Techno India NJR Institute of Technology

Design of Steel Structures Lab

(6CE4-22)

Session 2023-24

Rakesh Yadav
(Associate Professor)
Department of CE

Course Overview:

Student will learn basics of DSS from these 40 hours course. The subject has the significance to understand the types of structural steel members. Students will be able to analyze the structural behavior and design under the different loading like Gravity forces, lateral forces (wind & seismic loads), temperature effects, vibrations etc. Students will learn about the types of structural steel elements as connections, tension and compression members, members subjected to bending or beams, roof truss, steel bridges, steel tanks etc. with corelating to Indian standards. DSS is the main requirement for the job role in the companies like Tata Steel, Jindal steel & Power Ltd, L&T construction etc. Most of the questions asked during the placement drive for these Company are created from this subject. Student should learn and develop problem solving abilities using DSS in order to get a good job in top civil engineering company**.**

Course Outcomes:

CO.NO.	Cognitive Level	Course Outcome
1	**Analysis**	Learner will be able to solve the designing of tension and compression members.
2	**Evaluation**	Learner will be able to solve the designing of beams and beam columns.
3	**Synthesis**	Learner will be able to solve the designing of bolt and weld connections.
4	**Synthesis**	Learner will be able to solve the designing of the gantry girder.
5	**Application**	Classify and design the structural steel components of industrial building.

Prerequisites:

1. Analyze characteristics of water and wastewater
2. Students will develop an appreciation for the importance of environmental engineering as a major factor in preserving and protecting human health and the environment

Course Outcome Mapping with Program Outcome:

Course Outcome	PO 1	PO 2	PO 3	PO 4	PO 5	PO 6	PO 7	PO 8	PO 9	PO10	PO11	PO12	PSO 1	PSO 2	PSO 3
CO364.1	3	3	3	3	2	2	2	1	1	1	2	3	2	1	1
CO364.2	3	2	2	3	2	1	2	1	1	1	1	1	2	1	1
CO364.3	2	2	2	1	2	2	2	2	1	1	2	1	2	1	1
CO364.4	3	2	2	2	2	2	1	1	2	1	2	2	2	1	1
CO364.5	3	3	3	32	1	2	1	1	1	1	2	2	2	1	1
CO364 (AVG)	2.8	2.4	2.4	8.2	1.8	1.8	1.6	1.2	1.2	1	1.8	1.8	2	1	1

Faculty Lab Manual Link

1. https://r.search.yahoo.com/_ylt=AwrxzALl4qxhC3UAPWu7HAx.;_ylu=Y29sbwNzZzMEcG9zAzEEdnRpZAMEc2VjA3Ny/RV=2/RE=1638749029/RO=10/RU=https%3a%2f%2fwww.iare.ac.in%2fsites%2fdefault%2ffiles%2flab1%2fEnvironmental_Engineering%2520_Laboratory_Lab_MANUAL.pdf/RK=2/RS=wegI0PvdQ_xKJ3fWJJE2I P5K808-

Assessment Methodology:

1. Practical exam Of Environmental lab Experiment
2. Internal exams and Viva Conduct.
3. Final Exam (practical paper) at the end of the semester.

Techno India NJR Institute of Technology

Academic Administration of Techno NJR Institute

Syllabus Deployment

Name of Faculty	: Mr. Rakesh Yadav	Subject Code: 6CE4-22
Subject	: Steel Structures Design	
Department	: Civil Engineering	Sem: VI
Total No. of Labs Planned: 6		

COURSE OUTCOMES HERE (3 OUTCOMES)

At the end of this course students will be able to:

CO1. Able to get the knowledge about design of joints and design of structural steel members subjected to tensile and compressive force.

CO2. Able to design the beams and columns under various loading and supporting conditions..

CO3. Able to know the design of structural systems such as roof trusses.

Lab No.	Exp. No.	Name of Experiment
1	1	Case study of foot over bridges/truss- girder bridge in vicinity home town of the students, preferably in groups of 8-10 students. A report including photographs marked with names and section details of different members in it
2	1	Case study of foot over bridges/truss- girder bridge in vicinity /home town of the students, preferably in groups of 8-10 students. A report including photographs marked with names and section details of different members in it
3	1	Case study of foot over bridges/truss- girder bridge in vicinity home town of the students, preferably in groups of 8-10 students. A report including photographs marked with names and section details of different members in it
4	2	Case study of a structure using tubular sections or light gauge sections in vicinity /hometown of the students, preferably in groups of 8-10 students. A report including photographs marked with names, size and section details of different members in it

Techno India NJR Institute of Technology

Department of Civil Engineering

3rd Year - VI Semester: B.Tech. (Civil Engineering)

6CE4-22: Steel Structures Design

1. Find the shape factor of a rectangular section.

Solution:

(i) Shape factor for rectangular section of size $b \times d$. (Refer Fig. 10.13)

Yield moment $(M_y) = f_y \cdot z$ where f_y = yield stress and z = section modulus

$$= f_y \cdot \frac{I_{zz}}{y_{max}} \quad \text{Here, } I_{zz} = I_{xx}$$

$$= f_y \cdot \frac{bd^{3}/12}{d/2}$$

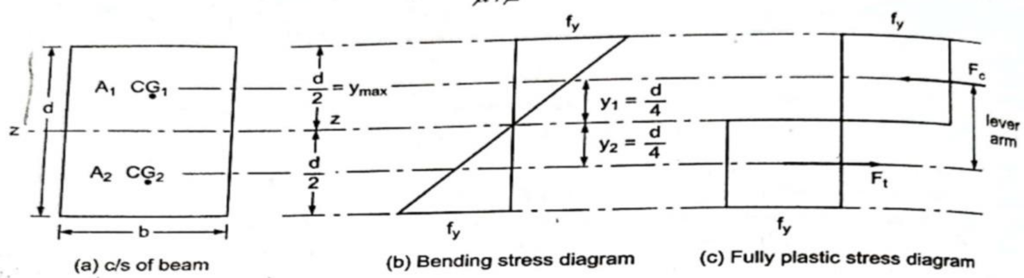


Fig. 10.13 Stress diagram for rectangular section

$$\therefore \quad M_y = f_y \cdot \frac{bd^2}{6}$$

Let, A_1 be the area under compression and A_2 be the area under tension respectively.

$$\therefore \quad F_c = F_t \qquad F_c = \text{compressive force}, \quad F_t = \text{tensile force}$$

$$\therefore \quad f_y \cdot A_1 = f_y \cdot A_2 \qquad \text{as } A_1 = A_2 = \frac{A}{2}$$

Now,

Plastic moment capacity is the moment of resistance when yield stress is 'f_y' at all fibres.

$$\therefore \quad M_p = F_c \cdot y_1 + F_t \times y_2$$

$$= f_y A_1 \cdot y_1 + f_y A_2 \cdot y_2$$

$$= f_y \cdot \frac{A}{2}\left[\frac{d}{4} + \frac{d}{4}\right]$$

$$= f_y \cdot \frac{A}{2} \cdot \frac{d}{2} \qquad \text{but} \Rightarrow A = bd$$

$$= f_y \cdot \frac{A \cdot d}{4}$$

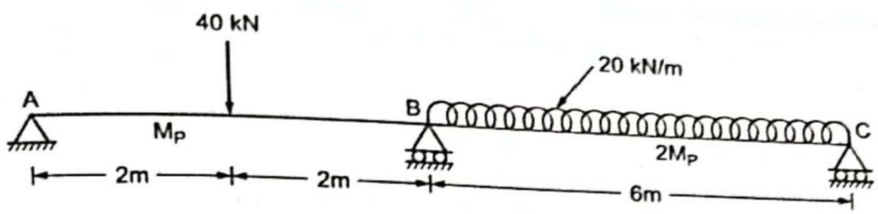
$$M_p = f_y \cdot \frac{bd^2}{4}$$

Hence, shape factor $(S) = \frac{M_p}{M_y}$

$$= \frac{f_y \cdot b\, d^2/4}{f_y \cdot b\, d^2/6} = \frac{6}{4}$$

$\therefore$ $S = 1.5$ For rectangular section.

