → made up of mild steel – square (or) hexagonal head, commonly used M16, M20, M24 etc
  nominal dia 12, 16, 20, 22, 24, 27, 30, 36 commonly
* Finished (Turned) Bolts, finished circular shape, Actual dimension 1.2 to 1.3 mm.
  mild steel, by ... hexagon head
* High Strength Friction Grip (HSFG) Bolts
  made from high strength steel rod, Surface – unfinished (Black Bolt)

  IS 3747

  dimensions – 16, 20, 24, 30, 36 mm

Classification of Bolts based on type of load transfer

* Bearing type → unfinished (black) bolt & finished (turned) Bolts
  * Transfer shear force from one member to other member.

* Friction Grip type → HSFG bolt
  * Transfer shear by friction

Dis-advantages of HSFG Bolt

* Material cost high
* Special attention is to be given to workmanship especially to give them right amount of tension.

Advantages of HSFG Bolt over Bearing Type Bolts

* Joints are rigid ie, no slip takes place in the joint
* As load transfer is mainly by friction, the bolts are not subjected to shearing and bearing stresses
* High static strength due to high frictional resistance
* High fatigue strength since nuts are prevented from loosening and stress concentrations avoided due to friction grip.
* smaller number of bolts result into smaller sizes of gusset plates.

Advantages of Bolted connections :-

* Making joints is noiseless
* Do not need skilled labour
* Needs less labour
* connections can be made quickly
* Structure can be put to use immediately
* Accomodates minor discrepancies in dimensions.
* Alternations, if any, can be done easily.
* Working area required in the field is less.

Dis-advantages of Bolted connections :-

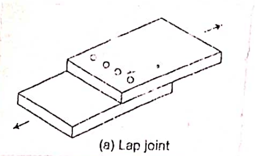
* Tensile strength is reduced considerably due to stress concentrations & reduction of area at the root of the threads.
* Rigidity of joints is reduced due to loose fit, resulting into excessive deflections.
* Due to vibrations nuts are likely to loosen, endangering the saftey of the structures.

Patterns of Riveted / Bolted joints :-

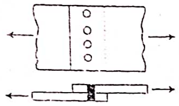
* chain riveting
* staggered riveting
* Diamond riveting
* staggered diamond riveting

Types of bolted connections :-

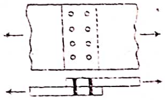
* Lap joints
* Butt joints



(a) Lap joint



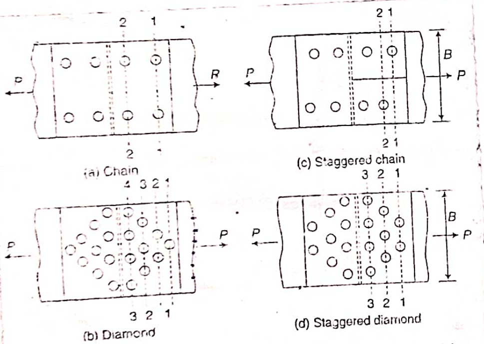
(b) Single bolted lap joint



(c) Double bolted lap joint

* single bolted lap joint
* Double bolted lap joint
* single cover butt joint
* single cover single butt joint
* single cover ~~strag~~ double bolted butt joint
* Double cover single bolted butt joint
* Double cover double bolted butt joint.

(d) Eccentricity in lap joint



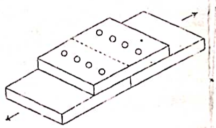
(a) Chain

(c) Staggered chain

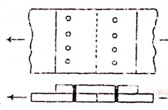
(b) Diamond

(d) Staggered diamond

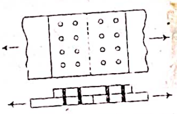
Fig. 4.2 *Rivet patterns, shown with the help of a single cover butt joint*



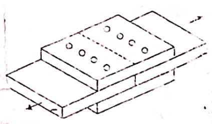
(e) Single-cover butt joint



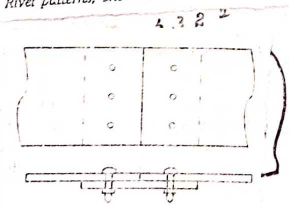
(d) Single-cover single bolted butt joint



(g) Single-cover double bolted butt joint

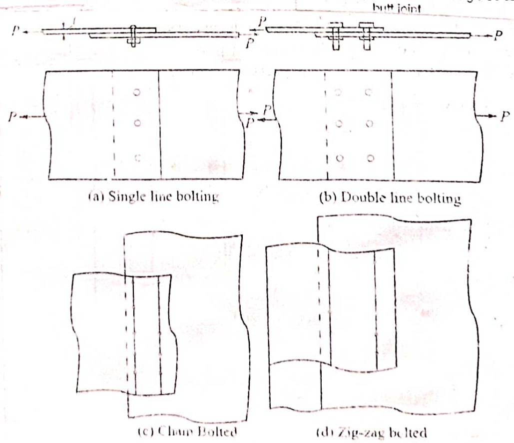


(h) Double-cover single bolted butt joint



(a) Single cover butt joint

(b) Double cover butt joint



(a) Single line bolting

(b) Double line bolting

(c) Chain Bolted

(d) Zig-zag bolted

Figure 3.9 Types of lap joints.

(*) vvI

Failure of Bolted or riveted Joints

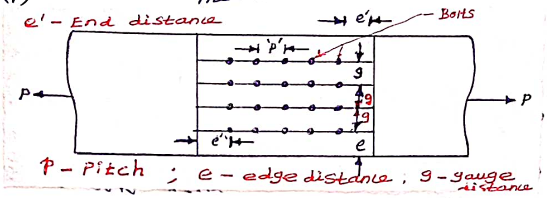
* Failure of bolts
    * shear failure of bolts
    * Bearing failure of bolts

* Failure of plates
    * Bearing failure of plates
    * Shear failure of plates
    * Tension failure of Plates.
    * splitting failure of plates.

Basic Definitions :-

IS: 800-2007
P.NO: 4
1.3.72

Pitch of bolts (p) :- → c/c spacing of the bolts in a row measured along the direction of load



IS: 800 - P.NO: 73
cl: 10.2.2

$p = 2.5d$
d → nominal dia of bolt

Gauge distance (g) :- it is the → distance b/w the two consecutive bolts of adjacent rows and it is measured at right angles to the direction of load.

Spacing b/w adjacent parallel bolts.

P.NO: 3
1.3.55 (Gauge)

Edge distance (e) :- → distance of centre of bolt hole from the adjacent edge of plate.

P.NO: 2
1.3.33

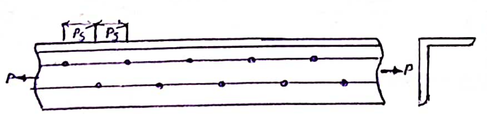
$e = 1.7 d_0$ d → dia of hole   cl: 10.2.4.2, P.NO: 74

End distance (e') → distance of the nearest bolt hole from the end of the plate

P.NO: 2
1.3.40

$e' = 1.7 d_0$ d → dia of hole   cl: 10.2.4.2, P.NO: 74

Staggered distance (P_s) :- → c/c distance of staggered bolts measured obliquely on the member.



Diameter of bolt & hole :-

Nominal size of bolts (d) in mm	12	14	16	20	22	24	30	36
Diameter of bolt hole (d_0) in mm	13	15	18	22	24	26	33	39

Material properties of bolts :- (IS 1367)

Grade 4.6 ; $f_y = 240\ N/mm^2$; $f_{ub} = 400\ N/mm^2$
Grade 4.8 ; $f_y = 320\ N/mm^2$; $f_{ub} = 420\ N/mm^2$
Grade 5.6 ; $f_y = 300\ N/mm^2$; $f_{ub} = 500\ N/mm^2$
Grade 5.8 ; $f_y = 400\ N/mm^2$; $f_{ub} = 520\ N/mm^2$

Fe 415 steel plate ; $f_y = 250\ N/mm^2$; $f_{ub} = 400$; $f_u = 410\ N/mm^2$

Design Tensile strength of Plates in a Joint

* Plates in a joint made with bearing bolts may fail under tensile force due to any one of the following

1) Bursting or shearing of the edge
2) crushing of plates
3) Rupture of plates

* The design tensile strength of plate is given by

(From IS 800-2007 ; Page No: 32)

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

Where,

IS: 800-2007
Page No: 30
Table: 5

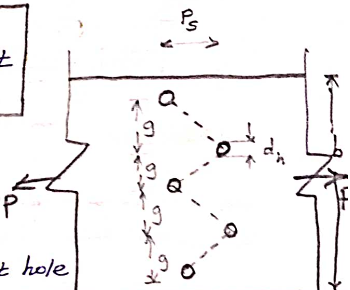
γ_{m1} = Partial saftey factor for failure at ultimate stress = 1.25

$f_u \rightarrow$ Ultimate stress of the material
From IS: 800-2007 ; Page No: 13 & 14 ; Table: 1

$A_n \rightarrow$ Net effective area of the member (Plate)

From IS: 800-2007 ; Page No: 33

$$\boxed{A_n = \left[b - n d_h + \sum \frac{P_{si}^2}{4 g_i} \right] t}$$



Where,

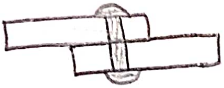
$b \rightarrow$ width of plate
$t \rightarrow$ thickness of Plate
$d_h \rightarrow$ diameter of bolt hole
$g \rightarrow$ gauge length b/w the bolt hole
$P_{si} \rightarrow$ Staggered pitch length b/w line of bolt hole
$n \rightarrow$ number of bolt holes in critical section
$i \rightarrow$ subscript for ~~[illegible]~~ summation of all inclined legs.

It may be noted that, if there is no staggered

$P_{si} = 0$

$\therefore$ $\boxed{A_n = (b - n d_h) t}$ which is the critical section

Lap joint



$n_n = 1$
$n_s = 0$

$n_n \rightarrow$ No. of shear plane with thread intercepting the shear plane

Double cover Butt joint

$n_n = 1$
$n_s = 1$

$n_s \rightarrow$ number of shear plane without thread intercepting the shear plane

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Type - I - Find the efficiency of the joint

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

* Strength of joint least value of the following three values

(i) Design strength of plate

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

IS: 800 - 2007 - P. NO: 32
clause : 6.3.1.

(ii) Design strength of the bolt in shear

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

IS: 800 - 2007 - P. NO: 75
clause: 10.3.3

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

(iii) Design strength of Bolt in Bearing

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

IS: 800 – 2007 ; P.NO: 75
Clause: 10.3.4

Nominal bearing strength $\quad V_{npb} = 2.5\, k_b\, d\, t\, f_u$

k_b is smaller of $\frac{e}{3d_o}$; $\frac{p}{3d_o} - 0.25$; $\frac{f_{ub}}{f_u}$; 1

$d \rightarrow$ nomina dia of bolt
$d_o \rightarrow$ dia of hole
$e \rightarrow$ end distance
$p \rightarrow$ pitch

* Strength of solid plate

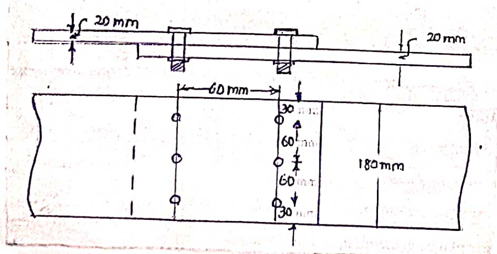
$$\boxed{T_{dg} = \frac{A_g \cdot f_y}{\gamma_{mo}}}$$

IS: 800 – 2007 ; page NO: 32
clause: 6.2

$f_y \rightarrow$ yield stress of material
$A_g \rightarrow$ gross area of c/s
$\gamma_{mo} \rightarrow$ Partial saftey factor for failure in tension by yielding.

Problem : 1

Find the efficiency of the lap joint as shown in figure.
M20 bolts of 4.6 and Fe410 (E250) plates are used.
Thickness of plate = 20 mm



Given Data:-

For M20 bolts of grade 4.6

Dia of bolt, $(d) = 20mm$

No. of bolt in critical section, $n = 3$

Dia. of bolt hole $(d_h) = 20 + \underbrace{2}_{\text{clearance}} = 22\,mm$ — IS:800-2007; P.NO: 73; Table: 19

4.6 grade Ultimate (Tensile stress) Strength $(f_{ub}) = 400\ N/mm^2$ → P.NO: 13; Table: 1

Partial saftey factor $(\gamma_{mb}) = 1.25$ → P.NO: 30; Table: 5

For plates, Fe410 (E250) → $f_u = 410\ N/mm^2$ → P.NO: 14 Table: 1

$\gamma_{m1} = 1.25$ → P.NO: 30 Table: 5

yield stress $f_y = 240\,N/mm^2$ → Page No: 14 Table: 1

Solution:-

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

Step: 1 – Strength of the joint:-

(i) Strength of the plate

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

IS 800 – 2007, Page No: 32 Clause: 6.3.1

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

$\left[P_{si} = 0\right]$ Since there is no stagered position of bolts.

$$\therefore A_n = \left[b - nd_h\right] t$$

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

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

t = thickness of thinner plate

$t = 20\,mm$

b = width of plate

$b = 180\,mm$

n = number of bolt holes in the weakest section

$n = 3$

$$T_{dn} = \frac{0.9\,A_n\,f_u}{\gamma_{m1}} = \frac{0.9 \times 2280 \times 410}{1.25} = 673056\,N$$

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

(ii) Design strength of bolt in shear (or) Shear capacity of Bolt :-

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

IS : 800 – 2007 ; P. No : 75
Clause : 10.3.3

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

For Lap Joint,

Number of shear planes at thread $n_n = 1$ per bolt

$\therefore$ Total $n_n = 1 \times 6 = 6$

Number of shear planes at shank $n_s = 0$ per bolt

$\therefore$ Total $n_s = 0$

$$\boxed{A_{nb} = 0.78\, A_{sb}}$$

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

$$A_{nb} = 245.045 \text{ mm}^2$$

no. of bolt

$$\therefore V_{nsb} = \frac{400}{\sqrt{3}} \left(6 \times 245.045 + 0 \right) = 339482 \text{ N}$$

$$V_{nsb} = 339.482 \text{ kN}.$$

$\therefore$ Design strength of bolt shear $V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{339.482}{1.25}$

$$\boxed{V_{dsb} = 271.586 \text{ kN}}$$

iii) Design strength of bolt in bearing :- (or) Bearing capacity of bolt

Refer IS : 800 – 2007 ; Page No : 75 ; clause : 10.3.4

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

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

k_b is smaller value of the following

edge distance given

(i) $\frac{e}{3d_0} = \frac{30}{3 \times 22} = 0.4545$

dia of hole

$d_0 = d_h$ pitch

(ii) $\frac{p}{3d_0} - 0.25 = \frac{60}{3 \times 22} - 0.25 = 0.6591$ least value

(iii) $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.9756$

(iv) 1

$\therefore k_b = 0.4545$

Nominal bearing strength of bolt,

$\therefore V_{npb} = 2.5\, k_b \cdot d \cdot t \cdot f_{ub}$

$= 2.5 \times 0.4545 \times 20 \times 20 \times 400$

$= 186345$ N per bolt

$V_{npb} = 186.345$ KN per bolt

$\therefore$ Design bearing strength of bolt

$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}} = \frac{186.345}{1.25} = 149.076$ kN per bolt

$\therefore$ Design strength for 6 bolt

For 6 bolt $= 6 \times 149.076$

$\boxed{V_{dpb} = 894.456 \text{ kN}}$

$\therefore$ strength of bolt

Least value of $V_{dsb} = 271.586$ KN

$V_{dpb} = 894.456$ KN

$\therefore$ strength of bolt $= 271.586$ KN.

Strength of Joint:-

Least value of

(i) Design strength of plate $T_{dn} = 673.056$ kN

(ii) Design strength of bolt $V_{dsb} = 271.586$ kN

$\therefore$

∴ Take the least value,

∴ Design strength of joint = 271.586 kN.

Step: 2 - Design strength of solid plate :-

Refer IS : 800 - 2007 ; Page No : 32 ; clause : 6.2

gross area of c/s

yield stress of material

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

Partial saftey factor for failure in tension by yielding

$A_g = b \cdot t$

$= 180 \times 20$

$A_g = 3600 \text{ mm}^2$

$f_y \rightarrow$ Refer IS 800 - 2007, P.NO : 14, Table : 1

based thickness (or) dia

$f_y = 240 \text{ N/mm}^2$

$\gamma_{mo} \rightarrow$ Refer IS : 800 - 2007, P.NO : 30; Table : 5

$\gamma_{mo} = 1.1$

∴ Design strength of member (solid plate)

$$T_{dg} = \frac{3600 \times 240}{1.1} = 818181.8 \text{ N}$$

$$\boxed{T_{dg} = 818.182 \text{ kN}}$$

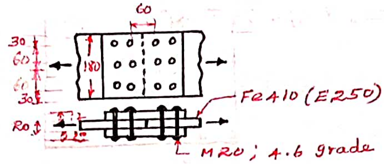
Step: 3 - Efficiency of the joint :-

$$\eta = \frac{\text{strength of joint}}{\text{Strength of Solid Plate}} \times 100 = \frac{271.586}{818.182} \times 100$$

$$\boxed{\eta = 33.19 \%}$$

Problem: 2

Find the efficiency of butt joint which made using two cover plates, ~~for~~ each of size 12mm and 6 numbers of bolt on each side. For M20 Bolt of grade 4.6 and Fe 410 (E250) plates are used.



Solution:-

For plates

$b = 180\ mm$

$t = 20\ mm$

$f_u = 410\ N/mm^2$ (P.NO: 14, Table: 1)

$\gamma_{m1} = 1.25$ (P.NO: 30, Table: 5)

Based on dia of bolt >20, 20-40 → $f_y = 240\ N/mm^2$ (Page NO: 14, Table: 1)

For Bolts

dia of bolt, $d = 20\ mm$

dia of hole, $d_0 = d_h = 20+2 = 22\ mm$ (P.NO: 73; Table: 19)

$f_{ub} = 400\ N/mm^2$ (P.NO: 13, Table: 1)

$\gamma_{mb} = 1.25$ (P.NO: 30; Table-5)

since the given joint is double cover butt joint.

For Butt joint

$n_n = 1$ for each bolt

$n_s = 1$ for each bolt.

Step: 1

Strength of joint

(i) - Strength of Plate :-

IS: 800-2007, clause 6.3.1; P.NO: 32

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

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

No Staggered is given

$$\therefore A_n = (b - nd_n)t$$

$$= (180 - 3 \times 22)\,20$$

$$A_n = 2280\ mm^2$$

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

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

(ii) Design strength of bolt in shear (or) Shear capacity of bolt :-

IS: 800-2007, P.NO: 75; clause: 10.3.3

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

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

For Butt joint :-

$n_n = 1$

$n_s = 1$

$\boxed{A_{nb} = 0.78\, A_{sb}}$

No. of bolt

$$= \frac{400}{\sqrt{3}}\left[\left(6 \times \frac{\pi \times 20^2}{4} \times 0.78\right) + \left(6 \times \frac{\pi \times 20^2}{4}\right)\right]$$

$$\boxed{V_{nsb} = 774.795 \text{ kN.}}$$

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{774.795}{1.25}$$

$$\boxed{V_{dsb} = 619.835 \text{ KN}}$$

(iii) <u>Design strength of bolt in bearing</u> (or) <u>Bearing capacity of Bolt:-</u>

Refer IS: 800 -2007, Page No: 75 ; clause: 10.3.4

$$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}} \qquad V_{npb} = 2.5\, k_b \cdot d \cdot t \cdot f_{ub}$$

$\underline{\underline{k_b}}$

(i) $\frac{e}{3d_0} = \frac{30}{3 \times 22} = 0.4545$

(ii) $\frac{p}{3d_0} - 0.25 = \frac{60}{3 \times 22} - 0.25 = 0.6591$

(iii) $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.9756$

(iv) 1

Taken least value

$\therefore \boxed{k_b = 0.4545}$

$$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}} = \frac{2.5 \times 0.4545 \times 20 \times 20 \times 400}{1.25} = \frac{186345}{1.25} \text{ N}$$

$V_{dpb} = 149076$ N for each bolt $= 149.076 \text{ kN} \times 6$

$$\therefore \boxed{V_{dpb} = 894.456 \text{ kN}}$$

Strength of bolt :-

smaller value of

$V_{dsb} = 619.835\ kN$

$V_{dpb} = 894.456\ kN$

$\therefore$ Strength of bolt $= 619.835\ kN$

Strength of joint

smaller value of

Strength of plate $= 673.056\ kN$

Strength of bolt $= 619.835\ kN$.

Strength of plate $>$ Strength of bolt

Hence safe

$\therefore$ Take Strength of joint $= 619.835\ kN$.

Step:2 - Strength of Solid plate (member):-

Refer IS:800-2007 ; P.No: 32 ; clause: 6.2

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

$A_g = b.t = 180 \times 20$

$= 3600\ mm^2$

$f_y = 240\ N/mm^2$ based on thick + dia (P.No: 14 ; Table: 1)

$\gamma_{mo} = 1.1$ (IS:800, 2007 ; P.No: 30 ; Table : 5)

$$= \frac{3600 \times 240}{1.1}$$

$$\boxed{T_{dg} = 818.182\ kN}$$

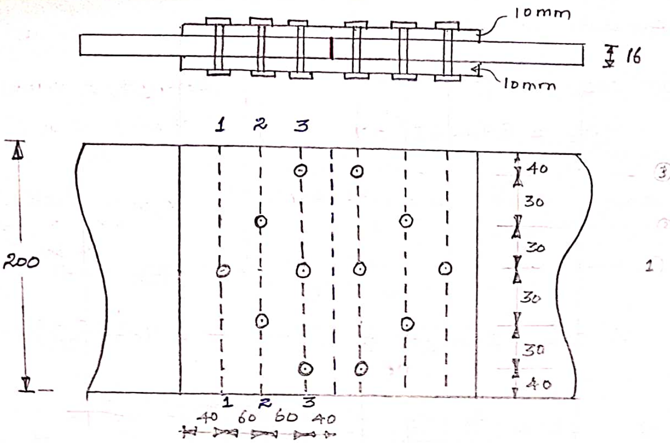
Step:3 - Efficiency of the joint:-

$$\eta = \frac{\text{Strength of joint}}{\text{Strength of Solid plate}} \times 100 = \frac{619.835}{818.182} \times 100$$

$$\boxed{\eta = 75.76\%}$$

Problem: 3

Find the maximum force which can be transferred through the double covered butt joint shown in fig. Find the efficiency of the joint also. Given M20 bolts of grade 4.6 and Fe 410 steel plates are used.



Given Data:-

For Bolt (M20 bolt, 4.6 grade)	For plate (Fe410)
$d = 20\,mm$	$b = 200\,mm$
$d_0 = 20 + 2 = 22\,mm$ (P.NO:73, table:19)	$t = 16\,mm$
$f_{ub} = 400\,N/mm^2$ (Table: 1 P.NO: 13)	$f_u = 410\,N/mm^2$ (P.NO:14, Table:1)
$\gamma_{mb} = 1.25$ (P.NO: 30, Table: 5)	$\gamma_{m_1} = 1.25$ (P.NO: 30 ; Table:5)
	$\gamma_{mo} = 1.10$ (P.NO: 30 ; Table:5)
	$f_y = 240\,N/mm^2$ (P.NO: 14; Table: 1)

Solution:-

Note: since the diamond pattern of bolting is given in this problem, first find the "failure of critical section" Then find the strength of bolt.

Step: 1

Shearing strength of Bolt:-

IS: 800-2007 ; P.NO: 75 ; clause: 10.3.3

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

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

For Double cover butt joint

$n_n = 1$

$n_s = 1$

$A_{nb} = 0.78\ A_{sb}$

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

$A_{nb} = 245.045\ mm^2$

Nominal strength of one bolt,

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

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

$$= \frac{400}{\sqrt{3}}\left[1 \times 245.045 + 1 \times 314.16\right]$$

$$= 129.14\ KN$$

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{129.14}{1.25} = 103.312\ KN \quad \text{For one bolt}$$

∴ Design shearing strength of bolt in the whole joint

$$= 6 \times V_{dsb}$$

$$= 6 \times 103.312$$

$$\boxed{V_{dsb_{whole}} = 619.872\ KN}$$

Step: 2 - Bearing capacity of Bolt :-

IS:800-2007 ; P.NO:75 ; clause: 10.3.4

$$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}} \qquad V_{npb} = 2.5\ k_b \cdot d \cdot t \cdot f_{ub}$$

k_b Value for 1-1	k_b Value for 2-2	k_b Value for 3-3
(i) $\dfrac{e}{3d_o} = \dfrac{100}{3 \times 22} = 1.515$	(i) $\dfrac{e}{3d_o} = \dfrac{40+30}{3 \times 22} = 1.06$	(i) $\dfrac{e}{3d_o} = \dfrac{40}{3 \times 22} = 0.606$
(ii) $\dfrac{p}{3d_o} - 0.25 = \dfrac{60}{3 \times 22} - 0.25 = 0.659$	(ii) $\dfrac{p}{3d_o} - 0.25 = \dfrac{60}{3 \times 22} - 0.25 = 0.659$	(ii) $\dfrac{p}{3d_o} - 0.25 = \dfrac{60}{3 \times 22} - 0.25 = 0.659$
(iii) $\dfrac{f_{ub}}{f_u} = \dfrac{400}{410} = 0.976$	(iii) $\dfrac{f_{ub}}{f_u} = \dfrac{400}{410} = 0.976$	(iii) $\dfrac{f_{ub}}{f_u} = \dfrac{400}{410} = 0.976$
(iv) 1	(iv) 1	(iv) 1
Least value ∴ $k_{b_1} = 0.659$	∴ $k_{b_2} = 0.659$	∴ $k_{b_3} = 0.606$

$V_{npb} = 2.5\ k_b \cdot d \cdot t \cdot f_{ub}$

For section ①-①, ②-② & ③-③ No. of bolt @ section ③-③

$V_{npb} = 1\ V_{npb\ ①-①} + 2V_{npb\ ②-②} + 3V_{npb\ ③-③}$

$\therefore V_{npb} = 1 \times 2.5\ k_{b_1} \cdot d \cdot t \cdot f_{ub} + 2 \times 2.5\ k_{b_2} \cdot d \cdot t\ f_{ub} + 3 \times 2.5\ k_{b_3} \cdot d \cdot t\ f_{ub}$

$= (1 \times 2.5 \times 0.659 \times 20 \times 16 \times 400) + (2 \times 2.5 \times 0.659 \times 20 \times 16 \times 400)$

$+ (3 \times 2.5 \times 0.606 \times 20 \times 16 \times 400)$

$= 211200 + 422400 + 585600$

$$\boxed{V_{npb} = 1219.2\ kN}$$

$$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}} = \frac{1219.2}{1.25}$$

$$\boxed{V_{dpb} = 975.36\ kN}$$

Step: 3 – Strength of Bolt :-

Shearing strength of bolt $V_{dsb} = 619.872\ kN$

bearing strength of bolt $V_{dpb} = 975.36\ kN$.

Take least value

$\therefore$ strength of Bolt $= 619.872\ kN$

Step: 4 :- – Strength of plate :-

IS 800 – 2007 ; P. NO : 32, clause : 6.3.1

$$T_{dn} = \frac{0.9\ A_n \cdot f_u}{\gamma_{m1}} \qquad A_n = \left[b - nd_n + \sum_i \frac{P_{si}^2}{4g_i}\right] t$$

$\therefore$ The given type of bolting is dimond pattern.

∴ The strength of plate should be checked in each section. ∴ $A_n = (b - n\,d_n)\,t$

Section ① - ①

$A_{n_1} = (b - n_1 d_n)\,t$

$= (200 - 1 \times 22)\,16$

$= 2848\ mm^2$

$T_{dn_1} = \dfrac{0.9\,A_{n_1}\,f_u}{\gamma_{m_1}}$

$= \dfrac{0.9 \times 2848 \times 410}{1.25}$

$\boxed{T_{dn_1} = 840.73\ kN}$

When this section fails, bolt in the section ①-① also has to fail. Hence strength of plate at section ②-②

Section ② - ②

$A_{n_2} = (200 - \overset{\text{No. of bolt @ section ②-②}}{2} \times 22)\,16$

$= 2496\ mm^2$

$T_{dn_2} = \dfrac{0.9 \times 2496 \times 410}{1.25}$

$= 736.82\ kN$

∴ $T_{dn_2\,fail} = T_{dn_2} +$ shearing strength of bolt ①-①

$= 736.82 + 103.312$

$\boxed{T_{dn_2} = 840.132\ kN}$

When the section ③-③ fails, bolt in the section ②-② & ①-① also has to fail

Section ③ - ③

$A_{n_3} = (200 - 3 \times 22)\,16$

$= 2144\ mm^2$

$T_{dn_3} = \dfrac{0.9 \times 2144 \times 410}{1.25} +$ shearing capacity of bolt ①① + shearing capacity of bolt in ②②

$= 632.91 + \underset{\text{No. of bolt in ①①}}{103.312} + (\underset{\text{No. of bolt in ②-②}}{2} \times 103.312)$

$\boxed{T_{dn_3} = 942.85\ kN}$

Above three, the least value is taken,

∴ Strength of plate = 840.132 kN.

Step:5

Strength of joint :-

Strength of bolt = 619.872 kN

Strength of plate = 840.132 kN.

∴ Take least value,

∴ $\boxed{\text{Strength of joint} = 619.872\ kN}$

$\therefore$ Max. design force safely transfered = 619.872 kN.

Step:6 — Design strength of solid plate:-

IS 800–2007 ; P.NO : 32 ; Clause : 6.2

$$T_{dg} = \frac{A_g \cdot f_y}{\gamma_{mo}} = \frac{(b \cdot t) \cdot f_y}{\gamma_{mo}}$$

$$= \frac{(200 \times 16) \times 240}{1.1}$$

$$\boxed{T_{dg} = 698.18 \text{ kN}}$$

Step:7 — Efficiency of joint

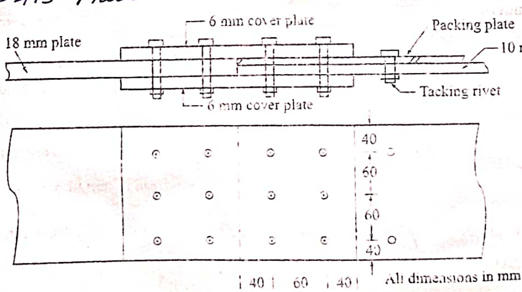
$$\eta = \frac{\text{Strength of joint}}{\text{Strength of solid plate}} \times 100 = \frac{619.872}{698.18} \times 100$$

$$\boxed{\eta = 88.78\ \%}$$

Type – II

Problem:4

Two cover plates, 10mm and 18mm thick are connected by a double cover butt joint using 6mm coverplates as shown in fig. Find the strength of the joint. Given M20 bolts of grade 4.6 and Fe 415 plates are used.



Given Data:-

For M20 Bolt, 4.6 grade

$d = 20mm$

$d_o = 20+2 = 22mm$

$f_{ub} = 400 N/mm^2$

$\gamma_{mb} = 1.25$

For Fe415 plates

$b = 200mm$

$t = 18 mm$

$f_u = 410 N/mm^2$

$\gamma_{m1} = 1.25$

$\gamma_{mo} = 1.10$

$f_y = 240 N/mm^2$

Packing plate thickness $= 18-10 = 8mm$

$e = 40mm$;

$p = 60mm$

$t = 10mm$

Step: 1 - Strength of plate

IS: 800-2007 ; P.NO: 32 ; clause: 6.3.1

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

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

No staggered is given

$$A_n = (b - nd_n)t$$

$$= (200 - 3\times22)10$$

$$A_n = 1340 mm^2$$

$$= \frac{0.9 \times 1340 \times 410}{1.25}$$

$$\boxed{T_{dn} = 395.57 KN}$$

Step:2 - Shearing strength of bolt:-

IS: 800-2007 ; P.NO: 75 ; clause: 10.3.3.

10.3.3.3 Packing Plates.

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} \cdot \beta_{pk}$$

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

For Double cover butt joint

$n_n = 1$

$n_s = 1$

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

$$A_{nb} = 0.78 \ A_{sb} = 0.78 \times 314.16 = 245.04 \ mm^2$$

Note :-

In this connection, packing plate of 8mm thickness is used. Hence there shall be reduction in the shear strength of bolt.

For design purpose tacking rivet is not considered

The reduction factor is given by

IS : 800 - 2007 ; P.No: 75 ; clause 10.3.3.3.

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

$$= (1 - 0.0125 \times 8)$$

$$\boxed{\beta_{pk} = 0.90}$$

$t_{pk} \rightarrow$ thickness of the thicker packing in mm

$t_{pk} = 8 \ mm$

$$V_{nsb} = \frac{400}{\sqrt{3}} \left[1 \times 245.04 + 1 \times 314.16 \right]$$

$$\boxed{V_{nsb} = 129.14 \ kN.}$$

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} \cdot \beta_{pk} = \frac{129.14}{1.25} \times 0.9$$

$$\boxed{V_{dsb} = 92.98 \ kN}$$ For 1 bolt

$\therefore$ For 6 bolt,

$$V_{dsb} = 6 \times 92.98$$

$$\boxed{V_{dsb} = 557.88 \ kN}$$

Step: 3 — Bearing strength of Bolt

IS : 800 - 2007 ; P. No : 75 ; clause : 10.3.4

$$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}}$$ $V_{npb} = 2.5\, k_b \cdot d \cdot t \cdot f_{ub}$

For k_b, least value of the following

$d = 20$ mm
$t = 10$ mm
$f_{ub} = 400$ N/mm^2

(i) $\frac{e}{3d_o} = \frac{40}{3 \times 22} = 0.61$

(ii) $\frac{p}{3d_o} - 0.25 = \frac{60}{3 \times 22} - 0.25 = 0.66$

(iii) $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.98$

(iv) 1

Take least value, $\therefore \boxed{k_b = 0.61}$

$$V_{npb} = 2.5 \times 0.61 \times 20 \times 10 \times 400$$

$$\boxed{V_{npb} = 122 \text{ kN}}$$

$$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}} = \frac{122}{1.25}$$

$$\boxed{V_{dpb} = 97.6 \text{ kN.}}$$ For 1 bolt

For 6 bolt, $V_{dpb} = 6 \times 97.6$

$$\boxed{V_{dpb} = 585.6 \text{ kN}}$$

Step: 4 - Strength of Bolt :-

Shearing strength of bolt = 557.88 kN

Bearing strength of bolt = 585.6 kN.

Take least value,

$\therefore$ $\boxed{\text{Strength of bolt} = 557.88 \text{ kN.}}$

Step: 5 - Strength of Joint :-

Strength of plate = 395.57 kN

Strength of bolt = 557.88 kN

Take least value,

$$\therefore \boxed{\text{Strength of joint} = 395.57 \text{ kN}}$$

Step: 6 – Strength of solid plate :-

IS: 800 – 2007 ; P. No: 32 ; clause: 6.2

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

$A_g = b.t$
$= 200 \times 18$
$= 3600 \text{ mm}^2$

$$= \frac{3600 \times 240}{1.1}$$

$$\boxed{T_{dg} = 785.45 \text{ kN}}$$

Step: 7 – Efficiency of Joint :-

$$\eta = \frac{\text{Strength of Joint}}{\text{Strength of solid plate}} \times 100 = \frac{395.57}{785.45} \times 100$$

$$\boxed{\eta = 50.36\%}$$

Type – III – Design of Bolted Joint

Procedure

(i) calculate the shearing strength of bolt.

(ii) calculate the bearing strength of bolt

(iii) calculate the strength of bolt (This is also called as Bolt value

(iv) Find the number of bolt in the joint.

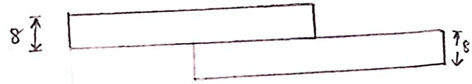
$$\boxed{\text{No. of bolt in a joint} = \frac{\text{Factored load}}{\text{bolt value}}}$$

(v) Fix the pitch distance (p) and Edge distance (e) and gauge distance (g) as per the code IS: 800 – 2007.

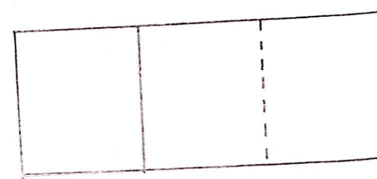
(vi) Arrange the bolts, and draw the diagram.

Problem: 5

Design a lap joint between the two plates of 100 mm × 8 mm size, and it has to transmit a factored load of 100 kN using black bolt of 12 mm dia, and 4.6 grade of steel and the plates are made up of steel grade Fe 410 (E 250).



Given data :-



For Bolt	For plate
4.6 grade	Fe 410 (E 250) grade steel
$d = 12 mm$	$b = 100 mm$
d_o (or) $d_n = 12 + 1 = 13 mm$ (P.No: 73, Table: 19)	$t = 8 mm$
	$f_u = 410 N/mm^2$
$f_{ub} = 400 N/mm^2$	$\gamma_{m1} = 1.25$
$\gamma_{mb} = 1.25$	$\gamma_{mo} = 1.10$
	$f_y = 250 N/mm^2$ (IS: 800 - 2007 ; P.No: 14 ; Table: 1)

Step: 1 – shearing strength of bolt :-

IS: 800 – 2007 ; P.No: 75 ; clause : 10.3.3.

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

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

For Lap joint, for each bolt

$n_n = 1$, $n_s = 0$

$A_{nb} = 0.78 A_{sb}$

$$V_{nsb} = \frac{400}{\sqrt{3}} \times \left[1 \times 0.78 \times \frac{\pi \times 12^2}{4} \right]$$

$$\boxed{V_{nsb} = 20.37 kN}$$

$$V_{dsb} = \frac{20.37}{1.25}$$

$$\boxed{V_{dsb} = 16.296 kN}$$

Step: 2 – Bearing strength of Bolt :-

IS: 800 – 2007 ; P.No: 75 ; clause : 10.3.4

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

$$V_{npb} = 2.5 \, k_b \cdot d \cdot t \cdot f_{ub}$$

k_b - For the least value of the following

(i) $\frac{e}{3d_o} = \frac{23}{3\times13} = 0.59$

(ii) $\frac{p}{3d_o} - 0.25 = \frac{30}{3\times13} - 0.25 = 0.52$

(iii) $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.98$

(iv) 1

Take Least value,

$\therefore \boxed{k_b = 0.52}$

IS : 800-2007 ; P.NO: 73, clause: 10.2.2

Min. spacing $(p) = 2.5$. dia of bolt

$= 2.5 \times 12$

$\boxed{p = 30mm}$

IS : 800-2007 ; P.NO: 74 ; clause: 10.2.4;

Min. end distance $(e) = 1.7\, d_o$

$= 1.7 \times 13$

$= 22.1$

say $\boxed{e = 23\ mm}$

$V_{npb} = 2.5\, k_b . d . t . f_{ub} = 2.5 \times 0.52 \times 12 \times 8 \times 400$

$\boxed{V_{npb} = 49.92\ kN}$

$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}} = \frac{49.92}{1.25}$

$\boxed{V_{dpb} = 39.936\ kN}$

Step: 3 – Bolt value (or) strength of Bolt :-

Least value of the following

shearing Strength of bolt = 16.296 kN.

Bearing Strength of bolt = 39.936 kN

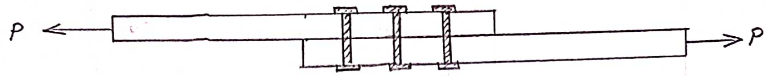
$\therefore$ $\boxed{\text{Bolt value (or) Strength of bolt} = 16.30\ kN.}$

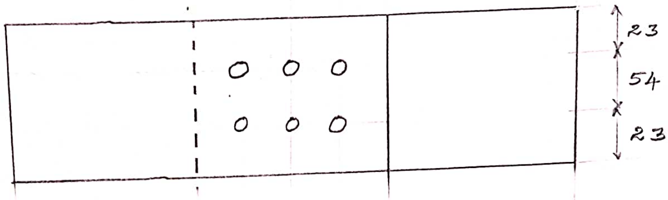
Step: 4 – Number of Bolt in Joint

No. of bolt $= \frac{\text{Factored load}}{\text{Bolt value}} = \frac{100}{16.30} = 6.13$

say. $\boxed{\text{No. of bolt} = 6\ \text{Nos.}}$

Step : 5 – Arrangement of Bolt :-





Average 3 no's of bolts in two rows, as chain pattern.

Step : 6 – check for strength of plate :-

IS : 800 – 2007 ; P. NO : 32 ; clause : 6.3.1

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

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

For no staggered condition

$$= (b - n d_n)\, t$$

$$= (100 - 2 \times 13)\, 8$$

$$A_n = 592 \text{ mm}^2$$

$$= \frac{0.9 \times 592 \times 410}{1.25}$$

$$\boxed{T_{dn} = 174.76 \text{ kN} > \text{factored load } (100 \text{ kN})}$$

Hence the design is safe.

