CBCS SCHEME

USN 1CR16CV073 **18CV61**

Sixth Semester B.E. Degree Examination, July/August 2022
Design of Steel Structural Elements

Time: 3 hrs. Max. Marks: 100

Note:* 1. *Answer any FIVE full questions, choosing ONE full question from each module.
2. *Use of IS 800-2007, steel table is permitted.*

Important Note : 1. On completing your answers, compulsorily draw diagonal cross lines on the remaining blank pages.
2. Any revealing of identification, appeal to evaluator and /or equations written eg. 42+8 = 50, will be treated as malpractice.

Module-1

1 a. What are the advantages and disadvantages of steel structures? **(10 Marks)**
b. Distinguish between working stress design and limit state design of steel structures. **(10 Marks)**

OR

2 a. Calculate the shape factor of triangle. **(10 Marks)**
b. Calculate 'M_p' for the continuous beam shown in Fig.Q2(b). Take load factor 1.5.

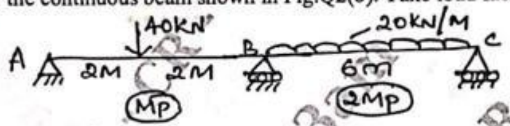

Fig.Q2(b) **(10 Marks)**

Module-2

3 a. Explain the failure modes of bolted connection. **(10 Marks)**
b. Design a bolted connection for a lap joint of plate thickness 16 mm and 12 mm to carry a factored load of 160 kN. Use M_{16} and 4.6 grade bolts. **(10 Marks)**

OR

4 a. What are the advantages and disadvantages of welded connection? **(10 Marks)**
b. A tie member of Truss consisting of angle section ISA 65 × 65 × 6 mm of Fe 410 grade is welded to 8 mm gusset plate. Design a weld to transmit a factored load of 150 kN. **(10 Marks)**

Module-3

5 a. Explain the failure modes of axial loaded column. **(10 Marks)**
b. Determine the design compressive strength of ISHB300@576.8 N/m, Length of column is 3.5 m and both ends are pinned. **(10 Marks)**

OR

6 Design a single angle discontinuous strut to carry a factored load of 65 kN. The length of strut is 3m, between inter section. It is connected to 12 mm thick gusset plate by 20 mm diameter, 4.6 grade bolts. **(20 Marks)**

Module-4

7 a. Explain the factors effecting strength of tension members. **(10 Marks)**
b. Design a tension member to carry factored load of 400 kN connected to shorter leg back to back. Length of member is 3m. **(10 Marks)**

OR

8 a. Explain Lug angles and column splices. (10 Marks)

b. Design slab base for a column made of ISHB250@536 N/m to carry axial working load of 520 kN. The grade of concrete is M_{20} and grade of steel Fe 410. (10 Marks)

Module-5

9 a. Explain the factors effecting lateral stability of beams. (10 Marks)

b. Calculate the load carrying capacity of laterally restrained simply supported beam with ISMB500@86.9 kg/m section for an effective span of 5m. (10 Marks)

OR

10 Design a steel beam section for supporting hall for the following data:

Clear span = 6.5 m

End bearing = 200 mm

c/c spacing of beams = 3 m

Live load on beams = 12 kN/m^2

Dead load = 3 kN/m^2 (20 Marks)

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Module 1
1a)
Advantages & Disadvantages of Steel Structures:
Advantages of Steel Structures:

- They are **super-quick** to build at the site, as a lot of work can be pre-fabbed at the factory.
- **Ease in expansion** of the structure.
- **Ease in repair** & rehabilitation or retrofitting.
- **Faster erection** of the structure.
- They are **flexible**, which makes them very good at resisting dynamic (changing) forces such as wind or earthquake forces. It can bend without cracking, which acts as a warning in seismic zones.
- A wide range of **ready-made structural sections** is available, such as I, C and angle sections.
- They can be made to take any kind of **shape** and clad with any type of
- A wide range of **joining methods** is available, such as bolting, welding, and riveting.
- Steel can be recycled. (New steel made from scrapped steel uses about one-third of the energy necessary for steel from virgin materials)