2. Calculate Plastic Moment Capacity required for Continuous beam with working load as shown in Fig.



Solution:

Load factor = 1.5

$\therefore$ Collapse load on $AB = 40 \times 1.5 = 60$ kN and

Collapse load on $BC = 20 \times 1.5 = 30$ kN/m

The loading is shown in Fig. 10.34.

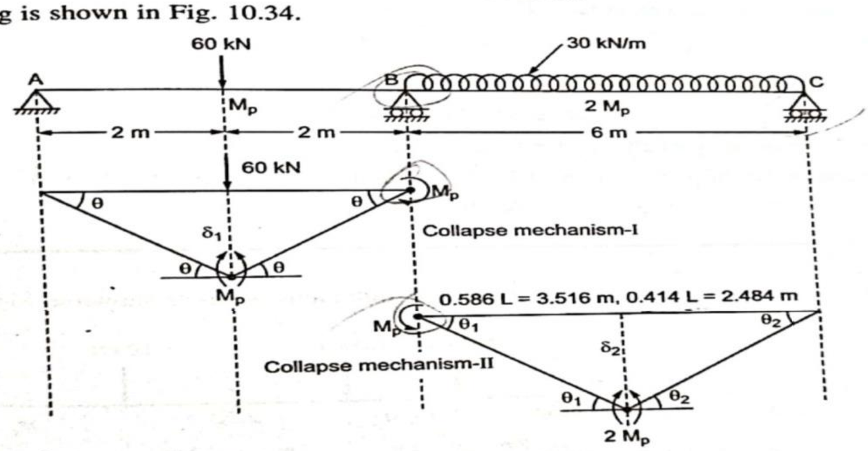


Fig. 10.34 Beam mechanism

(I) From Collapse Mechanism-I (Refer Fig. 10.34)

$$2\theta = \delta_1 = 2\theta$$

$$\text{Internal WD} = M_p\,(\theta + \theta) + M_p \cdot \theta = 3M_p \cdot \theta$$

$$\text{External WD} = 60 \times \delta_1 = 60 \times 2\theta = 120\,\theta$$

From the principle of virtual work

$$\text{Internal WD} = \text{External WD}$$

$$\therefore \quad 3M_p \cdot \not{\theta} = 120\,\not{\theta}$$

$\therefore \quad M_p = \frac{120}{3} = 40$ kNm

Plastic moment capacity of AB span = 40 kNm.

(II) From Beam Mechanism-II (Refer Fig. 10.34)

$$3.516\,\theta_1 = \delta_2 = 2.484\,\theta_2$$

$$\theta_1 = \frac{2.484\,\theta_2}{3.516} \quad \therefore \quad \theta_1 = 0.706\,\theta_2$$

$$\begin{aligned}\text{Internal WD} &= M_p \cdot \theta_1 + 2M_p\,(\theta_1 + \theta_2)\\ &= 0.706\,M_p \cdot \theta_2 + 2M_p\,(0.706\,\theta_2 + \theta_2)\\ &= 4.118\,M_p \cdot \theta_2\end{aligned}$$

$$\text{External WD} = (30 \times 6) \times \frac{\delta_2}{2} = 90 \times 2.484\,\theta_2 = 223.56\,\theta_2$$

Equating, internal WD and external WD

We have,

$$4.118\,M_p \cdot \cancel{\theta_2} = 223.56\,\cancel{\theta_2}$$

$$M_p = 54.288 \text{ kNm}$$

Plastic moment capacity of BC span = 54.288 kNm

$\therefore$ The higher value of plastic moment is the required moment capacity of beam

$\therefore \quad M_p = 54.288$ kNm

4 Determine plastic moment capacity of the section required for frame shown in given Fig. Load are working load factor = 1.75. Asume same M_p for all members

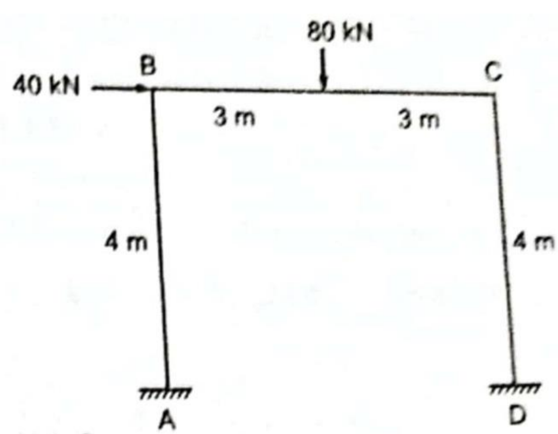


Fig. 10.37

(i) Beam mechanism (Refer Fig. 10.38)

$$\text{Internal WD} = M_p\,\theta + M_p\,\theta + M_p\,(\theta + \theta) = 4M_p\,\theta$$

$$\text{External WD} = 140 \times \delta \qquad \text{as, } \delta = 3\theta$$

$$= 420\,\theta$$

Eauation, Internal work done and external workdone

We have, $4\,M_p\theta = 420\,\theta$

$$M_p = \frac{420}{4} = 105 \text{ kNm}$$

$\therefore$ Plastic moment capacity of beam BC = 105 kNm

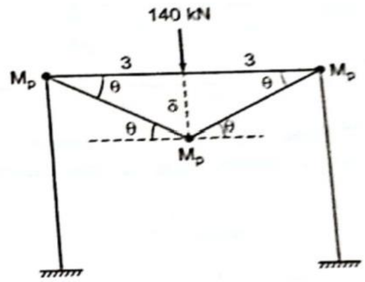


Fig. 10.38

(ii) Sway Mechanism (Refer Fig. 10.39)

$$\text{Internal WD} = M_p\theta + M_p\theta + M_p\theta + M_p\theta$$

$$= 4M_p\theta$$

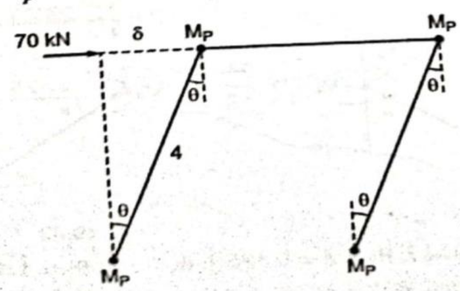
$$\text{External WD} = 70 \times \delta$$

$$= 70 \times 4\theta \qquad \text{as, } \delta = 4\theta$$

$$= 280\,\theta$$

Equating, $4\,M_p\theta = 280\,\theta$

$\therefore$ $M_p = 70$ kNm



(iii) Combined Mechanism (Refer Fig. 10.40)

$$\text{Internal WD} = M_p\theta + M_p(\theta + \theta) + M_p\theta + M_p\theta + M_p\theta = 6\,M_p\cdot\theta$$

$$\text{External WD} = 70 \times \delta_1 + 140 \times \delta_2;\ \delta_1 = 4\theta$$

$$= 70 \times 4\theta + 140 \times 3\theta;\ \delta_2 = 3\theta$$

$$= 700\,\theta$$

Equating, internal *WD* and external *WD*.

We have, $6\,M_p \cdot \theta = 700 \cdot \theta$

$$M_p = \frac{700}{6}$$

$\therefore$ $M_p = 116.67$ kNm

$\therefore$ Plastic moment capacity of frame is the highest of above

$\therefore$ $M_p = 116.67$ kNm

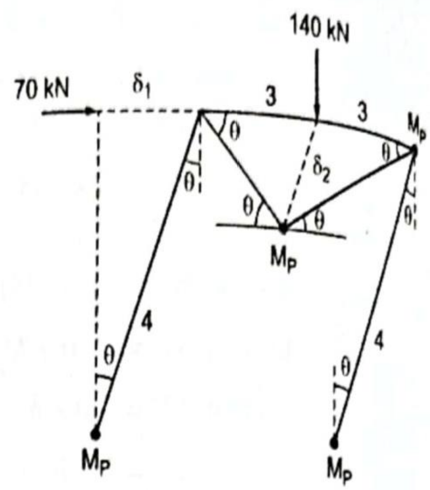


Fig. 10.40

5 **Explain the Types of Bolted connection and write the failure modes of bolted connections.**

Bolted Connections

Connections are always needed to connect two members. It is necessary to ensure functionality and compactness of structures. Prime role of connections is to transmit force from one component to another. Steel connections can be made by bolts or welds. Connections accounts for more than half cost of steel structure. Connections are designed more conservative than members because they are more complex.