Problem : 6

Design a lap joint b/w the two plates each of width 120mm, if the thickness of one plate is 16mm and the other is 12mm. The joint has to transfer a design load of 160kN. The plates are of Fe410 grade. Use bearing type bolts.

Assume M16 bolt 4.6 grade

16 | 12

Given Data :-

Assume M16 Bolt, 4.6 grade

shear strength 36.21 kN

t = 12 mm

d = 16 mm

For bolts.

Assume, $d = 16\,mm$

$d_o = 16+2 = 18\,mm$

$f_{ub} = 400\ N/mm^2$

$\gamma_{mb} = 1.25$

For plates

$b = 120\,mm$

$t_1 = 16\,mm$

$t_2 = 12\,mm$

$\therefore\ t = 12\,mm$ (least value)

$f_u = 410\ N/mm^2$

$\gamma_{m1} = 1.25$

$\gamma_{mo} = 1.10$

IS: 800-2007
P.NO: 74
10.2.4.2

edge distance $(e) = 1.7\,d_o = 1.7\times18 = 30.6$

say $\boxed{(e) = 32\,mm}$

P.NO: 73
Clause: 10.2.2

Pitch distance $(p) = 2.5d = 2.5\times16 = 40mm$

$\boxed{p = 40mm}$

Step: 1 :- Shearing Strength of bolt

IS: 800-2007 ; P.NO: 75 ; clause: 10.3.3

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

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

For lap joint, $\left.\begin{matrix} n_n = 1 \\ n_s = 0 \end{matrix}\right\}$ For each bolt

$$V_{nsb} = \frac{400}{\sqrt{3}}\left[1\times0.78\times\frac{\pi\times16^2}{4}+0\right]$$

$$\boxed{V_{nsb} = 36.22\ KN}$$

$$= \frac{36.22}{1.25}$$

$$\boxed{V_{dsb} = 28.98\ KN}$$

Step: 2 - Bearing strength of Bolt

IS: 800-2007 ; P.NO: 75 ; clause: 10.3.4

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

$$V_{npb} = 2.5\,k_b . d . t . f_{ub}$$

k_b For least value of following

(i) $\frac{e}{3d_o} = \frac{32}{3\times18} = 0.59$

(ii) $\frac{p}{3d_o} - 0.25 = \frac{40}{3\times18} - 0.25 = 0.49$

(iii) $f_{ub}/f_u = 400/410 = 0.98$

(iv) 1

Take least value,

$\therefore k_b = 0.49$

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

$\boxed{V_{npb} = 94.08\ KN}$

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

$$\boxed{V_{dpb} = 75.26 \text{ KN}}$$

Step: 3 – Strength of bolt (or) Bolt value :-

Shearing strength of bolt = 28.98 KN

Bearing strength of bolt = 75.26 KN

∴ least value,

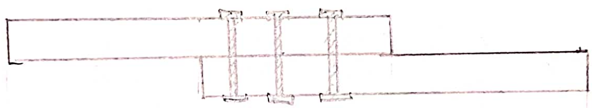
∴ $\boxed{\text{Bolt value (or) strength of bolt} = 28.98 \text{ KN}}$

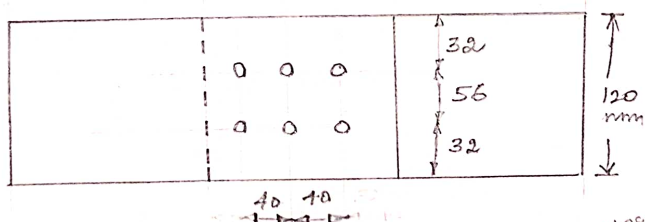
Step: 4 - No. of Bolts :-

$$n = \frac{\text{factored load}}{\text{Bolt value}} = \frac{160}{28.98} = 5.52$$

say, $\boxed{n = 6 \text{ nos.}}$

Step: 5 - Arrangement of Bolts :-





Step: 6 - Check for Strength of plate

IS: 800 – 2007 ; P.NO : 32 ; clause : 6.3.1

∴ no stagered condition

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

$$A_n = \left[b - nd_0 + \sum_i \frac{P_{si}^2}{4g_i}\right] t$$

$$= (b - nd_0)\, t$$

$$= (120 - 3\times18)\, 12$$

$$= 1008 \text{ mm}^2$$

$$= 0.9 \times 1008 \times 410 / 1.25$$

$$T_{dn} = 297.56 \text{ KN} > 160 \text{KN}$$

ie, $\boxed{\text{design load} > \text{factored load}}$

WELDED JOINT

Welding consists of joining two pieces of metal establishing a metallurgical bond b/w them.

Advantages of welded connections:-

* Due ~~to~~ without the absence of gusset plates, connecting angles etc., * welded structures are lighter.
* Welding is more adopted for riveting and bolting.
* This is possible to obtain 100% of efficiency for joint.
* It is ~~free~~ (less) from noise pollution
* Good aesthetic appearence
* Welded joints are air tight and water tight
* Welded joints are rigid.
* There is no mismatching of ~~bolt~~ holes. ~~in bolts~~.
* Alternation process will be easy.

Dis-advantages:-

* Un even heating and cooling of metal.
* There is a greater possibility of brittle fracture in welding.
* Sudden failures are possible (compared to bolted joint)
* Inspection of welded joint is expensive
* Skilled labours are required.
* Proper welding in field is difficult.

Types of welded joint:-

* Butt weld (or) groove weld
* Fillet weld
* Slot weld & plug weld

Important specification for welding :-

IS : 800 – 2007 ; P.No : 78 ; clause : 10.5

(i) End returns $\not<$ 2 times size of weld — around the corners, not less than

(ii) Lap length $\not<$ 4 t (or) 40mm

(iii) size of weld, Table : 21, P. No : 78

(iv) Effective throat thickness,

effective $t_t = k.s$

k → refer table : 22 ; P.No : 78

Generally, $\boxed{K = 0.7}$

(v) Intermediate weld $\not<$ 4.s (or) 40 mm

spacing b/w adjacent intermediate weld → compression = 12 t ; Tension = 16 t

Design strength of Fillet welds :-

IS : 800 – 2007 ; P.No : 79 ; clause : 10.5.7.1.1

$$f_{wd} = \frac{f_{wn}}{\gamma_{mw}} \cdot \text{Area}$$

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

f_u → smaller of the ultimate stress of the weld

γ_{mw} → Partial saftey factor. (Table : 5 : P.No : 30)

Area of weld $= L_w \cdot t_t$

L_w → Length of weld

t_t → throat thickness of weld

Full strength of the plate

IS : 800 – 2007 ; P.No : 32 ; clause : 6.2

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

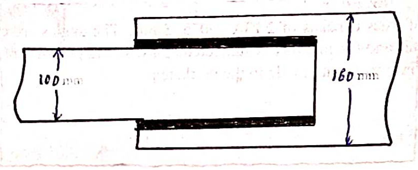
f_y → yield stress of materi[al]

A_g → gross area of c/s

γ_{mo} → Partial saftey factor for failure in tension by yielding (P.No : 30 : Table : 5)

Problem : 7

Design a suitable longitudinal fillet weld to connect the plates as shown in fig. to transmit a ~~the~~ full strength equal to the full strength of ~~the~~ small plate. Given: plates are 12 mm thick; grade of plates Fe 410 and welding to be made in workshop.



Given Data :-

Thickness of plate $t = 12\,mm$

Grade of plate Fe 410

$f_y = 250\ N/mm^2$

$\gamma_{mo} = 1.1$

breadth of plate $= 100\,mm$

$\gamma_{mw} = 1.25$ (P.No: 30, Table: 5)

Design the Longitudinal Fillet weld,

ie, To find L_w, s, t & full strength of the small plate ie, T_{dg}.

Step: 1 - Minimum size of weld (s) :-

Thickness of Plate $= 12\,mm$

Minimum size to be used $= 5\,mm$ (P.No: 78 ; IS: 800-2007 Table: 21)

edge thickness of plate $= 1.5\,mm$ (P.No: 79 ; clause: 10.5.8.1)

Maximum size of weld = Thickness of plate − edge thickness

$= 12 - 1.5$

$= 10.5\,mm$

$\therefore$ Use $\boxed{s = 10\,mm}$ Fillet weld

condition .

Full strength of weld = Full strength of the small plate

ie, Full Strength of weld $T_{dg} = \dfrac{A_g \cdot f_y}{\gamma_{mo}} = \dfrac{(b \cdot t) \cdot f_y}{\gamma_{mo}}$

$= \dfrac{100 \times 12 \times 250}{1.1}$

$$\boxed{T_{dg} = 272.73 \text{ kN}}$$

Step: 3 – Design Strength of weld:-

P.NO: 79
Clause 10.5.7.1.1

$$f_{wd} = \text{Area} \cdot \frac{f_{wn}}{\gamma_{mw}}$$

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

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

eff. Length of Weld = L_w

Assuming Normal weld,

$$\text{Area} = L_w \cdot t_t$$

$\therefore$ Throat thickness $t_t = k \cdot S$. (P.NO: 78 ; Table: 22)

$k = 0.7$
$S = 10$

$$= 0.7 \times 10$$

$$\boxed{t_t = 7 \text{ mm}}$$

$$f_{wd} = (L_w \cdot t_t) \cdot \frac{f_{wn}}{\gamma_{mw}} = \frac{L_w \cdot \times 7 \times \left(\frac{410}{\sqrt{3}}\right)}{1.25}$$

$$\boxed{f_{wd} = 1391.87 \, L_w}$$

Equating the condition,

strength of the Weld = strength of the plate

$$1391.87 \, L_w = 272.73 \times 10^3$$

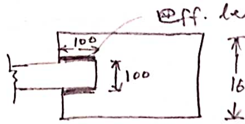
$$\therefore \boxed{L_w = 195.95 \text{ mm}}$$

Hence, welding on each side $= \frac{195.95}{2} = 97.9 \simeq 100$ mm

$\therefore$ provide eff. length of 100 mm on each side

Result:-

Length of weld = 100 mm (on each side)
size of weld = 10 mm
Throat thickness = 7.5 mm



Eff. length = 100 mm on each side

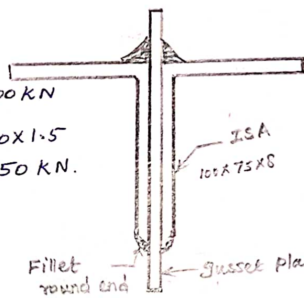
Problem: 8

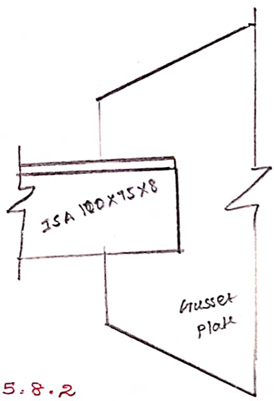
A tie member of a roof truss consists of 2 ISA 100 X 75 X 8 mm. The angles are connected to either side of a 10mm gusset plates and the member is subjected to a Working Pull of 300kN. Design the welded connection. Assume connections are made in the workshop.

Given Data:-

Working Pull (load) = 300 kN

$\therefore$ Factored load = 300 X 1.5
= 450 KN.





Step: 1 - Size of Weld

IS : 800 - 2007 ; P.No: 79 ; clause : 10.5.8.2

(i) At the rounded toe angle section should not exceed

size of weld (s) = $\frac{3}{4}$ thickness of section (not to exceed)

$$= \frac{3}{4} \times 8$$

$$\boxed{S = 6 \text{ mm}}$$

(ii) At top, the thickness should not exceed

$$S = t - 1.5 \quad \text{(IS: 800 - 2007 P.No: 79 ; clause 10.5.8.1)}$$

$$= 8 - 1.5$$

$$S = 6.5 \text{ mm}$$

$\therefore$ say (or) provide $\boxed{S = 6 \text{ mm}}$ Weld.

Each angle carries a factored load of
(or)
Load taken by each angle $= \frac{\text{Factored load}}{2} = \frac{450}{2}$

$$\boxed{\text{Load per one angle} = 225 \text{ kN}}$$

Step: 2 – Thickness of weld :-

$L_w \rightarrow$ total length of weld

Assume, Normal weld,

$\therefore t_t \rightarrow$ throat thickness

$$t_t = K \cdot S$$

$$= 0.7 \times 6$$

$$\boxed{t_t = 4.2 \text{ mm}}$$

Step: 3 – Design strength of weld :-

IS : 800 – 2007 ; P. NO : 79 ; clause : 10.5.7.1.1

$$f_{wd} = \text{Area of weld} \cdot \frac{f_{wn}}{\gamma_{mw}} \qquad f_{wn} = \frac{f_u}{\sqrt{3}} = \frac{410}{\sqrt{3}}$$

$$= (L_w \cdot t_t) \cdot \frac{(f_u/\sqrt{3})}{\gamma_{mw}} = \frac{(L_w \times 4.2) \times (410/\sqrt{3})}{1.25}$$

$$\boxed{f_{wd} = 795.35\, L_w}$$

Step: 4 – Eff. Length of weld (L_w)

Equating, ~~the~~

strength of weld = factored load

$$795.35\, L_w = 225 \times 10^3$$

$$\therefore L_w = 282.89$$

Say $\boxed{L_w = 283 \text{ mm}}$ for one angle

Step: 5 – Calculation of L_1 & L_2

Refer hand book SP: 6 (1) ; P. NO : 14

C_{xx} for ISA 100 × 75 × 8 mm $\rightarrow$ 3.1 cm

ie, $\boxed{C_{xx} = 31 \text{mm}}$

$L_1 \rightarrow$ Length of top weld

$L_2 \rightarrow$ Length of lower weld

To make centre of gravity of weld to coincide with that angle,

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

$$L_1 + L_2 = 283$$

$$L_1 = \frac{69 L_2}{31}$$

$$\therefore L_2 = 283 - l_1$$

$$L_1 = \frac{69}{31}(283 - l_1)$$

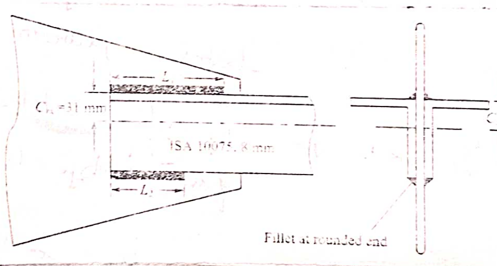
$$L_1 = 195.27 \text{ mm}$$

Say $\boxed{L_1 = 195 \text{ mm}}$

$$l_2 = 283 - l_1$$

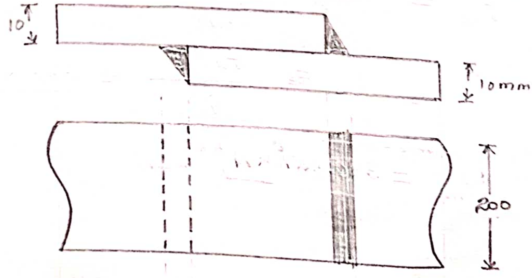
$$= 283 - 195$$

$$\boxed{l_2 = 88 \text{ mm}}$$



Problem: 9

Design the welded connection to connect two cover plates of width 200mm and thickness 10 mm for 100 percent efficiency.



Given Data :-

Thickness of plate $(t) = 10mm$

width of plate $(b) = 200 \text{ mm}$

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

Step: 1 : size of weld (S)

Minimum size of weld $(S_{min}) = 5 \text{ mm}$ (IS:800-2007, Table: 21) P.NO: 78

Maximum size of weld (S_{max}) = thickness of plate − edge thickness

P. NO: 79 Clause: 10.5.8.1

$= 10 - 1.5$

$= 8.5 \text{ mm}$

∴ use: $\boxed{\text{size of weld } S = 8 \text{ mm}}$

Step:2 : Throat thickness (t_t) :- & Eff. length of Fillet weld (L_w)

P.NO: 78 ; Table: 22

$t_t = k \cdot s = 0.7 \times 8 = 5.6 \text{ mm}$

$\boxed{t_t = 5.6 \text{ mm}}$

P.No: 78 Cl: 10.5.4.1

Eff. Length of fillet weld $(L_w) = 2(200 - 2S)$

$= 2(200 - 2 \times 8)$

$\boxed{L_w = 368 \text{ mm}}$

Step: 3 : Design strength of Fillet weld :-

P.NO: 79 ; Clause: 10.5.7.1.1

$$f_{wd} = \text{Area} \cdot \frac{f_{wn}}{\gamma_{mw}}$$

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

$\text{Area} = L_w \cdot t_t$

$$= L_w \cdot t_t \frac{f_u/\sqrt{3}}{\gamma_{mw}} = \frac{368 \times 5.6 \times 410/\sqrt{3}}{1.25}$$

$\boxed{f_{wd} = 390.26 \text{ kN}}$

Step:4 - Strength of Plate

IS:800-2007 ; P.NO:32 ; clause: 6.2

$$T_{dg} = \frac{A_g \cdot f_y}{\gamma_{m0}} = \frac{200 \times 10 \times 250}{1.1}$$

$$\boxed{T_{dg} = 454.55 \text{ kN}}$$

$T_{dg} \neq f_{wd}$

Step:5 - Strength of slot weld :-

$$\text{strength of slot weld} = \frac{f_{wn}}{\gamma_{mw}} = \frac{f_u/\sqrt{3}}{\gamma_{mw}}$$

$$= \frac{410/\sqrt{3}}{1.25}$$

$$= 189.37 \text{ N/mm}^2$$

$T_{dg} \neq f_{WD}$

∴ slot weld to be provide to resist a force

of = strength of plate - strength of weld

$$= 454.55 - 390.3$$

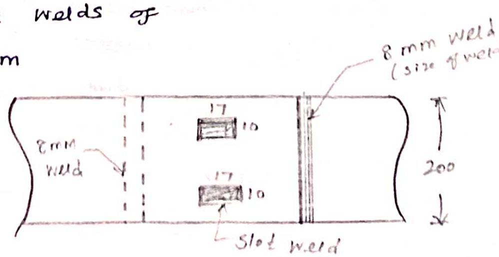
$$= 64.25 \text{ kN.}$$

∴ Area required for slot weld = $\dfrac{\text{strength of slot weld to be provided}}{\text{strength of slot weld}}$

$$= \frac{64.25 \times 10^3 \text{ N}}{189.37}$$

$$\boxed{A_{slot} = 339.5 \text{ mm}^2}$$

∴ Provide two slots welds of size 10mm x 17mm



Problem : 10

A tie member consists of two ISMC 250 (medium channel section). The channels are connected on either side of a 12 mm thick gusset plate. Design the welded joint to develop the full strength of the tie. However the overlap is to be limited to 400 mm.

Given Data:-

For ISMC 250, (From steel table / Hand book SP:6(1) P.NO:6)

thickness of web $t_w = 7.1\ mm$

thickness of flange $t_f = 14.1\ mm$

sectional area, $A = 3867\ mm^2$

Step: 1 - size of weld (S)

Minimum size of weld, $S_{min} = 3\ mm$ (IS:800 - 2007 ; P.NO:78 ; Table: 21) (Based on t_w is less than 10 mm)

Maximum size of weld, $S_{max} = t - 1.5$

$= 7.1 - 1.5$

$S_{max} = 5.6\ mm$

∴ Hence provide $\boxed{S = 4\ mm}$ weld

Step: 2 - Throat thickness of weld :-

P.NO:78 ; Table : 22

$t_t = k \cdot S = 0.7 \times 4$

$$\boxed{t_t = 2.8\ mm}$$

Step: 3 - strength of weld

P.NO:79 ; clause: 10.5.7.1.1

$$f_{wd} = \text{Area} \cdot \frac{f_{wn}}{\gamma_{mw}} = (L_w \cdot t_t) \cdot \frac{(f_u/\sqrt{3})}{\gamma_{mw}}$$

$$= (L_w \times 2.8) \times \left(\frac{410/\sqrt{3}}{1.25}\right)$$

$$\boxed{f_{wd} = 530.24\ L_w}$$

$L_w \rightarrow$ Length of weld

Given Data:-

edge distance $(e) = 50mm$

Vertical distance b/w bolts $(g) = 65mm$

horizontal distance b/w bolts $(p) = 55mm$

IS:800-2007
P.NO:73, Table:19

dia of bolt $(d) = 24mm$

dia of hole $(d_0) = d + \text{clearence} = 24+2 = 26mm$

IS:800-2007
P.NO:30; Table:5

$\gamma_{mb} = 1.25$

Step: 1 : Strength of Bolt in shear (V_{dsb})

IS 800-2007; P.NO:75; clause: 10.3.3

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

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

$n_n \rightarrow$ no. of thread in one portion

$n_n = 1$

$n_s \rightarrow$ no. of shank portion

$n_s = 0$

$f_{ub} \rightarrow$ for 4.6 grade

IS:800-2007
P.NO:13
Table:1

$f_{ub} = 400\ N/mm^2$

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

$$\boxed{A_{nb} = 352.864\ mm^2}$$

$$V_{nsb} = \frac{400}{\sqrt{3}}(1 \times 352.864 + 0)$$

$$V_{nsb} = \frac{400 \times 352.864}{\sqrt{3}}$$

$$\therefore V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{400 \times 352.864}{\sqrt{3} \times 1.25}$$

$$\boxed{V_{dsb} = 65.19\ kN}$$

Step:2 - Strength of bolt in bearing (V_{dpb})

IS:800-2007; P.NO:75; clause: 10.3.4

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

$$V_{npb} = 2.5\, k_b \cdot d \cdot t \cdot f_{ub}$$

k_b

(i) $\frac{e}{3d_0} = \frac{50}{3 \times 26} = 0.641$

(ii) $\frac{p}{3d_0} - 0.25 = \frac{55}{3 \times 26} - 0.25 = 0.455$

(iii) $\frac{f_{ub}}{f_u} = 400/410 = 0.976$

(iv) 1

Least value taken

$$\therefore \boxed{k_b = 0.455}$$

$$V_{dpb} = \frac{2.5 \times 0.455 \times 24 \times 8 \times 400}{1.25}$$

Assume 8 mm Tie bracket plates are used

$$\boxed{V_{dpb} = 71.64 \text{ kN.}}$$

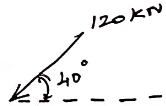
Step: 3 - Bolt value

Bolt value = least value of V_{dsp} & V_{dpb}

= 65.19 & 71.64

$\therefore$ $\boxed{\text{Bolt value} = 65.19 \text{ kN}}$.

Step: 4 - Resultant force (F_r)



$\therefore$ Horizontal force $F_{1h} = 120 \cos\theta$

$= 120 \times \cos 40$

$$\boxed{F_{1h} = 91.93 \text{ kN}}$$

$\therefore$ Vertical force $F_{1V} = 120 \sin 40$

$$\boxed{F_{1V} = 77.14 \text{ kN}}$$

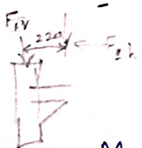
Step: 5 - check for bolt value:-

(a) Direct shear stress (F_1)

$$F_1 = \frac{\text{Vertical force}}{\text{No. of bolt}} = \frac{77.14}{4}$$

$$\boxed{F_1 = 19.28 \text{ kN}}$$

(b) Force in extreme bolt due to bending:



$$F_2 = \frac{M \cdot r_{max}}{\Sigma r^2}$$

load × distance

$$M = F_{1V} \times 220 - F_{1h} \times \left(\frac{65}{2} + 50\right)$$

Step:4 – Strength of plate (T_{dg})

IS: 800 – 2007 ; P.NO: 32 ; Clause: 6.2

Tensile strength of each Channel

$$T_{dg} = \frac{A_g \cdot f_y}{\gamma_{mo}} = \frac{3867 \times 250}{1.1}$$

$$\boxed{T_{dg} = 878.86 \text{ kN}}$$

Step:5 – Length of weld (L_w)

Equating,

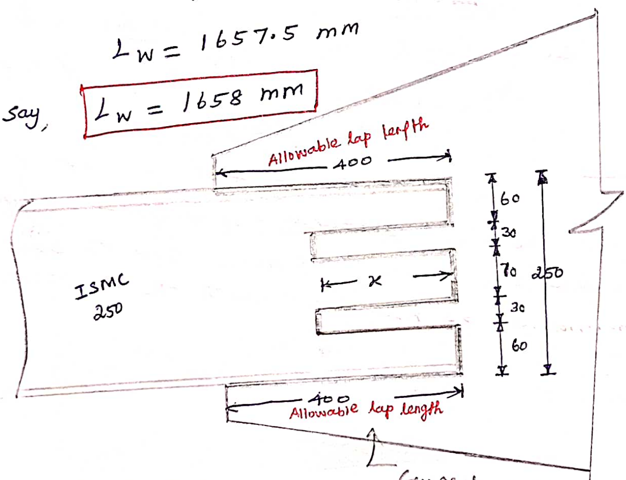
Strength of weld = Strength of plate

$$530.24\, L_w = 878.86 \times 10^3$$

$$\therefore L_w = 1.66 \times 10^3$$

$$L_w = 1657.5 \text{ mm}$$

say, $\boxed{L_w = 1658 \text{ mm}}$



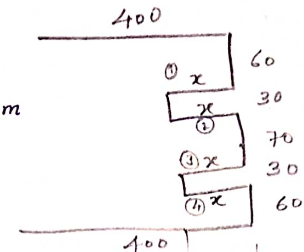
* The allowable length of lap is limited to 400 + 400 mm = 800 mm.

* It needs slot weld.

* Arrangement of slot weld as shown in fig., with two slots of length 'x', then.

$$L_w = 1658 \text{ mm}$$

Lap length limited = 400 mm



ie, $1658 - (400 + 400) = 858$ mm

Total length of weld

$$L_W = 400 + 400 + (250 - 2 \times 30) + 4x$$

$$1658 = 800 + 190 + 4x$$

$$\therefore \quad x = 167 \text{ mm}$$

Say $\boxed{x = 170 \text{ mm}}$ (size of slot)

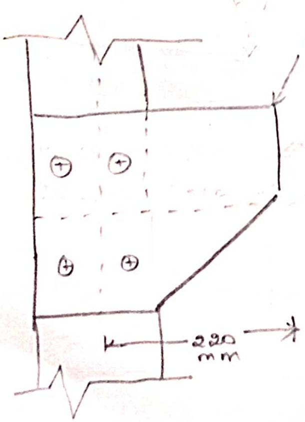
$L_w = 2(b-25)$

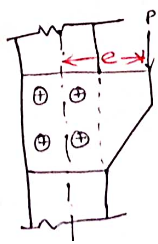
Type : IV

Eccentrically loaded connection.
(When the load lies in the plane of the weld)

Problem: 11

check the saftey of the bolted connection edge distance = 65mm ; horizontal 50mm, vertical distance b/w bolts = 55 mm ; Dia of bolts is 24 mm. distance b/w bolts = 55 mm ; Dia of bolts is 24 mm.
Inclination of the load with the horizontal is 40°.
(May/June 2012, Nov/Dec 2011)

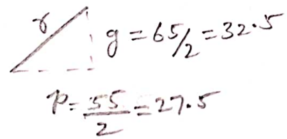




$$= 77.14 \times 10^3 \times 220 - 91.93 \times 10^3 \left(\frac{65}{2} + 50\right)$$

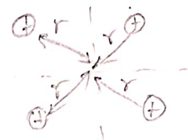
$$\boxed{M = 9.39 \text{ kN.m}}$$

$$r_{max} = \sqrt{\left(\frac{55}{2}\right)^2 + \left(\frac{65}{2}\right)^2} =$$



$$\boxed{r_{max} = 42.57 \text{ mm}}$$

$$\Sigma r^2 = 4 r^2 = 4 \times 42.57^2$$



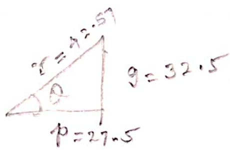
$$\boxed{\Sigma r^2 = 7249.81 \text{ mm}^2}$$

$$F_2 = \frac{M \cdot r_{max}}{\Sigma r^2} = \frac{9.39 \times 10^6 \times 42.57}{7249.81}$$

$\Sigma r = 4r$

$$\boxed{F_2 = 55.12 \text{ kN.}}$$

$$\cos\theta = \frac{27.5}{42.57}$$



$\cos\theta = \frac{\text{Adj}}{\text{hypt.}}$

$\sin\theta = \frac{\text{opp}}{\text{Adj}}$

$$\boxed{\cos\theta = 0.646}$$

Resultant force F_r

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

$$= \sqrt{19.28^2 + 55.12^2 + 2 \times 19.28 \times 55.12 \times 0.646}$$

$$\boxed{F_r = 69.16 \text{ kN.}}$$

$\therefore$ Resultant force F_r > Bolt value.

$\therefore$ Hence it is unsafe.

UNIT - III

TENSION MEMBERS

Types of sections — Net area — Net effective sections for angles and Tee in tension — Design of connections in tension members — use of lug angles — Design of tension splice — concept of shear lag.

Introduction :-

* A tension member is defined as a structural member, subjected to the force in the direction parallel to its longitudinal axis.

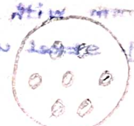
* A tension member is also called as ties (or) hangers.

* Tie ⟶ It is commonly used for the tension members in the roof truss

* Hangers ⟶ It is commonly used for suspension bridges

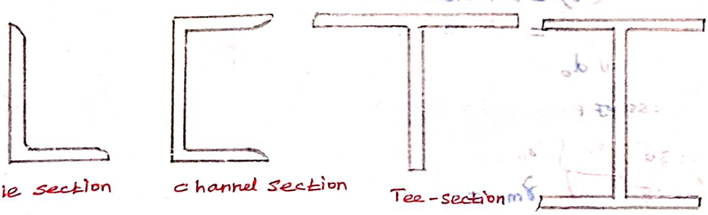
Types of c/s of tension members :-

1. wires and cables



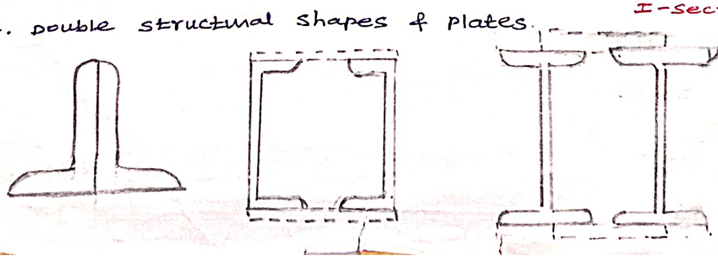
2. Rods & bars

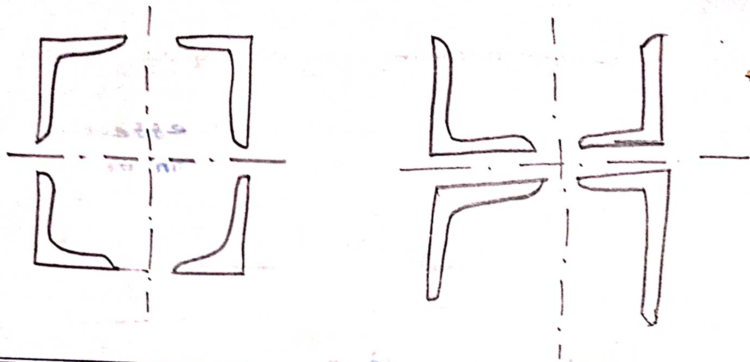
3. single structural shapes & plates.



Angle section   channel section   Tee-section   I-section

4. Double structural shapes & plates.



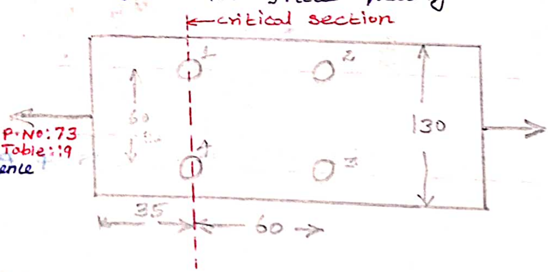


TYPE : 1 – Determination of Tensile strength of the Plate

$$\boxed{\text{factored design Tension } (T) < \text{Design strength of the member } (T_d)}$$

Problem : 1

Determine the design tensile strength of the plate 130 mm x 12 mm with the holes for 16 mm diameter bolts as shown in fig. steel used in of Fe 415 grade quality.



Given Data:-

dia of bolt $(d) = 16\,mm$

dia of holes $(d_o) = d +$ clearence (P.No:73, Table:19)

$= 16 + 2$

d_n (or) $d_o = 18\,mm$

Thickness of plate $(t) = 12\,mm$

P.No: 30, Table:5 ← $\begin{cases} \gamma_{mo} = 1.10 \\ \gamma_{m1} = 1.25 \end{cases}$

$f_y = 250\ N/mm^2$ — Table: 1 (dia of bolt = 16 less than 20; IS: 800-2007; P.No 14)

$f_u = 410\ N/mm^2$ — Table: 1

$f_{ub} = 400\ N/mm^2$ — (P.No: 13 Grade 4.6 Assum)

Step : 1 – Design strength Due to yielding :-

IS: 800 – 2007 ; P.No: 32 ; clause: 6.2

$$T_{dg} = \frac{A_g \cdot f_y}{\gamma_{mo}} = \frac{(130 \times 12) \times 250}{1.10} = 354.55\ kN.$$

$$\boxed{T_{dg} = 354.55\ kN}$$

Step: 2 – Design strength Due to Rupture

For plate

IS : 800 – 2007 ; P. No : 32 ; clause : 6.3.1

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

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

No stagered is given

$$\therefore\ A_n = (b - n d_n)\, t$$

$$= (130 - 2 \times 18)\, 12$$

$$\boxed{A_n = 1128\ mm^2}$$

$$= \frac{0.9 \times 1128 \times 410}{1.25}$$

$$\boxed{T_{dn} = 332.99\ kN}$$

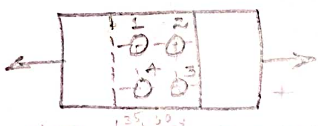
Step: 3 – Design strength Due to Block shear:-

IS : 800 – 2007 ; P. No : 33 ; clause : 6.4.1.

Bolted connection:-

$$T_{db} = \frac{A_{vg} \cdot f_y}{\sqrt{3} \cdot \gamma_{mo}} + \frac{0.9\, A_{tn} \cdot f_u}{\gamma_{m1}}$$

Refer IS : 800 – 2007 ; P. No : 34 ; Figure : 7 A



* shear along bolt line Parallel to the direction of external force

Code book

Min. gross area in shear (1-2 ; 3-4)

Min. gross area in shear along bolt line parallel to external force

$$= [2 \quad l] \times t$$

$$A_{vg} = [2 \times (35 + 60)] \times 12$$

length thickness

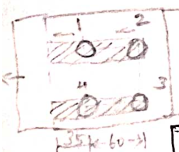


Fig. A

$$= (2 \times 95) \times 12$$

$$\boxed{A_{vg} = 2280\ mm^2}$$

Minimum Net area in shear along bolt line parallel to external force

Net area

length dia of hole thickness

$$A_{tn} = 2 [(35 + 60) - (1.5 \times 18)]\ 12$$

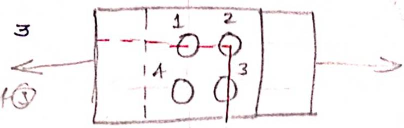
$$\boxed{A_{tn} = 1632\ mm^2}$$

= 1.5 dia of bolt

IS : 800 - 2007 ; P. NO : 34 ; Fig. 7B

Tensile length = Length b/w 2 to 3

= 60 mm



length b/w ② & ③

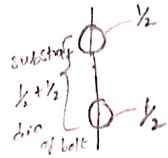
$A_{tg} = 60 \times t = 60 \times 12$

$\boxed{A_{tg} = 720\ mm^2}$

length, no. dia of hole of bolt, thick

$A_{tn} = (60 - 1 \times 18)\, t = (60-18) \times 12$

$\boxed{A_{tn} = 504\ mm^2}$



$$T_{db} = \frac{A_{vg} \cdot f_y}{\sqrt{3} \cdot \gamma_{mo}} + \frac{0.9\, A_{tn} \cdot f_u}{\gamma_{m1}}$$

$$= \frac{2250 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 504 \times 410}{1.25} = 299.17 + 148.78$$

$$\boxed{T_{db} = 447.95\ kN}$$

(or)

$$T_{db} = \frac{0.9\, A_{vn}\, f_u}{\sqrt{3} \cdot \gamma_{m1}} + \frac{A_{tg} \cdot f_y}{\gamma_{mo}}$$

$$= \frac{0.9 \times 1632 \times 410}{\sqrt{3} \times 1.25} + \frac{720 \times 250}{1.1} = 278.15 + 163.64$$

$$\boxed{T_{db} = 441.79\ kN}$$

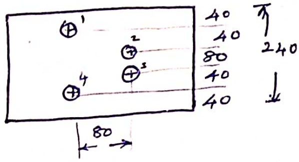
$$T < T_d$$

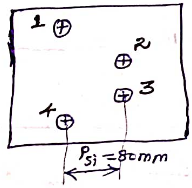
Hence safe.

Design tensile strength of the given joint = 332.99 kN (Rupture)

Problem : 2

A plate 240mm X 10mm has four staggered holes as shown in fig. The dia of hole is 17.50 mm. Locate the critical section and find minimum net area.





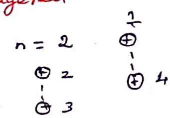
Given Data:-

$$d_o = 17.5 \text{ mm}$$
$$b = 240 \text{ mm}$$
$$t = 10 \text{ mm}$$
$$P_{si} = 80\text{mm}$$

Step : 1

(i) Along section 1-4 & 2-3

No stagered

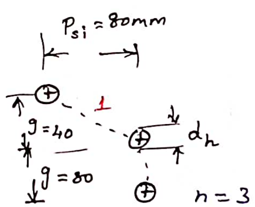


$$A_n = \left[b - n d_n + \cancel{\frac{\Sigma P_{si}^2}{4 g_i}}^{\,0} \right] t$$

$$= \left[240 - (2 \times 17.5) + 0 \right] 10$$

$$\boxed{A_n = 2050 \text{ mm}^2}$$

(ii) Along section 1-2-3 :-



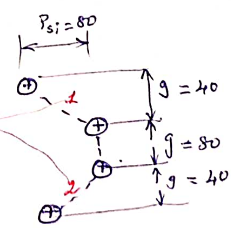
$$A_n = \left[b - n d_n + \frac{\Sigma P_{si}^2}{4 g_i} \right] t$$

inclination leg

$$= \left[240 - (3 \times 17.5) + \frac{1 \times 80^2}{40+80} \right] \times 10$$

$$\boxed{A_n = 2408.33 \text{ mm}^2}$$

(iii) Along section 1-2-3-4 :-



inclination leg.

$$A_n = \left[240 - (4 \times 17.5) + \frac{2 \times 80^2}{(40+80+40)} \right] \times 10$$

$$\boxed{A_n = 2500 \text{ mm}^2}$$

n = 4

Critical section :-

Minimum net area of the section is a critical.

$\therefore$ critical Section 1 - 4 & 2 - 3

Minimum Net area

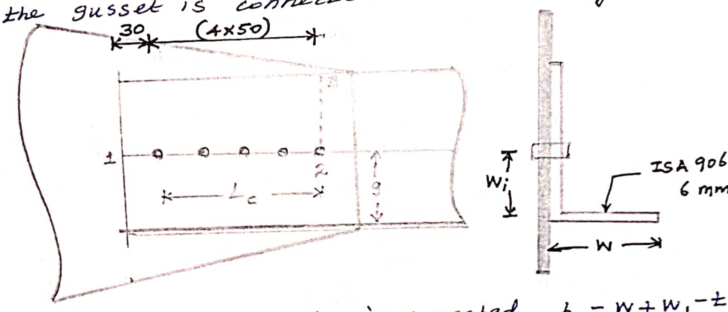
Minimum net area $= 2050\ mm^2$

Problem : 2

A single unequal angle ISA 9060, 6 mm is connected to a 10 mm gusset plates at the ends with 5 nos. of 16 mm bolts to transfer tension as shown in fig. Determine the design tensile strength of the angle.

(a) If the gusset is connected to 90 mm leg

(b) If the gusset is connected to 60 mm leg



$g = 50\ mm$, if 90 mm leg is connected, $b_s = w + w_1 - t$

$g = 30\ mm$; if 60 mm leg is connected.