Disadvantages of Steel Structures:

- Analysis approach and assumptions should be quite clear and definitive prior to structural system formation.
- Time required to **design connection** is more as compared to RC structures connection.
- **Cost** (especially in India) is high for structural steel compared to RC.
- **Skilled laborers** are required.
- Steel can soften and melt with exposure to extremely high temperatures. However, with the addition of passive fire protection, such as spray-on fireproofing, buildings built of structural steel can sustain greater temperatures and therefore, provide additional safety.
- They are prone to **corrosion** in humid or marine environments. Therefore, they need consistent maintenance.
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1 b)
A Civil Engineering Designer has to ensure that the structures and facilities he designs are (i) fit for their purpose (ii) safe and (iii) economical and durable. Thus safety is one of the paramount responsibilities of the designer. However, it is difficult to assess at the design stage how safe a proposed design will actually be. There is, in fact, a great deal of uncertainty about the many factors, which influence both safety and economy. The uncertainties affecting the safety of a structure are due to
· Uncertainty about loading
· Uncertainty about material strength and
· Uncertainty about structural dimensions and behaviour.
These uncertainties together make it impossible for a designer to guarantee that a structure will be absolutely safe. All that the designer can ensure is that the risk of failure is extremely small, despite the uncertainties.
An illustration of the statistical meaning of safety is given in Fig.2.1. Let us consider a structural component (say, a beam) designed to carry a given nominal load. Bending moments (B.M.) produced by loads are first computed. These are to be

compared with the resistance or strength (R.M.) of the beam. But the resistance (R.M.) itself is not a fixed quantity, due to variations in material strengths that might occur between nominally same elements. The statistical distribution of these member strengths (or resistances) will be as sketched in (a).

Similarly, the variation in the maximum loads and therefore load effects (such as bending moment) which different structural elements (all nominally the same) might encounter in their service life would have a distribution shown in (b). ***The uncertainty here is both due to variability of the loads applied to the structure, and also due to the variability of the load distribution through the structure.*** Thus, if a particularly weak structural component is subjected to a heavy load which exceeds the strength of the structural component, clearly failure could occur.

Unfortunately it is not practicable to define the probability distributions of loads and strengths, as it will involve hundreds of tests on samples of components. Normal design calculations are made using a single value for each load and for each material property and taking an appropriate safety factor in the design calculations. The single value used is termed as ***"Characteristic Strength or Resistance"*** and ***"Characteristic Load".***

Characteristic resistance of a material (such as Concrete or Steel) is defined as that value of resistance below which not more than a prescribed percentage of test results may be expected to fall. (For example the characteristic yield stress of steel is usually defined as that value of yield stress below which not more than 5% of the test values may be expected to fall). In other words, this strength is expected to be exceeded by 95% of the cases.

Similarly, ***the characteristic load is that value of the load, which has an accepted probability of not being exceeded during the life span of the structure.*** Characteristic load is therefore that load which will not be exceeded 95% of the time

2a)

Shape factor of various cross sections (conti.)

- Ex 3 Determine the shape factor for a triangular section of base b and height h as shown below.

Triangular section

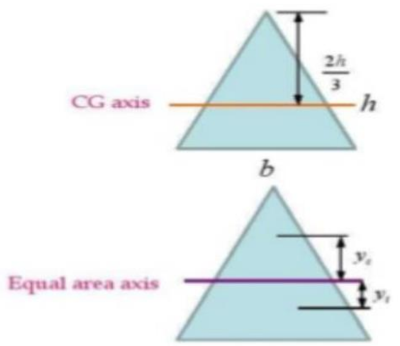
Elastic section modulus

$$Z=\frac{\left(\frac{bh^3}{36}\right)}{2h/3}=\frac{bh^2}{24}$$

Plastic section modulus

$$Z_p=\frac{A}{2}(y_c+y_t)$$

S = 2.346



Shape factor of various cross sections (conti.)

Ex 3 Determine the shape factor for a triangular section of base b and height h as shown below.