1. **Types of Bolts**

    1) Unfinished Bolt – ordinary, common, rough or black bolts
    2) High strength Bolt – friction type bolts

2. **Classifications of Bolted connections:**

    1) Based on Joint:
    2) Lap Joint
    3) Butt Joint
    4) Based on Load transfer Mechanism**:**
    5) Shear and bearing,
    6) Friction

3. **Grade classification of Bolts:**

    - The grade classification of a bolt is indicative of the strength of the material of the bolt. The two grades of bolts commonly used are grade 4.6 and 8.8.
    - For 4.6 grade 4 indicates that ultimate tensile strength of bolt = 4 × 100 = 400 N/mm2 and 0.6 indicates that the yield strength of the bolt is 0.6 × ultimate strength = 0.6 × 400 = 240 N/mm2

Most Imp. Failure of Bolted Joints.

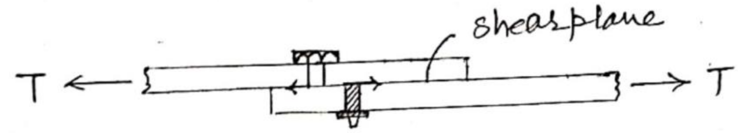
There are two categories of failure

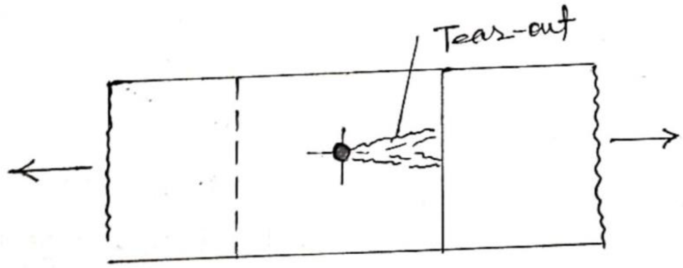
(a) Failure of Bolts. (b) Failure of plates.

(i) Shear failure :-

(a) Shear stresses are generated when plates slip due to applied force, Max^m shear stress in bolt may exceed to the nominal shear capacity

(b) When plate material is weaker than bolt than shear tear-out at end of the connected member take place.

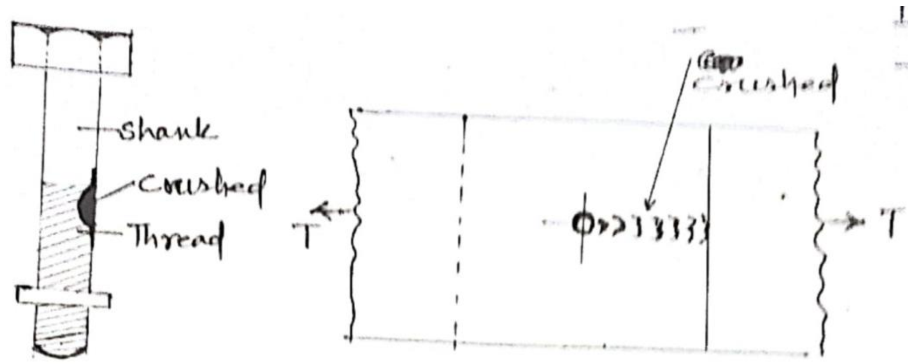




(ii) Bearing Failure

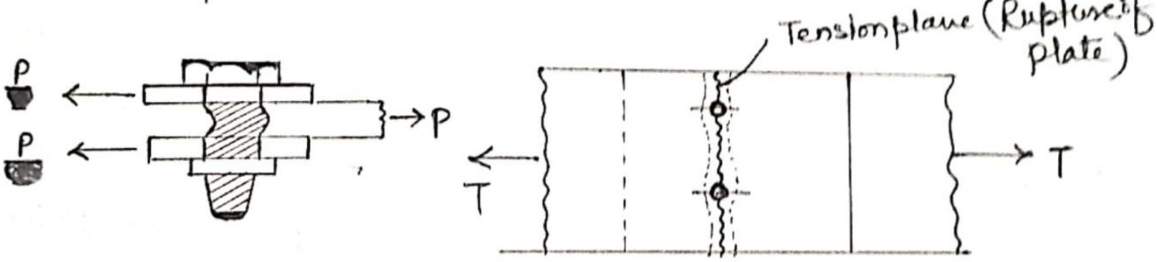
(a) Bolts are Crushed around the half Circumference. The plate having good strength in bearing which may press the bolt shank. so bolt deforms due to high local bearing stress.

(b) If plate material is Weaker, It gets crushed



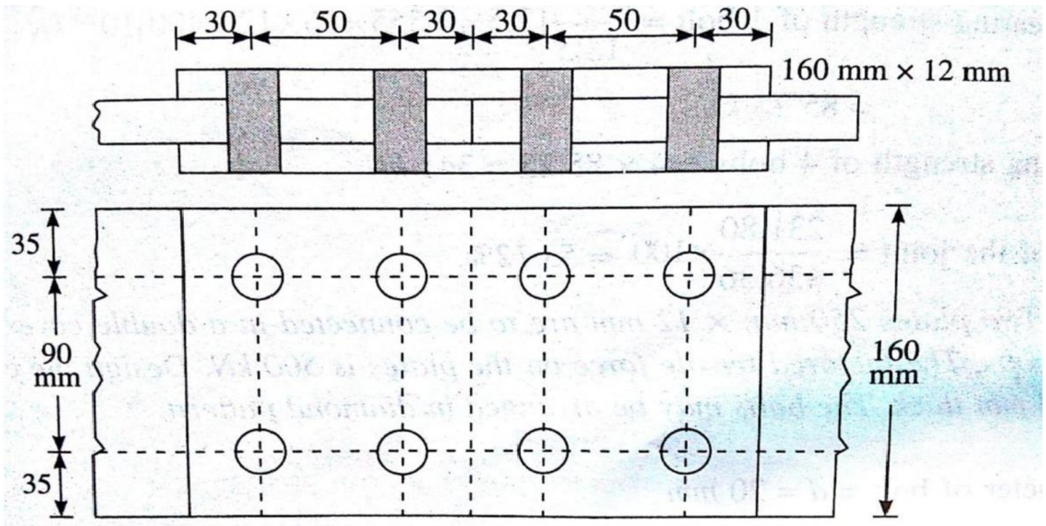
(iii) <u>Tensile Failure</u> :(a) Bolts are subjected to tension at tension plane (Stressed Area), also weaker than plate material.

(b) It occurs when plate is not sufficient flexible and weaker than Bolt material.



iv) <u>Block-shear Failure</u> :- In this failure, The failure of member occurs along a path involving tension on one plane and shear on perpendicular plane along the fasteners. When a block of material within bolted area breaks away from the remainder area. Such failure may appear.