Solution :-

Given Data

dia of bolt $(d) = 16\ mm$

P.No: 73 ; Table: 17

$\therefore$ dia of hole $(d_o) = 16 + 2 = 18\ mm$

$\gamma_{m0} = 1.1$

$\gamma_{m1} = 1.25$

$f_u = 410\ N/mm^2$

$f_y = 250\ N/mm^2$

$f_{ub} = 400\ N/mm^2$

Properties of ISA 9060, 6 mm

From SP: 6 (1) - 1964

P. NO : 14

Sectional area $A_g = 8.65\ cm^2 = 865\ mm^2$

$t = 6\ mm$

(a) Gusset is connected to 90 mm leg:-

Step: 1 – Design Tensile strength due to yielding

IS: 800–2007 ; P.NO: 32 ; clause: 6.2

$$T_{dg} = \frac{A_g \cdot f_y}{\gamma_{m0}} = \frac{865 \times 250}{1.1}$$

$$\boxed{T_{dg} = 196.59\ kN}$$

Step: 2 – Design Tensile strength Due to Rupture

For single angle section,

∴ IS: 800 – 2007 ; P.NO: 33 ; clause: 6.3.3.

$$T_{dn} = \frac{0.9\, A_{nc}\, f_u}{\gamma_{m1}} + \frac{\beta \cdot A_{go} \cdot f_y}{\gamma_{m0}}$$

$A_{nc} \rightarrow$ Net area of connect. leg

$A_{go} \rightarrow$ Gross area of outstanding leg

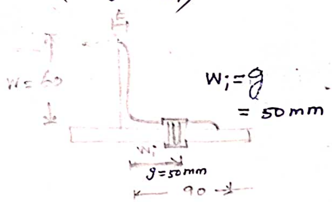
$\beta \rightarrow$

$$\beta = 1.4 - 0.076\,(w/t)\cdot\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L_c}\right) \leq \left(\frac{f_u\,\gamma_{m0}}{f_y \cdot \gamma_{m1}}\right) \geq 0.7$$

P.NO: 33 Fig: 6 $\longrightarrow$ $b_s = w + w_1 - t$

$$= 60 + 50 - 6$$

$$\boxed{b_s = 104\ mm}$$



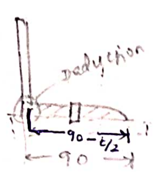
Given Fig. —— $L_c = 4 \times 50 = 200mm$

A_{nc} = Net area of connected leg

$$A_{nc} = \left(90 - \frac{t}{2} - \underset{\text{dia of hole}}{18}\right) t$$

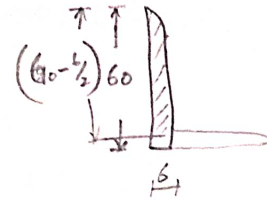
$$= \left(90 - \frac{6}{2} - 18\right) 6$$

$$\boxed{A_{nc} = 414\ mm^2}$$



$A_{go} \rightarrow$ Gross area of outstanding leg.

$$A_{go} = \left(60 - \frac{t}{2}\right) t$$



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

$$\boxed{A_{go} = 342 \ mm^2}$$

$$\beta = 1.4 - 0.076 \left(\frac{60}{6}\right)\left(\frac{250}{410}\right)\frac{104}{200}$$

$$\boxed{\beta = 1.16}$$

$$\frac{f_u \cdot \gamma_{mo}}{f_y \cdot \gamma_{m1}} = \frac{410 \times 1.1}{250 \times 1.25} = 1.44$$

$$\beta < \frac{f_u \cdot \gamma_{mo}}{f_y \cdot \gamma_{m1}} = 1.44 > 0.7$$

Hence it is OK.

$$\therefore \ T_{dn} = \frac{0.9 \ A_{nc} \cdot f_u}{\gamma_{m1}} + \frac{\beta \cdot A_{go} \ f_y}{\gamma_{mo}}$$

$$= \frac{0.9 \times 414 \times 410}{1.25} + \frac{1.16 \times 342 \times 250}{1.1}$$

$$= 122.21 + 90.16$$

$$\boxed{T_{dn} = 212.37 \ kN}$$

Step: 3 – Design strength due to Block shear

IS: 800 – 2007 ; P.NO: 33 ; Clause : 6.4.1

Block shear shall be taken as the smaller of

(i) $T_{db} = \dfrac{A_{vg} \cdot f_y}{\sqrt{3} \cdot \gamma_{m0}} + \dfrac{0.9\, A_{tn} \cdot f_u}{\gamma_{m1}}$

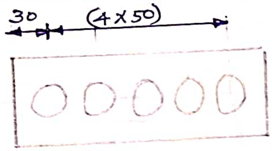
(ii) $T_{db} = \dfrac{0.9\, A_{vn}\, f_u}{\sqrt{3} \cdot \gamma_{m1}} + \dfrac{A_{tg}\, f_y}{\gamma_{m0}}$

P. No: 33
Clause: 6.4.1.

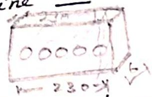
Failure may be taken along section ① – ② – ③

Refer IS: 800 – 2007 P. No: 34 ; Fig: 7A

Shear along bolt line parallel to the direction of external force



A_{vg} = Min. gross area in shear along bolt line ∥ to the direction of external force



$A_{vg} = [30 + (4 \times 50)]\, t = (30 + 200)\, 6 = 1380\ mm^2$

$\boxed{A_{vg} = 1380\ mm^2}$

$A_{vn} = [30 + (4 \times 50) - (4.5 \times d_0)]\, t$

$= [30 + 200 - (4.5 \times 18)] \times 6$

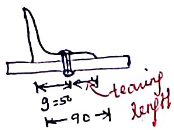
$\boxed{A_{vn} = 894\ mm^2}$

Shear along bolt line perpendicular to the direction of External force.

Tearing length in tension = connected leg distance – gauge distance

$= 90 - 50$

$= 40\ mm$



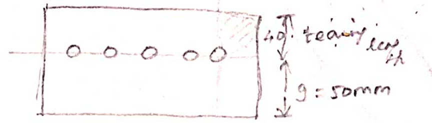
Minimum gross area in shear along bolt line ⊥r to the direction of ext. force

A_{tg} = Tearing length x thickness = 40 x 6

$\boxed{A_{tg} = 240\ mm^2}$

Min. Net area in shear ⊥ to the direction of external force

$$A_{tn} = (\underset{\text{Tearing length}}{40} - 0.5 \times \underset{\text{dia of hole}}{18}) \times \underset{\text{thickness}}{6}$$



$$\boxed{A_{tn} = 186\ mm^2}$$

(i) $$T_{db} = \frac{A_{vg} \cdot f_y}{\sqrt{3} \cdot \gamma_{mo}} + \frac{0.9\, A_{tn}\, f_u}{\gamma_{m_1}}$$

$$= \frac{1380 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 186 \times 410}{1.25} = 181.08 + 54.91$$

$$\boxed{T_{db} = 235.99\ kN}$$

(ii) $$T_{db} = \frac{0.9\, A_{vn} \cdot f_u}{\sqrt{3} \cdot \gamma_{m1}} + \frac{A_{tg} \cdot f_y}{\gamma_{mo}}$$

$$= \frac{0.9 \times 894 \times 410}{\sqrt{3} \times 1.25} + \frac{240 \times 250}{1.10} = 152.37 + 54.55$$

$$\boxed{T_{db} = 206.8\ kN}$$

Take least value

$$\therefore \boxed{T_{db} = 206.8 kN}$$

Strength of joint

$T_{db} = 206.8 kN$

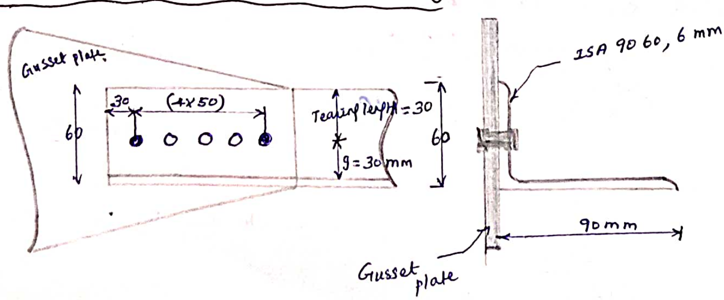
$T_{dn} = 212.37 kN$

$T_{dg} = 196.59 kN$

Take least value of three strength of joint

$\therefore$ Design Strength of the given joint = 196.59 kN (strength due to yielding)

(b) Gusset is connected to 60 mm leg



Step: 1 – Design Tensile strength Due to yielding

P.No: 32
Clause: 6.2

$$T_{dg} = \frac{A_g \cdot f_y}{\gamma_{mo}} = \frac{865 \times 250}{1.1}$$

$$\boxed{T_{dg} = 196.6 \text{ kN}}$$

Step: 2 – Design Tensile strength due to Rupture (Tearing at critical section)

For single Angles,

IS: 800–2007; P.NO: 33; clause: 6.3.3

18 → dia of hole

$$\text{Net Area connected leg } (A_{nc}) = \left(\begin{matrix}\text{length of}\\ \text{connected leg}\end{matrix} - 1 - \frac{t}{2}\right) t$$

$18 - 6/2$

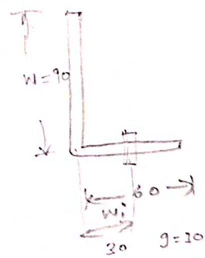
$$= \left(60 - 1 - \frac{6}{2}\right) \times 6 = 39 \times 6$$

$$\boxed{A_{nc} = 234 \text{ mm}^2}$$

$$\text{Gross area of outstanding leg } (A_{go}) = \left(\text{out standing length} - \frac{t}{2}\right) t$$

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

$$\boxed{A_{go} = 522 \text{ mm}^2}$$



$$b_s = W + W_1 - t$$

$$= 90 + 30 - 6$$

$$\boxed{b_s = 114 \text{ mm}}$$

$$L_c = 4 \times 50$$

$$\boxed{L_c = 200 \text{ mm}}$$

P.NO: 33; clause: 6.3.3

$$\beta = 1.4 - 0.076 \left(\frac{W}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L_c}\right) \leq \frac{f_u \cdot \gamma_{mo}}{f_y \cdot \gamma_{m1}} \geq 0.7$$

$$= 1.4 - 0.076 \times \left(\frac{90}{6}\right) \times \left(\frac{250}{410}\right) \times \left(\frac{114}{200}\right) \leq \left(\frac{400 \times 1.1}{250 \times 1.25}\right) \geq 0.7$$

$$\boxed{\beta = 1 \leq 1.44 \geq 0.77}$$

Hence it is ok

$$T_{dn} = \frac{0.9 \cdot A_{nc} \cdot f_u}{\gamma_{m1}} + \frac{\beta \cdot A_{go} \cdot f_y}{\gamma_{m0}}$$

P.No: 33
Clause: 6.3.3

$$= \frac{0.9 \times 342 \times 410}{1.25} + \frac{1 \times 522 \times 250}{1.1}$$

$$= 69.08 + 118.64$$

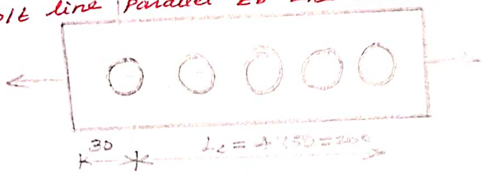
$$\boxed{T_{dn} = 187.72 \text{ kN}}$$

Step: 3 - Design strength due to block shear

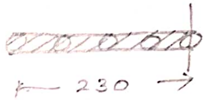
Failure may be taken along critical section ①-②-③ parallel to the direction of external force

Shear along bolt line | Parallel to the direction of external force

P.NO: 34
Fig: 7A



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



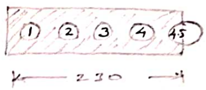
Gross area $\boxed{A_{vg} = 1380 \text{ mm}^2}$

Net area $A_{vn} = (230 - 4.5 \times 18)\, t$

dia of hole

thickness

$$= (230 - 4.5 \times 18)6$$



$$\boxed{A_{vn} = 894 \text{ mm}^2}$$

Shear along bolt line $\perp^r$ to the direction of external force

P.NO: 34, Fig. 7.B

Tearing length in Tension = connected leg distance − gauge distance

$$= 60 - 30$$

$$= 30 \text{mm}$$

Gross area, A_{tg} = Tearing length × thickness = 30×6

$$\boxed{A_{tg} = 180 \text{ mm}^2}$$

Net area

$$A_{tn} = (30 - 0.5 \times 18)\, t$$
$$= (30 - 0.5 \times 18) \times 6$$
$$\boxed{A_{tn} = 126\ mm^2}$$

Block shear strength is the smaller value of the following two values

(i) $$T_{db} = \frac{A_{vg} \cdot f_y}{\sqrt{3} \cdot \gamma_{m0}} + \frac{0.9\, A_{tn} \cdot f_u}{\gamma_{m1}}$$ (IS: 800-2007; P. No: 33; clause: 6.4.1)

2 152.37 + 40.91 193.28 kN

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

1 181.08 + 37.19 218.27 kN

$$= 181.08 + 37.19$$
$$\boxed{T_{db} = 218.27\ kN}$$

187.72 final kN

(ii) $$T_{db} = \frac{0.9\, A_{vn} \cdot f_u}{\sqrt{3} \cdot \gamma_{m1}} + \frac{A_{tg} \cdot f_y}{\gamma_{m0}}$$

$$= \frac{0.9 \times 894 \times 410}{\sqrt{3} \times 1.25} + \frac{126 \times 250}{1.10} = 152.37 + 40.91$$
$$\boxed{T_{db} = 193.28\ kN}$$

∴ Take, Least Value

∴ $$\boxed{T_{db} = 193.28\ kN}$$

Step: 4

Strength of member / Joint

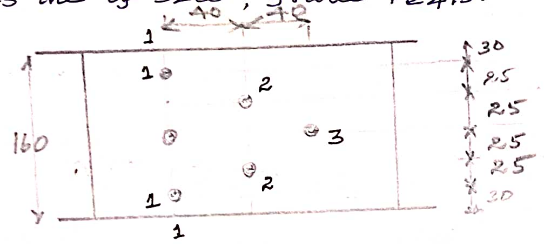
Least value of the following.

$$T_{dg} = 196.6\ kN$$
$$T_{dn} = 187.72\ kN$$
$$T_{db} = 193.28$$

∴ Design strength of member or Joint $= 187.72\ kN$

Problem : 4

Determine the design tensile strength of 160 mm × 8mm plate with the holes for 16 mm bolts as shown in fig. The given plates are of steel, grade Fe415.



Given Data :-

dia of bolt (d) = 16mm

P.NO : 73 ; Table : 19

dia of hole $(d_o) = (d_n)$ = d + clearence = 16 + 2 = 18 mm

P.NO : 30 ; Table : 5 ← $\gamma_{mo} = 1.1$

" ← $\gamma_{m1} = 1.25$

P.NO : 14 (Table : 1) → $f_u = 410\ N/mm^2$

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

Step : 1 - Design Tensile strength due to yieldiding

IS : 800 - 2007 ; P.NO : 32 ; clause : 6.2

$$T_{dg} = \frac{A_g \cdot f_y}{\gamma_{mo}} = \frac{(160 \times 8) \times 250}{1.1}$$

$$\boxed{T_{dg} = 290.91\ KN}$$

Step : 2 - Design strength due to rupture

For plates

IS : 800 - 2007 ; P.NO : 32 ; clause : 6.3.1

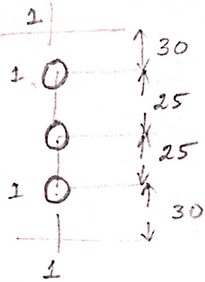
$$T_{dn} = \frac{0.9\ A_n \cdot f_u}{\gamma_{m1}}$$

P.NO : 33

$$A_n = \left[b - n.d_n + \Sigma_i \frac{P_{si}^2}{4g_i} \right] t$$

(The bolts are in stagered)

For section ①-①-①-①



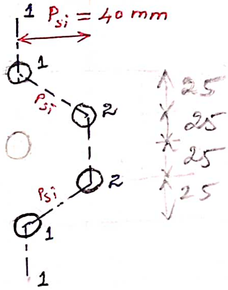
There is no-stagered

$$\therefore A_n = \left(b - \overset{\text{No. of bolt in critical section}}{n} d_n\right) t$$

$$= (160 - 3 \times 18)\, 8$$

$$\boxed{A_n = 848\ mm^2}$$

For section along ①-① - ② - ② - ①-①



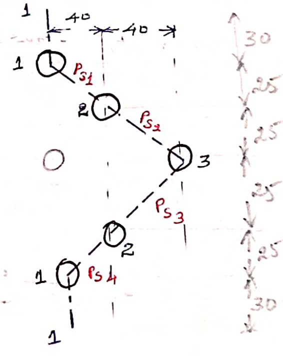
In this section is stagered,

$$\therefore A_n = \left[b - n d_n + \sum \frac{P_{si}^2}{4g_i}\right] t$$

$$= \left[160 - (4 \times 18) + \frac{2 \times 40^2}{4 \times 25}\right] \times 8$$

$$\boxed{A_n = 960\ mm^2}$$

For section along ①-①-②-③-②-①-①



In this section is stagered,

$$A_n = \left[160 - (5 \times 18) + \frac{4 \times 40^2}{4 \times 25}\right] \times 8$$

$$\boxed{A_n = 1072\ mm^2}$$

Net area, least value of three,

$$\therefore \boxed{A_n = 848\ mm^2}$$

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m_1}} = \frac{0.9 \times 848 \times 410}{1.25}$$

$$\boxed{T_{dn} = 250.33\ KN}$$

Step: 3 – Design strength due to Block shear

Note:-

* The bolts are in stagered pattern.
* Therefore, there is no need of checking block shear.
* Hence the design strength of the tension member is taken by the least value of yielding & rupture.

∴ $\boxed{\text{Design Strength of tension Member} = 250.33\ kN}$ Due to rupture

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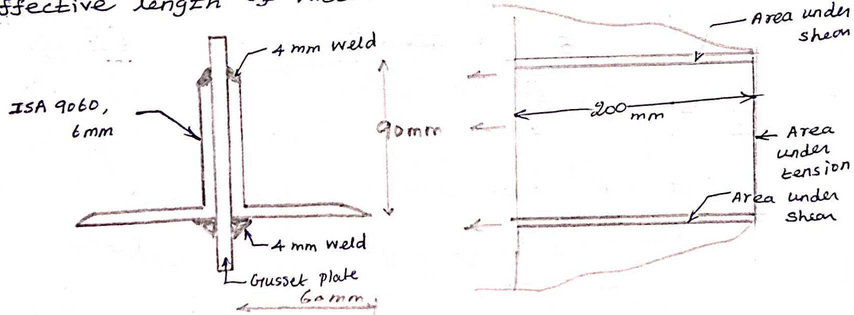
Problem : 5

WELDING. (ANGLE SECTION).

Determine the tensile strength of a roof truss member of 2 ISA 9060, 6mm connected to the gusset plate of 8 mm thickness by 4 mm weld as shown in fig. The effective length of weld is 200mm.



Given Data:-

$\gamma_{mo} = 1.1$

$\gamma_{m1} = 1.25$

$f_u = 410\ N/mm^2$

$f_y = 250\ N/mm^2$

Properties of ISA 9060, 6mm

~~SP(6)-1A~~ SP : 6 (1) - 1964

P.NO : 14

$A_g = 8.65\ cm^2 = 865\ mm^2$

$t = 6\ mm$

Step: 1 – Design Tensile strength due to yielding

IS:800–2007 ; P.No: 32 ; clause: 6.2

$$T_{dg} = \frac{A_g \cdot f_y}{\gamma_{mo}} = \frac{865 \times 250}{1.1}$$

$$\boxed{T_{dg} = 196.59 \text{ kN}}$$

For 2 ISA, $T_{dg} = 2 \times 196.59$

$$\boxed{T_{dg} = 393.18 \text{ kN}}$$

Step: 2 – Design Tensile strength due to Rupture

IS:800–2007 ; P.NO: 33 ; clause: 6.3.3.

single Angles,

$$T_{dn} = \frac{0.9\, A_{nc} \cdot f_u}{\gamma_{m1}} + \frac{\beta \cdot A_{go} \cdot f_y}{\gamma_{mo}}$$

$$\beta = 1.4 - 0.076 \left(\frac{W}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L_c}\right) \leq \left(\frac{f_u \cdot \gamma_{mo}}{f_y \cdot \gamma_{m1}}\right) \geq 0.7$$

$$= 1.4 - 0.076 \times \left(\frac{60}{6}\right) \times \left(\frac{250}{410}\right) \times \left(\frac{60}{200}\right) \leq \left(\frac{410 \times 1.1}{250 \times 1.25}\right) \geq 0.7$$

For fillet weld

P.NO: 33 ; Fig: 6

$b_s = W = 60$ mm

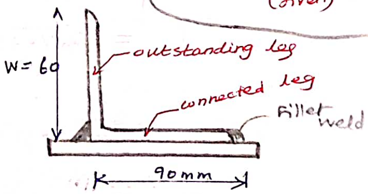
$t = 6$ mm

$L_c = 200$ mm (given)

$$\beta = 1.26 \leq 1.44 \geq 0.7$$

Hence it is ok

$$\therefore \boxed{\beta = 1.26}$$



Net area of connected leg

$$A_{nc} = \left(90 - \frac{t}{2}\right)6 = \left(90 - \frac{6}{2}\right)6$$

$$\boxed{A_{nc} = 522 \text{ mm}^2}$$

Gross area of outstanding leg

$$A_{go} = \left(60 - \frac{t}{2}\right)6 = \left(60 - \frac{6}{2}\right)6$$

$$\boxed{A_{go} = 342 \text{ mm}^2}$$

$$T_{dn} = \frac{0.9 \times 522 \times 410}{1.25} + \frac{1.26 \times 342 \times 250}{1.10}$$

$$= 154.09 + 97.94$$

$$\boxed{T_{dn} = 252.02 \text{ kN}}$$

For 2 ISA, $= 2 \times 252.02$

$$\boxed{T_{dn} = 504.05 \text{ kN}}$$

Step: 3 – Design Tensile strength due to block shear

The bolts are not given, Therefore, no importance of bolt line.

so considered one shear leg & one tension face failure can occur

IS: 800–2007 ; P. No: 33 ; clause: 6.4.1

minimum gross area = $\parallel$ el direction

minimum Net area

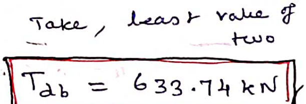
$A_{vg} = A_{vn} = 200 \times 6$

$= 1200 \text{mm}^2$

minimum gross area $\perp$ direction

$A_{tg} = A_{tn} = 90 \times 6$

$= 540 \text{ mm}^2$



(i) $$T_{db} = \frac{A_{vg} \cdot f_y}{\sqrt{3} \cdot \gamma_{m0}} + \frac{0.9\, A_{tn}\, f_u}{\gamma_{m1}}$$

$$= \frac{1200 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 540 \times 410}{1.25} = 316.87 \text{ kN}$$

For 2 ISA $\boxed{T_{db} = 633.74 \text{ kN}}$

(ii) $$T_{db} = \frac{0.9\, A_{vn}\, f_u}{\sqrt{3} \cdot \gamma_{m1}} + \frac{A_{tg} \cdot f_y}{\gamma_{m0}}$$

$$= \frac{0.9 \times 1200 \times 410}{\sqrt{3} \times 1.25} + \frac{540 \times 250}{1.1}$$

$$\boxed{T_{db} = 327.25 \text{ kN}}$$

For 2 ISA $\boxed{T_{db} = 654.5 \text{ kN}}$

Take, least value of two

$$\boxed{T_{db} = 633.74 \text{ kN}}$$

Step: 4 – Design strength of member

② $T_{dn} = 504.05$ kN

① $T_{dg} = 393.18$ kN

③ $T_{db} = 633.74$

Take least value

$\therefore$ $\boxed{\text{Design Strength of Tension member} = 393.18 \text{ kN}}$

Type – II

Design of Tension members

Step: 1 - Find the required gross area to carry the factored load considering the strength in yielding

$$\boxed{A_g = \frac{T_u}{(f_y/\gamma_{mo})} = \frac{1.1\, T_u}{f_y}}$$

$T_u \rightarrow$ factored tensile force

Step: 2 - select suitable shape of the section depending upon the type of the structure & location of the member such that gross area is 25 to 40% more than A_g calculate

Step: 3 – Determine the number of bolts or the welding required & arrange.

$$\text{No. of bolt} = \frac{\text{Strength of joint}}{\text{Bolt value}}$$

Step: 4 - Find the strength considering:

(a) strength in yielding of gross area
(b) strength in rupture of critical section
(c) strength in block shear.

Note:-

* Maintain the minimum edge distance & minimum pitch
* Strength in yielding is the least value, hence the design is safe.
* If A_g provided > A_g required

Step: 5 – The strength obtained should be more than factored tension.

If it is too much on higher side or the strength is less than factored ~~load~~ tension, the section may be suitably changed & checked.

strength obtained > factored tension

Step: 6 – check for slenderness ratio

IS: 800 – 2007

Problem : 1

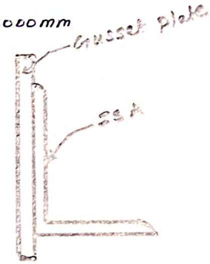
Design a single angle section for a tension member of a roof truss to carry a factored tensile force of 225 KN. The member is subjected to the possible reversal of stress due to the action of wind. The effective length of the member is 3 m. Use 20 mm shop bolts of grade 4.6 for the connection.

Given Data :-

Tensile force T_u = 225 KN

Eff. length = 3m = 3000mm

dia of bolt (d) = 20mm



Single Angle section

4.6 Grade

$f_u = 410\ N/mm^2$

$f_{ub} = 400\ N/mm^2$

$\gamma_{m_1} = 1.25$

$\gamma_{mo} = 1.1$

$\gamma_{mb} = 1.25$

$f_y = 240\ N/mm^2$

Step: 1 - Required Gross area (A_g req)

$$A_{g_{req}} = \frac{T_u}{(f_y/\gamma_{mo})} = \frac{225 \times 10^3\ N}{(240/1.1)}$$

$$\boxed{A_{g_{req}} = 990\ mm^2}$$

Step: 2 - selection of suitable section

Add 25% area to A_g calculation

$$\therefore A_{g_{req}} = 25\%\ \text{of}\ A_{g.req} + A_{g\,req}$$

$$= \frac{25}{100} \times 990 + 990$$

$$\boxed{A_{g_{req}} = 1237.5\ mm^2}$$

From SP:6 (1) - 1964 - P. No: 14

Try ISA 100 75, 8 mm, Gross Area, $\boxed{A_g = 1336\ mm^2}$ Provi

$\therefore A_{g\ provided} > A_{g\ req.}$

$1336\ mm^2 > 1237.5\ mm^2$

Hence it is ok.

Step: 3 - Determination of Number of bolts required

Factored load

$$\text{No. of bolts} = \frac{\text{Strength of joint}}{\text{Bolt value}}$$

$d = 20mm$

IS: 800: 2007 Table: 19, P. No: 73

$\therefore d_o$ (or) $d_n = d + \text{clearence} = 20 + 2 = 22\ mm$

Min. end distance $(e) = 1.7 \cdot d_o = 1.7 \times 22 = 37.4\ mm$

P. No: 74, clause: 10.2.4.2

say $\boxed{e = 40mm}$

Min. spacing (or) pitch $(p) = 2.5\ d = 2.5 \times 20 = 50\ mm$

P. No: 73 Cl: 10.2.2

say $\boxed{p = 60\ mm}$

Use Thickness of gusset plate = 10 mm

Thickness of angle section = 8 mm.

(a) Shearing strength of bolt in shear

IS: 800-2007; P. NO: 75; clause: 10.3.3

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

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

Lap joint, For single angle

$A_{sb} = \frac{\pi \times 20^2}{4} = 314.6\ mm^2$ $n_s = 0$

$n_n = 1$

$A_{nb} = 0.78 \times 314.6 = 245.39\ mm^2$

$$\therefore V_{dsb} = \frac{\frac{400}{\sqrt{3}} (1 \times 245.39 + 0)}{1.25} = \frac{56.67}{1.25}$$

$$\boxed{V_{dsb} = 45.30\ KN}$$

(b) Bearing strength of bolt

IS: 800 – 2007 ; P. No: 75
Clause: 10.3.4

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

$$V_{npb} = 2.5\, K_b\, d \cdot t \cdot f_{ub}$$

K_b least value of the following

(i) $\frac{e}{3d_0} = \frac{40}{3\times 22} = 0.606$

(ii) $\frac{p}{3d_0} - 0.25 = \frac{60}{3\times 22} - 0.25 = 0.66$

(iii) $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.976$

(iv) 1

Take $\boxed{K_b = 0.606}$

$$V_{dpb} = \frac{2.5 K_b \cdot d \cdot t \cdot f_{ub}}{\gamma_{mb}}$$

$$= \frac{2.5 \times 0.606 \times 20 \times 8 \times 400}{1.1}$$

$$V_{dpb} = \frac{97.6}{1.1}$$

$$\boxed{V_{dpb} = 78.08 \text{ kN}}$$

Bolt value

shearing strength of bolt $V_{dsb} = 45.3$ kN
bearing strength of bolt $V_{dpb} = 78.08$ kN

Take least value

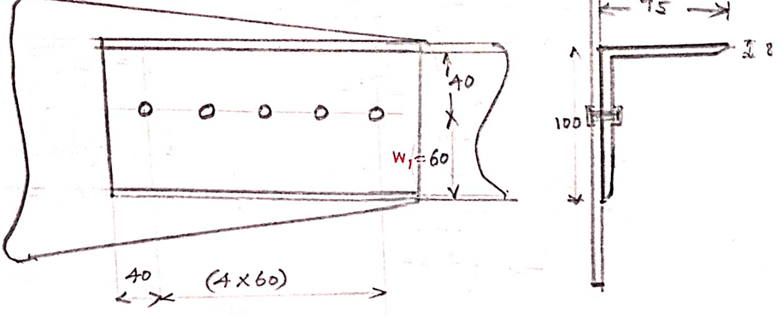
$\therefore \boxed{\text{Bolt Value} = 45.30 \text{ kN}}$

Number of bolt

$$\text{number of bolt} = \frac{\text{strength of joint (or) Factored load}}{\text{Bolt value}} = \frac{225 \times 10^3}{45.3 \times 10^3}$$

$$\boxed{\text{no. of bolt } (n) = 5}$$

Arrangement of Bolt



Step: 4 – Check for Design

(a) Strength due to yielding

P.NO: 32
Clause: 6.2

$$T_{dg} = \frac{A_{g\,pro} \cdot f_y}{\gamma_{mo}} = \frac{1336 \times 240}{1.1}$$

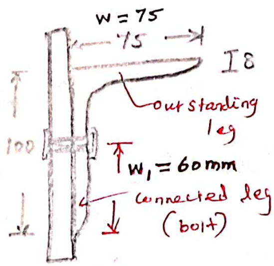
$$T_{dg} = 303.64 \text{ kN} > 225 \text{ kN}$$

Hence it is OK.

(b) Strength of plate in rupture

For single Angle, IS: 800 – 2007; P. NO: 33; Clause: 6.3.3

$$T_{dn} = \frac{0.9\, A_{nc} \cdot f_u}{\gamma_{m1}} + \frac{\beta \cdot A_{go} \cdot f_y}{\gamma_{mo}}$$



Area of connected leg, $A_{nc} = \left(100 - \frac{t}{2} - d_o\right)t$

$$= \left(100 - \frac{8}{2} - 22\right)8$$

$$\boxed{A_{nc} = 592 \text{ mm}^2}$$

Area of outstanding leg, $A_{go} = \left(75 - \frac{t}{2}\right)t = \left(75 - \frac{8}{2}\right)8$

$$\boxed{A_{go} = 568 \text{ mm}^2}$$

P.NO: 33
Cl: 6.3.3

$$\beta = 1.4 - 0.076\left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L_c}\right) \le \left(\frac{f_u \cdot \gamma_{mo}}{f_y \cdot \gamma_{m1}}\right) \ge 0.7$$

$w = 75$ mm ; $t = 8$ mm

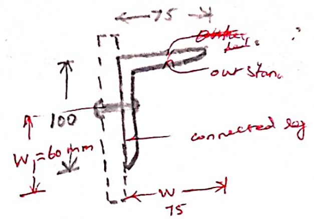
$$b_s = w + w_1 - t = 75 + 60 - 8$$

$$\boxed{b_s = 127 \text{ mm}}$$

$$L_c = 60 \times 4 \text{ nos} = 240 \text{ mm}$$

$$\boxed{L_c = 240 \text{ mm}}$$

$A_{nc} \rightarrow$ Net area of connected leg



$$A_{nc} = \left(100 - \frac{t}{2} - d_o\right) t$$

$$= \left(100 - \frac{8}{2} - 22\right) 8$$

$$\boxed{A_{nc} = 592 \text{ mm}^2}$$

$A_{go} \rightarrow$ Gross area of outstanding leg

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

$$\boxed{A_{go} = 568 \text{ mm}^2}$$

$$\beta = 1.4 - 0.076\left(\frac{W}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L_c}\right) \le \left(\frac{f_u \cdot \gamma_{mo}}{f_y \cdot \gamma_{m1}}\right) \ge 0.7$$

$$= 1.4 - 0.076 \times \left(\frac{75}{8}\right) \times \left(\frac{240}{410}\right) \times \left(\frac{127}{240}\right) \le \left(\frac{410 \times 1.1}{250 \times 1.25}\right) \ge 0.7$$

$$\beta = 1.17 \le 1.44 \ge 0.7$$

Hence it is ok

$$\boxed{\beta = 1.17}$$

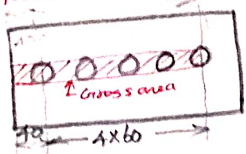
$$T_{dn} = \frac{0.9 A_{nc} f_u}{\gamma_{m1}} + \frac{\beta . A_{go} . f_y}{\gamma_{mo}}$$

$$= \frac{0.9 \times 592 \times 410}{1.25} + \frac{1.17 \times 568 \times 240}{1.1} = 174.76 + 151.0$$

$$\boxed{T_{dn} = 325.8 \text{ kN} > 225 \text{ kN}}$$

(c) strength due to Block shear

IS: 800-2007 ; P.NO: 33 ; clause: 6.4.1

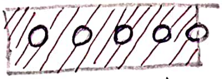


Shear along bolt line parallel to the external force

Min. Gross area, $A_{vg} = (40 + 240) \cdot t = 280 \times 8$

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

Min. Net area, $A_{vn} = (\overset{e}{40} + \overset{L_c}{240} - \overset{\text{No. of bolt in section}}{4.5} \times \overset{\text{dia of hole}}{22})\ \underset{\text{Thickness}}{8}$

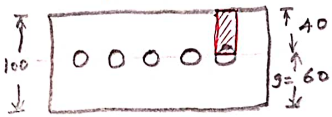


$$\boxed{A_{vn} = 1448\ mm^2}$$

Shear along bolt line perpendicular to the external force

Min. Gross area, $A_{tg} = \overset{\text{Tearing length}}{40} \times \overset{\text{Thickness}}{8}$

$$\boxed{A_{tg} = 320\ mm^2}$$



Min. Net. area, $A_{tn} = (40 - 0.5 \times 22)\ 8$

$$\boxed{A_{tn} = 232\ mm^2}$$

Clause: 6.4.1
P.NO: 33

(i) $T_{db} = \dfrac{A_{vg} \cdot f_y}{\sqrt{3} \cdot \gamma_{mo}} + \dfrac{0.9\ A_{tn} \cdot f_u}{\gamma_{m1}} = \dfrac{2240 \times 250}{\sqrt{3} \times 1.1} + \dfrac{0.9 \times 232 \times 410}{1.25}$

$= 293.92 + 68.49$

$$\boxed{T_{db} = 362.41\ kN} > 225\ kN$$

Hence safe.

(ii) $T_{db} = \dfrac{0.9\ A_{vn} \cdot f_u}{\sqrt{3} \cdot \gamma_{m1}} + \dfrac{A_{tg} \cdot f_y}{\gamma_{mo}} = \dfrac{0.9 \times 1448 \times 410}{\sqrt{3} \times 1.25} + \dfrac{320 \times 250}{1.1}$

$= 246.79 + 72.73$

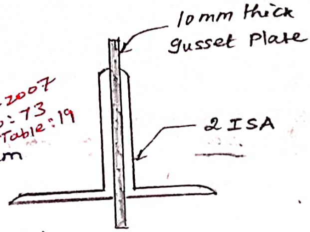
$$\boxed{T_{db} = 319.52\ kN} > 225\ kN$$

Hence it is ok

Problem: 2

Design the double angle tension member connected on each side of a 10mm thick gusset plate to carry an axial factored load of 375 kN. Use 20mm black bolts. Assume shop connection.

Given Data:-



Thickness of gusset plate $= 10\,mm$

Axial factored load $T_u = 375\,kN$

dia of bolt $d = 20\,mm$ (clearance IS: 800-2007 P-NO: 73 Table: 19)

$\therefore$ dia of hole $d_o = d_h = 20 + 2 = 22\,mm$

Table: 1 P-NO: 13 & 14
$f_{ub} = 400\,N/mm^2$
$f_u = 410\,N/mm^2$
$f_y = 240\,N/mm^2$

$\gamma_{mo} = 1.1$
$\gamma_{mb} = 1.25$
$\gamma_{m1} = 1.25$
$\rightarrow$ P. NO: 30 Table: 5

Step: 1 - Required Gross Area

$$A_{g\,cal} = \frac{T_u \cdot \gamma_{mo}}{f_y} = \frac{375 \times 10^3 \times 1.1}{240} = 1718.75\,mm^2$$

Add 25% area to A_g calculation

$$A_{g.\,req} = \frac{25}{100} A_{g\,cal} + A_{g.\,cal} = 1.25 \times 1718.75$$

$$\boxed{A_{g.\,req} = 2148.5\,mm^2}$$

For Double angle section

$$\therefore A_{g\,req} = \frac{2148.5}{2}$$

$$\boxed{A_{g.\,req} = 1074.25\,mm^2}$$

Step: 2 - Selection of suitable section:-

From SP:6(1)-1964 ; P.NO: 14

Try 2 ISA 9060, 8 mm $\qquad A_g = 11.37\,cm^2$

$$\therefore \boxed{A_{g\,provided} = 1137\,mm^2 > A_{g\,req.} = 1074.25\,mm^2}$$

Hence it is ok.

For Double Angle

$$A_{g\,provided} = 2 \times 1137$$

$$\boxed{A_{g\,provided} = 2274\,mm^2}$$

Step: 3 – No. of bolts

$$\text{No. of bolts } (n) = \frac{\text{Factored load}}{\text{Bolt value}}$$

(a) Shearing strength of bolt

IS: 800–2007; P.No: 75; clause: 10.3.3

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{\frac{f_{ub}}{\sqrt{3}}(n_n \cdot A_{nb} + n_s \cdot A_{sb})}{\gamma_{mb}}$$

$$= \frac{\frac{400}{\sqrt{3}}\left[(1 \times 245.04) + (1 \times 314.16)\right]}{1.25}$$

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

For Double Angle

$n_n = 1$

$n_s = 1$

$A_{nb} = 0.78\, A_{sb}$

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

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

$A_{nb} = 0.78 \times 314.16$

$\boxed{A_{nb} = 245.04 \text{ mm}^2}$

(b) Bearing strength of bolt

IS: 800–2007; P.No: 75; clause: 10.3.4

$$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}} = \frac{2.5\, K_b \cdot d \cdot t \cdot f_u}{\gamma_{mb}}$$

K_b is the smaller value of

(i) $\frac{e}{3d_o} = \frac{40}{3 \times 22} = 0.61$

(ii) $\frac{p}{3d_o} - 0.25 = \frac{60}{3 \times 22} - 0.25 = 0.66$

(iii) $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.976$

(iv) 1

$\therefore \boxed{K_b = 0.61}$

$$V_{dpb} = \frac{2.5 \times 0.61 \times 20 \times 8 \times 400}{1.25}$$

$$\boxed{V_{dpb} = 78.08 \text{ KN}}$$

min. end distance (e)

P.No: 74; Cl: 10.2.4.2

$e = 1.7\, d_o$

$= 1.7 \times 22$

$=$

say $\boxed{e = 40 \text{mm}}$

min. Pitch

P.No: 73; clause: 10.2.2

$p = 2.5\, d$

$= 2.5 \times 20$

$= 50$

say $\boxed{p = 60 \text{mm}}$

Take Least value of (a) & (b)

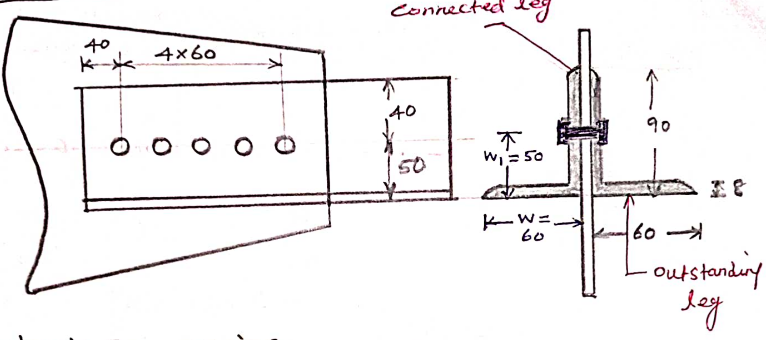
$\therefore \boxed{\text{Bolt value} = 78.08 \text{ KN}}$

No. of bolts :-

$$n = \frac{\text{Factored load}}{\text{Bolt value}} = \frac{375}{78.08}$$

say $\boxed{n = 5 \text{ bolts}}$

Arrangements of Bolt



Step: 4 – check for Design

(a) strength due to yielding

IS:800–2007; P.NO: 32; Clause: 6.2

$$T_{dg} = \frac{A_g \cdot f_y}{\gamma_{mo}} = \frac{2274 \times 240}{1.1}$$

$$\boxed{T_{dg} = 496.15\ kN} > 375\ kN$$

Hence ok.