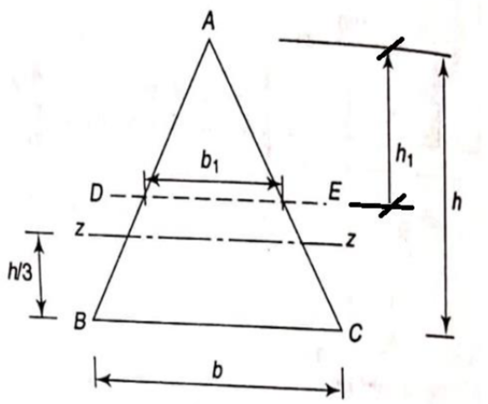


Fig. 8.35

Step 1: Calculate moment of inertia about zz axis

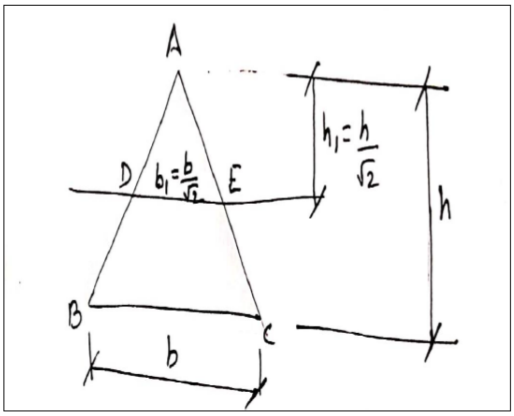
$$I_{ZZ}=\frac{bh^3}{36}$$

Step 2: Find elastic section modulus

$$Z_{ZZ}=\frac{I_{ZZ}}{y_{max}} \qquad y_{max}=\frac{2}{3}h$$

$$Z_{ZZ}=\frac{bh^2}{24}$$

Step 3: Find plastic neutral axis by dividing into equal area



Let DE be the plastic neutral axis

DE divides Δ ABC into two equal areas

Let Δ ADE $= A_1$

⏢ DECB $= A_2$

$A = A_1 + A_2$

But $A_1 = A_2$

Using similar triangles ADE and ABC,

$$\frac{h_1}{b_1} = \frac{h}{b} \Rightarrow \boxed{b_1 = \frac{b h_1}{h}}$$

$$A_1 = \frac{A}{2}$$

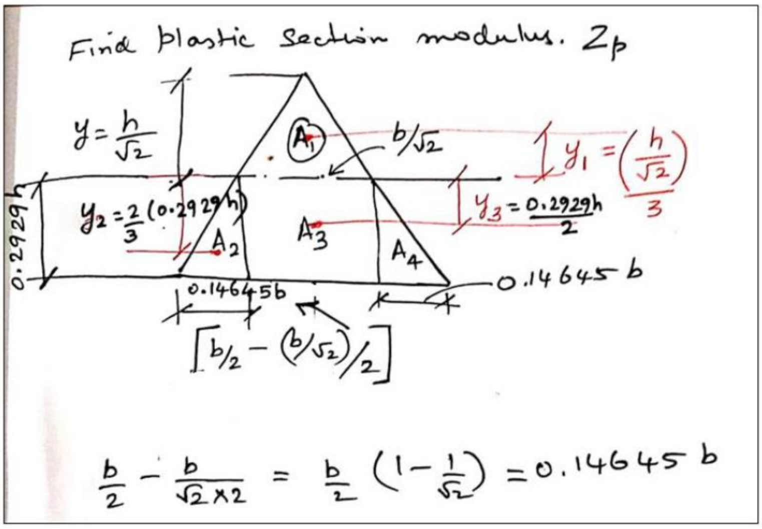
$$\frac{1}{2} b_1 h_1 = \frac{\left(\frac{1}{2} bh\right)}{2} = \frac{1}{4} bh$$

Substi $b_1 = \frac{b h_1}{h}$

$$\frac{1}{2} \times \frac{b h_1}{h} \times h_1 = \frac{1}{4} bh$$

$$h_1^2 = \frac{h^2}{2} \Rightarrow \boxed{h_1 = \frac{h}{\sqrt{2}}}$$

Step 4: Find plastic section modulus (Zp)