6. **Find the efficiency of the butt join shown in figure. Bolts are 16mm diameter of grade 4.6. Cover plates are 8mm thick.**



Shear Strength *(IS 800:2007, Clause 10.3.3, page no. 75)*

$$Vdsb = Vnsb\gamma mb$$

$Vnsb$= $fu\sqrt{3}\times$ $[nnAnb\ +nsAsb]$ $Vdsb$

= $400\sqrt{3}\times 1.25\times$ [(2 ×0.78 × π4× 162) +(0 × π4× 162)]

= 57.95 kN

Bearing Strength:*(IS 800:2007, Clause 10.3.4, page no. 75)*

$$Vdpb = Vnpb\gamma mb$$

Vnpb = 2.5kb dtfu

$Vdpb$ = 2.5 × 0.56 × 16 × 12 × 4101.25 = *88.16 kN*

e = 30mm

p = 50 mm

e/3do = 30/ (3 × 18) = 0.56;

p/3do – 0.25 = [50/ (3 ×18)] – 0.25 = 0.67;

fub/fu = 400/410 = 0.976;

Efficiency of *the* joint = [*Strength* of *the Joint per pitch length* / *Strength* of *solid plate per pitch Length*] ×100

Strength the joint per pitch length = 57.95 kN

Strength of *solid plate per pitch length* = $0.9 A_n f_u \gamma_{m1}$ (clause no. 6.3.1, page no. 32, IS 800:2007)

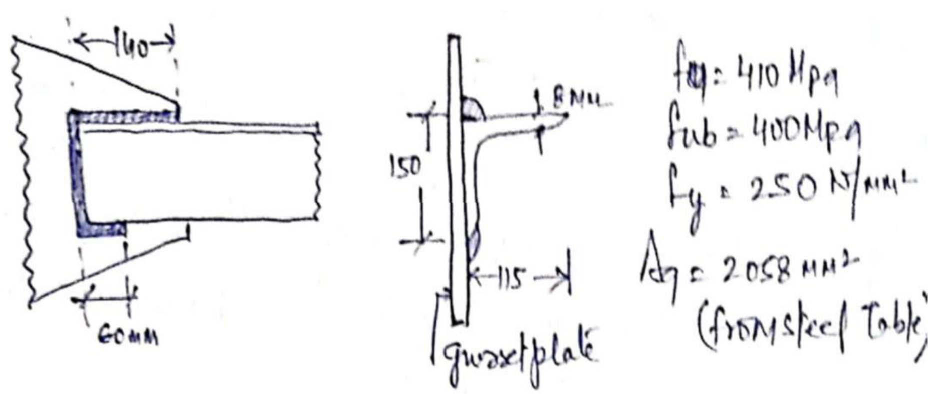
= 0.9× (50−16) ×12×4101.25

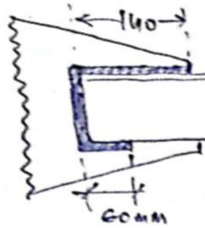
= 120.44 kN

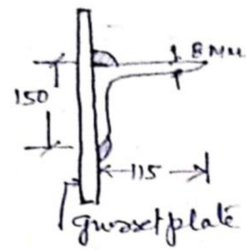
Efficiency of *the* joint = [57.95120.44] ×100

= 48.11 %

7. **Compute tensile strength of an angle ISA 150X115X8 mm of Fe 410 grade connected with gusset plate for a) Gross section yielding b) Net Section rupture.**







$f_u = 410$ Mpa
$f_{ub} = 400$ Mpa
$f_y = 250$ N/mm²
$A_g = 2058$ mm² (from steel Table)

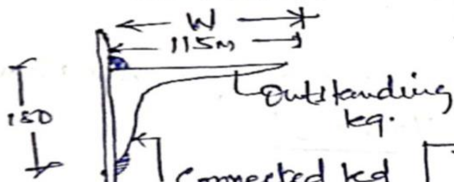
ANS

(1) Strength in Yielding of Gross Area :-

$$T_{dg} = \frac{A_g \cdot f_y}{\gamma_{m0}} = \frac{2058 \times 250}{1.10}$$

$$T_{dg} = 467.73 \text{ kN}$$

(2) Strength in fracture/Rupture ~~at Net Section~~ of Critical section :-



Shear width $(b_s) = 115$ mm
As per IS 800:2007
$b_s = W$

$$T_{dn} = \frac{0.9 A_{nc} \cdot f_u}{\gamma_{m1}} + \frac{\beta A_{go} \cdot f_y}{\gamma_{m0}} \quad \text{— (A)}$$

Weld Length $(L_e) = 140 + 150 + 60 = 350$ mm

$$\beta = \left\{1.4 - 0.076\left(\frac{115}{8}\right)\left(\frac{250}{410}\right)\left(\frac{115}{350}\right)\right\} \leq \left\{\frac{410}{250} \times \frac{1.10}{1.25}\right\} \geq 0.7$$

$$\beta = 1.1811 \leq 1.443 \geq 0.7 \quad \therefore \beta = 1.18$$

Now Net Area of Connected Leg $(A_{nc}) = \{150 - (8/2)\} \times 8 = 1168$ mm²

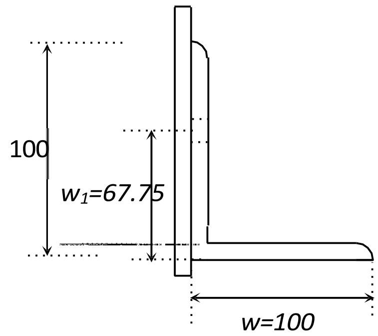
Net Area of Outstand leg $(A_{go}) = \{115 - (8/2)\} 8 = 888$ mm²

$$T_{dn} = \frac{0.9 \times 1168 \times 410}{1.25} + \frac{1.18 \times 888 \times 250}{1.10}$$

$$T_{dn} = 516.54 \text{ kN}$$

STRENGTH OF THE ANGLE (Minm Value) = **467.73 kN**

8. Design a single Equal angle 100 x 100 x 8 mm, connected to a gusset plate at the ends with 20mm diameter bolts with the connection length of 250mm to transfer tension.



Net area of connected leg *(A_{nc})* $= (100 - 8/2 - 21.5) \times 8$ = 596 mm^2

Gross area of outstanding leg *(A_{go})* = $(100 - 8/2) \times 8$ = 768 mm^2

Area of gross section *(A_g)* = $(100 + 100 - 8) \times 8$ = 1536 mm^2

Yield stress of steel *(f_y)* = 250 MPa

Ultimate stress of steel *(f_u)* = 410 MPa

Minimum End distance of fastener = 32.25 mm

Minimum Edge distance of fastener = 32.25 mm

Strength as governed by Rupture of Critical section:

Table 10.1

Shear Lag Distance *(b_s)* = 160 mm

Connection length *(L_c)* = 250 mm

Hence β $= 1.4 - 0.076 \times (w/t) \times (f_y / f_u) \times (b_s / L_c)$

= 1.4-0.076x((100/8)x(250/410)x(152/250) = 1.03

Clause 6.3.2

Tensile strength, T_{dn} = $0.9 \times A_{nc} \times (f_u / \gamma_{m1}) + \beta \times A_{g0} \times (f_y / \gamma_{m0})$

$$= 0.9 \times 596 \times \left(\frac{410}{1.25}\right) + 1.03 \times 768 \times \left(\frac{250}{1.1}\right) = 356 \text{ kN}$$

Strength as governed by Yielding of Gross section:

Tensile strength, T_{dg} $= A_g \times (f_y / \gamma_{m0}) = 1536 \times (250/1.1)$

= 349 kN

Hence yielding of gross area governs the member strength.

Therefore Design Tensile Strength of the member , T_d = 349 kN **Clause 6.2**

9. Design the tensile strength of section ISMB300 with gusset plate connected to the flange. The section is connected to end gusset plate by using four rows of 18 mm bolts at a section and a connection length of 100mm.