(b) strength due to rupture of critical section

IS:800–2007; P.NO: 33; Clause: 6.3.3

$$T_{dn} = \frac{0.9\, A_{nc} \cdot f_u}{\gamma_{m1}} + \frac{\beta \cdot A_{go} \cdot f_y}{\gamma_{mo}}$$

$$\beta = 1.4 - 0.076\left(\frac{W}{t}\right)\left(\frac{f_y}{f_u}\right)\cdot\left(\frac{b_s}{L_c}\right) \leq \left(\frac{f_u \cdot \gamma_{mo}}{f_y \cdot \gamma_{m1}}\right) \geq 0.7$$

$t = 8mm$
$W = 60\ mm$
$W_1 = 50\ mm$
$b_s = W + W_1 - t$
$= 60 + 50 - 8$
$\boxed{b_s = 102\ mm}$
$L_c = 4 \times 60$
$\boxed{L_c = 240mm}$

$$= 1.4 - 0.076 \times \left[\frac{60}{8} \times \frac{240}{410} \times \frac{102}{240}\right] \leq \frac{410 \times 1.1}{240 \times 1.25} \geq 0.7$$

$$\beta = 1.26 \leq 1.50 \geq 0.7$$

Hence it is ok

$$\therefore \boxed{\beta = 1.26}$$

Net area of connected leg, $A_{nc} = 2\left(90 - \frac{t}{2} - d_o\right) t$

$$= 2\left(90 - \frac{8}{2} - 22\right) 8$$

$$\boxed{A_{nc} = 1024\ mm^2}$$

Gross area of outstanding leg, $A_{go} = 2\left(60 - \frac{t}{2}\right) t$

$= 2\left(60 - \frac{8}{2}\right) 8$

$$\boxed{A_{go} = 896 \text{ mm}^2}$$

$$T_{dn} = \frac{0.9 \times 1024 \times 410}{1.25} + \frac{1.26 \times 896 \times 240}{1.1}$$

$$\therefore \boxed{T_{dn} = 548.6 \text{ kN}} > 375 \text{ kN}$$

Hence it is ok.

(c) Strength due to block shear

IS: 800-2007; P.No: 33; clause: 6.4.1.

Least value of the following two values

(i) $T_{db} = \frac{A_{vg} \cdot f_y}{\sqrt{3} \cdot \gamma_{mo}} + \frac{0.9 A_{tn} \cdot f_u}{\gamma_{m1}}$

$= \frac{2240 \times 240}{\sqrt{3} \times 1.10} + \frac{0.9 \times 232 \times 410}{1.25}$

$\boxed{T_{db} = 350.66 \text{ kN}}$

For Double angle, $\boxed{T_{db} = 701.32 \text{ kN}} > 375 \text{kN}$

Hence ok

(ii) $T_{db} = \frac{0.9 A_{vn} f_u}{\sqrt{3} \cdot \gamma_{m1}} + \frac{A_{tg} \cdot f_y}{\gamma_{mo}}$

$= \frac{0.9 \times 1448 \times 410}{\sqrt{3} \times 1.25} + \frac{320 \times 240}{1.10}$

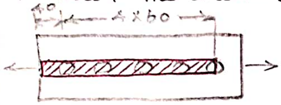
$\boxed{T_{db} = 316.61 \text{ kN}}$

For double angle, $\boxed{T_{db} = 633.22 \text{ kN}} > 375 \text{kN}$

Hence ok.

$\therefore$ Use 2 ISA 90×60×8 mm with 5 bolts of 20 mm dia,

Shear along bolt line Parallel to the external force



Min. Gross area,

$A_{vg} = (40 + 240) t$

$= 280 \times 8$

$\boxed{A_{vg} = 2240 \text{ mm}^2}$

Min. Net area

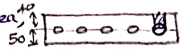
$A_{vn} = (40 + 240 - 4.5 \times 22) 8$ — No. of bolt, dia of hole, thick

$\boxed{A_{vn} = 1448 \text{ mm}^2}$

4.5 holes

Shear along bolt line $\perp^r$ to the external force.

Min. Gross Area,



$A_{tg} = 40 \times 8 = 320 \text{ mm}^2$

Min. Net area

$A_{tn} = (40 - 0.5 \times 22) 8$

$A_{tn} = 232 \text{ mm}^2$

Type – III
Design of Tension Member Splice

* If a single piece of required length is not available tension members are spliced to transfer required tension from one Piece to another.

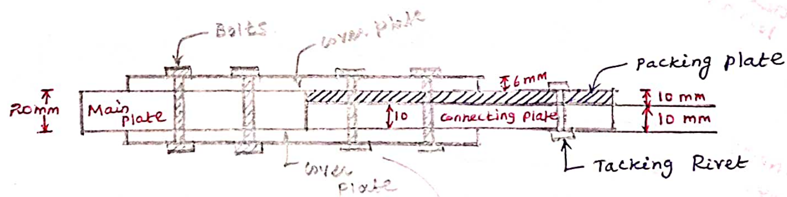
* The strength of the splice plates and the bolts/weld connecting them should have strength at least equal to the design load.

* When tension members of different thickness are to be connected, filler plates may be used to bring the members in level.

* The design shear capacity of bolts carrying shear through a packing plate in excess of 6mm shall be decreased by a factor. (IS: 800-2007 ; P.NO: 75 ; clause: 10.3.3.3)

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

$t_{pk} \rightarrow$ ~~thickness of~~ thickener packing Plate



Problem :-

Design a splice to connect a 300 x 20 mm plate with a 300 x 10 mm plate. The design load is 500 kN. Use 20 mm black bolts, fabricated in the shop.

Given Data:-

size of main plate = 300 x 20 mm

connecting plate size = 300 x 10 mm

Packing plate thickness required = Main plate thick – connecting Plate thick

= 20 – 10

= 10 mm.

Design load = 500 kN

dia of bolt $(d) = 20mm$ IS:800-2007; P.N: 73 Table: 19

dia of hole. d_h (or) $d_o = 20+2 = 22mm$

Assume, a Double cover butt joint of 6 mm cover plate to be used in top & bottom.

... **No. of bolts:-**

$$\text{No. of bolts} = \frac{\text{Factored (or) design load}}{\text{Bolt value}}$$

Step: 1 - Strength of bolt:-

(a) Shearing strength of bolt

* Packing plates are used in this connection
* The thickness of packing plate is more than 6mm.

IS:800 2007 P.NO: 75 Clause: 10.3.3 ∴ The strength will be decreased by (β_{pk}).

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

$$= 1 - 0.0125 \times 10$$

$$\boxed{\beta_{pk} = 0.875}$$

P.NO: 75 Clause: 10.3.3

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

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

$$= \frac{400}{\sqrt{3}}(1 \times 245.04 + 1 \times 314.16)$$

For Double cover butt Joint ∴ $n_n = 1$, $n_s = 1$

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

$A_{nb} = 0.78 A_{sb} = 245.04 mm^2$

$$\boxed{V_{nsb} = 129.14 kN}$$

$$V_{dsb} = \frac{0.875 \times 129.14}{1.25}$$

$$\boxed{V_{dsb} = 90.4 kN}$$

(b) Bearing strength of bolt

IS:800-2007; P.NO: 75 clause: 10.3.4

$$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}} = \frac{2.5 k_b . d . t . f_{ub}}{\gamma_{mb}}$$

k_b least value of following

(i) $\frac{e}{3d_o} = \frac{40}{3 \times 22} = 0.61$

(ii) $\frac{p}{3d_o} - 0.25 = 0.659$

(iii) $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.976$

(iv) 1

P.NO: 74 10.2.4.2

$e = 1.7 d_o$

$= 1.7 \times 22$

say $\boxed{e = 40mm}$

P.NO: 73; 10.2.2

$p = 2.5d$

$= 2.5 \times 20$

$= 50$

say $\boxed{p = 60mm}$

$$\therefore \boxed{k_b = 0.61}$$

$$\therefore V_{dpb} = \frac{2.5 k_b . d . t . f_{ub}}{\gamma_{mb}}$$

$$= \frac{2.5 \times 0.61 \times 20 \times 10 \times 400}{1.25}$$

$$\boxed{V_{dpb} = 97.6 kN}$$

∴ Bolt value is lesser value of ⓐ & ⓑ

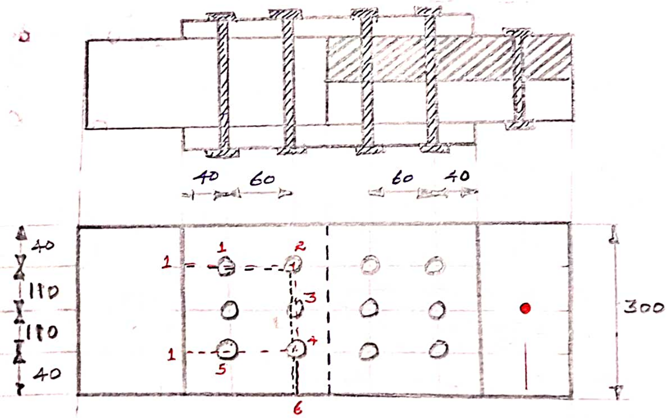
∴ $\boxed{\text{Bolt value} = 90.4 \text{ kN}}$

Step: 2 — No. of bolt :-

$$\text{No. of bolt} = \frac{\text{Factored load}}{\text{Bolt value}} = \frac{500}{90.4} = 5.5 \simeq 6$$

∴ $\boxed{\text{No. of bolts} = 6 \text{ Nos}}$

Step: 3 — Arrangement of Bolts



Step: 4 — Check for strength (Design)

(a) strength due to yielding IS: 800-2007 ; P.NO: 32 ; clause: 6.2

$$T_{dg} = \frac{A_g \cdot f_y}{\gamma_{m0}} = \frac{(300 \times 10) \times 240}{1.10} = 654.55 \text{ kN}$$

$\boxed{T_{dg} = 654.55 \text{ kN} > 500 \text{ kN}}$ Hence ok.

(b) strength due to rupture of critical section

IS: 800-2007 ; P.NO: 32 ; clause: 6.3.1

For Plate

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

P.NO: 33

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

The bolt is donot connected in stagered pattern

$$\therefore A_n = (b - n d_n) t = (300 - 3 \times 22)\, 10$$

$$\boxed{A_n = 2340\ mm^2}$$

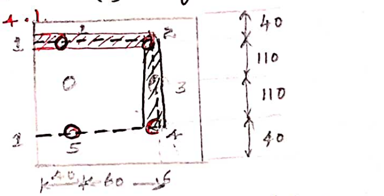
$$\therefore T_{dn} = \frac{0.9\, A_n \cdot f_u}{\gamma_{m_1}} = \frac{0.9 \times 2340 \times 410}{1.25}$$

$$\boxed{T_{dn} = 690.77\ kN > 500\ kN}$$ Hence ok.

(c) Strength due to Block shear

IS : 800 – 2007 ; P. NO : 33 ; clause : 6.4.1

(a) Along 0-①-②-④-⑤-⑥



Bolt line parallel to external force

$A_{vg} = 2(40+60) \times 10 = 2000\ mm^2$ (10 = Thickness)

$A_{vn} = 2(40+60 - 1.5 \times 22) \times 10 = 1340\ mm^2$ (22 = dia of hole)

Bolt line $\perp^r$ to the external force

$A_{tg} = 220 \times 10 = 2200\ mm^2$

$A_{tn} = (220 - 2 \times 22)\,10 = 1760\ mm^2$

(i) $$T_{db} = \frac{A_{vg} \cdot f_y}{\sqrt{3} \cdot \gamma_{mo}} + \frac{0.9\, A_{tn} \cdot f_u}{\gamma_{m_1}}$$

$$= \frac{2000 \times 240}{\sqrt{3} \times 1.1} + \frac{0.9 \times 1760 \times 410}{1.25}$$

$$= 781.9\ kN$$

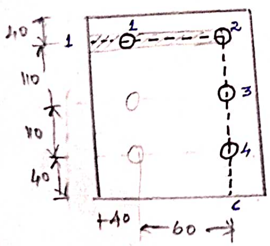
$$\boxed{T_{db} = 781.9\ kN > 500\ kN}$$ Hence ok

(ii) $$T_{db} = \frac{0.9\, A_{vn} \cdot f_u}{\sqrt{3} \cdot \gamma_{m_1}} + \frac{A_{tg}\, f_y}{\gamma_{mo}}$$

$$= \frac{0.9 \times 1340 \times 410}{\sqrt{3} \times 1.25} + \frac{2200 \times 240}{1.1}$$

$$\boxed{T_{db} = 728.38\ kN > 500\ kN}$$ Hence ok

Section Along 0-①-②-③-④-⑥



Bolt line parallel to ext. force

$A_{vg} = (60+40) \times 10 = 1000\ mm^2$

$A_{vn} = (60+40 - 1.5 \times 22) \times 10$

$A_{vn} = 670\ mm^2$

Bolt line $\perp^r$ to ext. force

$A_{tg} = (110+110+40) \times 10$

$= 2600\ mm^2$

$A_{tn} = (260 - 2.5 \times 22) \times 10$

$= 2050\ mm^2$

(i) $$T_{db} = \frac{A_{vg} \cdot f_y}{\sqrt{3} \cdot \gamma_{mo}} + \frac{0.9\, A_{tn} \cdot f_u}{\gamma_{m_1}} = \frac{1000 \times 240}{\sqrt{3} \times 1.1} + \frac{0.9 \times 2050 \times 410}{1.25}$$

$$\boxed{T_{db} = 758.8\ kN > 500\ kN}$$ Hence ok.

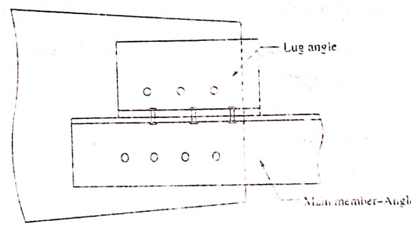
(ii) $$T_{db} = \frac{0.9\, A_{vn}\, f_u}{\sqrt{3} \cdot \gamma_{m_1}} + \frac{A_{tg} \cdot f_y}{\gamma_{mo}} = \frac{0.9 \times 670 \times 410}{\sqrt{3} \times 1.25} + \frac{2600 \times 240}{1.1}$$

$\boxed{T_{db} = 705.1\ kN > 500\ kN}$ Hence o k.

∴ Provide 6 nos of bolts in 20 mm dia & extra bolt in the cover plate on packing material.

Type-IV – LUG ANGLES

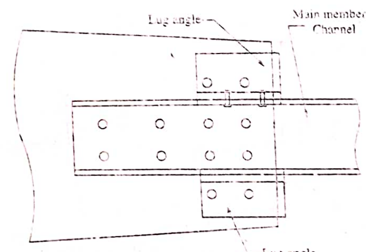
* Length of end connection of a heavily loaded tension member may be reduced by using lug angles.



(a)

* Using Lug angles there will be saving in gusset Plate, but it provide additional fasteners & angle required.

* So, now a days it is not used.



(b)

* IS: 800-2007, specifications for Lug angles are calculated, as P NO: 83, Clause: 10.12

* connection of lug angles, minimum two bolt, rivets or welds to be attached with gusset plate.

Design Lug angle

I → Design of main angle

* Selection of section using $A_{g_{req}} = \frac{\text{Factored load } (T_u)}{f_y/\gamma_{mo}}$
* Connection b/w main angle & gusset plate

Bolt value IS: 800-2007, P.NO: 83 $T_m = \frac{T_u}{2}$

* Shearing capacity of bolt
* bearing capacity of bolt

P.NO: 33 $A_n = (b_1 + b_2 - \gamma_2 - d_h)t$

* No. of bolt
* Arrangement of bolt
* check for yield strength
* check for Rupture $> T_u$ given

II. Design of lug angle

* Load on Lug angle = Load on main angle

$T_L = T_u$.

$T_{LB} = 1.2 T_u$

* Selection of section

$A_{req} = \frac{T_{LB}}{f_y/\gamma_{mo}}$

* connection

III. Design of connection b/w Lug & main angle

Problem: 1

A tension member of a roof truss carries a factored axial tension of 430 kN. Design the section and its connection

(a) without using lug angles

(b) using lug angles.

Given Data:-

Factored axial tension = 430 kN.

$\gamma_{mo} = 1.1$

$\gamma_{m1} = 1.25$

$\gamma_{mb} = 1.25$

$f_{ub} = 400\ N/mm^2$

Assume 4.6 grade bolt

Assume 20 mm dia bo

$\therefore d_o = 20 + 2 = 22mm$

$d = 20\ mm$

$f_y = 240\ N/mm^2$

$f_u = 410\ N/mm^2$

(a) without using lug angle

Step: 1 - Gross area required:-

$$A_{g.req} = \frac{T_u \cdot \gamma_{mo}}{f_y} = \frac{430 \times 10^3 \times 1.1}{240} = 1892\ mm^2$$

$$\boxed{A_{g\ req} = 1892\ mm^2}$$

25% extra $A_{g\ req} = 2462\ mm^2$

Step: 2 - select suitable section

From SP:6(D-1964; P.NO: 10 (Equal angle section)

Try, ISA 125×100/100, 10 mm Angle section,

$A_g = 1903\ mm^2$

$$\therefore \boxed{A_{g\ provided} = 1903\ mm^2 > A_{g\ req}}$$ Hence safe

Step: 3 - Bolt value

(a) shearing strength of bolt · IS:800-2007; P.NO: 75; clause: 10.3.

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{\frac{f_{ub}}{\sqrt{3}}(n_n \cdot A_{nb} + n_s \cdot A_{sb})}{\gamma_{mb}}$$

$$= \frac{\frac{400}{\sqrt{3}}(1 \times 0.78\ \pi \times 20^2/4 + 0)}{1.25}$$

$$\boxed{V_{dsb} = 45.30\ kN}$$

For single angle

$\therefore n_n = 1$

$n_s = 0$

$A_{sb} = \pi \times 20^2/4$

$A_{nb} = 0.78\ A_{sb}$

(b) Bearing strength of bolt   IS : 800 – 2007 ; P. No : 75 ; clause : 10.3.4

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

$e_{min} = 1.5\, d = 1.5 \times 20$
$= 30\ mm$

$P_{min} = 2.5\, d = 2.5 \times 20$
$= 50\ mm$

$$V_{dpb} = 2.5\, k_b \cdot d \cdot t \cdot \frac{f_{ub}}{\gamma_{mb}}$$

$$= 2.5 \times 0.4545 \times 20 \times 10 \times \frac{400}{1.25}$$

$$V_{dpb} = 72.72\ kN$$

k_b — Take least value

(i) $\frac{e}{3 d_0} = \frac{30}{3 \times 22} = 0.4545$

(ii) $\frac{p}{2 d_0} - 0.25 = \frac{50}{2 \times 22} - 0.25 = 0.68$

(iii) $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.98$

(iv) 1

$\therefore k_b = 0.4545$

Take least value.

$\therefore$ $\boxed{\text{Bolt value} = 45.30\ kN}$

Note:- In case single shear, bolt value is usually governed by value in single shear.

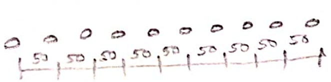
No. of bolt $= \frac{430}{45.3} = 9.57 \simeq 10$ nos

(a) connection without Lug angle

(i) No. of bolt required $= \frac{\text{Factored load}}{\text{Bolt value}} = \frac{430 \times 10^3}{45.3 \times 10^3}$

$= 9.5$ nos

Provide 10 bolts.



(ii) Length of connection,

$L_c = 9 \times 50 = 450\ mm$

(iii) $15\, d = 15 \times 20 = 300\ mm$

(iv) Check

$L_c > 15 d$

$450\ mm > 300\ mm$

It is long connection

(v) $\beta_{lj} = 1.075 - 0.005 \times \frac{L_j}{d}$

IS:800-2007
P. No: 75
Cl: 10.3.3.1

$= 1.075 - 0.005 \times \frac{450}{20}$

$= 0.9625$

(vi) shear strength of bolt (after reducing for long connection)

$= \beta_{lj} \times$ Bolt value

$= 0.9625 \times 45.3$

$= 43.57$ kN.

(vii) No. of bolts required $= \frac{\text{Factored load}}{\text{Shear strength of Bolt}}$

$= \frac{430 \times 10^3}{43.57 \times 10^3} = 9.87 < 10$ provided

Hence 10 bolts are sufficient

Strength due to Yielding

(viii) Yield strength $= \frac{A_g \cdot f_y}{\gamma_{mo}} = \frac{1903 \times 250}{1.1}$

P.No: 32, Cl: 6.2

$= 432.5$ kN > 430 kN.

Hence ok.

Rupture Strength

$T_{dn} = \frac{0.9 A_{nc} f_u}{\gamma_{m1}} + \beta \cdot A_{go} \cdot f_y / \gamma_{mo}$

P.No: 33
Cl: 6.3.3

$A_{nc} = \left(100 - \frac{10}{2} - 22\right) 10 = 730 \text{ mm}^2$

$b_s = w + w_1 - t$
$= 100 + 50 - 10$
$= 140$

$A_{go} = \left(100 - \frac{10}{2}\right) 10 = 950 \text{ mm}^2$

$\beta = 1.4 - 0.076 \left(\frac{95}{10}\right) \times \frac{250}{410} \times \frac{130}{450} = 1.2728 > 0.7$ &

$< \frac{f_u \cdot \gamma_{mo}}{f_y \cdot \gamma_{m1}}$

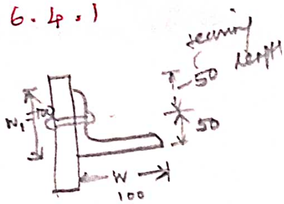
$\beta = 1.2728$

Strength in rupture $= \frac{730 \times 0.9 \times 410}{1.25} + \frac{1.2728 \times 950 \times 250}{1.1}$

$= 490.305$ kN > 430 kN

Hence ok

Block shear Strength — P.No: 33, Cl: 6.4.1



$$A_{vg} = (450+30)\,10 = 4800\ mm^2$$

$$A_{tg} = \overset{50}{70} \times 10 = 700\ mm^2$$

$$A_{vn} = (480 - 9.5 \times 22)\,10 = 2710\ mm^2$$

$$A_{tn} = \left(70 - \frac{22}{2}\right) \times 10 = 490\ mm^2$$

$$\text{Block shear Strength} = \frac{4800 \times 250}{\sqrt{3} \times 1.1} + \frac{490 \times 0.9 \times 410}{1.1} = 774.49\ kN$$

(or)

$$= \frac{0.9 \times 2710 \times 410}{\sqrt{3} \times 1.25} + \frac{700 \times 0.9 \times 250}{1.1} = 605.06\ kN$$

∴ Take least value

∴ Block shear Strength = 605.06 kN

∴ Strength of angle = 432.5 kN > 430 kN Hence OK.

(b) Connection with Lug angle

(i) Load on connected leg

Gross area of connected leg = Gross area of outstanding leg

∴ Load is shared equally

ie, Load in outstanding leg = load in connected leg

$$= \frac{430}{2} = 215\ kN$$

* ∴ Lug angle is to be designed to take a load of atleast 20% more than the force Main plate load

$$= 1.2 \times 215 = 258\ kN.$$

Gross area of Lug angle required $= \dfrac{\overset{\text{load}}{258 \times 1000}}{(250/1.1)} = 1135\ mm^2$ (converted into Newton; f_y, γ_{m0})

SP:6(1)-1964 — Provide ISA 100100, 6 mm

$$\therefore A_{g\,provided} = 1167\ mm^2$$

$\therefore$ The strength of lug angle in rupture =

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

$$= \frac{0.9 \times \overbrace{(100+100-10-22)}^{A_n}\, 6 \times 410}{1.25}$$

$$= 297.56 \text{ kN} > 258 \text{ kN}$$

Hence ok.

Bolt Value :-

(i) In single shear = 45.27 kN

(ii) In bearing $= \dfrac{2.5 K_b \cdot d \cdot t \cdot f_{ub}}{\gamma_{m1}}$

$$= \frac{2.5 \times 0.4545 \times 20 \times 6 \times 400}{1.25}$$

$$= 43.63 \text{ kN}$$

Take least value $\therefore$ Bolt value = 43.63 kN.

No. of bolts

$$n_{req} = \frac{\text{factored load}}{\text{Bolt value}} = \frac{258}{43.63} = 5.91$$

Provide 6 - nos bolt.

Connection of Main angle to gussetplate

Design force for connected leg $= 1.4 \times 215$

40% extra $\therefore$ No. of bolts requied to connect lug angle with main angle

$$= \frac{1.4 \times 215}{43.63} = 6.89$$

Provide 7 bolts.

No. of bolts for Connection of main angle to gusset plates.

Force to be transfered = 215 kN

Bolt value ϕ = 45.27 kN.

$$\text{No. of bolt} = \frac{215}{45.27} = 4.75$$

Provide 5 nos.

$$n = \frac{T_{Lm}}{\text{bolt value}}$$

$$T_{Lm} = 1.4\, T_{LL}$$

Required length of Gusset plate

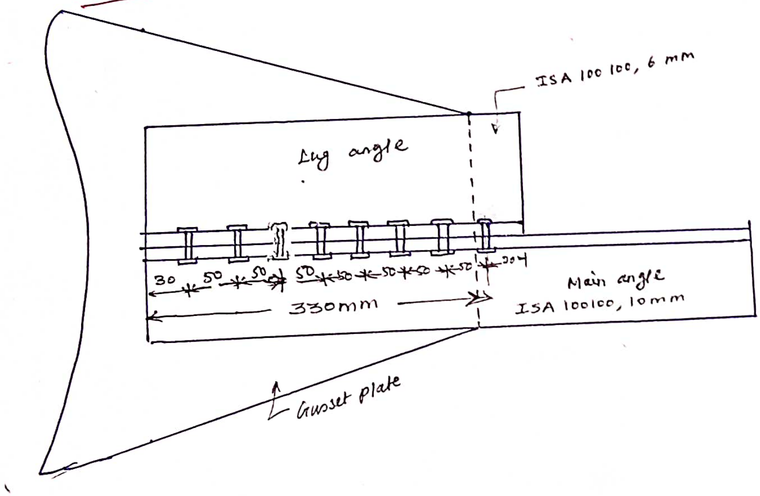
$$= g + (7-1) \times p$$

$$= 30 + (7-1) \times 50)$$

$$= 330 \text{ mm}$$ (compared to 480 mm required without lug angle)

(Block shear strength may be checked)

Connection detail



Lug angles

2) A roof truss member carries a tensile load of 300 kN. Design the section and its connection using Lug angles.

Step: 1 – Given Data

Tensile load $(T) = 300\,kN$

$\therefore$ Ultimate load $(T_u) = 1.5T = 450\,kN$

Assume

Fe410 steel grade & M20, 4.6 bolts.

$f_u = 410\,N/mm^2$ } – Table: 1 – P.NO: 14

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

$f_{ub} = 400\,N/mm^2$ – Table 1 – P.NO: 13

$\gamma_{m0} = 1.1$ } P.NO: 30 Table: 5

$\gamma_{m1} = 1.25$

$\gamma_{mb} = 1.25$

I. Design of main angle

a) Selection of Section

$$A_{g.req} = \frac{T_u}{f_y/\gamma_{m0}} = \frac{450\times10^3}{250/1.1} = 1980\,mm^2$$

$\therefore$ SP: 6(1) – 1964, P.NO: 10 (Equal angle Section)

Try ISA 110 110, 10 mm

Properties

$A_{g_{pro}} = 2106\,mm^2 > A_{g_{req}}$. Hence safe.

$t = 10\,mm$

b) Connection b/w main angle & gusset plate

Provide 20 mm dia bolt ; $d = 20\,mm$; $d_0 = d + \text{clearance}$ – P.NO: 73 Table: 19

$= 20 + 2$

$d_0 = 22\,mm$

IS: 800 – 2007 ; P.NO: 83 ; Cl: 10.12.1

Lug angle connection, the main angle & Lug angle shares the load equally at joint

$\therefore$ Load on bolt $T_m = \frac{T_u}{2} = 225\,kN$

(i) Shearing capacity of bolt – IS: 800-2007 ; P.NO: 75, Cl: 10.3.3

The main angle is connected with gusset plate

$\therefore$ only single shear occurs. $\therefore n_n = 1$ & $n_s = 0$

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{f_{ub}/\sqrt{3}\,(n_n A_{nb} + n_s A_{sb})}{\gamma_{mb}}$$

$f_{ub} = 400$, $n_n = 1$, $A_{nb} = 245$, $n_s = 0$, $\gamma_{mb} = 1.25$

$$\boxed{V_{dsb} = 45.2\,kN}$$

$A_{sb} = \pi d^2/4 = \pi \times 20^2/4$

$A_{nb} = 0.78\,A_{sb}$

$= 245\,mm^2$

$p_{min} = 2.5d = 2.5\times20 = 50$ say 60mm

K_b least value of following

$e_{min} = 1.7d_0 = 1.7\times22 = 37.4 \simeq 40mm$

(i) $\frac{e}{3d_0} = \frac{40}{3\times22} = 0.61$

(ii) $\frac{p}{3d_0} - 0.25 = \frac{60}{3\times22} - 0.25 = 0.659$

(iii) $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.976$

(iv) 1

say $\boxed{K_b = 0.61}$

(ii) bearing capacity of bolt

IS:800-2007; P.NO:75, Cl:10.3.4

$$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}} = \frac{2.5\,k_b\cdot d\cdot t\cdot f_{ub}}{\gamma_{mb}}$$

0.61, 20, 10, 400, 1.25

$$\boxed{V_{dpb} = 97.6\,kN}$$

(iii) Bolt value

(i) shear capacity $V_{dsb} = 45.2\,kN$

(ii) bearing capacity $V_{dpb} = 97.6\,kN$

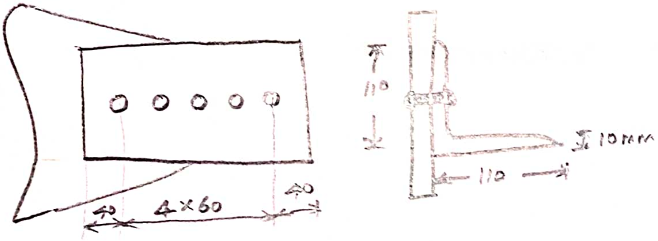
Take least value

$\therefore$ $\boxed{\text{Bolt value} = 45.2\,kN}$

(iv) No. of bolt

$$n = \frac{\text{Load on main angle}}{\text{Bolt value}} = \frac{225\,kN}{45.2\,kN} = 4.9 \simeq 5\text{ Nos.}$$

(v) Arrangement of bolt



(c) check for main angle design

(i) check for yield strength IS:800-2007; P.NO:32, Cl:6.2

$A_g = 2106\,mm^2$

$$T_{dg} = \frac{A_g\cdot f_y}{\gamma_{mo}} = \frac{2106\times250}{1.1} = 478.6\,kN > T_u$$

, Hence safe

(ii) check for Rupture Strength IS:800-2007; P.NO:33, Cl:6.3.1

$$T_{dn} = \frac{\alpha A_n f_u}{\gamma_{m1}} = \frac{0.8\times1920\times410}{1.25}$$

$T_{dn} = 503.8\,kN > T_u$

Hence safe.

$\alpha = 0.8$ (for more than 4 bolt)

$A_n = (b - nd_h)\,t$

$= (b_1 + b_2 - nd_h)\,t$

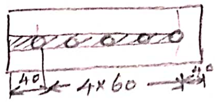
$= [(110+110) - (5\times22)]\times10$

$= 1920\,mm^2$

(iii) Check for block shear strength   IS:800-2007 ; P.NO: 33, Cl: 6.4.1

min gross area   Shear along parallel to external force

$$A_{vg} = (40+240)\,t = 280 \times 10 = 2800\ mm^2$$

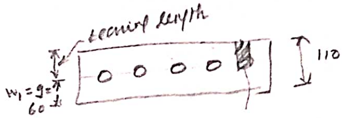


min net area

$$A_{vn} = (280 - 4.5 \times d_0)\,t = (280 - 4.5 \times 22)\,10$$
$$= 1810\ mm^2$$

Shear along $\perp^r$ to the external force

$$A_{tg} = \text{tearing length} \times \text{thickness}$$
$$= (110 - w_1) \times 10 = (110 - 60) \times 10$$
$$= 500\ mm^2$$



$$A_{tn} = (50 - 0.5 d_h)\,t = (50 - 0.5 \times 22)\,10 = 390\ mm^2$$

a) $$T_{db} = \frac{A_{vg} \cdot f_y}{\sqrt{3} \cdot \gamma_{m0}} + \frac{0.9\, A_{tn}\, f_u}{\gamma_{m1}} =$$

b) $$T_{db} = \frac{0.9\, A_{vn}\, f_u}{\sqrt{3} \cdot \gamma_{m1}} + \frac{A_{tg} \cdot f_y}{\gamma_{m0}} =$$

II: Design of Lug angle

(i) Load on Lug angle

Load on Lug angle = Load on main angle = 225 kN

20% extra

P.NO: 75 Cl: 10.3.3   Load on bolt of Lug angle (T_{LB}) = $1.2\, T_L = 1.2 \times 225 = 270\ kN$.

a) selection of section

$$A_{g\,req} = \frac{T_{LB}}{f_y/\gamma_{m0}} = \frac{270 \times 10^3}{250/1.1} = 1188\ mm^2$$

Select ISA 110 110, 8 mm

$A_{g\,pro} = 1702\ mm^2 > A_{g\,req}$. Hence ok.

$t = 8mm$

$b_1 = b_2 = 110\ mm$.

(b) connection of Lug angle & gusset plate

Bolt value = 45.2 kN (From main angle)

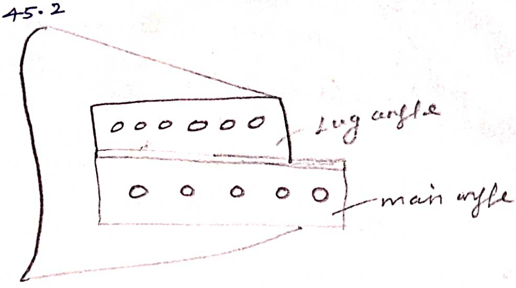
single Lug angle is connected with gusset plate directly,

∴ only single shear takes

NO. of bolt

$$n = \frac{T_{LB}}{\text{Bolt value}} = \frac{270}{45.2} = 5.97 \simeq 6 \text{ nos.}$$

Arrangement of bolt



Check for

same

III. Connection of main angle to gusset plate.

(i) Force to be transfered = 225 kN

Design for connected leg = 40% extra force to be transfered

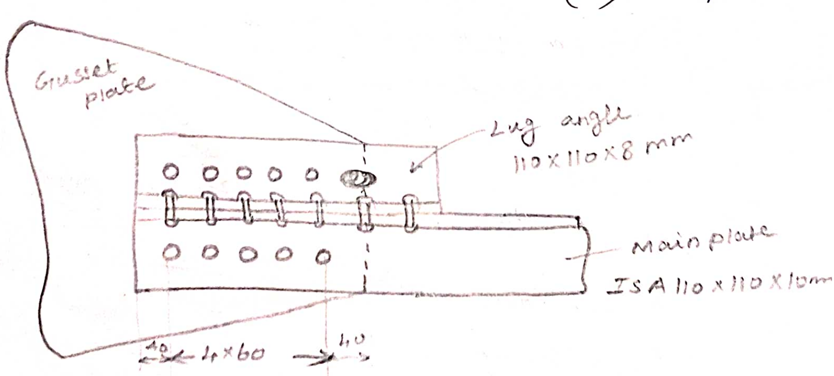
= 1.4 × 225

= 315 kN.

(ii) No. of bolt (Lug angle & main angle)

$$n = \frac{315 \times 10^3}{\text{Bolt value}} = \frac{315 \times 10^3}{45.2 \times 10^3} = 6.96 \simeq 7 \text{ nos.}$$

(iii) Required Length of gusset plate = 40 + (7−1)60 = 400 mm



I

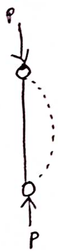
UNIT - IV

COMPRESSION MEMBERS

Introduction

* Many structural members are in compression.
* Vertical compression members in buildings are called columns, posts or stanchions.
* Compression members in trusses are called struts.

Care of Engineers



* To transfer load axially unexpected eccentricity of load is unavoidable due to imperfection.
  (This eccentricity causes lateral Bending moment)
* Axial compression increases the lateral deflection increases resulting into additional bending stresses.
  (it is called buckling of columns)
* Load carrying capacity depends upon the end conditions & also on slenderness ratio of the column sections.

(i) Eff. length based upon end conditions

(ii) Finding Design stress based on slenderness ratio

Type - I - Determination of Design for Axial compression load

IS: 800 - 2007 ; P.NO: 34 ;

7.1.

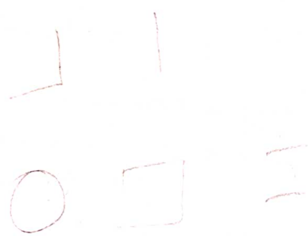
7.1.1

7.1.2

7.1.2.1

7.1.2.2

Com comp



Problem:

Determine the design axial load capacity of the column ISMB 350. If the length of the column is 3 m and its both ends pinned.

Given Data:-

Assume, for rolled steel sections,

$f_y = 250\ N/mm^2$

$f_u = 410\ N/mm^2$

$E = 2 \times 10^5\ N/mm^2$

Step ①. given data & choose coeff. properly below.

End condition: Both ends are pinned (hinged).

(IS:800-2007 P.NO: 45 ; Table: 11)

Step ② eff. length

$\therefore$ eff. length $KL = L = 3\ m$

Properties of ISMB 350 [SP:6(1)-1964 ; P.NO: 2]

$t_f = 14.2\ mm$

$b_f = 140\ mm$

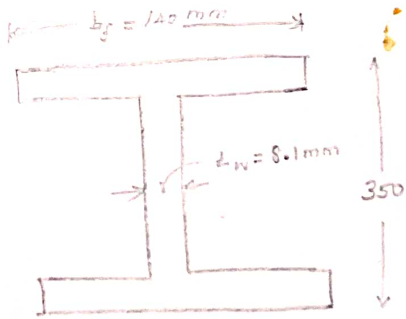
$t_w = 8.1\ mm$

C/s Area $= 66.71\ cm^2 = 6671\ mm^2$

$Wt = 52.4\ kg/m = 524\ N/m$

$r_{xx} = 14.29\ cm = 142.9\ mm$

$r_{yy} = 2.84\ cm = 28.4\ mm$



1 kg = 9.81 N

Step: 1 - Calculation of Design compressive strength:

Step ③ calculate λ_{yy} & λ_{zz}

Non-dimensional effective slenderness ratio

$$\lambda_{yy} = \sqrt{f_y/f_{cc}} = \sqrt{\frac{f_y \cdot (KL/r_{yy})^2}{\pi^2 E}}$$

IS: 800-2007
P.NO: 34.
Clause: 7.1.2.1

$KL = 3m$
$E = 2 \times 10^5\ N/mm^2$; $r_{yy} = 28.4\ mm$;

$$\lambda_{yy} = \sqrt{\frac{250 \times (3000/28.4)^2}{\pi^2 \times 2\times10^5}}$$

$$\boxed{\lambda_{yy} = 1.19}$$

<u>calculate λ_{zz}</u>

$$\lambda_{zz} = \sqrt{\frac{f_y \cdot (KL/r_{zz})^2}{\pi^2 E}} = \sqrt{\frac{250 \times \left(\frac{3000}{142.9}\right)^2}{\pi^2 \times 2\times10^5}}$$

$$\boxed{\lambda_{zz} = 0.24}$$

Step④ Buckling class & imperfection factor α.

<u>Step: 2 - calculation of Impervious factor (α)</u>

To classify the buckling class

Refer. IS: 800-2007; P.NO: 44; Table: 10

(i) $\frac{h}{b_f} > 1.2$

$$\frac{h}{b_f} = \frac{350}{140} = 2.5 > 1.2$$

Hence buckling about ZZ-axis is in class 'a'

(ii) $t_f \leq 40\,mm$

$$t_f = 14.2\,mm < 40\,mm$$

Hence buckling about yy axis is in class 'b'.