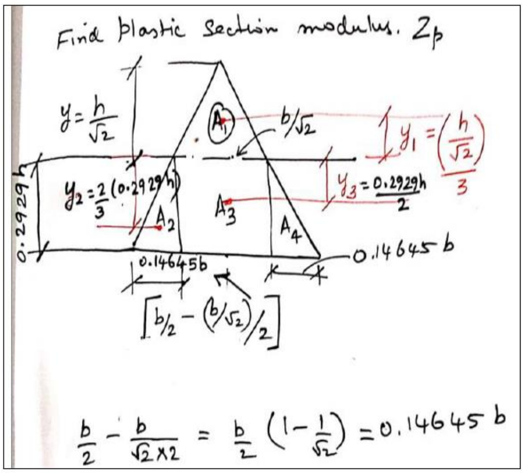


$Z_p = \Sigma (A_i y_i)$

A_i	y_i	$A_i y_i$
$A_1 = \frac{1}{2}\frac{b}{\sqrt{2}}\frac{h}{\sqrt{2}} = \frac{bh}{4}$	$y_1 = \frac{(h/\sqrt{2})}{3}$	$A_1 y_1 = \frac{bh^2}{12\sqrt{2}}$
$A_2 = \frac{1}{2} \times 0.14645b \times 0.2929h = 0.0214466\, bh$	$y_2 = \frac{2}{3} \times 0.2929h = 0.195267h$	$A_2 y_2 = \frac{bh^2}{238.79}$
$A_3 = \frac{b}{\sqrt{2}} \times 0.2929h$	$y_3 = \frac{0.2929h}{2}$	$A_3 y_3 = \frac{bh^2}{32.969}$
$A_4 = 0.0214466bh$	$y_4 = 0.195267h$	$A_4 y_4 = \frac{bh^2}{238.79}$

$$\Sigma A_i y_i = \frac{bh^2}{10.2425}$$

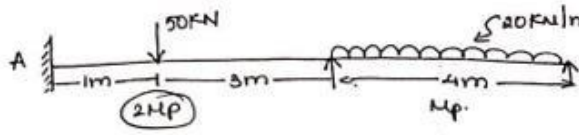
$$\text{Shape Factor} = S = \frac{\text{plastic section modulus } (Z_p)}{\text{Elastic section modulus } (Z_e)} = \frac{bh^2/10.2425}{bh^2/24} = 2.34318$$


Find plastic section modulus. Z_p
$y = \frac{h}{\sqrt{2}}$
A_1
$b/\sqrt{2}$
$y_1 = \frac{\left(\frac{h}{\sqrt{2}}\right)}{3}$
$y_2 = \frac{2}{3}(0.2929h)$
$y_3 = \frac{0.2929h}{2}$
0.2929h
A_2
A_3
A_4
0.14645 b
0.146645b
$\left[b/2 - (b/\sqrt{2})/2\right]$
$\frac{b}{2} - \frac{b}{\sqrt{2}\times 2} = \frac{b}{2}\left(1 - \frac{1}{\sqrt{2}}\right) = 0.14645\, b$

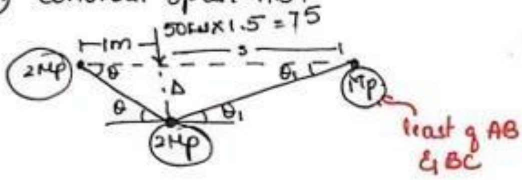
2b)

Solved problems

1. Analyse the continous beam shown. Calculate maximum plastic moment. Take load factor = 1.5.



① Consider span AB.