Section properties:

Gross area A = 5626 mm^2

Depth of section D = 300

mm Breadth of flange

b=140 mm

Thickness of flange t_f

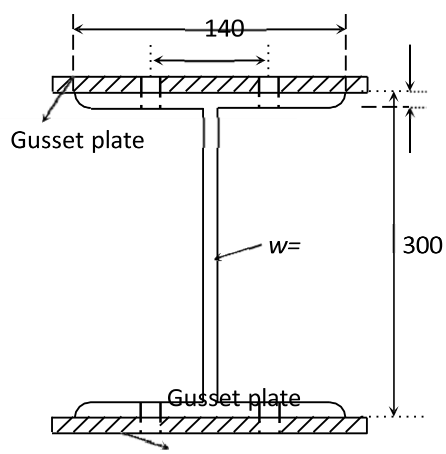
=12.40 mm

Thickness of web t_w = 7.5 mmNet

area of connected leg (A_{nc})

= 5626-(300-2*12.4)*7.5-4*19.5*12.4

= 2594.80 mm^2



Area of outstanding leg (A_{go}) = $(300 - 2 \times 12.4) \times 7.5$ = 2064 mm^2

Tensile strength by yielding of gross section:

$T_{dg} = A_g \times f_y / \gamma_{m0} = 5626 \times 250 / (1.1 \times 1000)$ = ***1279 kN***

Tensile strength by rupture of critical section:

w_1 = (g)/2 = 40 mm; *w = 300/2 = 150 mm* **Clause 6.2**

$b_s = 150 + 40 - (12.4+7.5)/2 = 180.05mm$; $L_c = 100$ mm

$\beta = 1.4 - 0.076 \times (w/t) \times (f_y / f_u) \times (b_s / L_c)$

$= 1.4 - 0.076 \times (150/7.5) \times (250/410) \times (180.05/100) = -0.268 < 0.7$

Hence β = 0.7

Clause 6.3.3

$T_{dn} = 0.9 \times A_{nc} \times (f_u / \gamma_{m1}) + \beta \times A_{g0} \times (f_y / \gamma_{m0})$

$= ((0.9 \times 2594.8 \times 410/1.25) + (0.7 \times 2064 \times 250/1.1))/1000$

= *1094.35 kN*

The tensile strength of the section is 1094 kN.

10. Design the compressive strength of the column section ISLB 500 @0.75 kN/m with the effective length of the column as 5 m. assume the buckling axis as y-y axis and basic yield strength (fy) as 250 mPa.

Section Properties:

Gross area of the section (A_g)= *95.50x10* $^2mm^2$

Depth of section (*D*) = *500 mm*

Full Width of flange (b_f)= *180*

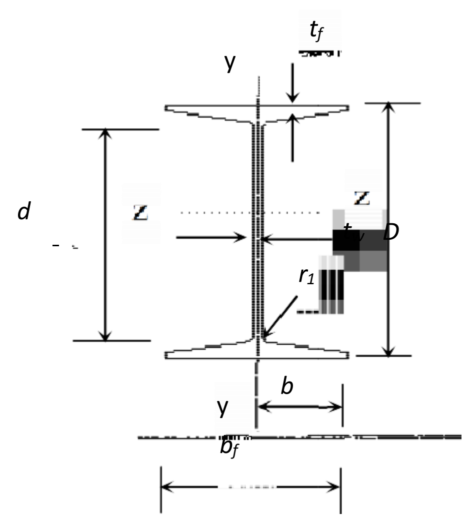
mm Thickness of flange

(t_f)=*14.1 mm*Thickness of web

(t_w)= *9.2 mm* Radius of

gyration *(r_y)= 33.4 mm*Radius

at root (r_1)= *17 mm*



Yield stress (f_y) = 250 m*Pa*

Section classification:

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

Flange:

$b = b_f / 2 = 180 / 2 = 90$ mm

$b / t_f = 90 / 14.1 = 6.38 < 9.4\ \varepsilon$ (**Plastic**)

∴ Flange is fully effective

Web:

$d = D - 2 \text{ x } t_f - 2 \text{ x } r_1$

$= 500 - 2 \times 14.1 - 2 \times 17 = 437.8 \ mm$

$d / t_w = 437.8 / 9.2 = 47.6 > 42\ \varepsilon$ (**Slender**)

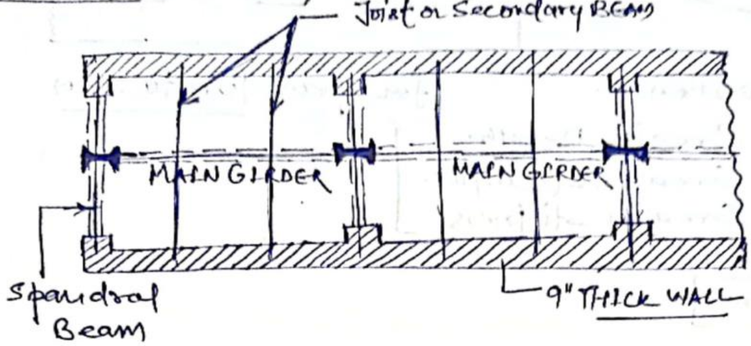
∴Web is not fully effective Section is classified as **Slender**

Net area of the section $= A_g - (((d / t_w) - 42) \text{ x } t_w \text{ x } t_w)$

$= (95.50 \times 10^2) - ((47.6 - (42 \times 1)) \times 9.2 \times 9.2) = 90.77 \text{x} 10^2 \ mm^2$

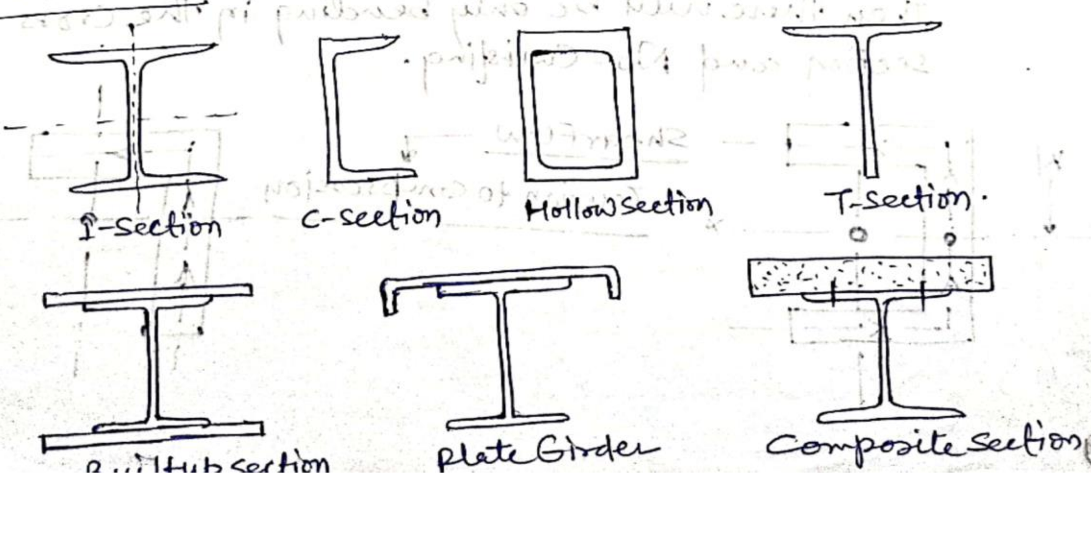
Design of Beams

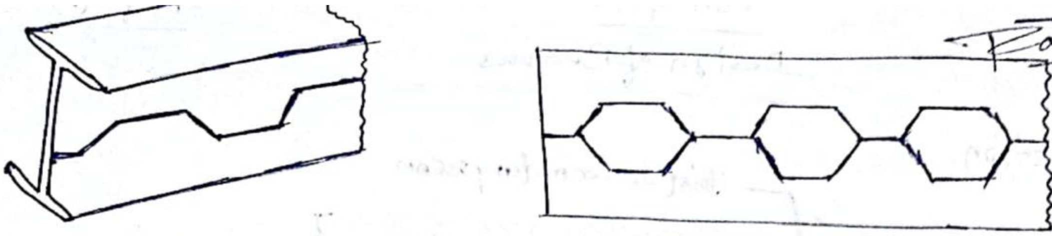
Introduction



1) Joist :- Kind of Secondary Beam which supports only flooring.
2) Girder :- It is main Beam which Support Only Number of Joists or Secondary Beams.
3) Spandral Beams :- It is a Outer Beam at floor level that carry a part of floor load.
4) Purlins :- A beam which carry the roof loads in trusses.
5) Lintel :- This is a beam spanning door opening, window opening.
6) Stringer Beam :- A longitudional beams. Used in Bridges floors and steel stairs.

Beam-Sections





CASTELLATED BEAMS.

for span 10m to 20m

Advantages (1) Greater Depth.
(2) Greater strength.
(3) Greater Stiffness.