From IS: 800-2007; page No: 35; Table: 7

For ZZ axis, class 'a' $\boxed{\alpha = 0.21}$

For yy axis, class 'b' $\boxed{\alpha = 0.34}$

Step: 3 – classiffication of ϕ

IS: 800–2007 ; P. NO: 34 ; clause: 7.1.2.1.

$$\phi_{yy} = 0.5\left[1 + \alpha_{yy}(\lambda_{yy} - 0.2) + \lambda_{yy}^2\right]$$

$$= 0.5\left[1 + 0.34\left(1.19 - 0.2\right) + 1.19^2\right]$$

$$\boxed{\phi_{yy} = 1.38}$$

$$\phi_{zz} = 0.5\left[1 + \alpha_{zz}(\lambda_{zz} - 0.2) + \lambda_{zz}^2\right]$$

$$= 0.5\left[1 + 0.21\left(0.24 - 0.2\right) + 0.24^2\right]$$

$$\boxed{\phi_{zz} = 0.53}$$

Step: 5

Check : By stress Reduction factor 'χ' Method

P.NO: 34;
Cl: 7.1.2.1.

$$f_{cd} = \frac{\chi \cdot f_y}{\gamma_{m0}}$$

For stress reduction factor,
P.NO: 37
IS: 800–2007 ; table: 8b

For Buckling about yy-axis
buckling class - b.

$$\frac{KL}{r_y} = \frac{3000}{28.4}$$

$$= 105.63$$

$f_y = 250$

χ

$100 \rightarrow 0.520$
$110 \rightarrow 0.458$

$$\chi = 0.458 + \frac{(0.520 - 0.458)}{(110 - 100)} \times (110 - 105.63)$$

$$\boxed{\chi = 0.485}$$

$$f_{cd} = \frac{\chi \cdot f_y}{\gamma_{mo}} = \frac{0.485 \times 250}{1.1}$$

$$\boxed{f_{cd} = 110.23 \ N/mm^2} > 109.33 \ N/mm^2$$

Hence the design is safe.

Step:4

Step:4 - calculation of Design comp. strength (f_{cd})

In yy-direction

IS: 800-2007
P.NO: 34
Clause: 7.1.2.1

$$f_{cd\,yy} = \frac{f_y/\gamma_{mo}}{\phi_{yy} + [\phi_{yy}^2 - \lambda_{yy}^2]^{0.5}}$$

$$= \frac{250/1.1}{1.38 + [1.38^2 - 1.19^2]^{0.5}}$$

$$\boxed{f_{cd\,yy} = 109.33 \ N/mm^2}$$

In zz-direction:-

$$f_{cd\,zz} = \frac{f_y/\gamma_{mo}}{\phi_{zz} + [\phi_{zz}^2 - \lambda_{zz}^2]^{0.5}}$$

$$= \frac{250/1.1}{0.53 + [0.53^2 - 0.24^2]^{0.5}}$$

$$\boxed{f_{cd\,zz} = 266.69 \ N/mm^2}$$

Take least value

$\therefore$ Design Compressive Stress $f_{cd} = 109.33 \ N/mm^2$

Step:6 – Design Compressive Strength

IS: 800-2007 ; P.NO: 34 ; clause: 7.1.2

$$P_d = A_e \cdot f_{cd} = 6671 \times 109.33$$

$$\boxed{P_d = 729.3 \text{ kN}}$$

Problem: 2

Determine the design axial load capacity of the column ISHB 300 @ 577 N/m, if the length of column is 3m and its both ends hinged/pinned.

l = 4m

ISMB 150

$KL = L/\sqrt{2}$

2nd condition: one end fixed other end hinged

Given data:-

Assume For rolled steel sections.

$f_y = 250 \text{ N/mm}^2$

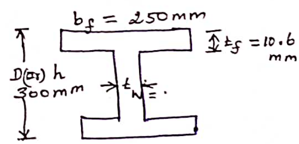
$f_u = 410 \text{ N/mm}^2$

$E = 2 \times 10^5 \text{ N/mm}^2$

$L = 3\text{m}$

End condition : Both ends pinned/hinged

$\therefore L = KL = 3\text{m} = 3000 \text{ mm}$



Properties of ISHB 300 @ 577 N/m

$b_f = 250$ mm ; $t_f = 10.6$ mm

$h = 300$mm ; $t_w =$

$r_{xx} =$; $r_{yy} =$

$A_e = A = \cancel{7g}$ C/s area $= 7484 \text{ mm}^2$; wt = 577 N/m.

Step: 1 – Calculate λ_{yy} + λ_{zz}

(i) Non-dimensional effective slenderness ratio

IS: 800-2007 ; P.NO: 34 Cl: 7.1.2.1

$$\lambda_{yy} = \sqrt{\frac{f_y}{f_{cc}}} = \sqrt{\frac{\underbrace{f_y}_{250 \text{ N/mm}^2} \cdot \left(\frac{\overbrace{KL}^{3000\text{mm}}}{r_{yy}}\right)^2}{\pi^2 \underbrace{E}_{2\times10^5 \text{ N/mm}^2}}} = 0.624$$

$$=$$

$$\lambda_{zz} = \sqrt{f_y \cdot \left(\frac{KL}{r_{zz}}\right)^2 / \pi^2 E} = \cancel{0.621}\ 0.26$$

* Step: 2 - Impervious factor (α)

To classify the buckling class

IS:800-2007; P.NO: 44, Table: 10

(i) $\frac{h}{b_f} = \frac{300}{250} = 1.2 \leq 1.2$ Hence buckling about z-z axis in a class 'b'

(ii) $t_f = 10.6 \leq 100mm$

buckling about yy-axis in a class 'c'

From IS:800-2007; P.NO:35; Table: 7

For zz-axis, class b, $\boxed{\alpha_{zz} = 0.34}$

for yy-axis, class c, $\boxed{\alpha_{yy} = 0.49}$

* Step:3 - classification of ϕ

IS: 800-2007; P.NO:34, Cl: 7.1.2.1

$$\phi_{yy} = 0.5\left[1 + \alpha_{yy}(\lambda_{yy} - 0.2) + \lambda_{yy}^2\right]$$

$$=$$

$$\phi_{zz} = 0.5\left[1 + \alpha_{zz}(\lambda_{zz} - 0.2) + \lambda_{zz}^2\right]$$

$$=$$

* Step: 4 - calculation of Design compressive strength IS:800-2007; P.NO: 34; Cl: 7.1.2.1

In yy-direction

$$f_{cd_{yy}} = \frac{f_y/\gamma_{mo}}{\phi_{yy} + \left[\phi_{yy}^2 - \lambda_{yy}^2\right]^{0.5}}$$

$$=$$

In zz-direction

$$f_{cd_{zz}} = \frac{f_y/\gamma_{mo}}{\phi_{zz} + \left[\phi_{zz}^2 - \lambda_{zz}^2\right]^{0.5}}$$

$$=$$

Take least value of above two

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

* Step: 5 - check: by stress reduction factor 'χ' method IS:800-2007; P.NO:34 Cl: 7.1.2.1

$$f_{cd} = \chi . f_y / \gamma_{mo}$$

$\chi \rightarrow$ stress reduction factor

IS: 800-2007; P.NO: 37, table: 8(b)

yy-axis class (b)

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

$f_y = 250 N/mm^2$, $\gamma_{mo} = 1.1$

$\chi \rightarrow$

Problem : 3

In a truss a strut 3m long consists of two angles ISA 100100, 6 mm. Find the factored strength of the member if the angles are connected on both sides of 12mm gusset by

(i) One bolt
(ii) two bolt
(iii) welding, which makes the joint rigid.

Solution :-

Given data

Properties of section From SP:6(1)-1964 ; P.NO: 10

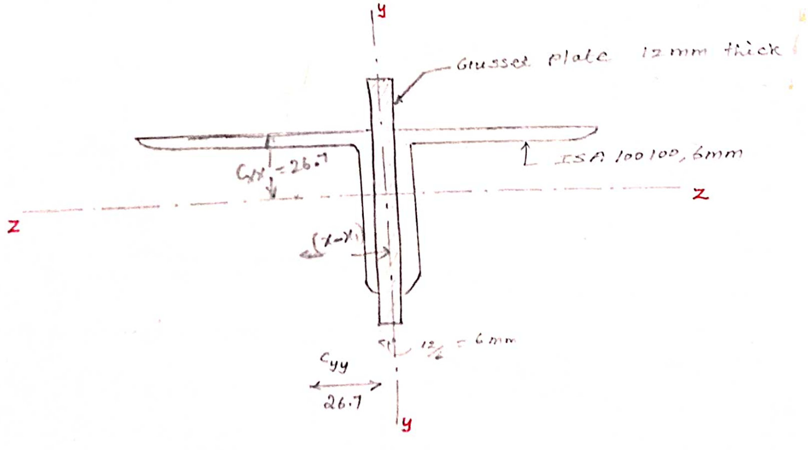
Angle section, ISA 100100, 6mm

c/s area $= 11.67\ cm^2 = 1167\ mm^2$

$C_{xx} = C_{yy} = 2.67\ cm = 26.7\ mm$

$I_{xx} = I_{yy} = 111.3\ cm^4 = 111.3 \times 10^4\ mm^4$

$r_{xx} = r_{yy} = 3.09\ cm = 30.9\ mm$



Iyy of the Member

$h=(\bar{y}-y_1)$ $h^2=$

$$I_{yy} = 2\left[I_{yy} \text{ of one angle} + \text{Area of one angle} \times \left[C_{yy} + \frac{\text{Gusset plate thick}}{2}\right]^2\right]$$

$$= 2\left[I_{yy} \text{ of one angle} + A h^2\right]$$

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

$$I_{yy} \text{ of the member} = 472.17\times10^4 \ mm^4$$

$$\therefore \; r_{yy} = \sqrt{\frac{I_{yy}}{A}} = \sqrt{\frac{472.17\times10^4}{1167\times2}} = 44.98$$

Two angles

$r_{xx} = r_{zz} = 30.9mm$

$\therefore$ r_{zz} is governing the strength of member

Take, $r_{min} = 30.9mm$

case:(i) when one bolt (a single bolt) is used for connection

Assume - both ends are pinned (or) hinged. $\therefore KL = L = 3000mm$

IS:800-2007 P.No: 48

$r = r_{zz} = 30.9 \ mm$

$KL = L = 3000 \ mm$

IS: 800-2007 Table: 10 P.NO: 44

$$\therefore \frac{KL}{r} = \frac{3000}{30.9} = 97$$

P.NO:44

The member belongs to buckling class c (because angle section)

From IS: 800-2007; Table: 9 (c) P.NO: 42

for $\frac{KL}{r} = 97$ & $f_y = 250 \ N/mm^2$

f_{cd}

90 → 121

100 → 107

$$\therefore f_{cd} = 107 + \left(\frac{121-107}{100-90}\right) \times (100-97)$$

$$\boxed{f_{cd} = 111.2 \ N/mm^2}$$

∴ Design comp. strength,

IS: 800-2007
P.NO: 34
Clause: 7.1.2

$$P_d = A_e \cdot f_{cd} = 2 \times 1167 \times 111.2$$

$$\boxed{P_d = 259.54 \text{ kN}}$$

case: (ii) When two bolts are used

The eff. length is reduced. It may be taken as

equal to or not less than 2 bolts, eff. length taken as 0.7 to 0.85L

IS: 800-2007
P.No: 48
Cl: 7.5.2.1

0.85 times actual length

$$KL = 0.85L = 0.85 \times 3000 = 2550 \text{ mm}$$

$$\therefore \frac{KL}{r} = \frac{2550}{30.9} = 82.5$$

From IS: 800-2007; Table: 9C; P.NO: 42

$\frac{KL}{r} = 82.5$; $f_y = 250 \text{ N/mm}^2$

From table;

f_{cd} for $\frac{KL}{r} = 80 \rightarrow 136 \text{ N/mm}^2$

f_{cd} for $\frac{KL}{r} = 90 \rightarrow 121 \text{ N/mm}^2$

$$\therefore f_{cd} \text{ for } \frac{KL}{r} = 82.5 \Rightarrow 121 + \left(\frac{121 \sim 136}{90 \sim 80}\right) \times (90 \sim 82.5)$$

$$\boxed{f_{cd} = 132.25 \text{ N/mm}^2}$$

P.NO: 34

Design comp. strength, $P_d = A_e \cdot f_{cd} = 2 \times 1167 \times 132.25$

$$\boxed{P_d = 308.67 \text{ kN}}$$

Case: (iii) - Rigid joint by welding

P.NO: 48

Eff. Length $\quad KL = 0.7L = 0.7 \times 3000 = 2100 \text{ mm}$

$$\frac{KL}{r} = \frac{2100}{30.9} = 67.96$$

From IS: 800 - 2007; Table: 9C; P.NO: 42;

For $\frac{KL}{r} = 60 \Rightarrow f_{cd} = 168 \text{ N/mm}^2$

For $\frac{KL}{r} = 70 \Rightarrow f_{cd} = 152 \text{ N/mm}^2$

$\therefore$ For $\frac{KL}{r} = 67.96 \Rightarrow f_{cd} = 152 + \left(\frac{168 \sim 152}{60 \sim 70}\right) \times (70 \sim 67.96)$

$$\boxed{f_{cd} = 155.26 \text{ N/mm}^2}$$

P.NO: 34

Design Comp. Strength, $P_d = A_e \cdot f_{cd}$

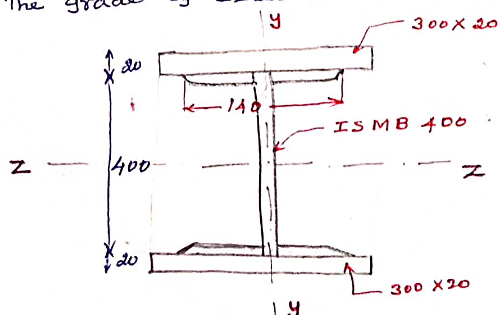
$$= 2 \times 1167 \times 155.26$$

$$\boxed{P_d = 362.39 \text{ kN}}$$

Problem:

Determine the load carrying capacity of the column section as shown in fig. if its actual length is 4.5 m. Its one end may be assumed fixed and the other end hinged. The grade of steel is Fe 415 ie (E250)

Strem reduction factor method



Given Data,

Properties of ISMB 400

P.NO: 2 ; SP: 6(1) – 1964

$h = 400\ mm$; $b_f = 140\ mm$

$t_f = 16\ mm$;

c/s Area $= 78.46\ cm^2$

$A_e = 7846\ mm^2$

$I_{zz} = 20458.4\ cm^4$

$I_{zz} = 204.584 \times 10^6\ mm^4$

$I_{yy} = 622.1\ cm^4$

$I_{yy} = 622.1 \times 10^4\ mm^4$

Step: 1 – Buckling class

For Built up section,

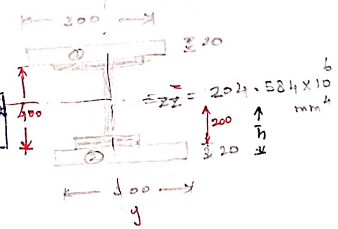
IS: 800 – 2007 ; P.NO: 44 ; Table: 10

Buckling class $= C$

Step: 2 – sectional properties

$$I_{zz} = 204.584 \times 10^6 + \left[2 \times 300 \times 20 \times \left(200 + \frac{20}{2}\right)^2\right]$$

$(A\bar{h}^2)$



$$\boxed{I_{zz} = 733.784 \times 10^6\ mm^4}$$

$$I_{yy} = 622.1 \times 10^4 + 2\,\frac{bd^3}{12}$$

(Note: M.I of plate about its own axis neglected)

$$= 622.1 \times 10^4 + 2 \times \frac{20 \times 300^3}{12}$$

$$\boxed{I_{yy} = 96.221 \times 10^6\ mm^4}$$

$\therefore\ I_{yy} < I_{zz}$

$\therefore$ Take I_{yy} values,

Buckling about yy-axis governs the design

Area = $A_{member} + A_{plate} = 7846 + 2[300 \times 20] = 19846\ mm^2$

$$\gamma = \gamma_{yy} = \sqrt{\frac{I_{yy}}{A}} = \sqrt{\frac{96.221 \times 10^6}{7846 + 2(300 \times 20)}} = \sqrt{\frac{96.221 \times 10^6}{19846}}$$

$$\gamma_{min} \quad \boxed{\gamma = \gamma_{yy} = 69.63\ mm}$$

End condition :- one end fixed & other end is free

∴ Eff. length $KL = 0.8L = 0.8 \times 4500 = 3600\ mm$

P.NO: 45 — Table: 11

Slenderness ratio $\frac{KL}{\gamma} = \frac{3600}{69.63} = 57.70$

Step: 3 – Design Comp. Stress (f_{cd})

From IS: 800 – 2007 ; Table: 9C ; P.NO: 42

For $\frac{KL}{\gamma} = 50 \Rightarrow$ ~~183~~ $f_{cd} = 183\ N/mm^2$

For $\frac{KL}{\gamma} = 60 \Rightarrow f_{cd} = 168\ N/mm^2$

∴ $\frac{KL}{\gamma} = 57.70 \Rightarrow f_{cd} = 168 + \left(\frac{183-168}{50 \sim 60}\right) \times (60 - 57.7)$

$$\boxed{f_{cd} = 180.45\ N/mm^2}$$

Design. Comp. Strength, $P_d = A_e \cdot f_{cd} = 19846 \times 180.45$

P.NO: 34,

$$\boxed{P_d = 3581.21\ KN}$$

Load (working load) carrying capacity of the column

$$= \frac{3581.21}{1.5} = 2387.47\ kN$$

Type-II

Design of compressive Members

Design procedure:-

Step: 1 – Design compressive stress should be assumed as per given load condition.

(a) Rolled steel sections $f_{cd} = 135\ N/mm^2$; slenderness ratio varies from 70 to 90

$f_{cd} = 90\ N/mm^2$; slenderness ratio varies from 110 to 130

(b) Angle section

(c) compressive member carrying a large load, $f_{cd} = 200\ N/mm^2$

Step: 2 – Calculate the effective sectional area required,

P.NO:34 → $P_d = A_e \cdot f_{cd}$ i.e., $\boxed{A_{e_{req}} = \dfrac{P_d}{f_{cd}}}$

Step: 3 – Select a suitable section to give the effective area and calculate r_{min}.

Step: 4 – Determine the effective length, based on the given end conditions and types of connections.

Step: 5 – Find the slenderness ratio (λ), design strength (f_{cd}) and load carrying capacity (P_d).

Step: 6 – Revise the section, if calculated P_d less than given design load.

Problem :-

Design a steel column (stanchion) I-section 3.5 m long, in a building subjected to a factored load of 550 kN. Both ends of the stanchion are effectively restrained in direction and position. (Both ends are fixed) use Fe410 steel grade.

Given Data :-

Factored load = 550 kN

Length of column (L) = 3.5 m = 3500 mm

Step : 1 - Design compressive stress (f_{cd})

For I - section, $\boxed{f_{cd} = 135 \ N/mm^2}$

Step : 2 - Sectional area required (A_{req})

$$A_{req} = \frac{P_d}{f_{cd}} = \frac{550 \times 10^3}{135}$$

$$\boxed{A_{req} = 4074 \ mm^2}$$

Step : 3 - Select a suitable section

From SP:6 (1) - 1964, Page No:4

select, ISHB 150 @ 346 N/m.

$A = 4408 \ mm^2$

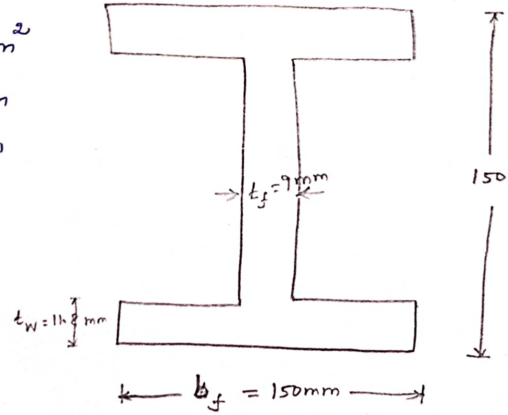
$r_{zz} = 60.9 \ mm$

$r_{yy} = 33.5 \ mm$

$t_f = 9 \ mm$

$t_w = 11.8 \ mm$

$b_f = 150 \ mm$



Step: 4 – Effective length

End condition : Both ends are fixed.

From IS : 800 – 2007, P.NO: 45 ; Table : 11

$$kL = 0.65L = 0.65 \times 3500 = 2275 \, mm.$$

Step: 5 – slenderness ratio (λ) , design strength (f_{cd}), P_d

$$\lambda = \sqrt{\frac{f_y \cdot \left(\frac{kL}{r}\right)^2}{\pi^2 E}}$$

(i) Buckling class → IS : 800 – 2007 ; P.NO : 44 ; Table : 10   I-section

1. $\frac{h}{b_f} > 1.2$

$$\frac{150}{150} = 1 \not> 1.2$$

2. $\frac{h}{b_f} \leq 1.2$

$$1 < 1.2$$

$$t_f \leq 100 mm$$
$$9 < 100 mm$$

∴ This section buckled under class 'c'

(ii) Imperfection factor (α)   IS : 800 – 2007 ; P. NO : 35 ; Table : 7

$$\boxed{\alpha = 0.49}$$

(iii) slenderness ratio (λ)   IS : 800 – 2007 P. NO : 34 ;

$$\lambda_{yy} = \sqrt{f_y \cdot \left(\frac{kL}{r_{yy}}\right)^2 / \pi^2 E}$$

$$= \sqrt{250 \times \left(\frac{2275}{33.50}\right)^2 / \pi^2 \times 2 \times 10^5}$$

$$\boxed{\lambda_{yy} = 0.76}$$

P.NO: 34

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

$$= 0.5\left[1 + 0.49(0.76 - 0.2) + 0.76^2\right]$$

$$\boxed{\phi = 0.93}$$

IS: 800-2007
P.NO: 34

(v) Design compressive stress (f_{cd})

$$f_{cd} = \frac{f_y / \gamma_{mo}}{\phi + \left[\phi^2 - \lambda^2\right]^{0.5}} = \frac{250/1.1}{0.93 + \left[0.93^2 - 0.76^2\right]^{0.5}}$$

$$\boxed{f_{cd} = 155.03 \text{ N/mm}^2}$$

IS 800-2007
P.NO: 34

(vi) Design compressive Strength (P_d)

$$P_d = A_e \cdot f_{cd} = 4408 \times 155.03$$

$$\boxed{P_d = 683.37 \text{ kN}} > 550 \text{ KN}.$$

Hence the design is safe.

Problem: 2

Design a column to support a factored load of 1050 kN. The column has an effective length of 7 m with respect to 'z'-axis, and 5 m with respect to y-axis. Use Fe 410 steel grade.

Given Data

Factored load = 1050 kN

eff. length (z) axis = 7 m

eff. length (y-axis) = 5 m.

$f_y = 250\ N/mm^2$

$\gamma_{mo} = 1.1$

$E = 2 \times 10^5\ N/mm^2$

Step: 1 - Design compressive stress (f_{cd})

Provide I-section as column.

For I-section $\boxed{f_{cd} = 135\ N/mm^2}$

Step: 2 - sectional Area req. (A_{req})

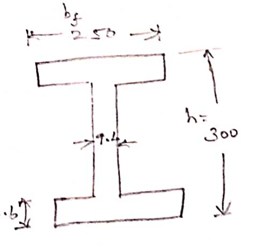
$$A_{req} = \frac{P_d}{f_{cd}} = \frac{1050 \times 10^3}{135}$$

$$\boxed{A_{req} = 7777.78\ mm^2}$$

Step: 3 - suitable section

From SP: 6(1) - 1964 ; P. No: 4

Provide, ISHB 300 @ 630 N/m



$A = 8025\ mm^2$; $b_f = 250\ mm$

$t_w = 9.4\ mm$; $t_f = 10.6\ mm$

$r_{zz} = 127\ mm$; $r_{yy} = 52.9\ mm$

Step: 4 - Eff. Length

Assume both ends are fixed.

But, the eff. length are given

$$KL_{zz} = 7000 \text{ mm}$$

$$KL_{yy} = 5000 \text{ mm}$$

Step: 5 - Slenderness ratio (λ), comp. stress (f_{cd}), comp. Strength (P_d)

(a) Slenderness ratio

IS: 800-2007 ; P.NO: 20; Table: 3 - (i)

Max. eff. Slenderness ratio = 180

For zz axis

$$\frac{KL_{zz}}{r_{zz}} = \frac{7000}{127}$$

$$= 55.12 < 180$$

Hence ok

For yy axis

$$\frac{KL_{yy}}{r_{yy}} = \frac{5000}{52.9}$$

$$= 94.52 < 180$$

Hence ok

(b) Buckling class

IS: 800-2007 ; P.NO: 44 ; Table: 10

(i) $\frac{h}{b_f} > 1.2 \quad = \frac{300}{250} = 1.2$ Not ok

(ii) $\frac{h}{b_f} \leq 1.2 \quad = 1.2$ ok

$t_f < 100$

$10.6 < 100$ It is ok

yy

$\therefore$ $\boxed{\text{Buckling class - c (yy axis)}}$

$\boxed{\text{Buckling class - b (zz axis)}}$

(c) Imperfection factor (α)

IS: 800-2007 ; P.NO: 35; Table: 7

$\boxed{\alpha_{yy} = 0.49}$

class - c

$\boxed{\alpha_{zz} = 0.34}$

Class - b

$$\lambda_{yy} = \sqrt{\frac{f_y \left(KL/r_{yy}\right)^2}{\pi^2 E}}$$

$$= \sqrt{\frac{250 \times 94.52^2}{\pi^2 \times 2 \times 10^5}}$$

$$\boxed{\lambda_{yy} = 1.06}$$

$$\lambda_{zz} = \sqrt{\frac{f_y \left(\frac{KL}{r_{zz}}\right)^2}{\pi^2 E}}$$

$$= \sqrt{\frac{250 \times 55.12^2}{\pi^2 \times 2 \times 10^5}}$$

$$\boxed{\lambda_{zz} = 0.62}$$

$$\phi_{yy} = 0.5\left[1 + \alpha_{yy}\left(\lambda_{yy} - 0.2\right) + \lambda_{yy}^2\right]$$

$$= 0.5\left[1 + 0.49\left(1.06 - 0.2\right) + 1.06^2\right]$$

$$\boxed{\phi_{yy} = 1.27}$$

$$\phi_{zz} = 0.5\left[1 + \alpha_{zz}\left(\lambda_{zz} - 0.2\right) + \lambda_{zz}^2\right]$$

$$= 0.5\left[1 + 0.34\left(0.62 - 0.2\right) + 0.62^2\right]$$

$$\boxed{\phi_{zz} = 0.76}$$

Design compressive stress (f_{cd})

$$f_{cd} = \frac{f_y/\gamma_{mo}}{\phi_{yy} + \left[\phi_{yy}^2 - \lambda_{yy}^2\right]^{0.5}}$$

$$= \frac{250/1.1}{1.27 + \left[1.27^2 - 1.06^2\right]^{0.5}}$$

$$\boxed{f_{cd} = 115.39 \ N/mm^2}$$

Design compressive stress (f_{cd})

$$f_{cd} = \frac{f_y/\gamma_{mo}}{\phi_{zz} + \left[\phi_{zz}^2 - \lambda_{zz}^2\right]^{0.5}}$$

$$= \frac{250/1.1}{0.76 + \left[0.76^2 - 0.62^2\right]^{0.5}}$$

$$\boxed{f_{cd} = 189.47 \ N/mm^2}$$

Design compressive strength (P_d)

$$P_d = A_e . f_{cd} = 8025 \times 115.39$$

$$\boxed{P_d = 926 \ KN} \not> 1050 \ KN$$

Design compressive strength (P_d)

$$P_d = A_e . f_{cd} = 8025 \times 189.47$$

$$\boxed{P_d = 1520.5 \ KN} > 1050 \ KN$$

Hence the design is safe in ZZ-direction.

Problem:

A column 4 m long has to support a factored load of 6000 kN, The column is effectively held at both ends and restrained in direction at one of the ends. Design the column using beam sections and plates.

Given Data:

Length of column $(L) = 4\,m = 4000\,mm$

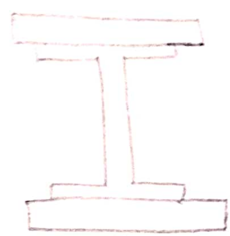
factored load $= 6000\,kN$

End condition : one end is fixed, other end is hinged.

Step: 1 - Design comp. stress (f_{cd}):

For heavy load,

$\therefore \boxed{f_{cd} = 200\ N/mm^2}$



Step: 2 - Sectional area required (A_{req})

$$A_{req} = \frac{P_d}{f_{cd}} = \frac{6000 \times 10^3}{200}$$

$$\boxed{A_{req} = 30{,}000\ mm^2}$$ For whole area

Step: 3 - selection of suitable section

From SP:6(1)-1964 ; Page NO: 4

Select, ISHB 450 @ 925 N/m,

$A = 11789\ mm^2$; $b_f = 250\,mm$; $t_f = 13.7\,mm$

$t_w = 11.3\,mm$; $r_{zz} = 185\,mm$; $r_{yy} = 50.8\,mm$

$I_{zz} = 40349.9 \times 10^4\ mm^4$; $I_{yy} = 3045 \times 10^4\ mm^4$

Area of I-section $= 11789\ mm^2$

Area of two cover plates $= 30{,}000 - 11789$

$= 18211\ mm^2$

$\therefore$ Area for one plate $= \frac{18211}{2} = 9105.5\ mm^2$

Assume thickness of plates $= 20\ mm$

Area of plate $= b \cdot t$

$9105.5 = b \times 20$

$\therefore\ b = 455.3\ mm$

say $\boxed{b = 500\ mm}$

Provide 500×20 mm plates.

check for overhang IS:800-2007; P.No: 74 clause: 10.2.3.

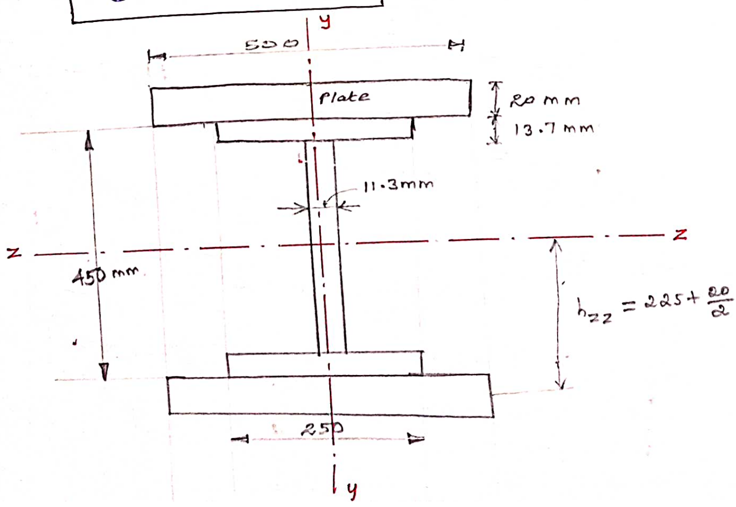
overhang $\frac{500-250}{20} = 12.5 < 12t$

Total Area Provided =

A_e = Area of I-section + Area of Top plate + Area of bottom plate

$= 11789 + (2 \times 500 \times 20)$

$\boxed{A_e = 31789\ mm^2}$



Step: 4 – Effective length

End condition : one end fixed , other end is hinged.

Is : 800 – 2007 ; Page No : 45 ; Table : 11

$$KL = 0.8L = 0.8 \times 4000$$

$$\boxed{KL = 3200\,mm}$$

Step: 5 – To find r_{zz} & r_{yy}

$$r_{zz} = \sqrt{\frac{I_{zz}}{A}} \quad ; \quad r_{yy} = \sqrt{\frac{I_{yy}}{A}}$$

$I_{zz} = I_{zz}$ main section $+ I_{zz}$ for two plates

$$= 40349.9 \times 10^4 + 2\left[\frac{bd^3}{12} + Ah^2\right]$$

$$= 40349.9 \times 10^4 + 2\left[\frac{500 \times 20^3}{12} + (500 \times 20) \times \left(225 + \frac{20}{2}\right)^2\right]$$

$$= 40349.9 \times 10^4 + 1.105 \times 10^9$$

$$\boxed{I_{zz} = 1508.67 \times 10^6\ mm^4}$$

$$r_{zz} = \sqrt{\frac{I_{zz}}{A}} = \sqrt{\frac{1508.67 \times 10^6}{31789 + (2 \times 500 \times 20)}}$$

$$\boxed{r_{zz} = 217.85\,mm}$$

$I_{yy} = I_{yy}$ main section $+ I_{yy}$ for two plates.

$$= 3045 \times 10^4 + 2\left[\frac{db^3}{12} + Ah^2\right]$$

$$= 3045 \times 10^4 + 2\left[\frac{20 \times 500^3}{12} + A(0)^2\right]$$

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

$$r_{yy} = \sqrt{\frac{I_{yy}}{A}} = \frac{447.12 \times 10^6}{11789 + (2 \times 500 \times 20)}$$

$$\boxed{r_{yy} = 118.59 \text{ mm}}$$

Step:6 – To find λ_{yy}

$$\lambda_{yy} = \sqrt{\frac{f_y \cdot (KL/r_{yy})^2}{\pi^2 E}} = \sqrt{\frac{250 \times \left(\frac{3200}{118.59}\right)^2}{\pi^2 \times 2 \times 10^5}}$$

$$\boxed{\lambda_{yy} = 0.30}$$

Step:7 – Imperfection factor (α)

Buckling class. IS: 800-2007; P.NO: 44 ; Table: 10

For Built up member

Buckling class at any axis = C

Imperfection factor $\boxed{\alpha = 0.49}$

Step: 8 – calculation of ϕ

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

$$= 0.5\left[1 + 0.49(0.3 - 0.2) + 0.3^2\right]$$

$$\boxed{\phi = 0.57}$$

Step: 9 – Design compressive stress (f_{cd})

IS:800–2007 ; P.NO: 34

$$f_{cd} = \frac{f_y/\gamma_{mo}}{\phi + [\phi^2 - \lambda^2]^{0.5}} = \frac{250/1.1}{0.57 + [0.57^2 - 0.3^2]^{0.5}}$$

$$\boxed{f_{cd} = 215.5 \ N/mm^2}$$

Step: 10 – Design compressive strength (P_d)

IS:800–2007 ; P.NO: 34

$$P_d = A_e \cdot f_{cd} = 31789 \times 215.5$$

$$\boxed{P_d = 6850.5 \ kN} > 6000 \ kN.$$

Hence the design is safe.

Type–III – Loading through only one leg

{IS:800-2007
P.NO: 48,
Clause: 7.5.1.2}

Problem:-

Calculate the compressive resistance of a 200 X 200 X 18 mm angle, assuming that the angle is loaded through only one leg. When it is connected by

(i) two bolts at the end.

(ii) one bolt at the end

(iii) Welded at each end.

Assume the length of member is 3m ; Yield stress f_y = 250 MPa

Given Data:-

$L = 3 \ m = 3000 \ mm$

$f_y = 250 \ N/mm^2$

Angle = 200 X 200 X 18 mm

ie, ISA 200 X 200 X 18

Properties of section ISA 200 X 200 X 18 mm

From SP:6 (1) - 1964 ; Page No: 10

Area = 68.81 cm^2 = 6881 mm^2 ; Wt = 54 kg/m = 540 N/m

$\gamma_{xx} = \gamma_{yy} = 6.13$ cm = 61.3 mm

$\gamma_{vv} = 3.9$ cm = 39 mm

case:1 - connected by two bolts at end

a) At fixed end condition

IS: 800 - 2007 ; P. No: 48 ; table: 12

$k_1 = 0.2$; $k_2 = 0.35$; $k_3 = 20$

Slenderness ratio

$$\lambda_e = \sqrt{k_1 + k_2 \lambda_{vv}^2 + k_3 \cdot \lambda_\phi^2}$$ ← P. No: 48

$$\lambda_{vv} = \frac{l/\gamma_{vv}}{\xi \sqrt{\frac{\pi^2 E}{250}}}$$

$$\xi = \left(\frac{250}{f_y}\right)^{0.5} = \left(\frac{250}{250}\right)^{0.5}$$

$$\boxed{\xi = 1}$$

$$\lambda_{vv} = \left(\frac{3000}{39}\right) \Big/ \left(1 \cdot \sqrt{\frac{\pi^2 \times 2 \times 10^5}{250}}\right)$$

$$\boxed{\lambda_{vv} = 0.87}$$

$b_1 = 200$ mm
$b_2 = 200$ mm
$t = 18$ mm

$$\lambda_\phi = \frac{(b_1 + b_2)/2t}{\xi \sqrt{\frac{\pi^2 E}{250}}} = \frac{(200+200)/2 \times 18}{1 \sqrt{\frac{\pi^2 \times 2 \times 10^5}{250}}}$$

$$\boxed{\lambda_\phi = 0.125}$$

$$\lambda_e = \sqrt{k_1 + k_2 \quad \lambda_{vv}^2 + k_3 . \lambda_\phi^2}$$

$$= \sqrt{0.2 + (0.3 \times 0.87^2) + (20 \times 0.125^2)}$$

$$\boxed{\lambda_e = 0.88}$$

Buckling class → IS:800-2007 ; P.NO:44 ; Table:10

For any angle section – class 'c'.

$\therefore$ Imperfection factor $\boxed{\alpha = 0.49}$ → IS:800-2007 P.NO:35, Table:7

P.NO:34, $$\phi = 0.5[1 + \alpha(\lambda_e - 0.2) + \lambda_e^2]$$

$$= 0.5[1 + 0.49(0.88 - 0.2) + 0.88^2]$$

$$\boxed{\phi = 1.05}$$

P.NO: 34, $$f_{cd} = \frac{f_y/\gamma_{mo}}{\phi + [\phi^2 - \lambda_e^2]^{0.5}} = \frac{250/1.1}{1.05[1.05^2 - 0.88^2]^{0.5}}$$

$$\boxed{f_{cd} = 140.05 \ N/mm^2}$$

P.NO: 34 $$P_d = A_e . f_{cd} = 6881 \times 140.05$$

$$\boxed{P_d = 963.68 \ kN}$$

(b) At hinged end condition

From IS:800-2007; P.NO: 48; Table: 12

One Bolt only hinged. $K_1 = 0.70$; $K_2 = 0.6$; $K_3 = 5$

$$\xi = \left(\frac{250}{f_y}\right)^{0.5} = \left(\frac{250}{250}\right)^{0.5}$$

$$\boxed{\xi = 1}$$

$$\boxed{\lambda_{vv} = 0.87} \qquad \boxed{\lambda_\phi = 0.125}$$

$$\lambda_e = \sqrt{K_1 + K_2 \cdot \lambda_{vv}^2 + K_3 \cdot \lambda_\phi^2}$$

$$= \sqrt{0.7 + 0.6 \times 0.87^2 + 5 \times 0.125^2}$$

$$\boxed{\lambda_e = 1.11}$$

P.NO:44 Buckling class (for any angle section) = 'c'

P.NO:35 Imperfection factor $\alpha = 0.49$

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

$$= 0.5\left[1 + 0.49(1.11 - 0.2) + 1.11^2\right]$$

$$\boxed{\phi = 1.34}$$

$$f_{cd} = \frac{f_y/\gamma_{mo}}{\phi + \left[\phi^2 - \lambda_e^2\right]^{0.5}} = \frac{250/1.1}{1.34 + \left[1.34^2 - 1.11^2\right]^{0.5}}$$

$$\boxed{f_{cd} = 108.71\ N/mm^2}$$

welded at each side equal to two bolt are connected at each end.

Design a single angle discontinuous strut, carrying a factored load of 65 kN at one of its length. Assume that the distance b/w the joint is 2.5 m. Take $f_y = 250$ MPa.

Given Data

Factored load $= 65$ kN

$f_y = 250\ N/mm^2$

distance b/w joint $= 2.5\ m = 2500\ mm$

Step: 1 - Design compressive stress (f_{cd})

For angle Section, $\boxed{f_{cd} = 90\ N/mm^2}$

Assume two bolts are connected at its end and assume fixed end condition.

Step: 2 - Area required (A_{req})

$$A_{req} = \frac{P_d}{f_{cd}} = \frac{65 \times 10^3}{90} =$$

$$\boxed{A_{req} = 722.22\ mm^2}$$

Step: 3 - selection of suitable section:-

From SP: 6 (1) - 1964 - Page No: 8

select ISA $65 \times 65 \times 6$ mm

$A = 744\ mm^2$; $r_{vv} = 12.5$ mm

Step: 4 - Slenderness ratio (λ_e), ϕ

Assume two bolts are connected at its end & also assume fixed end condition.