$\Delta = 1\theta = 3\theta_1$

$\therefore \theta = 3\theta_1$

(or) $\boxed{\theta_1 = \frac{1}{3}\theta}$

$EWD = 75 \times \Delta = 75 \times \theta = 75 \times 3\theta_1$ ——— ①

$IWD = 2M_p(\theta) + 2M_p(\theta+\theta_1) + M_p\theta_1$

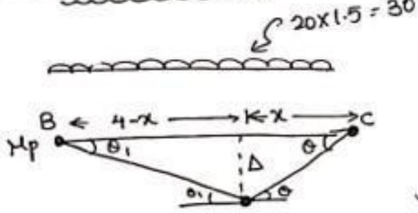
$= 15 M_p\theta_1$ ——— ②

Equating ① & ②

$75 \times 3\theta_1 = 15 M_p \theta_1$

$\boxed{M_p = 15 \text{ KN-m}}$

② Consider span BC



it is like a propped cantilever with UDL

$\therefore W_c = \frac{11.656 \, M_p}{L}$

when $W_c = w \times l$.

$\therefore M_p = \frac{W_c \times l}{11.656} = \frac{wl \times l}{11.656} = \frac{30 \times 4^2}{11.565} = 41.18$ KN-m.

$\boxed{M_p = 41.18 \text{ KN-m}}$

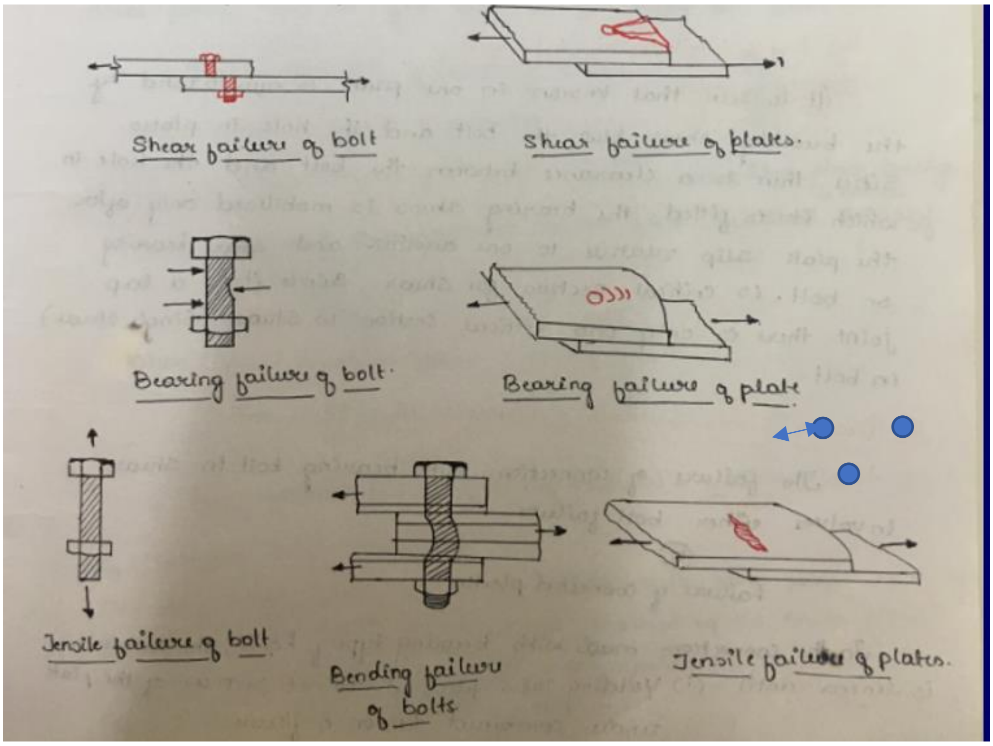
$\therefore$ Finally plastic moment = 41.18 KN-m.

2b)

Module 2

3a) Behaviour of bolted joints:

1. Shear failure of bolt
2. Shear failure of plate
3. Bearing failure of bolt
4. Bearing failure of plate
5. Tensile failure of bolt
6. Tensile failure of plate



Terminologies

Pitch of the bolt (p): c/c spacing of the bolt in a row, measured along the direction of load

Gauge distance(g): C/C b/w 2 consecutive bolts of adjacent row, measured perpendicular to load