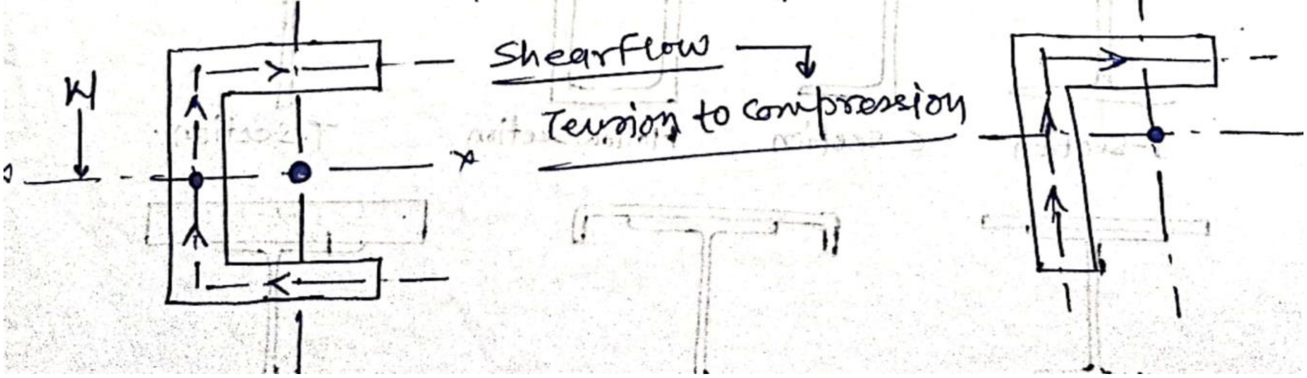
Bending of Beams

* Shear Centre :- It is a point of Intersection of the bending axis and the plane of the transverse section. Shear Centre is also known as the Centre of twist.

Condn a) In Case of Beam having Two axis of Symmetry, The shear Centre Coincides with Centroid.

b) In case of Beam section having One-Axis of Symmetry, The shear centre does not Concide with the centroid.

= (c) When load passes through the shear Centre Then there will be only bending in the Cross section and No-twisting.



Determine the max^m UDL for a simply supported beam of span 8m. The beam is ISMB 300 and the ends are restrained against Torsion but Compression flange is latterally Unsupported.

Given ISMB 300 @ 44.2 kg/m. (from steel Table)

Area (A) = 56.26×10^2 mm²

Total Depth of section (D) = 300 mm

Width of Flange (b_f) = 140 mm

Thickness of Web (t_w) = 7.5 mm

r_{min} = 28.4 mm

Plastic sect^n Modulus (Z_p) = 651.74×10^3 mm³

* Cond^n :- Ends are restrained against torsion but Compression flange is laterally Unsupported.

∴ Effective Length for Simply Supported Beam

(As per clause 8.3.1, Table-15)

$KL = 1 \times 8 = 8\text{m}$ (Normal Cond^n)

Step-1 Calculate Critical Stress (f_{crb})

from table-14

$$\frac{KL}{r_{min}} = \frac{8 \times 10^3}{28.4} = 281.69$$

$$\frac{h}{t_f} = \frac{300}{12.4} = 24.194$$

$\frac{KL}{r}$	h/t_f = 20	25
280	91.1	74.7
290	87.7	71.8

I^{st} Interpolation w.r.t $\frac{h}{t_f}$

$$f_{crb} = 91.1 - \left[\frac{(91.1 - 74.7)(24.19 - 20)}{(25 - 20)}\right]$$

$$= 77.36 \ N/mm^2$$

II^{nd} Interpolation w.r.t $\frac{h}{t_f}$

$$f_{crb} = 87.7 - \left[\frac{(87.7 - 71.8)(24.19 - 20)}{(25 - 20)}\right]$$

$$= 74.38 \ N/mm^2$$

Final f_{crb} $= 77.36 - \left[\frac{(77.36 - 74.38)(281.69 - 280)}{(290 - 280)}\right]$

$$\boxed{f_{crb} = 76.86 \ N/mm^2}$$

step-II Calculate Design Bending Compressive stress (f_{bd}) For Rolled Section $\boxed{\alpha_{LT} = 0.21}$

from Table-13 a;

f_{crb}	f_y (250)
80	63.6
70	56.8

By Interpolation

$$f_{bd} = 63.6 - \left[\frac{(63.6 - 56.8)(80 - 76.86)}{(80 - 70)}\right]$$

$$\boxed{f_{bd} = 61.46 \ N/mm^2}$$

Pandian

Step-III Section classification

from Table-(2) $\epsilon = \sqrt{\frac{250}{250}} = 1$

(i) $\frac{b}{2t_f} = \frac{140}{12.4 \times 2} = 5.65 < 9.4\epsilon$

(ii) $\frac{d}{t_w} = \frac{h - 2(t_f + R)}{t_w} = \frac{300 - 2(12.4 + 10)}{7.5}$

$= 34.03 < 84\epsilon$ ∴ plastic section

step-IV Design of Bending Strength of Laterally Unsupported Beam.

Clause 8.2.2 $M_d = \beta_b \cdot Z_p \cdot f_{bd}$

$\{\beta_b = 1\}$

∴ $M_d = 651 \times 10^3 \times 61.46$

$M_d = 40 \cdot kN \cdot m$

step-V calculate the load (w)

$\text{Max}^m \text{ Moment} = \frac{wl^2}{8}$

∴ $M \simeq M_d = \frac{wl^2}{8}$

∴ $40 = \frac{w(8)^2}{8}$

$w = 5 \; kN/m$ Ans

5.9 COLUMN BASE

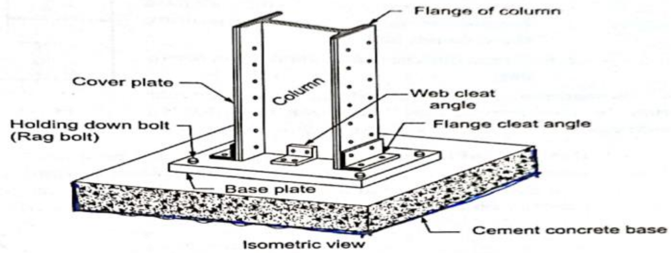
Foundation is necessary for a column to distribute the column load on sufficient area of the soil so that the pressure on base should not exceed the bearing capacity of the soil. It is also important that the column load should be applied on sufficient area of the concrete foundation so that the stress in concrete should be within the bearing strength of the concrete. A steel plate is therefore used to distribute the column load on sufficient area of concrete foundation.

Base plate may be used of the following types depending on the load on the column:

(i) Slab base
(ii) Gusseted base

5.9.1 Slab Base

For columns carrying small loads, slab bases are used. It consists of a base plate (placed underneath a machined column end) and cleat angles. The machined column end transfers the load to the slab base by direct bearing. The column end is connected to base plate by welding or by means of bolted angle iron cleats. No gusset plates are required for connecting the slab base. Fig. 5.15 shows the details of slab base.



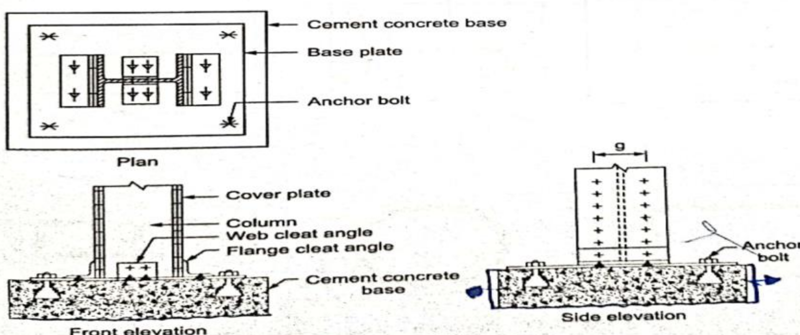


Fig. 5.15 Slab base

5.9.2 Gusseted Base

For columns carrying heavy loads, gusseted bases are used. The loads are transmitted to the base plate through the gusset plates attached to the flanges of the column by means of angle iron cleats (also called gusset angles). In addition to the gusset plates, cleat angles are used to connect the column to base plate. The thickness of base plate in this case will be less than the thickness of the slab base for the same axial load as the bearing area of the column on base plate increases by the gusset plate.