From IS: 800 - 2007 ; Page NO: 48 ; Table: 12

$$k_1 = 0.20 \quad ; \quad k_2 = 0.35 \quad ; \quad k_3 = 20$$

P.NO: 48 $$\xi = \left(\frac{250}{f_y}\right)^{0.5} = \left(\frac{250}{250}\right)^{0.5}$$

$\boxed{\xi = 1}$ yield stress ratio

ISA $65 \times 65 \times 6$ mm

$b_1 = 65$

$b_2 = 65$

$t = 6$ mm

P.NO: 48 $$\lambda_{vv} = \frac{l/r_{vv}}{\xi\sqrt{\frac{\pi^2 E}{250}}} = \frac{\left(\frac{2500}{12.5}\right)}{1\sqrt{\frac{\pi^2 \times 2 \times 10^5}{250}}}$$

in minor axis

$$\boxed{\lambda_{vv} = 2.25}$$

P.NO: 48 $$\lambda_\phi = \frac{(b_1+b_2)/2t}{\xi.\sqrt{\pi^2 E/250}} = \frac{(65+65)/2\times6}{\xi.\sqrt{\frac{\pi^2 \times 2 \times 10^5}{250}}}$$

$$\boxed{\lambda_\phi = 0.122}$$

equivalent slenderness ratio

P.NO: 48 $$\lambda_e = \sqrt{k_1 + k_2.\lambda_{vv}^2 + k_3.\lambda_\phi^2}$$

$$= \sqrt{0.20 + 0.35 \times 2.25^2 + 20 \times 0.122^2}$$

$$\boxed{\lambda_e = 1.51}$$

P.NO: 34 $$\phi = 0.5\left[1 + \alpha(\lambda - 0.2) + \lambda^2\right]$$

$$= 0.5\left[1 + 0.49(1.51 - 0.2) + 1.51^2\right]$$

$$\boxed{\phi = 1.96}$$

<u>Buckling class</u> P.NO: 44

for any angle section

$\boxed{\text{Class} - C}$

<u>Imperfection factor (α)</u>

for class - c P.NO: 35

$\boxed{\alpha = 0.49}$

Step:5 - Design comp. stress (fcd)

P.NO:34

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

$$= \frac{250/1.1}{1.96\left[1.96^2 - 1.51^2\right]^{0.5}}$$

$$\boxed{f_{cd} = 70.81\ N/mm^2}$$

Step:6 - Design comp. Strength (P_d)

IS: 800-2007 ; P.NO: 34

$$P_d = A_e . f_{cd} = 744 \times 70.81$$

$$P_d = 53\ kN \ngtr 65\ kN$$

Hence the Section of revised.

Choose ISA 65 X 65 X 8 mm

Repeat step 3 to 6

TYPE-4 - Built up column : (Latticed columns)

* The size & shape of standard rolled steel section are limited, because of the limitations of the rolling wheels.
* By rolled steel sections do not furnish the required sectional area or When a special shape or large radius of gyration in two different direction, effective column section is fabricated.
* For economical design of heavily loaded columns, least radius of gyration of column section is increased to maximum ($r_{yy} \geq r_{zz}$).
* To achieve this condition the rolled sections are kept away from the centroidal axis of the column. They are connected by some connecting system known as "Latticed System".
* The commonly used lattice are
  (a) Lacing bars
  (b) Batten plates
  (c) Lacing with battens.

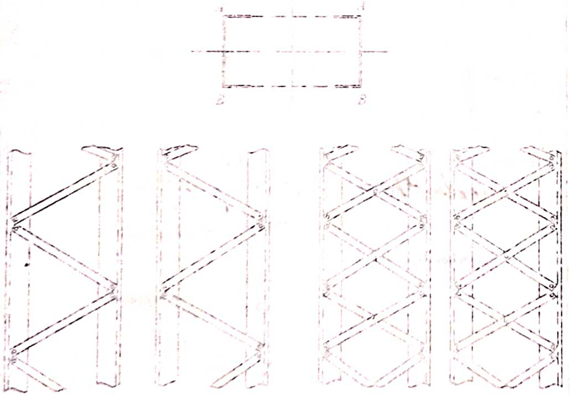
load carrying capacity

* To achieve maximum value for minimum radius of gyration, without increasing the area of the section, a number of elements are placed away from the principal axis using suitable lateral systems.

* The commonly used lateral systems are
  (a) Lacing (or) Latticing
  (b) Battening.

Lacing :-

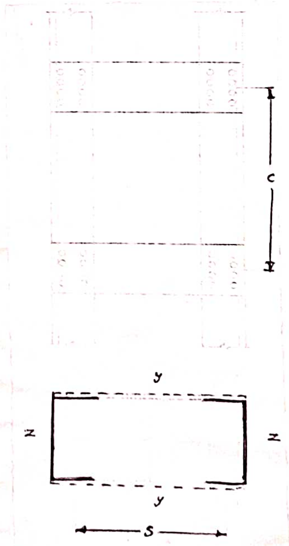
* Rolled steel flats and angles are used for lacing.
* One can use single lacing (or) double lacing system.
* The lacings are subjected to shear forces due to horizontal forces on columns.



Lacing on Face A | Lacing on Face B | Lacing on Face A | Lacing on Face B

Preferred Lacing Arrangement | Preferred Lacing Arrangement

(A) Single Laced System | (B) Double Laced System



Battened Column

Battens :-

* Instead of lacing one can use battens to keep members of columns at required distances.

Design of laced (or) Latticed columns

Part - I - Design of column Member

Step: 1 - Assume design compressive stress

The grade of steel $f_y = 250$ MPa,

range of $\boxed{f_{cd} = 125 - 175 \text{ MPa}}$

Step: 2 - Find the required c/s area

$$\boxed{A_{req} = \frac{P_u}{\text{Assume } f_{cd}}}$$

Step: 3 - Selection of suitable section consisting of two channels, or two angles or four angles or two I-sections with or without extra plates.

Step: 4 - The sections are so spaced, the radius of gyration about the YY- axis is equal to or greater than above zz-axis

$$\boxed{r_{yy} \geq r_{zz}}$$

Step: 5 - The effective length of column is estimated and the Slenderness ratio is determined. $\frac{KL}{r_{zz}}$

Step: 6 - The effective slenderness ratio of the laced column 1.05 times of the actual max. slenderness ratio.

IS: 800 - 2007 ; P. No: 48 ; clause : 7.6.1.5

Step: 7 - Find the design compressive stress for the estimated slenderness ratio.

Step: 8 - compute the design compressive strength, it should be more than the factored load.

$$\boxed{P_{d(\text{estimated})} > P_{u(\text{factored load})}}$$

Part - II - Design of Lacing

Step: 9 - Fix the angle of inclination (θ) of the lacing bar, with the longitudinal axis of the component. It should be kept b/w $40^\circ - 70^\circ$ (assume $\theta = 45^\circ$)

IS: 800 - 2007 ; Page NO: 50 ; clause: 7.6.4

Step: 10 - calculation of maximum spacing of lacing bars. It should not be greater than 50 (or) 0.7 times of the slenderness ratio.

IS: 800 - 2007 ; Page No: 50 ; clause: 7.6.5.1

Step: 11 - calculate the shear force of the lacing member

ie, $\boxed{V_t = 2.5\% \text{ of axial force in column}}$

IS: 800 - 2007 ; P.NO: 50 ; clause: 7.6.6.1

Step: 12 - calculate the compressive force in the lacing bar which is equal to

$$\boxed{\text{Comp. force} = \frac{V_t}{N} \cdot \operatorname{cosec}\theta}$$ For single lacing

$$\boxed{\text{comp. force} = \frac{V_t}{2N} \operatorname{cosec}\theta}$$ For double lacing.

Step: 13 - Find the width of the lacing bar.

It should not be less than 3 times of nominal dia of bolt

IS: 800 - 2007 ; P.NO: 50 ; clause: 7.6.2

$$\boxed{\text{Width of lacing bar} = 3d}$$ dia of bolt/rivet

step: 14 – Find Thickness of the lacing bar not less than

* For single lacing system, $\boxed{t \nless \frac{1}{40} \cdot l_{eff.}}$

* For Double lacing system $\boxed{t \nless \frac{1}{60} l_{eff.}}$

IS : 800 – 2007 ; P. No : 50 ; clause : 7.6.3

step: 15 – Find the Slenderness ratio of lacing bar, which should be less than 145.

IS: 800-2007 ; P. No : 50 ; clause : 7.6.6.3

step: 16 – calculate the design compressive strength of the lacing flat. It should be more than the force in the flat.

IS : 800 – 2007 ; P. No : 34 & 35 ; clause : 7.1.2 & Clause : 7.1.2.1 & Table : 7

step: 17 – The tensile strength of the flat is calculated. and it should be more than the force in the lacing flat

IS : 800 – 2007 ; P. No : 32 ; clause : 6.2 & 6.3.1

Part – III – Design of connection

step: 18 – calculate the strength of the bolt or rivet

Shearing strength V_{nsb}, bearing strength V_{npb} take least value

IS : 800 – 2007 ; P. No : 75 ; clause : 10.3.3 & 10.3.4

step : 19 – No. of bolt :

$$\boxed{\text{No. of bolt} = \frac{\text{Factored load}}{\text{strength of bolt}}}$$

UNIT IV Compression members (Cont)

Problem:-

Design a laced column with two channels back to back of length 10 m to carry an axial factored load of 1100 KN. The column may be assumed to have restrained in position but not in direction at both ends. (hinged end). Provide single lacing system.

Given Data :-

Length of column $(L) = 10\,m = 10{,}000\,mm$

Factored load $(P) = 1100\,kN$

End condition → Both ends are hinged

Arrangement → Two channels back to back

Part - I - Design of column member

Step : 1 - Design compressive stress (f_{cd})

Assume, $f_{cd} = 125\,N/mm^2$; $f_y = 250\,N/mm^2$

Step : 2 - Required c/s area

$$A_{req} = \frac{P_u}{f_{cd}} = \frac{1100 \times 10^3}{125}$$

$$\boxed{A_{req} = 8800\,mm^2}$$

For one channel $A_{req} = \frac{8800}{2}$

$$\boxed{A_{req} = 4400\,mm^2}$$

Step : 3 - Selection of suitable section

From SP:6 (1) - 1964 ; P. No: 6 ;

Take, ISMC 300 @ 35.8 kg/m

Area $= 45.64\,cm^2 = 4564\,mm^2$

For 2 channels $= 2 \times 4564 = 9128\,mm^2$

$$\boxed{Area_{prov} = 9128\ mm^2 > A_{req}\ (8800\ mm^2)}$$

Hence it is ok.

$b_f = 90mm$; $t_f = 13.6\ mm$; $t_w = 7.6\ mm$

$C_{yy} = 23.6mm$; $I_{zz} = 6362.6 \times 10^4\ mm^4$

$I_{yy} = 310.8 \times 10^4\ mm^4$; $r_{zz} = 118.1\ mm$; $r_{yy} = 26.1\ mm$

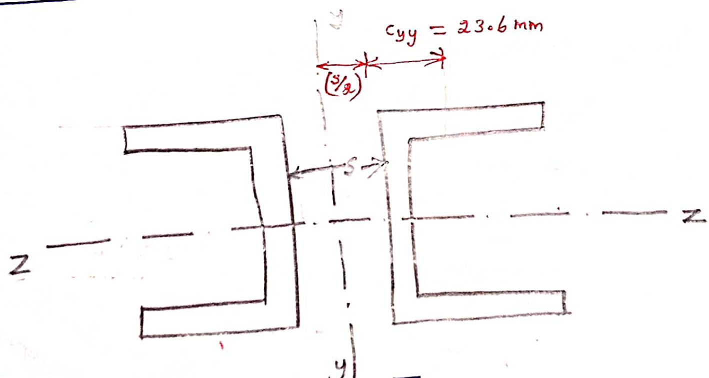
Step : 4 - Condition with radius of gyration :-

$$r_{yy} \geq r_{zz}$$

$r_{yy} = 26.1\ mm$; Let, $r_{min} = r_{zz} = 118.1\ mm$

$r_{zz} = 118.1\ mm$

Calculation of spacing b/w the members



$$I_{zz} = 2\left[I_{zz\,(single)} + Ah^2\right]$$

$$= 2\left[6362.6 \times 10^4 + (4564 \times 0^2)\right]$$

$$\boxed{I_{zz} = 127.26 \times 10^6\ mm^4} \text{ ---------- } (1)$$

$$I_{yy} = 2\left[I_{yy\,(single)} + Ah^2\right]$$

$$= 2\left[310.8\times10^4 + 4564 \times \left(C_{yy} + \frac{S}{2}\right)^2\right]$$

$$I_{yy} = 2\left[310.8\times10^4 + 4564 \times \left(23.6 + \frac{S}{2}\right)^2\right] \quad \text{— (2)}$$

Equating equation (1) & (2)

$$2\left[310.8\times10^4 + 4564\left(23.6 + \frac{S}{2}\right)^2\right] = 127.26\times10^6$$

$$310.8\times10^4 + 4564\left(23.6 + \frac{S}{2}\right)^2 = \frac{127.26\times10^6}{2}$$

$$\frac{127.26\times10^6}{2} - 310.8\times10^4 = 4564\left(23.6 + \frac{S}{2}\right)^2$$

$$\frac{63.63\times10^6 - 310.8\times10^4}{4564} = \left(23.6 + \frac{S}{2}\right)^2$$

$$\frac{60.522\times10^6}{4564} = \left(23.6 + \frac{S}{2}\right)^2$$

$$13260.74 = \left(23.6 + \frac{S}{2}\right)^2$$

$$115.16 = 23.6 + \frac{S}{2}$$

$$\therefore \boxed{S = 183.12\ mm} \qquad \text{say } \boxed{S = 200\ mm}$$

Step: 5 – Slenderness Ratio

Both ends are hinged (IS: 800 – 2007 ; F.NO: 45 ; Table: 11)

$$\therefore KL = L$$

$$\boxed{KL = 10{,}000\ mm}$$

Slenderness ratio, $\frac{KL}{r_{zz}} = \frac{10{,}000}{118.1} = 84.67$

Take max. value

$r_{zz} = 118.1\ mm$

$$\boxed{\frac{KL}{r_{zz}} = 84.67}$$

Step: 6 - Effective Slenderness Ratio

P.NO: 48
Clause: 7.6.1.5

$$\text{Eff. Slenderness ratio} = 1.05 \frac{KL}{r_{zz}}$$

$$= 1.05 \times 84.67$$

$$= 88.9$$

Step: 7 - Design compressive stress (f_{cd})

P. NO: 34 ; Clause: 7.1.2.1

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

$$= \sqrt{\frac{250 \times 88.9^2}{\pi^2 \times 2 \times 10^5}}$$

$$\boxed{\lambda = 1}$$

P.NO: 34 ; Clause: 7.1.2.1

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

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

$$\boxed{\phi = 1.196}$$

Buckling class

P.NO: 44

For Channel Section

Buckling class - C

Imperfection factor

$\alpha = 0.49$ (P.NO: 35)

P.NO: 34 ; Clause: 7.1.2.1

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

$$= \frac{250/1.1}{1.196\left[1.196^2 - 1^2\right]^{0.5}}$$

$$\boxed{f_{cd} = 122.7 \ N/mm^2}$$

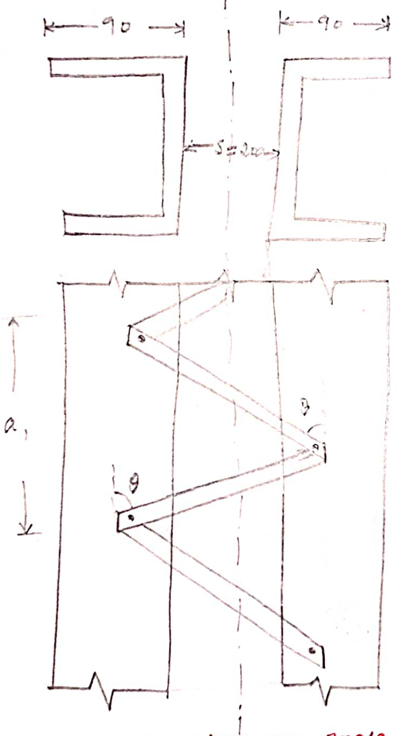
Step: 8 – Design comp. strength (P_d)

$$P_d = A_e \cdot f_{cd} = (2 \times 4564) \times 122.7$$

$$\boxed{P_d = 1120\ KN} > 1100\ KN$$

Part – II – Design of Lacing

Providing single Lacing system



Step: 9 – Determination of Angle (θ) of Lacing Flat

From IS: 800 – 2007 ; P.No: 50 ; Clause: 7.6.4

For, single Lacing system 40° to 70°

Take $\boxed{\theta = 45^\circ}$

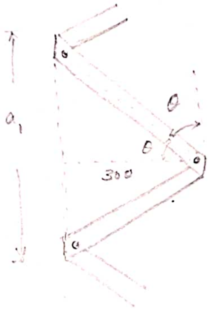
Step: 10 – calculation of Vertical spacing :-

Horizontal length of lacing flat $= s + g + g$

$g \rightarrow$ gauge distance, assume, $\boxed{g = 50\,mm}$

$\therefore$ Horizontal length $= s + g + g = 200 + 50 + 50$

$= 300\,mm$



From fig.

$$\tan\theta = \frac{a_1/2}{300}$$

$$\tan 45 = \frac{a_1/2}{300}$$

Vertical spacing: $\boxed{a_1 = 600\,mm}$

Step: 13 – Minimum width of lacing flat

IS: 800 – 2007 ; P. 110 : 50 ; Clause : 7.6.2

Minimum Width of lacing flat $= 3d$ ← nominal diameter of bolt/rivet

Assume, dia of bolt = 16 mm

$= 3 \times 16$

$= 48$

Say $\boxed{b = 50\,mm}$

Step: 14 – Minimum thickness of flat

IS : 800 – 2007 ; P. 110 : 50 ; clause : 7.6.3

For single lacing system,

$$t \nless \frac{1}{40}\, l_{eff}$$

$$t \nless \frac{1}{40} \times 425$$

$$t < 10.625\,mm$$

Minimum 10.625 mm ;

So $\boxed{\text{Provide 12 mm thickness}}$

$$\frac{300}{l} = \cos\theta$$

$$\therefore \cos 45 = \frac{300}{l}$$

$$\therefore l = \frac{300}{\cos 45}$$

$$\therefore l = 424.26$$

Say $\boxed{l = 425\,mm}$

∴ Provide 12 mm thickness plate with 50 mm width

$$\boxed{t = 12mm}$$

Step: 11 – Shear force in Lacing member:

IS:800-2007 - P.No: 50; clause: 7.6.6.1

$V_t = 2.5\%$ of axial force

$$= \frac{2.5}{100} \times 1100$$

$$\boxed{V_t = 27.5 \text{ kN}}$$

Transverse shearforce in each channel $= \frac{27.5}{2}$

$$\boxed{V_t = 13.75 \text{ KN}}$$

Step: 12 – Compressive force in Lacing flat

For single lacing,

$$\text{Comp. force in Lacing flat} = \frac{V_t}{N} \operatorname{cosec}\theta$$

$$= \frac{13.75}{1} \cdot \operatorname{cosec} 45$$

$$= \frac{13.75}{1} \times \frac{1}{\sin 45}$$

$$\boxed{\text{Comp. force in Lacing Flat} = 19.45 \text{ kN}}$$

Step: 15 – Slenderness ratio for Lacing bar

size of Lacing bar $b = 50$; $t = 12$ mm

$$r_{min} = \sqrt{\frac{I}{A}} \qquad I = \frac{bd^3}{12} = \frac{50 \times 12^3}{12} = 7200 \text{ mm}^4$$

$$= \sqrt{\frac{7200}{(50 \times 12)}} = \sqrt{\frac{7200}{600}}$$

$$\boxed{r_{min} = 3.46 \text{ mm}}$$

Slenderness ratio $= \frac{KL}{r} = \frac{400}{3.46}$

$$\boxed{\lambda = 115.61} < 145$$

IS : 800 : 2007
P.NO : 50 ;
Clause : 7.6.6.3

Hence it is ok

Step: 16 – Design compressive strength (P_d)

IS : 800 - 2007 ; P.NO : 45

For angle section, $\boxed{\text{Buckling class} = C}$

IS : 800-2007 ; P.NO : 35 ; Table : 7

Imperfection factor, for class c ;

$$\boxed{\alpha = 0.49}$$

P.NO : 34 →

$$\lambda_e = \sqrt{\frac{f_y/\left(\frac{KL}{r}\right)^2}{\pi^2 E}} = \sqrt{\frac{250 \times 115.61^2}{\pi^2 \times 2 \times 10^5}}$$

$$\boxed{\lambda = 1.3}$$

P.NO : 34 →

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

$$= 0.5\left[1 + 0.49\left(1.3 - 0.2\right) + 1.3^2\right]$$

$$\boxed{\phi = 1.61}$$

P.NO : 34

$$f_{cd} = \frac{f_y/\gamma_{mo}}{\phi + \left[\phi^2 - \lambda^2\right]^{0.5}} = \frac{250/1.1}{1.61 + \left[1.61^2 - 1.3^2\right]^{0.5}}$$

$$\boxed{f_{cd} = 88.79 \ N/mm^2}$$

$$P_d = A \cdot f_{cd} = 600 \times 88.79$$

$$\boxed{P_d = 53.27 \text{ kN}} > 19.45 \text{ kN}$$

Hence it is ok.

Step:17 - Tensile Strength of Lacing Flat

IS: 800 - 2007 ; Page No: 32

(i) Design strength due to yielding

$$T_{dg} = \frac{A_g \cdot f_y}{\gamma_{mo}} = \frac{600 \times 250}{1.1}$$

$$\boxed{T_{dg} = 136.36 \text{ kN}}$$

(ii) Design strength due to rupture

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

$$A_n = \left(b - n d_n + \cancel{\sum \frac{P_{si}^2}{4 g_i}}^{\,0}\right) t$$

$$A_n = [50 - 1(18)]\, 12$$

$$\boxed{A_n = 384 \text{ mm}^2}$$

$b = 50$
$n = 1$
$d_n = 16 + 2 = 18$ mm
$t = 12$ mm
$f_{ub} = 400$ mm
$\gamma_{m1} = 1.25$

$$T_{dn} = \frac{0.9 \times 384 \times 400}{1.25}$$

$$\boxed{T_{dn} = 110.6 \text{ kN}} > 19.45 \text{ kN}$$

Hence it is ok.

Part-III - Design of connections

Step: 18 - Strength of bolt

(i) Shearing strength of bolt

$$V_{nsp} = \frac{f_{ub}}{\sqrt{3}}\left(n_n \cdot A_{nb} + n_s \cdot A_{sb}\right)$$

It is in double shear

$$n_n = 1 \; ; \quad n_s = 1$$

$$A_{sb} = \frac{\pi d^2}{4} = \frac{\pi \times 16^2}{4} = 201.1 \ mm^2$$

$$A_{nb} = 0.78 \ A_{sb} = 0.78 \times 201.1 = 156.86 \ mm^2$$

$$V_{nsb} = \frac{400}{\sqrt{3}}\left[(1 \times 156.86) + (1 \times 201.1)\right]$$

$$\boxed{V_{nsb} = 82.67 \ kN}$$

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{m1}} = \frac{82.67}{1.25}$$

$$\boxed{V_{dsb} = 66.14 \ kN}$$

(ii) Bearing strength of bolt

$$V_{npb} = 2.5 \, K_b \cdot d \cdot t \cdot f_{ub}$$

Refer IS : 800 - 2007 clause : 10.2.4.2

$e = 1.7 \, d_o = 1.7 \times 18$

$e = 30.6 \ mm$

say $\boxed{e = 31 mm}$

Refer IS : 800 - 2007 clause : 10.2.2

$P = 2.5d = 2.5 \times 16$

$\boxed{P = 40 mm}$

k_b

(i) $\dfrac{e}{3d_o} = \dfrac{31}{3 \times 18} = 0.57$

(ii) $\dfrac{P}{3d_o} - 0.25 = \dfrac{40}{3 \times 18} - 0.25 = 0.49$

(iii) $\dfrac{f_{ub}}{f_u} = \dfrac{400}{410} = 0.976$

(iv) 1

Take least value, $\boxed{K_b = 0.49}$

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

$$\boxed{V_{npb} = 94.08 \text{ kN}}$$

$$V_{dpb} = \frac{V_{npb}}{\gamma_{m1}} = \frac{94.08}{1.25}$$

$$\boxed{V_{dpb} = 75.26 \text{ kN}}$$

Take least value above three

$$\boxed{\text{strength of bolt} = 66.14 \text{ kN}}$$

Step: 19 - No. of bolt

$$\text{No. of bolt} = \frac{\text{factored load}}{\text{strength of bolt}} = \frac{19.45 \times 10^3}{66.14 \times 10^3}$$

$$= 0.29$$

Say, $\boxed{\text{no. of bolt} = 1}$

Result:

Provide 50 ISF 12 flat @ 45° inclination then to the centre of gravity and connect with one bolt of 16 mm dia at each of channel end.

Design of column Bases

* column bases are to transmit the small loads such as concrete or masonry blocks.

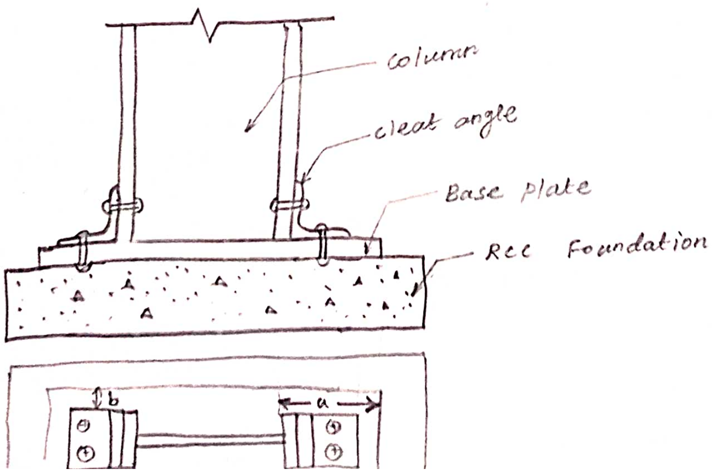
* The column bases spread the loads on wider area so that the intensity of varing pressure on the foundation block. is with in the bearing strength.

* **Types of column base**

    (i) slab base
    (ii) Gusseted base

*** slab Base :-**

* These are used in columns, carrying small loads.
* In this type of column is directly connected to base plate through cleat angles (shown in fig)
* The load is transfered to the base to the bearings.

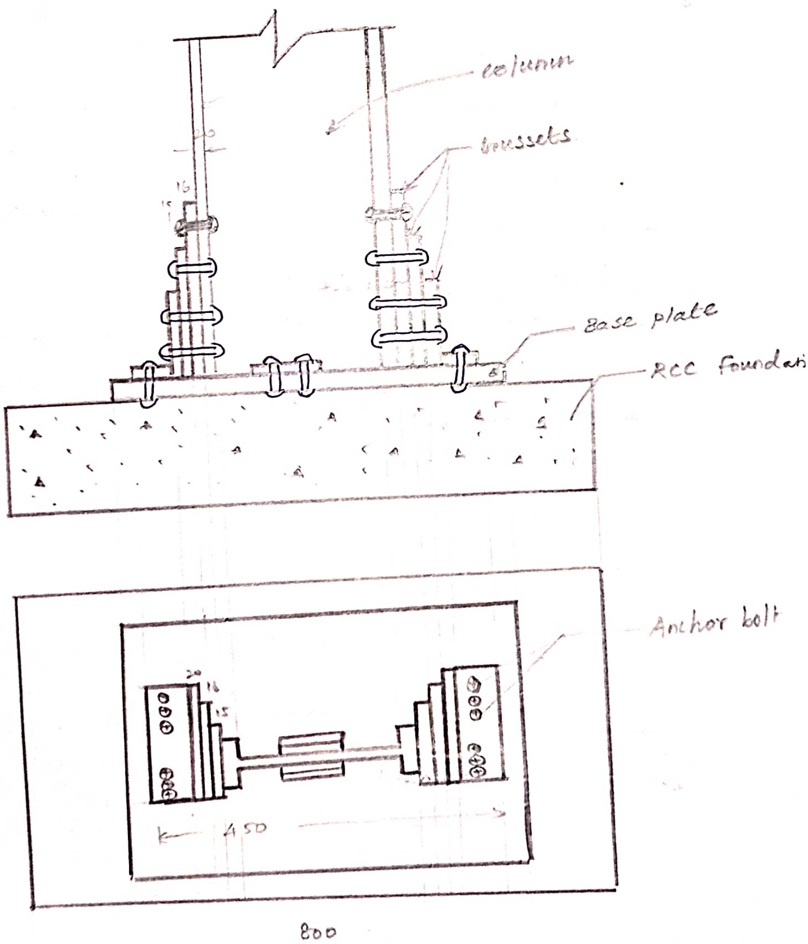


* Gusseted Base

* For columns carrying heavy loads then the gusseted plates are used.

* In this plate, the column is connected to base plate through gussets.

* The load is transferred to the base partly through bearing and partly through gussets.



Type - VI - Design of slab Base

(IS: 800-2007
P. NO: 46
Clause: 7.4)

* The Design of slab base consists in finding the size & thickness of slab base.

* It is assumed that the pressure is distributed uniformly under the slab base.

Part - I - Design of column Base

step : 1 - size of base plate

* Find the bearing strength of concrete $= 0.45 f_{ck}$

* Area of base plate required $= \dfrac{P_u}{0.45 f_{ck}}$ ← factored load

* select the size of base plate.

For the economy, as far as possible, if the projections

$$a = b$$

step: 2 - Thickness of base plate :-

* Find the intensity of Pressure

$$W = \frac{P_u}{\text{Area of base plate}}$$

* Minimum thickness required

IS: 800-2007 →
P. NO: 47
clause: 7.4.3.1

$$t_{min\ req}\ (\text{or})\ t_s = \left[\frac{2.5\,w\,(a^2 - 0.3b^2)\,\gamma_{mo}}{f_y}\right]^{0.5} > t_f$$

t_s → Thickness of base plate

t_f → Thickness of flange

step: 3 - connections

* If bolted connection, is to be used for connecting column to base plate, use 2 ISA 6565, 6 mm thick angles with 20 mm bolts.

* If welded connection, is to be used for connecting column to base check the weld length of fillet welds.

* connect base plate to foundation concrete, using 4 nos 20 mm diameter and 300 mm long anchor bolts.

Problem

Design a slab base for a column ISHB 300 @ 577 N/m carrying an axial factored load of 1000 kN. M_{20} concrete is used for the foundation. Provide welded connection between column and base plate.

Given Data:-

ISHB 300 @ 577 N/m

Factored load = 1000 kN

Grade of concrete $(f_{ck}) = 20\ N/mm^2$

Type of connection = welded connection.

Part - I - Design of Base Plate

Step: 1 - size of base plate

(i) Bearing strength of concrete

Bearing strength of concrete $= 0.45\ f_{ck} = 0.45 \times 20$

$= 9\ N/mm^2$

(ii) Area of base plate required.

$$A_{b.req} = \frac{\text{Factored load } (P_u)}{\text{Bearing Strength of concrete}} = \frac{1000 \times 10^3}{9}$$

$$\boxed{A_{b\ req} = 111.11 \times 10^3\ mm^2}$$

Hence provide 360 mm X 310 mm size of plate.

$\therefore$ A_b provided $= 360 \times 310 = 111.6 \times 10^3 \ mm^2$

$\therefore$ A_b req $<$ A_b provided

Hence ok.

Step: 2 - Thickness of base plate

(i) Intensity of pressure,

$$W = \frac{P_u}{A_{b\,(provided)}} = \frac{1000 \times 10^3}{111.6 \times 10^3}$$

$$\boxed{W = 8.96 \ N/mm^2}$$

(ii) Minimum thickness of base plate

From IS: 800 - 2007 : P. No: 47 ; clause: 7.4.3.1.

$$t_s = \sqrt{\frac{2.5 W (a^2 - 0.3 b^2) \gamma_{mo}}{f_y}} > t_f$$

From SP: 6(1)-1964

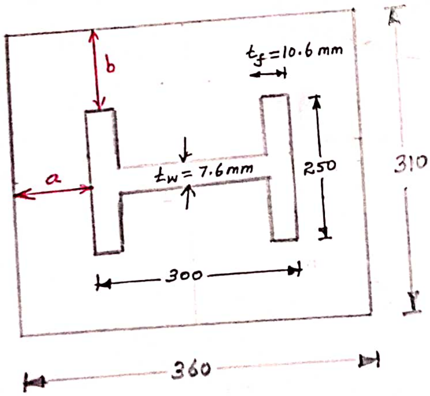
P.NO: 4

for ISHB 300 @ 577 N/m

$t_f = 10.6 \ mm$

$t_w = 7.6 \ mm$

$b_f = 250 \ mm$



$$a = \frac{360 - 300}{2} = 30 mm$$

$$b = \frac{310 - 250}{2} = 30 mm$$

$$t_s = \sqrt{\frac{2.5 \times 8.96\left(30^2 - 0.3 \times 30^2\right) \times 1.1}{250}} > t_f$$

$$t_s = 7.88 \text{ mm} \ngtr 10.6 \text{ mm}$$

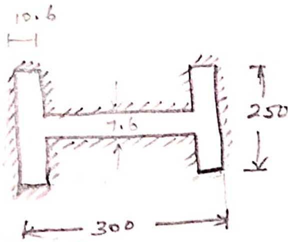
∴ Hence provide thickness of plate = 12 mm.

∴ connecting 360 X 310 X 12 mm plate to concrete foundation and use 4 Nos of hanger bolts of 20mm dia & 300 mm long.

Part – II – Design of connection

Assume the column to be connected with base plate by fillet weld.

* Total length available for welding



$$\text{Length of weld} = 2\left[250 + (250 - 7.6) + 300 - 2 \times 10.6\right]$$

$$= 1542.4 \text{ mm}$$

IS:800-2007
P.NO: 79
Clause: 10.5.7.1.1

Eff. Length of weld

* Strength of weld.

$$\text{Strength of weld} = \frac{f_u}{\sqrt{3}.\gamma_{mw}} . A_{throat}$$

Area of weld

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

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

size of weld.

$$\text{Area of weld} = 0.7\, S . L_e$$

eff. length

Size of weld (S)

IS:800-2007, P.No: 78 clause: 10.5.2.

min = 2.4 mm

∴ Provide $\boxed{S = 6 \text{ mm}}$

$\therefore$ Design Strength, based on that condition

$$\underbrace{(0.7\, s \,.\, L_e)}_{\text{Area of weld}} \frac{f_u}{\sqrt{3} \cdot \gamma_{mw}} = \text{factored load}$$

$$0.7 \times 6 \times L_e \times \frac{410}{\sqrt{3} \times 1.25} = 1000 \times 10^3$$

$$\therefore \boxed{L_e = 1257\ mm}$$

Check

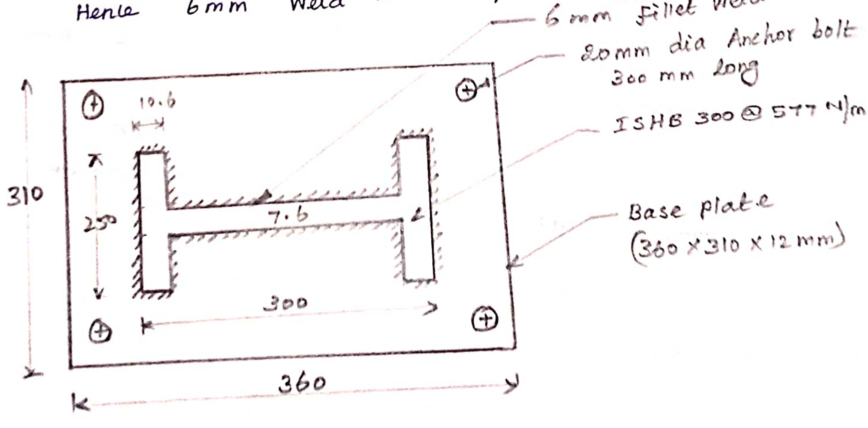
Note: After deducting for end return of the weld at the rate of twice the size of weld at each end.

$$\text{Available eff. length of weld} = \left[\text{Available length of weld} - 2 \times \text{size of weld} \times \text{No. of end returns}\right] > L_e$$

$$= \left[1542.4 - (2 \times 6 \times 12)\right] > 1257$$

$$= 1398.4 > 1257\ mm$$

Hence 6 mm weld is adequate.



Type – VII – Design of ~~gusseted~~ gusset Base

* IS: 800 – 2007 specified that the gusset plates, angle cleat stiffeners and fastenings etc., in combination with the bearing area, shall be sufficient to take the loads, bending moments and reactions to the base plate without exceeding specified strength.

Design procedure :-

Step: 1 –

$$\text{Area of base plate} = \frac{\text{Factored load}}{\text{Bearing Strength of Concrete}} = \frac{P_u}{0.45 f_{ck}}$$

Step: 2

Assume various members of gusset base

(a) Thickness of gusset plate = 16 mm (assumed)

(b) Size of gusset angle is assumed such that its vertical leg ~~is~~ can accommodate two bolt in one vertical line

corresponding to this leg the other leg is assumed in which one bolt can be provided.

(c) Assume, Thickness of angle is approximately equal to the thickness of gusset plate.

Step: 3 – Width of gusset plate is kept such that ~~is~~ ~~with~~ b/w just project outside the gusset angle and hence length will be,

$$\text{Length} = \frac{\text{Area of Plate}}{\text{Width}}$$

Step:4 – When the end of the column is assumed to complete bearing on the base plate 50% of load is assumed to be transferred by the bearing and another 50% by the fastenings.

Step:5 – Thickness of base plate

It is computed by flexural strength at the critical sections.

Problem:-

Design a gusseted base for a column ISHB ~~4~~50 @ 855 N/m with two cover plates 250 mm × 20mm carrying a factored load of 2500 kN. The column is to be supported on concrete pedastral to be built with M_{20} concrete. Take effective height of column is 4 m.

Given Data

ISHB ~~4~~50 @ 855 N/m

Factored load = 2500 kN

Height of column = 4 m = 4000 mm

Cover plate size = 250 × 20mm.

Step: 1 – properties of section

SP:6 (1) – 1964 : P.No: 4

$b_f = 250$ mm

$t_f = 13.7$ mm

$t_w = 9.8$ mm

Step: 2 – Required Area of base plate

Grade of concrete, $f_{ck} = 20\ N/mm^2$

$$\text{Area req} = \frac{\text{Factored load}}{\text{Bearing strength of concrete}} = \frac{P_u}{0.45 f_{ck}} = \frac{2500 \times 10^3}{0.45 \times 20}$$

$$\boxed{A_{req} = 277.77 \times 10^3\ mm^2}$$

Step: 3 – Selection of suitable section

(i) select section ISA 150 X 115 X 15 mm (SP:6(1)-1964 P.NO: 14)

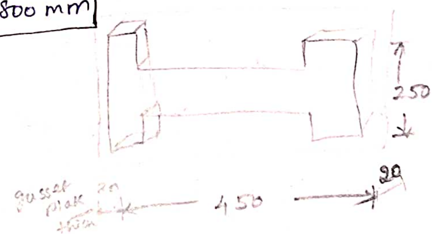
(ii) select gusset plate thickness = 16 mm

(iii) ∴ Minimum length required for Base plate } = $\underset{\text{cover plate width}}{450} + (\underset{\text{thick}}{2 \times 20}) + (\underset{\text{gusset plate thick}}{2 \times 16})$

$+ (\underset{\text{Angle thick}}{2 \times 15}) + (\underset{\text{angle width}}{2 \times 115})$

$= 782 \text{ mm}$

∴ say $\boxed{\text{Length of Base plate} = 800 \text{ mm}}$



(iv) Breadth of base plate

$$\text{Breadth} = \frac{\text{Area of base plate}}{\text{Length of base plate}} = \frac{277.77 \times 10^3}{800}$$

$$B = 347.2$$

Say $\boxed{B = 350 \text{ mm}}$

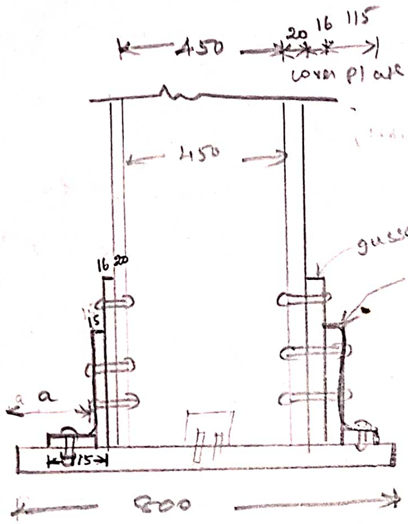
∴ Area provided $= l.B = 800 \times 350 = 280 \times 10^3 \text{ mm}^2$

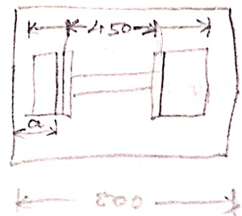
∴ $A_{req} < A_{provide}$, Hence ok.