Edge distance(e): distance from bolt to edge of the plate

Shear connection with bearing type of bolt: page 74 of IS 800: 2007, clause 10.3

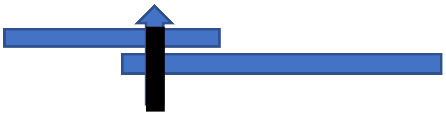
1. Force transfer of bearing type of bolt -
2. Design shear strength of bearing type of bolt – failure at bolt and failure at plate
   a. Yielding takes place at the net section of the plate under combined tension and flexure
   b. Shearing takes place in bolt in shear plane

c. Failure of bolt takes place in bearing
d. Failure of plate takes place in bearing
e. Block shear failure

3b)

Solution:

a) Lap joint



Strength of bolt in single shear: (assume fully threaded bolt)

$$V_{dsb} = \frac{f_{ub}}{\sqrt{3}}(n_n A_{nb} + n_s A_{sb})\ /\gamma_{mb}$$

$$= \frac{400}{\sqrt{3}}(1 * 0.78 * pi * 16 * 16/4 + 0)\ /1.25 = 56.58\text{kN}$$

Bolts in Bearing = V_{dpb} = (2.5K_b *d t* f_u)/ γ_{mb}

K_b *is least of* e/3d_o, p/3d_o- 0.25 , fub/fu , 1 from the IS 800:2007 pg no 75

Assumed e=40mm, p= 50mm, d_o= 20+2=22mm

40/ 3*22= 0.60

50/3*18-0.25= 0.50

400/410= 0.97

1

V_{dpb} = (2.5**0.50*16*12*410)/ 1.25 = 98.40kN*

4a)

Advantages of welding

- Welding will enable direct transfer of stress between members eliminating gusset and splice plates necessary for bolted structures. Hence, the weight of the joint is minimum.
- When we see tension members, the absence of holes improves the efficiency of the section.
- It requires less fabrication cost compared to other methods due to handling of less parts and elimination of operations like drilling, punching etc. and consequently less labor leading to economy.

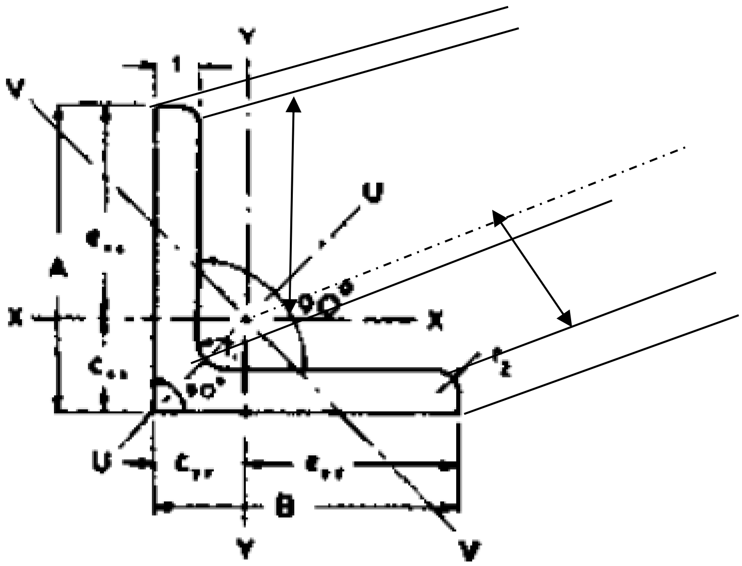
- Welding offers air tight and water tight joining and hence is ideal for oil storage tanks, ships etc.
- Welded structures also have a neat appearance and enable the connection of complicated shapes.
- Welded structures are more rigid compared to structures with riveted and bolted connections.
- Generally welded joints are as strong as or stronger than the base metal, thereby placing no restriction on the joints. Stress concentration effect is also considerably less in a welded connection.

- **Disadvantages of welding**
- It requires skilled manpower for welding as well as inspection.
- Also, non-destructive evaluation may have to be carried out to detect defects in welds.
- Welding in the field may be difficult due to the location or environment.
- Welded joints are highly prone to cracking under fatigue loading.
- Large residual stresses and distortion are developed in welded connections.