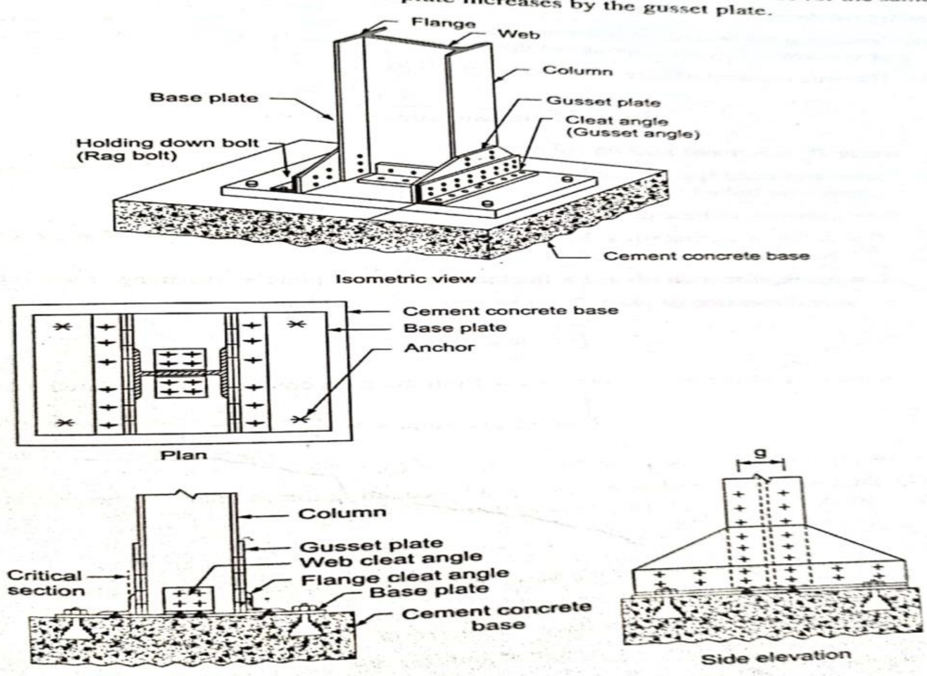


Fig. 5.18 Gusseted base

Problem 5.9

Design the base plate for an ISHB 300 @ 618 N/m column to carry a factored load of 1000 kN. Assume Fe 410 grade steel and M 25 grade concrete.

Solution: (i) Given data

Design the base plate = ?

Column ISHB 300 @ 618 N/m

Factored load = 1000 kN

$= 1000 \times 10^3$ N

For, Steel Fe 410; $f_y = 250$ N/mm^2

Concrete, M 25; $f_{ck} = 25$ N/mm^2

(ii) Properties of ISHB 300 @ 618 N/m (Refer steel table)

$A = 8025$ mm^2

$h = 300$ mm $\quad b_f = 250$ mm

$t_f = 10.6$ mm $\quad t_w = 9.4$ mm

$I_{zz} = 129.50 \times 10^6$ mm^4, $\quad r_{zz} = 127.0$ mm

$I_{yy} = 22.46 \times 10^6$ mm^4 $\quad r_{yy} = 52.9$ mm

(iii) Bearing strength of concrete

$= 0.45\, f_{ck}$

σ_b $= 0.45 \times 25 = 11.25$ N/mm^2 $\qquad \sigma_b = \frac{P}{A}$

$\therefore$ Required area of base plate $= \dfrac{1000 \times 10^3}{11.25} = 88.89 \times 10^3$ mm^2

Use a rectangular base plate of size 400 × 350 mm with area 1,40,000 mm^2

If ISHB 300 is kept at the centre of base plate, as shown in Fig. 5.31.

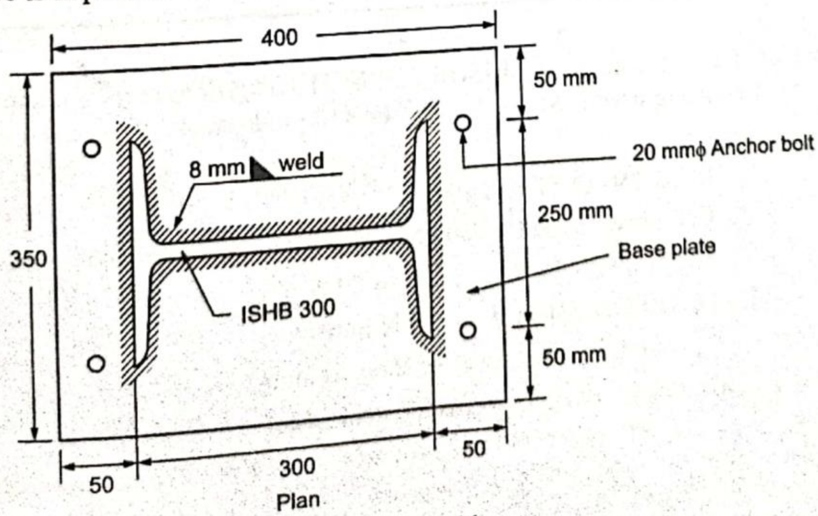


Fig. 5.31

The projections of 50 mm is available on each side as shown in Fig. 5.31 above.

The bearing pressure of concrete

$$w = \frac{P}{\text{Area of base plate}}$$

$$= \frac{1000 \times 10^3}{400 \times 350}$$

$$= 7.15 \text{ N/mm}^2 < 11.25 \text{ N/mm}^2 \quad \therefore \quad \text{Safe.}$$

Projections $a = b = 50$ mm

(iv) The thickness of base plate: (Refer IS:800-2007)

Fwd = 0.7t S

$$t_s = \sqrt{\frac{2.5\, w\, (a^2 - 0.3b^2)\, \gamma_{mo}}{f_y}}$$

$$= \sqrt{\frac{2.5 \times 7.15 \times (50^2 - 0.3 \times 50^2) \times 1.10}{250}}$$

$$= 11.75 \text{ mm} > t_f = 10.6 \text{ mm} \quad \therefore \quad \text{Safe}$$

= say 12 mm

$\therefore$ Provide the base plate for ISHB 300 @ 618 N/m column is 400 × 350 × 12 mm.

(v) Welded connection

Use 8 mm fillet weld all around the column section to hold the base plate in position.

Total length available for welding = 2 × 250 + 2 × 300 – 2 × 10.6 – 2 × 9.4 = 1060 mm

or 2(250 + 300 – 10.6 – 9.4) = 1060 mm

Capacity of 8 mm weld = 0.7 × 8 × 189 = 944.8 N/mm

$\therefore$ Length of weld required = 1000 × 10^3/944.8 = 1058 mm < 1060 mm

$\therefore$ 8 mm the fillet weld is adequate.

Problem 5.10

Design the base plate for a column made of ISHB 250 @ 51.10 kg/m to carry a compressive load of 780 kN. The grade of concrete used is M 20. Assume Fe 410 grade steel. *[MTU 2011-12]*

Solution: (i) Given data:

ISHB 250 @ 51.10 kg/m

Compressive load = 780 kN = 780 × 10^3 N

For M 20 grade concrete, $f_{ck} = 20$ N/mm^2

Fe 410 grade steel, $f_y = 250$ N/mm^2

$\gamma_{mo} = 1.1$, $\gamma_{mw} = 1.25$ (For shop welding)

(ii) Properties of ISHB 250 @ 51.10 kg/m (Refer steel table)

$A = 64.96$ cm^2, $h = 250$ mm $b = 250$ mm

$t_f = 9.7$ mm $t_w = 6.9$ mm

$I_{xx} = 7736.5$ cm^4 $r_{zz} = 10.91$ cm, $r_{yy} = 5.49$ cm.

(v) Welded connection

Assume 8 mm thick fillet weld all around the column to hold it in position.