(v) Pressure under the base plate,

$$\text{pressure under the base plate} = \frac{\text{Load}}{\text{Area Provided}} = \frac{2500 \times 10^3}{280 \times 10^3}$$

$$= 8.93 \text{ N/mm}^2$$





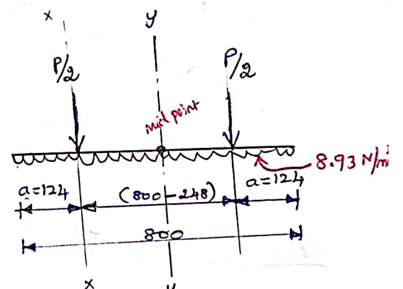
Step:4
Projections 'a'

$$a = \frac{800 - 450 + (20+16+15)2}{2}$$

$$\boxed{a = 124\ mm}$$

Thickness of base plate

Step: 5 - B.M. calculation



B.M @ section XX , $M_{xx} = 8.93 \times 124 \times \frac{124}{2}$

$$\boxed{M_{xx} = 68.65 \times 10^3\ N.mm}$$

Determination of Max. B.M.

B.M @ section YY   Max. B.M will be @ center.

load per mm width column width

$P = 8.93 \times 450 \times 1$

$\boxed{P = 4018.5\ N}$

$$M_{yy} = \left(8.93 \times 400 \times \frac{400}{2}\right) - \frac{4018.5}{2} \times (400 - 124)$$

$$= 190.99 \times 10^3\ N.mm$$

Equating moment of resistance to B.M

P.53 8.2.1.2 ie, $1.2 \times \underbrace{\frac{1}{6} b t^2}_{Ze} \frac{f_y}{\gamma_{mo}} = B.M$

$$1.2 \times \frac{1}{6} \times 1 \times t^2 \times \frac{250}{1.1} = 190.99 \times 10^3$$

$$\boxed{t = 64.8\,mm}$$

~~Hence use 65 mm thickness of base plate~~

~~∴ Use 800 X 350 X 65 mm size of base plate.~~

Use 65 mm base plate of size 850 X 350 mm

Step:6 –

Assuming ends of columns are faced for complete bearing, the connection b/w gusset plate and column will be designed for 50 percent of axial load

$$\text{Design load} = \frac{50}{100} \times 2500 = 1250\,kN.$$

$$\text{Load on each splice} = \frac{1250}{2} = 625\,kN.$$

Step: 7 – Design of connections

Assume 24 mm dia bolt in single shear

(i) Shearing strength of bolt IS: 800: 2007 – P.No: 75 Clause: 10.3.3

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{\frac{f_{ub}}{\sqrt{3}} \cdot (n_n \cdot A_{nb} + n_s \cdot A_{sb})}{\gamma_{mb}}$$

$$= \frac{\frac{400}{\sqrt{3}} (1 \times 352.87 + 0)}{1.25} = \frac{81.49}{1.25}$$

$$\boxed{V_{dsb} = 65.19\,kN}$$

In single shear

$n_n = 1$

$n_s = 0$

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

$= 452.4\,mm^2$

$A_{nb} = 0.78\,A_{sb}$

$= 0.78 \times 452.4$

$A_{nb} = 352.87\,mm^2$

(ii) Bearing Strength of Bolt   P.NO: 75 ; clause: 10.3.4

$$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}} = \frac{2.5 k_b \cdot d \cdot t \cdot f_{ub}}{\gamma_{mb}}$$

$$= \frac{2.5 \times 0.583 \times 24 \times 65 \times 400}{1.25}$$

$$= 727.58 \text{ kN.}$$

k_b (Take Least Value)

(i) $\frac{e}{3d_0} = \frac{50}{3 \times 26} = 0.641$

(ii) $\frac{P}{3d_0} - 0.25$ $\frac{65}{3 \times 26} - 0.25$

(iii) $\frac{f_{ub}}{f_u} = 0.583$

(iv) 1

0.583

$e = 1.7 d_0$
$= 1.7 \times 26$
$= 44.2$

say 50

$P = 2.5 d.$
$= 2.5 \times 24$
$= 60$

say 65 mm

Due to heavy load on column the bearing strength of bolt will also be more than the shearing strength of bolt.

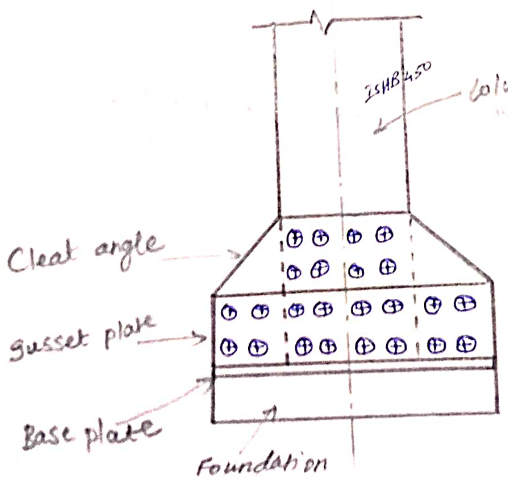
$\therefore$ Bolt value $= 65.19$ kN.

No. of Bolt

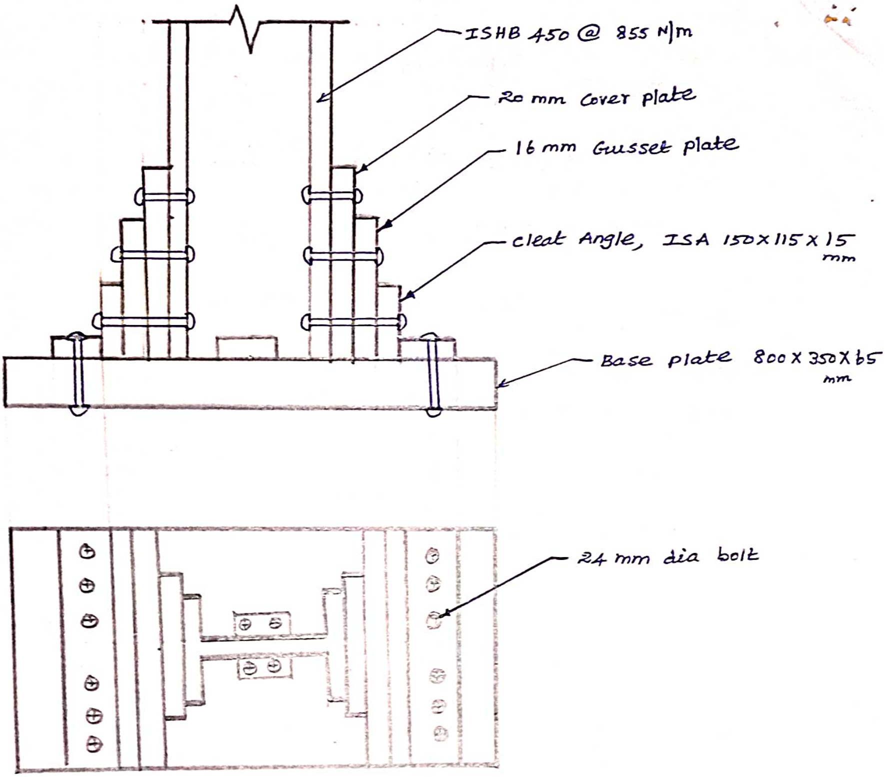
$$n = \frac{\text{factored load}}{\text{Bolt value}} = \frac{625}{65.19} = 9.5$$

say $\boxed{n = 10 \text{ nos}}$

For arrangement purpose, No. of bolt $= 16$ nos.



Provide 24 mm dia bolts of 16 nos for connecting column to gusset plate & another 8 bolts to connect cleat angle to gusset plate

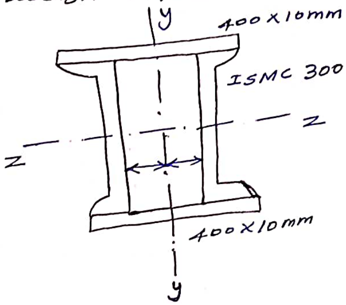


H.W

Design a gusseted base for a column ISHB 350 @ 710 N/m with two plates 450mm X 20mm carrying a factored load of 3600 kN. The column is to be supported on concrete pedastal to be built with M_{20} concrete.

1) Fig. 13(a) shows a built up column section. The column has an effective length of 4.75 m. Find the design compressive load for the column. Take $f_y = 250\ N/mm^2$.

May/June 2016



Given Data

Effective length $(KL) = 4.75\,m$

$f_y = 250\ N/mm^2$

* Properties of ISMC 300

From SP:6(1) – 1964, P.No: 6

c/s area $= A = 4564\ mm^2$

Depth $h = 300\,mm$

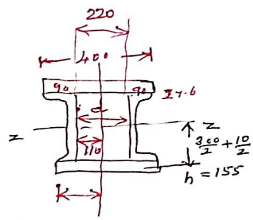
$b_f = 90\,mm$; $t_f = 13.6\,mm$

$t_w = 7.6\,mm$

$c_{yy} = 23.6\,mm$

$I_{zz} = 6362.6 \times 10^4\ mm^4$

$I_{yy} = 310.8 \times 10^4\ mm^4$



* Compound section properties

Combined area $A = A_{\text{channel section}} + A_{\text{plate}}$

$= 2\left[4564 + (400 \times 10)\right]$

$$\boxed{A = 17128\ mm^2}$$

$I_{zz} = I_{zz\ \text{channel}} + I_{zz\ \text{plate}}$ $\quad bd^3/12 + Ah^2$

$= 2\left[6362.6 \times 10^4 + \left(\frac{400 \times 10^3}{12}\right) + 400 \times 10 \times 155^2\right]$

$\qquad 33333.33 \quad + \quad 96100000$

$$\boxed{I_{zz} = 319.52 \times 10^6\ mm^4}$$

$I_{yy} = I_{yy\ \text{channels}} + I_{yy\ \text{plate}}$ $\quad \left(\frac{d}{2} + c_{yy}\right)^2$ (h)

$= 2\left[310.8 \times 10^4 + \underset{A}{4564}\left(\frac{400-180}{2} + c_{yy}\right)^2 + \frac{db^3}{12}\right]$

$= 2\left[310.8 \times 10^4 + 4564\left(110 + 23.6\right)^2 + \frac{10 \times 400^3}{12}\right]$

$\qquad 81462653.44 \qquad 53333333.33\ (53.33 \times 10^6)$

$$\boxed{I_{yy} = 275.8 \times 10^6\ mm^4}$$

Take least value

$\therefore$ $$\boxed{I_{min} = 275.8 \times 10^6\ mm^4}$$

* Slenderness ratio

$$\text{slenderness ratio} = \frac{KL}{r_{min}}$$

$$r_{min} = r_{yy} = \sqrt{\frac{I_{min}}{A}} = \sqrt{\frac{275.8 \times 10^6}{17128}} =$$

$$\boxed{r_{min} = 126.89 \text{ mm}}$$

$$\text{Slenderness ratio} = \frac{KL}{r_{min}} = \frac{4750}{126.89} = 37.43$$

* Buckling classification

From IS : 800-2007 ; P.NO : 44, Table : 10

For built up section, Buckling class = 'C'

* Design compressive stress (f_{cd})

from IS : 800-2007 ; P.NO : 42 ; Table : 9 (C)

For, $\frac{KL}{r} = 30 \longrightarrow f_{cd} = 211 \text{ N/mm}^2$

$f_y = 250 \text{ N/mm}^2$, $\frac{KL}{r} = 40 \longrightarrow f_{cd} = 198 \text{ N/mm}^2$

$\therefore \frac{KL}{r} = 37.43 \longrightarrow 211 - \left(\frac{211-198}{40-30}\right) \times (37.43 - 30)$

$$\therefore \boxed{f_{cd} = 201.34 \text{ N/mm}^2}$$

* Design compressive strength (P_d)

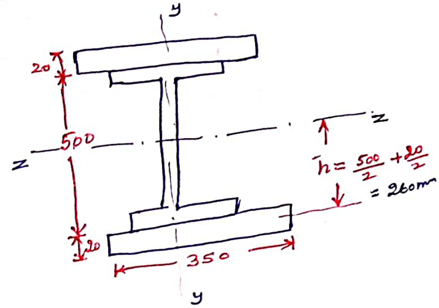
IS : 800-2007 ; P.NO : 34,

$$P_d = A_e \cdot f_{cd} = 17128 \times 201.34$$

$$\boxed{P_d = 3448.5 \text{ KN}}$$

— × — × — × — × — × — × —

1) Calculate the compressive resistance of a compound column consisting ISMB 500 with one cover plate 350 X 20 mm on each flange and having a length of 5m. Assume that the bottom of column is fixed and top is rotation fixed, translation free. Take $f_y = 250\ N/mm^2$. $\boxed{\text{NOV/DEC 2016}}$



Given Data

$f_y = 250\ N/mm^2$

$L = 5m$

End condition : $KL = 1.2L$

P.NO: 45
Table: 11

* **Properties of ISMB 500**

From SP: 6(1) – 1964, P.NO: 2

$A = 110.74\ cm^2 = 11074\ mm^2$

$h = 500mm$; $b_f = 180mm$; $t_f = 17.2mm$

$I_{zz} = I_{xx} = 45218.3 \times 10^4\ mm^4$; $t_w = 10.2mm$

$I_{yy} = 1369.8 \times 10^4\ mm^4$;

$r_{zz} = r_{xx} = 20.21cm = 202.1\ mm$

$r_{yy} = 3.52\ cm = 35.2\ mm.$

* **Compound section properties**

Combine area $A = A_{I\text{-section}} + A_{plate} = 11074 + 2(350 \times 20)$

$$\boxed{A = 25074\ mm^2}$$

$$I_{zz} = I_{zz\,c.n} + \left[\overset{No}{2} \times \overset{\text{Area of plate}}{(350 \times 20)} \times \overset{h^2}{\left(\frac{500}{2} + \frac{20}{2}\right)^2}\right]$$

$$= 45218.3 \times 10^4 + \left[\underbrace{2 \times 350 \times 20 \times 260^2}_{9464 \times 10^5}\right]$$

$$\boxed{I_{zz} = 1398.583 \times 10^6\ mm^4}$$

$$I_{yy} = I_{yy\,cn} + 2\frac{db^3}{12} = 1369.8 \times 10^4 + \underbrace{\left(2 \times \frac{20 \times 350^3}{12}\right)}_{142916666.67}$$

$$\boxed{I_{yy} = 156.615 \times 10^6\ mm^4}$$

Take minimum value

$$\therefore \boxed{I_{min} = I_{yy} = 156.615 \times 10^6\ mm^4}$$

* **Slenderness ratio**

$$\text{Slenderness ratio} = \frac{KL}{r_{min}}$$

$$r_{min} = r_{yy} = \sqrt{\frac{I_{yy}}{A}} = \sqrt{\frac{156.615 \times 10^6}{25074}}$$

$$\boxed{r_{min} = r_{yy} = 79.03 \text{ mm}}$$

$$\text{Slenderness ratio} = \frac{KL}{r_{min}} = \frac{1.2 \times 5000}{79.03} = 75.92$$

* **Buckling classification**

from IS:800-2007 ; P.NO: 44, Table: 10

For built up section, Buckling class = 'C'

* **Design compressive stress (f_{cd})**

IS:800-2007 ; P.NO: 42, Table: 9 (C)

For $\frac{KL}{r} = 70 \longrightarrow f_{cd} =$

$\frac{KL}{r} = 80 \longrightarrow f_{cd} =$

$\therefore \frac{KL}{r} = 75.92 \longrightarrow f_{cd} =$

$\therefore f_{cd} =$

* **Design compressive strength (P_d)**

IS:800-2007 ; P.NO: 34,

$P_d = A_e \cdot f_{cd} = 25074 \times$

$P_d =$

—— x —— x —— x —— x —— x —— x —— x

UNIT : V DESIGN OF BEAMS.

BEAMS: Beam is a structural member with length considerably larger than cross-sectional dimensions subject to lateral loads which give rise to bending moment shear forces in the member.

Purlin:

* It is rest b/w the trusses and to support roof sheets on beams.
* For this purpose angles or channels are commonly used.
* T-Sections are used in water tanks to support steel plates.
* In buildings, I-Sections are commonly used as beams.
* For Heavy loads, I-Sections with additional Plates connected on Flanges are used.
* If still heavier sections are required built up Sections like Plate girders are used.

Types of Beams:

* Based on lateral supports to compression flanges there are mainly two types of Beams.

(i) Laterally supported Beams.

(ii) Laterally unsupported Beams

* If the compression flanges are Laterally supported by flooring, it is mainly subjected to bending and shear.

* If the compression flange of beam is not laterally supported, the lateral buckling of the compression flange reduces the load carrying capacity of the beam.

Classifications of cross section:

* When the plastic analysis is used, the members should be capable of forming plastic hinges with sufficient rotation capacity without local buckling.

* Hence it is necessary to see that plate elements of a cross section do not buckle locally due to compressive stresses before plastic hinges are formed.

* The local buckling can be avoided before the limit state is achieved by limiting the width to thickness ratio of each element of a cross section subjected to compression due to axial force, moment or shear.

* On the basis, IS 800 - 2007, classifies various cross - section as follows.

(i) class 1 - plastic cross section
(ii) class 2 - Compact cross section.
(iii) class 3 - Semi compact cross section.
(iv) class 4 - Slender cross sections.

Type I. Design of Laterally supported Beam.

PROBLEM NO: 1: Design a simply supported beam of 5m span, carrying a reinforced concrete floor, cable of providing lateral restrained to the top compression flange. The UDL is made up of 20 kN/m. imposed load and 20 kN/m Dead load. Assume Fe410 grade of steel.

Given Data:

Type of Beam = Simply supported.

Span (l) = 5m.

Imposed load = 20 kN/m.

Dead load = 20 kN/m.

Grade of steel = Fe410.

solution:

step 1: load calculation:

Factored imposed load = 1.5×20 = 30 kN/m.

Factored Dead load = 1.5×20 = 30 kN/m

Total : 60 kN/m

Step 2: B.M and S.F calculation.

$$B.M \rightarrow \frac{wl^2}{8} \Rightarrow \frac{60 \times 5^2}{8} \Rightarrow 187.5 \text{ kN m.}$$

$$S.F \rightarrow \frac{wl}{2} \rightarrow \frac{60 \times 5}{2} \Rightarrow 150 \text{ kN.}$$

Step 3: Required Section Modulus:

$$Z_p = \frac{M}{f_y} \cdot \gamma_{m0} \Rightarrow \frac{187.5 \times 10^6 \times 1.1}{250}$$

$$Z_p = 825 \times 10^3 \text{ mm}^3 \Rightarrow 825 \text{ cm}^3.$$

Step ④. Selection of a suitable section:

IS 800 - 2007, P. No: 138, Table: 46.

Try. ISLB 350, $Z_p = 851.11 cm^3$

Properties of ISLB 350:

$w = 49.5$ kg/m. $\Rightarrow$ 495 N/m.

$b_f = 165$ mm, $t_f = 11.4$ mm

$t_w = 7.4$ mm, $Z_e = 751.9 cm^3$.

Step ⑤: check the class of section.

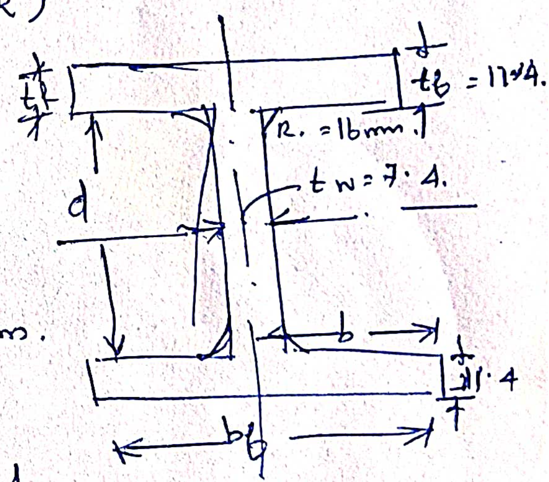
SP 6 (1) - 1964, P. No; 2

ISLB : 350, radius of the root 'R' = 16mm.

Depth of web, $d = h - 2(t_f + R)$

$d = 350 - 2(11.4 + 16)$

$\boxed{d = 295.2 mm}$



$b = \frac{b_f}{2} = \frac{165}{2} \Rightarrow 82.5$ mm.

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

Refer. Table 2. P. 18.

$\frac{b}{t_f} \Rightarrow \frac{82.5}{11.4} = 7.23 < 9.4\varepsilon.$

$\frac{d}{t_w} = \frac{295.2}{7.4} = 39.89 < 84\varepsilon$

Hence the section classified as plastic section.

step (6) : check the adequacy of section including self wt. of the Beam.

Self wt of Beam = 495 N/m

Factored self wt = 1.5 × 495 ⇒ 742.5 N/m.

= 0.743 kN/m

Total load acting on the beam. (including self wt of Beam).

= 60 + 0.743

= 60.743 kN/m

$$\text{Max. B.M} = \frac{wl^2}{8} \Rightarrow \frac{60.743 \times 5^2}{8} \Rightarrow 189.82 \text{ kNm.}$$

$$\text{max. S.F (V)} = \frac{wl}{2} \Rightarrow \frac{60.743 \times 5}{2} \Rightarrow 151.86 \text{ kN.}$$

Required Section modulus:

$$Zp_{req} = \frac{M}{f_y} \cdot \gamma_{m0} \Rightarrow \frac{189.82 \times 10^6}{250} \times 1.1$$

$$Zp_{req} = 835.21 \text{ cm}^3.$$

Zp (provided) > Zp req.

$851.11 \text{ cm}^3 > 835.21.$

Hence ok.

step (7): check for Design shear strength.

Refer IS 800 - 2007, P. No: 59, clause. 8.4

$V \leq V_d.$

$$V_p = \frac{A_v \cdot f_{yw}}{\sqrt{3}}$$

Where,

$A_v = h \times t_w$

$= 350 \times 7.4$

$A_v = 2590 \text{ mm}^2$

$f_{yw} = 250 \text{ N/mm}^2$

Refe. P. No: 59, clause 8.4.1

$$V_p = \frac{A_v \cdot f_{yw}}{\sqrt{3}}$$

$$= \frac{2590 \times 250}{\sqrt{3}} \Rightarrow 373.83 \times 10^3 \text{ N.}$$

$$\boxed{V_p = 373.83 \times 10^3 \text{ N}}$$

$V_n = V_p$.

$$V_d = \frac{V_n}{\gamma_{m0}} = \frac{373.83 \times 10^3}{1.1} \Rightarrow 339.85 \text{ kN.}$$

$V = 151.86 < V_d = 339.8$ kN.

actual. design shear.

Hence it is ok.

Step ⑧: check for Design Bending strength.

IS 800 - 2007, P. No: 52, clause 8.2, 8.2.1.3

$M \leq M_d$.

(i) $0.6\, V_d > V$

$0.6 \times 339.85 > 151.86$

$203.91 \text{ kN} > 151.86 \text{ kN}$.

Hence ok.

Take. $M_d = \frac{\beta_b \cdot Z_p \cdot f_y}{\gamma_{m0}}$ → P. 53, 8.2.1.2

where. $Z_p = 851.11 \times 10^3 \text{ mm}^3$

$\beta_p = 1$.

$$\therefore M_d = \frac{1 \times 851.11 \times 250}{1.1} \Rightarrow 193.43 \text{ kN.m.}$$

$$M_d \leq \frac{1.2\, Z_e \cdot f_y}{\gamma_{m0}} \Rightarrow 193.43 \leq \frac{1.2 \times 751.9 \times 10^3 \times 250}{1.1}$$

$193.43 < 205.1$

Hence ok.

step ⑨ : check for deflection :

DL = 20+20.

For S.S. Beams.

$I_{zz} = 13158.3 \times 10^4 \ mm^4$

$$\delta_{max} = \frac{5wl^4}{384 E \cdot I} \Rightarrow \frac{5 \times 20 \times 5000^4}{384 \times 2 \times 10^5 \times 13158.3 \times 10^4}$$

$\delta_{max} = 6.20 \ mm$

Allowable max. deflection $= \frac{L}{300} \Rightarrow \frac{5000}{300} = 16.67 \ mm.$

$\delta_{max} < \delta_{allowable}$.

Hence safe.

PROBLEM NO: ②: Design a Simply supported beam of eff. span 1.5 m carrying a factored concentrated load of 360 kN at midspan. Assume the beam is laterally restrained.

Given Data:

Type of beam = S.S.

L = 1.5 m

W = 360 kN.

Solution: step ①

1. Load calculation..

factored concentrated load. = 360 kN.

② Max. B.M. $= \frac{Wl}{4} = \frac{360 \times 1.5}{4} \Rightarrow 135 \ kN.m.$

③ Section modulus required.

$$Z_p = \frac{M}{f_y} \cdot \gamma_{m0}. \Rightarrow \frac{135 \times 10^6 \times 1.1}{250}$$

$$\boxed{Z_p = 594 \times 10^3 \ mm^3}$$

Step ④: Select a trial section ISMB 350 @ 524 N/m. (from steel table P. No. 2).

$t_w = 8.1$, $t_f = 14.2\,mm$, $b_f = 140\,mm$,

$D = 350\,mm$, $Z_{ez} = 779\,cm^3 \Rightarrow 779 \times 10^3\,mm^3$

$Z_p = 889.57\,cm^3 \Rightarrow 889.57 \times 10^3\,mm^3$.

Radius of root = 14 mm.

$I_{zz} = 13630.3 \times 10^4\,mm^4$,

Now, $d = h - 2(t_f + r)$

$= 350 - 2(14.2 + 14)$

$\boxed{d = 293.6\,mm}$.

Step ⑤: Section classification.

(from IS 800 - 2007, P. No: 18, table. 2)

$b/t_f = \dfrac{140/2}{14.2} = 4.93 < 9.4$.

$d/t_w = \dfrac{350}{8.1} = 43.21 < 84$.

Step ⑥: Adequacy of section including self weight.

Self wt = 524 N/m

= 0.524 kN/m.

factored self wt = 1.5×0.524 = 0.786 kN/m.

Total factored load = 360 + 0.786

= 360.786 kN

Max. B.M = $\dfrac{Wl}{4} \Rightarrow \dfrac{360.786 \times 1.5}{4} \Rightarrow 135.29$

$\boxed{M_{max} = 135.29\,kN.m}$

Section Modulus:

$$Z_p = \frac{M}{f_y} \times \gamma_{M0} \Rightarrow \frac{135.29 \times 10^6 \times 1.1}{250}$$

$$Z_p = 595.30 \times 10^3 \ mm^3 < 889.57 \times 10^3 \ mm^3$$

Hence safe.

(7) Check for shear strength.

$$\text{Design shear force} = \frac{W}{2} + \frac{wl}{2}$$

point load. self wt.

$$V = \frac{360}{2} + \frac{0.786 \times 1.5}{2}.$$

$$\underline{V = 180.59 \ kN}$$

∴ Design shear strength, $V_d = \frac{A_v \times f_y}{\sqrt{3} \times \gamma_{M0}}$. → P. 59.

$$V_d = \frac{(h \times t_w) \times f_y}{\sqrt{3} \times \gamma_{M0}}$$

$$= \frac{(350 \times 8.1) \times 250}{\sqrt{3} \times 1.1} \Rightarrow 372 \times 10^3 \ N.$$

$$V_d = 372 \times 10^3 \ N. > 180.59 \ kN.$$

Hence safe.

(8) Check for design moment.

$$0.6 V_d = 0.6 \times 372 \Rightarrow 223.2 \ kN \not< 180.59 \ kN.$$

ie. $V < 0.6 V_d$.

(9) $\therefore M_d = \dfrac{\beta_p \cdot Z_p \cdot f_y}{\gamma_{M0}}$ → P. No: 53.

$= \dfrac{1 \times 889.57 \times 10^3 \times 250}{1.1}$

$= 202.18 \times 10^6$ N.mm

$= 202.18$ kN.m.

$\leq 1.2 \times Z_e \times f_y \times \dfrac{1}{\gamma_{M0}} \Rightarrow \dfrac{1.2 \times 779 \times 10^3 \times 250}{1.1}$

$= 212.45$ kN.m.

$\therefore M_d = 202.18 \leq 212.45$ kN m.

Hence safe.

(10) Check for deflection.

$\delta = \dfrac{Wl^3}{48EI} \Rightarrow \underline{360}$

Working load $= \dfrac{360}{1.5} = 233.33$ kN.

$\delta = \dfrac{233.33 \times 10^3 \times 1500^3}{48 \times 2 \times 10^5 \times 13630.3 \times 10^4}$

$\delta_{max} = 0.60$ mm.

~~Max.~~ defln. allowed $= \dfrac{\text{Span}}{360} \Rightarrow \dfrac{1500}{360} = \underline{\underline{5\text{mm}}}$.

$\therefore \delta_{max}$ 0.6mm $< \delta_{allow}$ 5mm.

Hence safe.

Type II Laterally unsupported Beams.

* when a steel Beam is subjected to bending stress under the application of load, the compression flange of a beam tends to buckle horizontally.

- On the other hand, the bottom flange being tension and other to remain straight.
- since two flanges and the web actually single rigid unit, when the buckling occurs consequently.
- It will lead to entire c/s to rotate.
- Thus the beam deflects laterally and which causes the beam to twist.

Ref. IS-800-2007, P.54, 8.2.2.

PROBLEM NO: 3 :

Design a Laterally unrestrained (unsupported) beam to carry an udl of 50 kN/m. The beam is supported for a length of 1.5m.

Given Data:

udl = 50 kN/m.

l = 1.5m.

Solution:

step 1: calculation of load.

factored load = 1.5 x 50 kN => 75 kN m

= $\boxed{75 \text{ kN m}}$

Step ②: calculation of B.M and S.F

$$\text{Max. B.M} = \frac{wl^2}{8} = \frac{75 \times 1.5^2}{8} \Rightarrow \underline{21.09 \text{ kN.m}}$$

$$\text{Max S.Force} = wl/2 \Rightarrow \frac{75 \times 1.5}{2} \Rightarrow 56.25 \text{ kN}.$$

Step ③: Req. plastic section modulus.

$$Z_{p\,req} = 1.3 \times \frac{M}{f_y} \cdot \gamma_{M0} \Rightarrow \frac{1.3 \times 21.09 \times 10^6 \times 1.1}{250}$$

$$Z_{p\,req} = 120.63 \times 10^3 \text{ mm}^3.$$

Step ④: select a suitable section.

Ref. IS 800-2007, P.No: 139, Table 46.

select, ISMB 175 @ 19.3 kg/m. — steel table P.2.

$A = 24.62 \text{ cm}^2 = 2462 \text{ mm}^2$, $D = 175 \text{ mm}$.

$b_f = 90 \text{ mm}$, $t_f = 8.6 \text{ mm}$, $t_w = 5.5 \text{ mm}$.

$Z_e = 145.4 \text{ cm}^3$, $Z_p = 166.08 \text{ cm}^3$

From SP (6) (1), — 1964, P. No: 4,

$I_{zz} = 1272 \text{ cm}^4$, $I_{yy} = 85 \text{ cm}^4$

$r_{zz} = 71.9 \text{ mm}$, $r_{yy} = 18.6 \text{ mm}$.

$r_1 = 10 \text{ mm}$.

Step ⑤: classification of section.

From IS 800-2007, P.No. 18, Table 2

$$\epsilon = \sqrt{\frac{250}{f_y}} \Rightarrow \sqrt{\frac{250}{250}} \Rightarrow 1.$$

$b/t_f = \frac{90/2}{8.6} \Rightarrow 5.23 < 9.4\varepsilon.$ | $b = b_f/2 = 90/2 = 45\,mm$

$d/t_w = \frac{137.8}{5.5} = 25.05 < 84\varepsilon.$

Hence the section is plastic. (Class I)

Step ⑥ check for shear strength.

IS 800 : 2007, P. No. 59, clause 8.4.

$V \leq Vd.$

Design strength $V_p = \frac{A_v \cdot f_{yw}}{\sqrt{3}} = \frac{(175 \times 5.5) \times 250}{\sqrt{3}}$

$$\boxed{V_p = 138.92\,kN}$$

$V_n \leq V_p.$

$\therefore\ V_d = \frac{V_n}{\gamma_{m0}} \Rightarrow \frac{138.92}{1.1} \Rightarrow 126.3\,kN.$

$V < Vd.$

$56.25 < 126.3\,kN.$ Hence ok.

Step ⑦: Calculation & Lateral torsional buckling moment ~~check for Bending stress.~~

IS - 800 : 2007, P. No : 54, clause 8.2.2.

$Md = \beta_b \cdot Z_p \cdot f_{bd}.$ $\beta_b = 1.$

$f_{bd} =$ $G = \frac{E}{2(1+\mu)} \Rightarrow \frac{2 \times 10^5}{2(1+0.3)}.$

$G = 76.92 \times 10^3\ N/mm^2$

$$I_t = \frac{\Sigma b_i t_f^3}{3} + \frac{(D - t_f) t_w^3}{3}$$

$\Sigma b_i = 2b_f = 180mm$

$D = 175 mm.$

$$= \frac{180 \times 8.6^3}{3} + \frac{(175 - 8.6) \times 5.5^3}{3}$$

$$\boxed{I_t = 47.39 \times 10^3 \ mm^4}$$

$$I_W = (1 - \beta_f) \cdot \beta_f \cdot I_y \cdot h_f^2$$

$$= (1 - 0.5) \times 0.5 \times 85 \times 10^4 \times 166.4^2$$

$$\boxed{I_W = 5.88 \times 10^9 \ mm^4}$$

$\beta_f = 0.5$ (Constant

$h_f = D - t_f$

$= 175 - 8.6$

$h_f = 166.4 mm$

Ref. P.No: 54, 8.2.2.1

$$M_{cr} = \sqrt{\frac{\pi^2 E I_y}{(L_{LT})^2} \left[G \cdot I_t + \frac{\pi^2 \cdot E I_W}{(L_{LT})^2} \right]}$$

$$\sqrt{\left(\frac{\pi^2 \times 2 \times 10^5 \times 85 \times 10^4}{1500^2}\right) \left[\left(76.92 \times 10^3 \times 47.39 \times 10^3\right) + + \left(\frac{\pi^2 \times 2 \times 10^5 \times 5.88 \times 10^9}{1500^2}\right) \right]}$$

$$\therefore \boxed{M_{cr} = 81.02 \times 10^6 \ N.mm.}$$

Step ⑧: calculation of Moment carrying capacity.

$$M_d = \beta_b \times Z_p \times f_{bd}$$

Non dimensional Slenderness ratio — P.54, clause 8.2.2.

$$\lambda_{LT} = \sqrt{\frac{Z_p \times f_y}{M_{cr}}}$$

$$= \sqrt{\frac{166.08 \times 10^3 \times 250}{81.02 \times 10^6}}$$

$$\boxed{\lambda_{LT} = 0.72}$$

Imperfection factor. — P.54, 8.2.2.

$$\phi_{LT} = 0.5\left[1 + \alpha_{LT}\left(\lambda_{LT} - 0.2\right) + \lambda_{LT}^2\right]$$

where, $\alpha_{LT} = 0.21$ for rolled steel section.

$\alpha_{LT} = 0.49$ for welded steel section.

$$\therefore \cancel{\chi_{LT}} = \frac{1}{\phi}$$

$$\phi_{LT} = 0.5\left[1 + 0.21\left(0.72 - 0.2\right) + 0.72^2\right]$$

$$\boxed{\phi_{LT} = 0.81}$$

$$\therefore \chi_{LT} = \frac{1}{\phi_{LT} + \left(\phi_{LT}^2 - \lambda_{LT}^2\right)^{0.5}}$$

$$= \frac{1}{0.81 + \left(0.81^2 - 0.72^2\right)^{0.5}}$$

$$\chi_{LT} = 0.85 < 1$$

$$\therefore f_{bd} = \frac{\chi_{LT} \cdot f_y}{\gamma_{M0}} = \frac{0.847 \times 250}{1.1}$$

$$\boxed{f_{bd} = 192.30 \ N/mm^2}$$

$\beta_b = 1.0$ for plastic and compact section.

$$\therefore M_d = \overset{\beta_p \cdot Z_p \cdot f_{bd}}{1 \times 106.08 \times 10^3 \times 192.3}$$

$$M_d = 31.94 \times 10^6 \ N.mm.$$

M_d 31.94 kN m $>$ Max. B.m 21.09 kN m.

Hence safe.

$M_d > M$.

Step (8) check for deflection.

$$\delta = \frac{5wl^4}{384 \cdot EI_z} \Rightarrow \frac{5 \times 50 \times 1500^4}{384 \times 2 \times 10^5 \times 1272 \times 10^4}$$

$$\delta = 1.3 \ mm.$$

$$\text{Max. Allowable} = \frac{\text{Span}}{300} \Rightarrow \frac{1500}{300} \Rightarrow 5mm.$$

$$\delta_{max.} < \delta_{allowable}.$$

Hence ok.

Step (9). check for web buckling.

from IS 800, 2007, P. No: 67.

$$A_b = (b_1 + n_1) t_w$$

Where $b_1 = (b_f - t_w)/2$

$= \frac{90 - 5.5}{2} \Rightarrow 45.25mm.$

$n_1 = B/2 = \frac{175}{2} = 87.5mm.$

$\therefore A_b = (42.25 + 87.5) \times 5.5$

$= 713.63\ mm^2$

$I = \frac{b_1 \cdot t_w^2}{12} \Rightarrow \frac{42.25 \times 5.5^3}{12} \Rightarrow 585.78\ mm^4$

$I = 585.78\ mm^4.$

$A = b_1 \times t_w$

$= 42.25 \times 5.5$

$A = 232.38\ mm^2$

$r_{min} = \sqrt{I/A} = \sqrt{\frac{585.78}{232.38}}$

$r_{min} = 1.59mm.$

$\lambda = l_{eff}/r_{min} \Rightarrow$ P. 66, 8.7.1.5.

$KL = 0.7L$

$= \frac{0.7 \times 137.8}{1.59}$

$\lambda = 60.75$

from IS-800-2007, P. No: 56.

$f_{bd} = 163.6 - \frac{(163.6 - 152.3) \times (310 - 291.4)}{300 - 250}$

$\boxed{f_{bd} = 161.66 N/mm^2}$

Strength against web buckling

$= A_b \times f_{bd}$

$= 713.63 \times 161.6$

$= 115.36 \times 10^3$ kN.

$= 115.36 \times 10^3$ kN. $>$ 56.25 kN.

Hence the section is safe against web buckling.

Step (10) check for web bearing.

P. No. 67 of IS 800 : 2007.

$F_w = (b_1 + n_2) t_w \cdot f_y / \gamma_{M0}$.

where $b_1 = 42.25$ mm.

$n_2 = 2.5 (t_f + k)$

$= 2.5 (8.6 + 10)$

$= 46.5$ mm

$\therefore F_w = (42.25 + 46.5) \times 5.5 \times 250/1.1$

$= 110.93 \times 10^3$ kN

$= 110.93$ kN $>$ 56.25 kN.

Hence the section is safe against web buckling.

Problem No: (4)

calculate the moment carrying capacity of a Laterally unrestrained ISMB 400 member of length. 3m.

(Assignment)

Available in spiral notes.

TYPE III : DESIGN OF PLATED BEAMS (BULT UP BEAMS).

* For long span and heavy load, a large Bending moment are generated
* when the available rolled beam section do not have sufficient strength to resist the external B.M.
* They may be reinforced along the entire length or part of it.
* Also the depth of the beam may be restricted due to the head room requirements
* The above situation, it become necessary to provide built up or compound beams.
* The design of builtup beam is a trial and error procedure.
* A suitable I - Section is choosen and then the remaining modulus of section is produced by the plates
* The plates are attached to the compression and tension flanges of the beam section.
* Finally the compound section is checked for bending of shear strength.

PROBLEM NO: (8): Design a steel beam section for supporting a roof of a big hall for the following data and applied the usual check. Assume the grade of steel is Fe410.

Clear span := 6.5m

End Bearing = 150mm

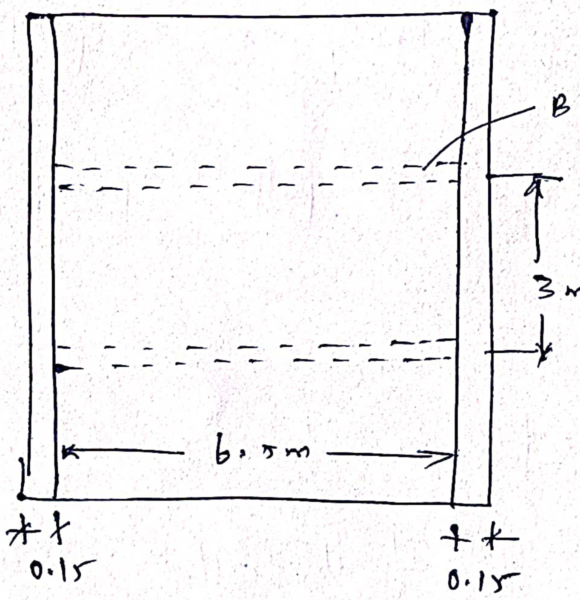
c/c spacing of Beam = 3m.

Imposed load on Beam = $10 kN/m^2$

Dead load on Beam (including selfwt) = $4 kN/m^2$

Restriction on the depth of Beam 375mm.

<u>Note:</u> The compression flange of the beam laterally supported throughout the span.



<u>Solution:</u>

<u>Step (1):</u> calculation of loads

Factored Imposed load = 1.5×10 = $15 kN/m^2$

Factored dead load = 1.5×4 = $6 kN/m^2$

<u>Given:</u> c/c spacing of Beam = 3m.

UDL/m length = $(15\times3) + (6\times3)$ = 63 kN/m.

∴ Total load Per m length = <u>63 kN/m.</u>

Effective span = clear span + 2 ($\frac{1}{2}$ x bearing).

$= 6.5 + 2\left(\frac{1}{2} \times 0.5\right)$

$= \underline{6.65m}$

Step ②: Max. B.M & S.F calculation.

$$B.M = \frac{wl^2}{8} = \frac{63 \times 6.65^2}{8} = 348.25\,kNm.$$

$$S.F = \frac{wl}{2} = \frac{63 \times 6.65}{2} = 209.48\,kN.$$

Step ③: Required plastic section modulus.

$$Zp_{req} = \frac{M \cdot \gamma_{mo}}{fy} = \frac{348.25 \times 10^6 \times 1.1}{250}$$

$= 1.53 \times 10^6\ mm^3$

$= 1532.3\ cm^3$

Note:

since the depth of beam is restricted to 375mm.

Try ISWB 350 @ 558.19 N/m.

$Zp = 995.49\ cm^3$ — from. SP(6) 1.

Zp req'd for plate = Zp_{req} for builtup section − Zp for ISWB 350 section

$= 1532.3 - 995.49$

$$\boxed{Zp_{req.}\text{ for plate} = 536.81\ cm^3}$$

Area of plate : $Ap = \frac{Zp}{h} \Rightarrow \frac{536.81 \times 10^3}{350}$

$\therefore$ Aplate. $= 1533.74\ mm^2$ → for two plates.

Assuming the width of the Top and bottom cover plate is equal to the width of beam flange.

b = 200mm.

Thickness of cover plate, $t_p = \frac{A_p}{b_f} \Rightarrow \frac{1533.74/2}{200}$

$t_p = \underline{\underline{3.8 mm}}$.

Say $\boxed{t_p = 8mm}$

Hence provide 8mm thick & 200mm width flange of cover plates at top and bottom.

Zp (provided actually by builtup section).

= Zp of rolled section + Zp of plates.

$\boxed{Z_p = A_p \times h}$

$= 995.49 \times 10^3 + \overset{A_p}{(200 \times 8)} \underset{d + t_p}{(350 + 8)}$

$$\boxed{Z_{p \, (\text{Builtup section})} = 1.57 \times 10^6 \, mm^3}$$

Total depth of builtup section = D + 2t

= 350 + (2×8)

= $\underline{\underline{366 mm}}$

366mm < 375mm. (restriction given as depth of Beam).

Hence ok.

<u>Step ④</u>: <u>Selection of properties of section</u>

IS 800: 2007, P.No: 138 or 139, Table 46.

Take. ISWB 350 @ 558.19 N/m.

D = 350mm., b_f = 200mm, t_f = 11.4mm.

t_w = 8mm, I_{zz} = 15521.7 cm4, I_{yy} = 1175.9 cm^4

R = r_1 = 12mm, Z_e = 887 cm^3, Z_p = 995.49 cm^3

Step (5): Section classification.

$d = D - 2(t_f + R) \Rightarrow 350 - 2(11.4 + 12)$

$\therefore \boxed{d = 303.2\,mm}$

$\xi = \sqrt{\frac{250}{f_y}} \Rightarrow \sqrt{\frac{250}{250}} = 1.$

Ref. IS 800 : 2007, P. No: 18, Table : 2

(i) $b/t_f = \frac{b_f/2}{t_f} \Rightarrow 8.77 < 9.4\,\xi.$

(ii) $d/t_w = \frac{303.2}{8} = 37.9 < 84\,\xi.$

Hence the section is classified as plastic section (class I)

Step (6): check for shear strength.

IS 800 - 2007, P. No: 59, clause : 8.4.

$A_v = h \times t_w.$

$V \leq V_d.$

$V_p = \frac{A_v \cdot f_{yw}}{\sqrt{3}} = \frac{(350 \times 8) \times 250}{\sqrt{3}}$

$\boxed{V_p = 404.14\ kN}$

$V_n = V_p.$

$\therefore V_d = \frac{V_n}{\gamma_{m0}} = \frac{404.14}{1.1} \Rightarrow 367.4\ kN.$

$V < V_d.$

$209.48 < 367.4\ kN$ Hence Ok.

check for high & low shear.

$V < 0.6\,V_d.$

$209.48\ kN < 0.6 \times 367.4$

$\therefore 209.48\ kN < 220.44\ kN.$

Hence ok.

Step (7): check for Bending shear.

Ref. IS 800 - 2007, P.No: 53, clause 8.2.1.2

$$Md = \frac{\beta_b \cdot Z_p \cdot f_y}{\gamma_{m0}} < 1.2\, Z_e \cdot f_y \cdot \frac{1}{\gamma_{m0}}$$

$$= \frac{1 \times 1.57 \times 10^6 \times 250}{1.1} < \frac{1.2 \times 1.41 \times 10^6 \times 250}{1.1}$$

$$\boxed{Md = 356.82} < 384.55\ kN \cdot m$$

Hence it is ok.

$$\boxed{Z_e = \frac{I_{zz}}{\frac{h}{2} + t_w}}$$

I_{zz} = I_{zz} of rolled section + I_{zz} of plates.

$$= 15521.7 \times 10^4 + 2(Ak^2)$$

$$= 15521.7 \times 10^4 + 2\left[(8 \times 200) \times \left(\frac{350}{2} + 8/2\right)^2\right]$$

$$\boxed{I_{zz} = 257.75 \times 10^6\ mm^4.}$$

Builtup Section

$$Z_e = \frac{I_{zz}}{h/2 + t_w} = \frac{257.75 \times 10^6}{\frac{350}{2} + 8} \Rightarrow 1.41 \times 10^6\ mm^3$$

$\therefore$ Md > M.

i.e. 356.82 kNm > 342.25

Hence ok.

Step (8): check for Deflection.

$$\boxed{\delta = \frac{5wl^4}{384\, EI}}$$

$$\delta = \frac{5 \times (10\times3) \times (6.65\times10^3)^4}{384 \times 2\times10^5 \times 257.75\times10^6}$$

$\delta = 14.82\,mm.$

Max. allowable deflection $= \frac{Span}{300} = \frac{6650}{300} = 22.17\,mm$

$\delta < \delta_{max}.$

$14.82 < 22.17\,mm$ Hence ok.

Step (9): check for web crippling. / web bearing.

Assume stiff bearing as 100mm P. No: 67, 8.7.4.

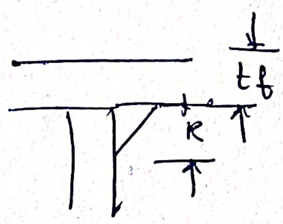
$\therefore$ Bearing strength, $F_w = (b_1 + n_2) t_w \cdot \frac{f_{yw}}{\gamma_{M0}}$

$b_1 = 100mm$, (width of stiff bearing assumed)

P.No: 66, Fig: 15

$n_2 = 2.5\,(t_f + R)$

$= 2.5\,(11.4 + 12)$



$\boxed{n_2 = 58.5\,mm}$

$F_w = (100 + 58.5) \times 8 \times \frac{250}{1.1}$

$\boxed{F_w = 288.18\,kN}$

$F_w = 288.18 > V = 209.48$

Hence ok.

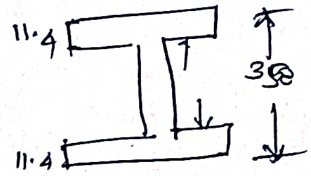
Hence the web is ok.

Step (10): check for web buckling at support.

IS 800 - 2007, P.No: 67, clause: 8.7.3.1

$F_{cd(w)} = A_b \cdot f_{cd}$

$= (b_1 + n_1) t_w \times F_{cd}$



$\underline{\underline{f_{cd}}}$. $\lambda = \frac{l_{eff} \text{ for web}}{r_{min}} \Rightarrow \frac{0.7\, l_{eff} \text{ for web}}{r_{min}}$

$\lambda = \frac{327.2 \times 0.7}{2.31}$

$\boxed{\lambda = 99.15}$

$r_{min} = \sqrt{I/A}$

$= \sqrt{\frac{4266.67}{800}}$

$r_{min} = 2.31 mm$

$I = \frac{b \cdot t_w^3}{12}$

$= \frac{100 \times 8^3}{12} = 4266.67 mm^4$

$A = b_1 \cdot t_w$

$= 100 \times 8$

$A = 800 mm^2$

IS 800-2007, P.No: 42, Table: 9C

$f_y = 250 N/mm^2$

$\frac{KL}{r}$	f_{cd}
= 100 →	107 N/mm^2
90 →	121 N/mm^2

For 99.15 → $f_{cd} = 121 - \left(\frac{121 \sim 107}{100 \sim 90}\right) \times (90 \sim 99.15)$

$\boxed{f_{cd} = 108.19 N/mm^2}$

strength against web buckling.

$F_{cd(w)} = (b_1 + n_1) \cdot t_w \cdot f_{cd}$

$= (100 + 183) \times 8 \times 108.19$

$= 244942.16 N$

$F_{cd(w)} = 244.94 kN > 209.48 kN$

Hence ok.

$n_1 = D/2$

$= 350 + (2 \times 8)$

$= \frac{356}{2}$

$n_1 = 183 mm$

Buckling resistance

$= f_{cd} \cdot A_b$.

$A_b = (b_1 + n_1) t_w$

PROBLEM NO: 2: Design a S.S beam of 7 m span carrying a reinforced concrete floor capable of providing lateral restraint to the top compression flange. The total udl is made up of 100 kN DL including self wt plus 150 kN imposed load. In addition, the beam carries ~~carries~~ a point load at mid span made up of 50 kN dead load and 50 kN imposed load (assuming a stiff bearing length of 75 mm).

Step 1: Load calculation.

udl

Factored Dead load $= 1.5 \times 100 = 150$ kN

Factored Live load $= 1.5 \times 150 = 225$ kN

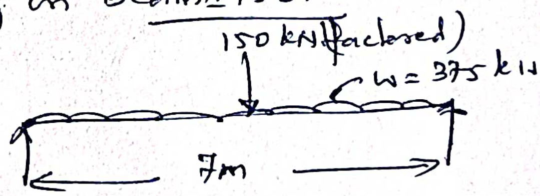
Total udl acting on beam $= 375$ kN.

Concentrated load:

Factored D.L $= 1.5 \times 50 = 75$ kN.

" L.L $= 1.5 \times 50 = 75$ kN

Total concentrated load acting on Beam $= 150$ kN



Step 2: Max. S.F & B.M

$$B.M = \frac{WL^2}{8} + \frac{WL}{4}$$

$$= \frac{(wl).l}{8} + \frac{WL}{4} \Rightarrow \frac{375 \times 7}{8} + \frac{150 \times 7}{4}$$

$$\boxed{B.M = 590.63 \text{ kN.m}}$$

$$S.F = \frac{wl}{2} + \frac{W}{2} \Rightarrow \frac{375}{2} + \frac{150}{2}$$

$$\boxed{S.F = 262.5 \text{ kN}}$$

Other steps are same as previous problem.

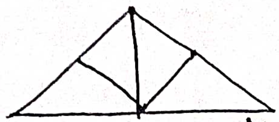
$$\delta = \frac{5wl^4}{384.EI} + \frac{Wl^3}{48EI} = 7.91 \text{ m}$$

allowable $\delta_{max} = \frac{L}{300} = 23.3$ mm

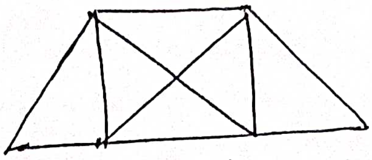
DESIGN OF ROOF TRUSSES.

Roof trusses: Roof trusses are supported on walls or a series of columns.

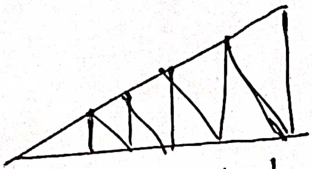
Types of Roof trusses:



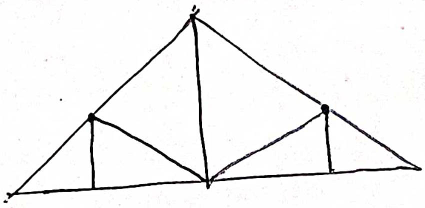
King Post truss.
(upto 6m span)



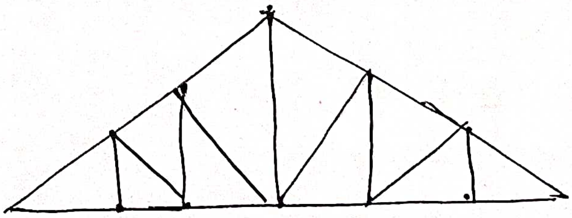
Queen Post truss.
(6m to 9m span)



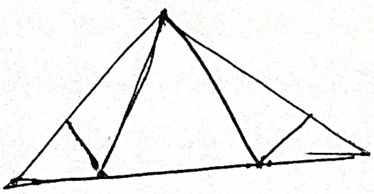
North light truss.
(8m to 10m)



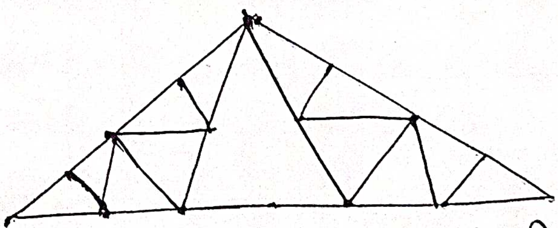
Howe - Triangle 4 panel.
(6m to 15m span)



Howe - Triangle 6 panel.
(12m to 24 m span)

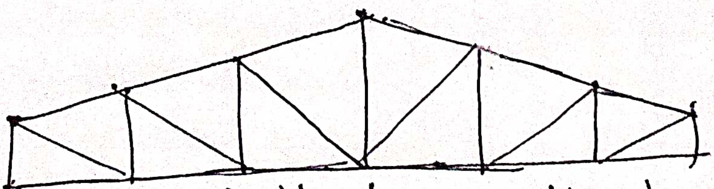


(9m span)



(12 m to 18 m span)

Fink or French trusses.



Pratt truss - 16m to 30m.

PROBLEM NO: ①: A roof truss is to be built in Lucknow for an industry. The size of shed is 24 x 40m. The height of the building is 12m. Determine the basic wind pressure.

Given Data:

Place of construction = Lucknow.

Size of shed = 24 m x 40 m.

Height of building = 12m.

Solution: step ①: Basic wind speed (V_b)

IS : 875 (Part III) - 1987, P.No: 53, clause 5.2

The basic wind speed for Lucknow = 47m/sec.

Step ②: calculation of Design wind speed. (V_z)

IS 875 (Part III) - 1987, P.No: 8, clause 5.3.

$V_z = V_b \cdot k_1 \cdot k_2 \cdot k_3.$

$k_1 \rightarrow 1.0$ (IS. 875, Part III, 1987, P. 11, Table 1)

$k_1 \rightarrow$ risk co-efficient.

$k_2 \rightarrow$ Terrain, height and structure size factor.

IS - 875 (Part III) - 1987, P. No: 8, clause : 5.3.2.1

Assuming, category 3.

IS 875 (part III) - 1987, P. No: 11, class 5.3.2.2.

Assuming class. B.

Refer: Table :2, P. No: 12

10m $\rightarrow k_2 = 0.88$

15m $\rightarrow k_2 = 0.94$

For 12m $\rightarrow$ 0.904.

$\boxed{k_2 = 0.904\ m}$

$k_3 \rightarrow$ Topography factor.

Refer IS 875 (Part III) - 1987, P.No: 56.

$k_3 = 1 + Cs.$ where. $C \rightarrow$ slope

$S \rightarrow$ factor.

$C \rightarrow \frac{z}{L} \Rightarrow$ height of hill

$\because$ There is no hilly terrain.

$\therefore Cs = 0.$

$\therefore \boxed{k_3 = 1}$

Design wind speed. (V_z)

$V_z = V_b \cdot k_1 \cdot k_2 \cdot k_3.$

$= 47 \times 1 \times 0.904 \times 1$

$\boxed{V_z = 42.488\ m/sec}$

Step 3: Design wind Pressure. (P_z)

IS 875 (Part III) - 1987, P.No: 12.

$P_z = 0.6\ V_z^2 \Rightarrow 0.6 \times (42.488)^2$

$P_z = 1083.14\ N/m^2$

$\boxed{P_z = 1.083\ kN/m^2}$

PROBLEM NO: (2): Determine the basic wind intensity for a industrial building situated in chennai.

(i) life of the structure = 50 year

(ii) Terrain category = 2

(iii) size of building = 20 × 40 m.

(iv) Height of the building = 10 m

(v) Topography slope < 3°.

(vi) slope of ground 1 in 4.

Solution: step (1): Basic wind speed (V_b)

Is : 875 (Part III) - 1987, P.No: 53, clause : 5.2

The Basic wind speed for chennai = 50 m/sec.

Step (2): calculation of Design wind speed (V_z)

Is 875 (part III) - 1987, P.No: 8, clause 5.3.

$V_z = V_b \cdot k_1 \cdot k_2 \cdot k_3$

$k_1 \longrightarrow$ Risk co-efficient.

Refer. P. No: 11, Table 1. IS 875, Part III.

$\boxed{k_1 = 1}$

k_2: Terrain, ht of structure, size factors.

P. 11, clause 5.3.2.2.

Given category is '2'. $\boxed{\text{class B}}$

$\therefore \boxed{k_2 = 0.98}$ Page - 12, Table 2

$k_3 \rightarrow$ Topography factor.

$k_3 = 1 + CS$

$C < 3^\circ$

$\therefore \; Cs = 0$

$< 3^\circ$.

$\therefore \; \boxed{k_3 = 1}$

Design wind speed. $(V_z) = V_b \cdot k_1 \cdot k_2 \cdot k_3.$

$\therefore \; V_z = 50 \times 1 \times 0.98 \times 1$

$\boxed{V_z = 49 \text{ m/sec}}$

Step ③: Design wind pressure. (P_z)

Ref. IS 875 (Part III) 1987, P. No: 12,

$P_z = 0.6 V_z^2 = 0.6 \times 49^2$

$P_z = 1440.6 \text{ N/m}^2$

$\boxed{P_z = 1.44 \text{ KN/m}^2}$

PROBLEM NO: ③ A Power Plant structure having maximum dimension more than 60m is proposed to be built on down hill side near Dehradun. The height of the hill is 400m with a slope of 1 in 3. If the location is 250m from the crest of the hill on downward slope, and is at a height of 9m. Determine the design wind Pressure.

Given Data:

Type of structure = Power plant.

place of " = Dehradun.

Height of hill = 400m.

Slope = 1 in 3.

Height of structure = 9m.

located distance from crest = 250 m.

Solution: step ①: Basic wind speed (V_b)

Ref IS 875 (part III) -1987, P. No: 53, clause 5.2

Basic wind Speed for Dehradun $\boxed{V_b = 47\,m/sec}$

step ② - Design wind speed. (V_z)

Risk co-efficient. (k_1)

IS: 875 (P. III) - 1987, P. No: 11, table: 11.

for Power plant - Probable life of structure is taken as 100 years.

$\therefore \boxed{k_1 = 1.07}$

Terrain, Height & Structure size factor (k_2)

IS - 875 (pt. III) - 1987, P. No: 8, clause 5.3.2.1.

Since the ht of building = 9m, belong to category 2

Ref. IS 875, P. No: 11, clause 5.3.2.2

size of building is more than 60.

$\therefore \boxed{\text{class. C}}$ → Refer. Table 2

$\boxed{k_2 = 0.93}$

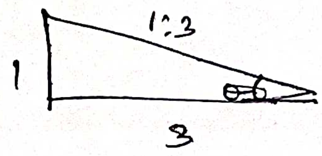
* Topography factor (k_3)

Slope = 1 in 3, Ref. IS 875 -1987, P.No: 56.

$k_3 = 1 + Cs.$

(i) slope. (a) $3^\circ < \theta < 17^\circ$

(b) $> 17^\circ$.



$\therefore \theta = 18^\circ 26' > 17^\circ$.

$\tan\theta = \frac{1}{3}$

$\theta = 18^\circ 26'$

$\therefore C = 0.36$

To find 's'

Z = ht of hill = 400m.

H = 9m , X = 250m.

Ref. 875 (Pt III) -1987, P. No: 55,

$$Le = \frac{Z}{0.3} = \frac{400}{0.3} \Rightarrow 1333.3 \text{ m}.$$

$$\frac{H}{Le} = \frac{9}{1333.33} \Rightarrow 0.00675$$

$$\frac{X}{Le} = \frac{250}{1333.33} \Rightarrow 0.1875$$

Ref. 875, 1987 - P.No: 57, fig: 14.

$\boxed{S = 1}$

$k_3 = 1 + C.S \Rightarrow 1 + 0.36 \times 1$

$\boxed{k_3 = 1.36.}$

Design wind speed. (V_z) $\Rightarrow V_b . k_1 . k_2 . k_3$.

$= 47 \times 1.07 \times 0.93 \times 1.36$

$\boxed{V_z = 63.60 \text{ m/sec}}$

Step ③: Design wind Pressure (P_z)

$P_z = 0.6\, V_z^2 = 0.6 \times 63.60^2$

$= 2427\ N/m^2$

$\boxed{P_z = 2.427\ kN\ m^2}$.

PROBLEM: ④: Determine the design loads on the Purlins of an industrial building near Visakhapatnam.

The Data are,

class of building = General with life of 50 yrs.

Terrain category : 2

maximum dimension : 40m

width of building : 15m

Height of structure : 10m

Height of eve level : 8m

Topography = θ less than ~~3~~ 3°

Permeability = medium.

span of truss = 15m

pitch = 1/5

sheeting : Ac sheets.

spacing of purlin = 1.35m

spacing of truss = 4 m.

solution:

step ①: calculation of Design wind Pressure:

(a) Basic wind speed (V_b)

Is 875 (part III) – 1987, P.No: 53, clause : 5.2,

for Visakhatnam, $\boxed{V_b = 50 m/sec}$

(b) Design wind speed (V_z)

(i) calculation of Risk - co-efficient (k_1)

Is. 875, Part (III) – 1987, P.No: 11, Table-1

$\boxed{k_1 = 1}$

(ii) calculation of terrain, height & structure size factor (k_1)

Is - 875, (Part III) – 1987, P. No: 8, clause 5.3.2.1

~~k~~

Given, $\boxed{\text{category : 2}}$

Is. 875, Part III – 1987, P. No: 8, clause 5.3.2.2.

~~Given~~ a $\boxed{\text{class : B}}$

Is. 875 : Part III, P. No: 12, Table: 2,

$$\boxed{k_2 = 0.98}$$

(iii) calculation of topography factor (k_3)

Is. 875 – Part III, P. No: 56,

$$\boxed{k_3 = 1 + C_s}$$

Topography less than 3°, $\boxed{C = 0}$

$$\boxed{k_3 = 1}$$

Design wind load (V_z)

$$V_z = V_b \cdot k_1 \cdot k_2 \cdot k_3$$

$$= 50 \times 1 \times 0.98 \times 1$$

$$\boxed{V_z = 49 \text{ m/sec}}$$

Design wind pressure. (P_z)

$$P_z = 0.6\, V_z^2$$

$$= 0.6 \times (\cancel{4}49)^2 \Rightarrow 1440.6 \text{ N/m}^2$$

$$\boxed{P_z = 1.44 \text{ kN/m}^2}$$

H.W A roof truss shed is to be built in Lucknow for an industry. The size of shed is 30m x 40m. The ht of building is 14m. Determine the basic wind speed.

Ans:

Design wind speed : $\boxed{V_z = k_1 \cdot k_2 \cdot k_3 \cdot V_b}$

IS. 875 (part 3) — 1987, P. No: 8,

$V_b \rightarrow$ Basic wind speed. P.No:53, Appendix - A,

$k_1 \rightarrow$ Risk - co-efficient, Assume life of structure → 50yrs

Lucknow — $V_b = 47\, m/s$.

(IS: 875 Part 3 - 1987, P. No: 11, table: 1)

$\boxed{k_1 = 1}$

k_2 = Terrain & Height factor. P. No: 8,

(Shed is an industrial area).

∴ Terrain category = 3.

Ref. IS875 - part(II) (greatest horizontal, vertical, dimension is 40m)

∴ class. B.

IS 875, (pt: 3) — 1987, PNo: 12, table: 2,

Ht. (40m) Terrain category (3) → B class.

30m → 1.03.

50m → 1.09

$\boxed{40m \rightarrow 1.06}$

$k_3 \rightarrow$ Topography factor. $\underline{k_3}$

$\boxed{k_3 = 1 + C.s}$

$k_3 = 1.0$.

$\boxed{k_3 = 1}$

[hills, cliffs, are not present in that area]

$\boxed{\theta = 8 = 0}$

$V_z = V_b \cdot k_1 \cdot k_2 \cdot k_3$

$= 43.616 \text{ m/s}$.

$P_z = 0.6 V_z^2 \Rightarrow 1141.4 \text{ N/m}^2$.

$$\boxed{P_z = 1.1414 \text{ kN/m}^2}$$

DESIGN OF PURLINS:

* Purlin is a member which rest b/w roof trusses and support roof sheeting.

* I - sections, channels, angles, cold formed c or z - sections are commonly used as purlins.

* The Purlins are spaced on main rafter of trusses such that the sheets overlap on them.

* For A.c. sheets. the Overlap of atleast 150 mm. is to be ensured.

* Hence one can see in most of the building Purlins are spaced at 1.35m to 1.40m. when A.c sheets are of length 1.65m

* Purlins are connected to main rafter of the trusses with their web normal.

* cleat angles are used for the connection.

* Hence the Z-Z & Y-Y axes purlins are also inclined to vertical to horizontal.

* If θ is the inclination of main rafter to the horizontal, then the component of dead load & live load on Purlins in directions normal & perpendicular to sheeting are $W\cos\theta$ & $W\sin\theta$ respectively.

* windload is always taken as normal to the rafter. Hence it is having bending only about Z-Z axis of the purlins.

* The effective length of purlins may be taken as centre to centre distance b/w the supports and assume that purlins act as beams.

* Simply supported purlins, $B.M = \frac{Wl^2}{8}$

* Continuous purlins, $B.M = \frac{Wl^2}{10}$

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PROBLEM NO: (5): Design an angle iron purlin for a trussed roof from the following Data.

(i) Span of roof truss = 12m.

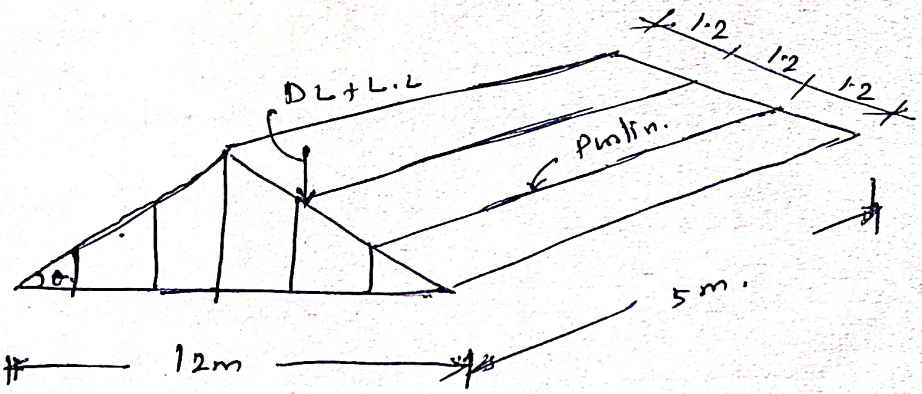
(ii) Spacing of roof truss = 12m.

(iii) Spacing of purlins along the slope of roof truss } = 1.2m

(iv) Slope of roof truss = 1V to 2H

(V) Wind load on roof surface normal to roof = 1.04 kN/m^2

(iii) Vertical load from roof sheeting = 0.20 kN/m^2



Step (1): selection of a trial section.

Assume, minimum depth of purlin $= \frac{L}{45} = \frac{5000}{45}$

$= 111.11$ mm,

∴ Assume, minimum width of purlin $= \frac{L}{60} = \frac{5000}{60}$

$= 83.33$ mm

Hence Try $\boxed{\text{ISA } 125 \times 95 \times 8 \text{ mm}}$ angle section

Step (2) Dead load, Live load, windload calculation.

D.L from roofing = 0.2 kN/m^2

Assume L.L. on roof = 0.6 kN/m^2

Total. 0.8 kN/m^2

Total vertical downward load on purlin per meter length } $= 0.8\ kN/m^2 \times 1.20\ m$

$= 0.96\ kN/m$

Step ④ : calculation of load along Roofing or sheeting and load Normal to roofing.

(i) Factored (D.L + L.L) along sheeting

$= 1.5 \times 0.96 \times \cos 26^\circ 56'$

$W = 1.288\ kN/m.$

(ii) Factored (D.L + L.L) normal to sheeting $= 1.5\,(0.2 \cos 26^\circ 56' - 1)$

$= -1.23\ kN/m$

$W = 1.23\ kN/m$ (outward)

Take greater value. $\boxed{W = 1.288\ kN/m}$

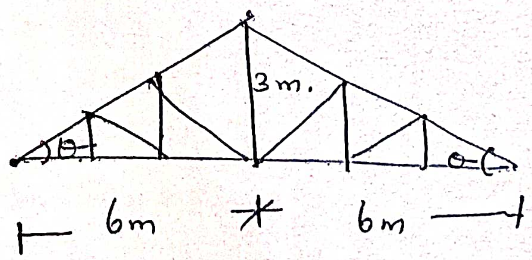
Step ④: calculation of Inclination (θ)

Given 1V to 2H

$\tan\theta = \frac{V}{H} \cdot \frac{Opp.}{Adj}$

$\tan\theta = 3/6.$

$\therefore \boxed{\theta = 26.56^\circ}$



Step ⑤: calculations of B.M.

Max. B.M for Purlin, $M = \frac{Wl^2}{10}$ → for continuous Beam.

$= \frac{1.288 \times 5^2}{10}$

$\boxed{M = 3.22\ kN.m}$

step ⑥: classification of section. $\boxed{\text{ISA } 125\times95\times8 \text{ mm}}$

(i) $\frac{b}{t_f} = \frac{95}{8} \Rightarrow 10.5 < 15.7\varepsilon.$

(ii) $d/t_w = \frac{125}{8} = 15.625 < 15.7\varepsilon.$

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

$\boxed{\varepsilon = 1}$

∴ This section is semicompact section (class III)

step ⑦: Design Bending Moment.

IS 800:2007 P. 53 8.2.1.2

$$M_d = \frac{\beta_b \cdot Z_p \cdot f_y}{\gamma_{m0}}$$

$\beta_b = 1$

$\beta_b = \frac{Z_e}{Z_p}$

$1 = \frac{Z_e}{Z_p}.$

$\therefore Z_p = Z_e.$

Ref. sp 6(1), -1964, angle section.

$Z_e = 30.6 \text{ cm}^3$

$\therefore Z_p = Z_e = 30.6 \text{ cm}^3 = 30.6\times10^3 \text{ mm}^3$

$$M_d = \frac{1\times 30.6\times10^6 \times 250}{1.1}$$

$$\boxed{M_d = 6.95 \text{ kN.m} > M = 3.22 \text{ kN.m}.}$$

Hence it is ok

∴ Hence Provide ISA 125 × 95 × 8 mm.

PROBLEM NO: ⑥: symmetric trusses of span 20 m and height 5 m are spaced at 4.5 m centre to centre. Design channel section. Purlins to be placed at 1.4 m distances to resist the following loads.

wt of sheeting including bolts = 171 N/m²

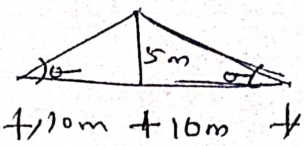
Live load = 0.4 kN/m²

wind load = 1.2 kN/m², suction.

Solution: step ①: Inclination (θ)

Ht of truss = 5m

Span of truss = 20m



$\tan\theta = \frac{V}{H} = \frac{5}{10}$

$\boxed{\theta = 26.56^\circ}$

Step ②: Design for DL + L.L

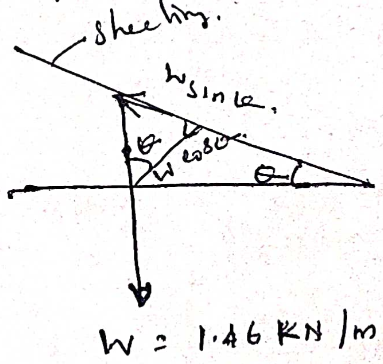
D.L

D.L from sheeting = 171 N/m²

Assume self wt of purlin = 125 N/m²

∴ Total D.L = 171 + 125 = 296 N/m²

= 0.296 kN/m²



L.L.

Live load = 0.4 kN/m²

∴ Factored DL + L.L = 1.5 (DL + L.L)

= 1.5 (0.296 + 0.4)

= 1.044 kN/m²

Total factored (D.L + L.L) vertically down load.

= 1.044 × 1.4 m.

= 1.46 kN/m.

Load normal to sheeting $= 1.46 \cos\theta = 1.46 \times \cos 26.56$

$$\boxed{W_z = 1.306 \text{ kN/m}}$$

Load in the direction parallel to sheeting $= 1.46 \sin\theta$

$= 1.46 \times \sin 26.56$

$$\boxed{W_y = 0.653 \text{ kN/m}}$$

Step ③ B.M. calculation. Span. 4.5m.

$$M_z = \frac{Wl^2}{8} = \frac{1.306 \times 4.5^2}{8} = 3.306 \text{ kN.m}$$

$$M_y = \frac{Wl^2}{8} = \frac{0.653 \times 4.5^2}{8} = 1.653 \text{ kN.m}$$

Step ④: Shear force calculation.

$$F_z = \frac{Wl}{2} = \frac{1.306 \times 4.5}{2} = 2.939 \text{ kN}.$$

$$F_y = \frac{Wl}{2} = \frac{0.653 \times 4.5}{2} = 1.469 \text{ kN}.$$

Step ⑤: Z_{pz} required.:

To account biaxial bending.

depth of trial section.

$$Z_{pz.req} = \frac{M_z}{f_y} \cdot \gamma_{mo} + 2.5 \frac{d}{b} \cdot \frac{M_y}{f_y} \cdot \gamma_{mo}$$

breadth of trial section.

Try ISMC 100

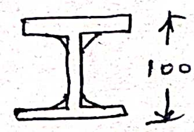
$h = D = 100mm$

$b_f = 50mm$, $t_f = 7.5mm$,

$t_w = 4.7mm$, $Z_e = 37.3 \times 10^3 \text{ mm}^3$

$Z_{pz} = 43.83 \times 10^3 \text{ mm}^3.$

From SP.6(1) 1964.



$r_1 = R = 9mm$

$d = D - 2(t_f + R) = 100 - 2(7.5+9).$

$\boxed{d = 67mm}$

$b_f = b = 50mm$

$$Z_{pz\,req} = \left(\frac{3.306\times10^6 \times 1.1}{250}\right) + \left(\frac{2.5\times 67\times 1.653\times 10^6 \times 1.1}{50 \times 250}\right)$$

$Z_{pz\,req} = 42.18\times10^3\,mm^3 < Z_{pz\ provided} = 43.825\times10^3\,mm^3$

Hence ok.

<u>step 6</u> — Check for shear.

$$V_{dz} = \frac{f_y}{\sqrt{3}} \cdot \frac{1}{\gamma_{M0}} \cdot A_{vz}$$

$$V_d = \frac{V_n}{\gamma_{M0}} = \frac{V_p}{\gamma_{M0}} = \frac{A_v \cdot f_y}{\sqrt{3}\cdot \gamma_{M0}}$$ $h = D = 100mm$

$A_{vz} = h \cdot t_w$ (hot rolled)

$$= \frac{250}{\sqrt{3}} \times \frac{1}{1.1} \times (100 \times 4.7)$$

$\boxed{V_{dz} = 61.67\,kN > F_z = 2.939\,kN}$

Hence the section is adequate to resist shear.

$$V_{dy} = \frac{f_y}{\sqrt{3}} \cdot \frac{1}{\gamma_{M0}} \cdot A_{vy}$$ $A_{vy} = 2b \cdot t_f.$

$$= \frac{250}{\sqrt{3}} \times \frac{1}{1.1} \times 2 \times 50 \times 7.5$$

$V_{dy} = 94.41\,kN > 1.469\,kN.$

Hence the section is adequate, to resist shear.

Step 7 section classification

IS 800 - 2007, P. No: 18, Table: 2.

(i) $b/t_f = \frac{50}{7.5} = 6.67 < 9.4\varepsilon$

(ii) $d/t_w = \frac{67}{4.7} = 16.45 < 42\varepsilon$

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

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

Hence it is plastic section. (class I)

Step ⑧: Design capacity of the section in both axes.

$$M_{dz} = \frac{Z_{pz} \cdot f_y}{\gamma_{M0}} \leq 1.2\, Z_{ez} \cdot \frac{f_y}{\gamma_{M0}}$$

$$= \frac{43.825 \times 10^3 \times 250}{1.1} < \frac{1.2 \times 37.3 \times 10^3 \times 250}{1.1}$$

$$M_{dz} = 9.955 \text{ kN.m} < 10.172 \text{ kN.m}$$

Hence ok.

$$M_{dy} = \frac{Z_{py} \cdot f_y}{\gamma_{M0}} \leq 1.5\, Z_{ey} \cdot \frac{f_y}{\gamma_{M0}}$$

From steel Table (SP.6(1) - 1964)

$Z_{py} = 14208.25 \text{ mm}^3$

$$\therefore M_{dy} = \frac{14208.25 \times 250}{1.1} < \frac{1.5 \times 13.1 \times 10^3 \times 250}{1.1}$$

$$M_{dy} = 3.229 \text{ kN.m} < 4.466 \text{ kN.m}$$

Hence ok

Check for biaxial bending.

$$\frac{M_z}{M_{dz}} + \frac{M_y}{M_{dy}} \leq 1$$

$$\frac{3.306}{9.955} + \frac{1.653}{3.229} < 1.$$

$0.844 < 1$ Hence ok

Step ⑨: Check for wind condition. (forces)

At this stage live load is not to be considered since wind force is suction and hence critical load condition is when there is no live load.

For this conditions of loading:

Factored D.L $= 1.5 \times (0.296 \times 1.5)$

$= 0.666$ kN/m.

wind load (WL) $= 1.2$ kN/m^2 given

$= 1.2 \times 1.5 = 1.8$ kN/m.

Factored wind force $= 1.5 \times 1.8 = 2.7$ kN/m. (suction)

= it act normal to sheeting

Load normal to sheeting $= -2.7 +$ factored DL

$= -2.7 + 0.666 \cos 26.56$

$W_{zz} = -2.104$ kN/m (outward)

Load parallel to sheeting $= 0.666 \times \sin 26.56$

$= 0.30$ kN/m.

$$M_{zz} = \frac{wl^2}{8} = \frac{2.104 \times 4.5^2}{8} = 5.21 \text{ kNm}$$

$$M_{yy} = \frac{0.3 \times 4.5^2}{8} \Rightarrow 0.76 \text{ kN.m}$$

M_{dz} for Laterally supported compression flange is to be found.

∴ eff length of simply supported channel purlin is established loading.

$$(\because KL = 1.22)$$

$$\lambda = \frac{KL}{r_y} = \frac{1.2L}{r_{yy}} = \frac{1.2 \times 4500}{14.9} = 362.42$$

r_{min}

$$\frac{h}{t_f} = \frac{100}{7.5} = 13.33$$

$$f_{crb} \Rightarrow h/t_f.$$

$$I_z = 416.4 \times 10^4 \text{ mm}^4$$

$$w = 1.306 \text{ kN/m} = 1.306 \text{ N/mm}.$$

$$\therefore \delta = \frac{5wl^4}{384 EI} = \frac{5 \times 1306 \times 4500^4}{384 \times 2 \times 10^5 \times 416.4 \times 10^4}$$

$$\delta = 8.4 \text{ mm}$$

Permisable deflection. $\delta_{max} = \frac{L}{150} \Rightarrow \frac{4500}{150} = 30 \text{ mm}$

$\delta < \delta_{max}$ Hence safe.

∴ Provide ISMC 125 as purlin.

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