Total length available for welding (Refer Fig. 5.32)

$$L = 2(250 + 250 - 9.7 - 6.9) = 966.8 \text{ mm}$$

$$\text{Length of weld required} = \frac{\text{Load}}{\text{Strength of weld}}$$

$$\text{Strength of 8 mm weld} = 0.7 \times 8 \times 189 = 944.8 \text{ N/mm}$$

$$\therefore \quad \text{Length of weld required} = \frac{780 \times 10^3}{944.8} = 806.78 < 966.8 \text{ mm} \quad \therefore \text{Safe}$$

∴ Fillet weld of 8 mm thick is adequate.

Problem 5.11

Design a gusseted base for a column ISHB 350@710 N/m with two plates 450 mm × 20 mm carrying a factored load of 2500 kN. The column is to be supported on concrete pedestal with M 20 grade concrete.

Solution: (i) Given data:

Column ISHB 250 @ 710 N/m

Factored laod $P_u = 2500 \text{ kN} = 2500 \times 10^3$ N

Plate size = 450 mm × 20 mm

For M 20 grade concrete $f_{ck} = 20 \text{ N/mm}^2$

(ii) Properties of ISHB 350 @ 710 N/m

$A = 92.21 \text{ cm}^2$

$h = 350$ mm, $b_f = 250$ mm

$t_f = 11.6$ mm, $t_w = 10.1$ mm

(iii) Bearing strength of concrete = $0.45 f_{ck}$ (As per IS:456-2000)

$= 0.45 \times 20 = 9 \text{ N/mm}^2$

∴ Required area of base plate

$$A = \frac{P_u}{\text{Bearing strength}}$$

$$= \frac{2500 \times 10^3}{9}$$

$$= 277.78 \times 10^3 \text{ mm}^2$$

Assuming, ISA 150 × 150 × 15 mm and 16 mm gusset plate

$$\text{Width of base plate required} = 350 + 2(20 + 16 + 150)$$

$$= 722 \text{ mm say } 750 \text{ mm}$$

$$\therefore \quad \text{Length of base plate} = \frac{277.78 \times 10^3}{750}$$

$$= 370.37 \text{ mm say } 500 \text{ mm}$$

∴ Size of base plate = 750 × 500 mm

∴ Area provided = 375000 > 277.78×10^3 ∴ OK

Now,

$$\text{Pressure under base plate} = \frac{2500 \times 10^3}{375000}$$

$= 6.67\ \text{N/mm}^2 < 9\ \text{N/mm}^2$ ($\sigma_{cbc} = 9$ MPa, for M 20 conc.) ∴ Safe.

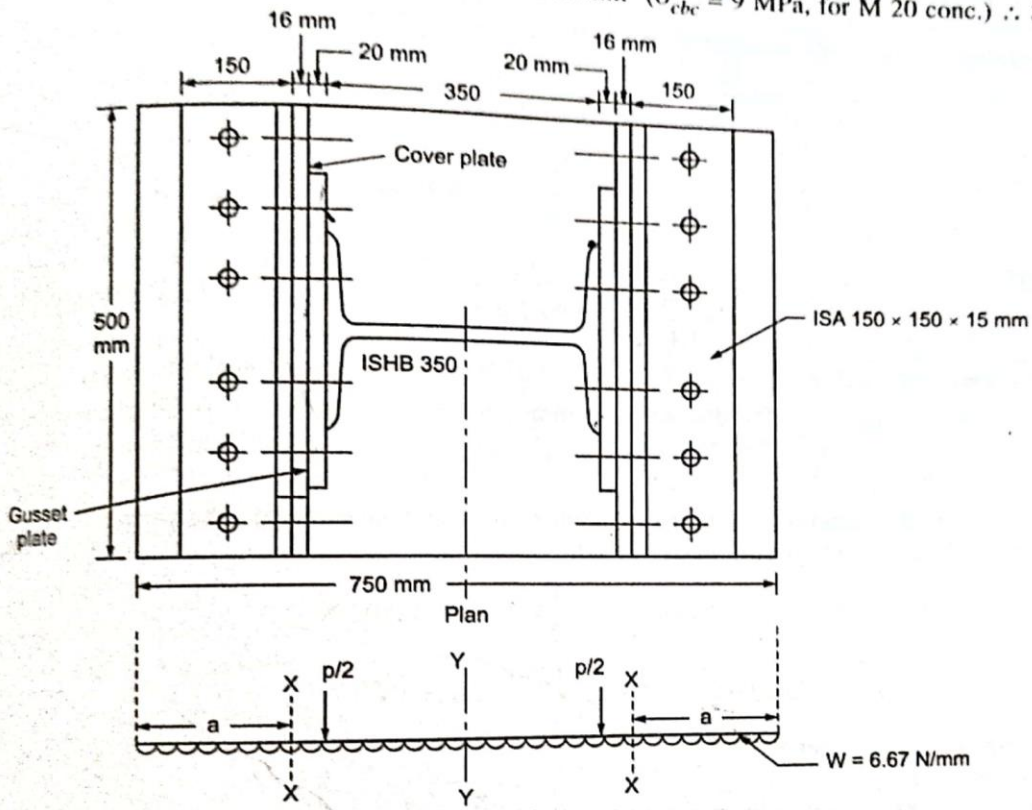


Fig. 5.33 Plan and loading diagram

$$a = \frac{750 - 2(20 + 16) - 350}{2}$$

$= 164$ mm

∴ B.M. at the section *X-X* per mm width

$$\text{B.M.}_{(X\text{-}X)} = \frac{Wa^2}{2} = \frac{6.67 \times 164^2}{2} = 89.7 \times 10^3\ \text{Nmm}$$

Reaction $\frac{p}{2}$ due to upward earth pressure

$$\frac{p}{2} = \frac{W \times L}{2} = \frac{6.67 \times 750}{2} = 2501.25\ \text{N}$$

$$\text{B.M. at section } Y\text{-}Y = 6.67 \times 375^2 - \frac{p}{2} \times \frac{350}{2}$$

$$= 468.98 \times 10^3 - 2501.25 \times 175 = 31.26 \times 10^3 \text{ Nmm}$$

$$\text{Design B.M.} = 89.7 \times 10^3 \text{ Nmm}$$

$$\therefore \quad \text{Bending strength} = \frac{f_y}{\gamma_{mo}} = \frac{250}{1.1} = 227.27 \text{ N/mm}^2$$

Equating moment of resistance to design B.M.

$$\frac{1.2 \cdot f_y \cdot Z_e}{\gamma_{mo}} = \text{B.M.}$$

$$Z_e = \frac{bt^2}{6} \qquad b = 1 \text{ mm}$$

$$\therefore \quad \frac{1.2 \times 250 \times \frac{t^2}{6}}{1.1} = 89.7 \times 10^3$$

$\therefore$ $t = 44.42$ mm say 56 mm (available thickness of plate as per steel table)

$\therefore$ Provide base plate of 750 × 500 × 56 mm

(iv) Connection

Assuming ends of columns are faced for complete bearing and the connection between gusset plate and column will be designed for 50% of axial load.

$$\text{Design load} = \frac{1}{2} \times 2500 = 1250 \text{ kN}$$

$$\text{Load on each face} = \frac{1250}{2} = 625 \text{ kN}$$

Using 20 mm diameter shop bolts,

(a) Strength of bolt in single shear $= 0.78 \times \frac{\pi}{4}(20)^2 \times \frac{400}{\sqrt{3} \times 1.25}$

$$= 45.27 \times 10^3 \text{ N}$$

(b) Strength of bolt (20 mm) in bearing (16 mm gusset plate)

$$= \frac{2.5 \times 0.5 \times 20 \times 16 \times 400}{1.25}$$

$$= 128 \times 10^3 \text{ N}$$

$$\therefore \quad \text{Bolt value} = 45.27 \times 10^3 \text{ N}$$

$$\text{No. of bolts required} = \frac{625 \times 10^3}{45.27 \times 10^3}$$

$$= 13.8 \text{ Nos say 16 Nos.}$$

Provide 16 No. of bolts 20 mm dia. as shown n Fig. 5.34 for connecting column to gusset plate.