4b)

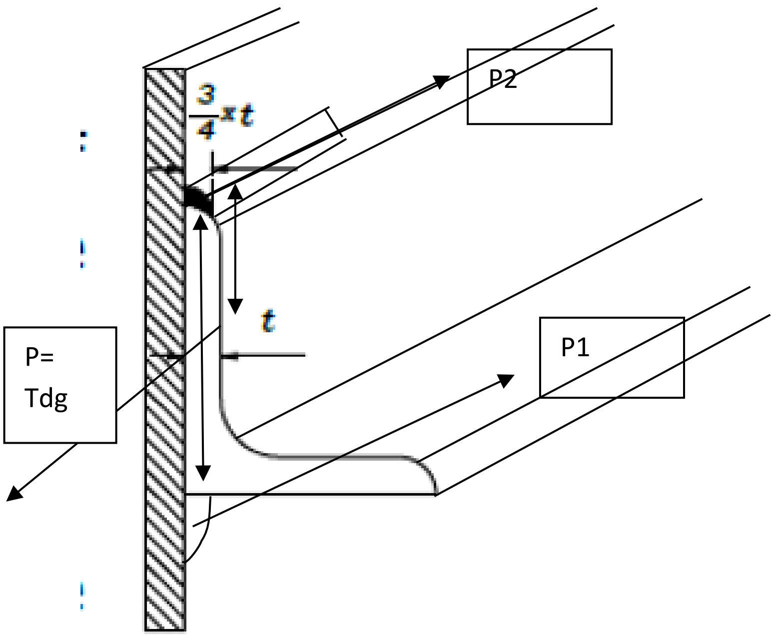
4b) A tie member of a truss consisting of an angle section ISA 65 x 65 x 6 of Fe 410 grade is welded to an 8 mm gusset plate. Design a weld to transmit a load of 150. Assume shop welding.

SP 6



Cl: 6.2, P- 32

Tension capacity of the angle= $T_{dg} = \frac{A_g \times f_y}{\lambda_{mo}} = \frac{1047 \times 250}{1.1}$ = 237.95kN



Size of the weld , D= ¾*t = ¾*6= 4.5mm say 4mm which is greater than 3mm

Strength of bottom weld = p1= 0.707*D*l_1*$\frac{410}{\sqrt{3}*1.25}$

P1=0.707*4*l_1*$\frac{410}{\sqrt{3}*1.25}$

P1=535.54l_1 N

Strength of top weld = p2= 0.707*D*l_2*$\frac{410}{\sqrt{3}*1.25}$

P2= 0.707*4*l_1*$\frac{410}{\sqrt{3}*1.25}$

P2= 535.54l_2

P=P1+P2

237.954*10^3= 535.54l_1+535.54l_2

Distributing weld in such a way that c.g of the weld coincides with that of the angle section.

Taking the moment wrt to one of the force , wrt P2

P1*90 = P*65.8

$535.54 * l_1 * 90 = 237.95 * 65.8$

$l_1 = 237.95*65.8/535.54*90$

$l_1 = 324.84mm = 325mm$

on substituting the l_1 in P1

$P1 = 535.54*325 = 174.050*10^3$ N

P1+P2= P

$174.05*10^3 + P2 = 237.95*10^3$

$P2 = 63.9*10^3$ N

Wkt $535.54 l_2 = P2$

Therefore = $l_2 = 63.9*10^3/535.54 = 119.31mm$

Effective length of *l1* = 325 + 2 x 4 = 333 mm say 335 mm

Effective length of *l2* = 120 + 2 x 4 = 128 mm say 130 mm

Module 3

5a)

- **□ Crushing failure:- This type of failure occur in short column Such member has a critical load cause material failure.**
- **□ Buckling failure :- This type of failure occur in long column**
- **Such member has a critical load which cause elasti instability due to which the member fail.**
- **□□ Mixed mode of failure:- The above two failure occur in the extreme cases. For all intermediate value of slenderness ratio the column fail due to combined effect. Most of th practical column fail in this mode.**
- **Flexural buckling**
- **Torsional Buckling**

Flexural- Torsional Buckling

5b)

Q.1. Design a "angle strut" using single angle section to carry a load of 150 kN. Use M_{20} property class 5.6 bolts. The length of the member is 2.5m.

Factored load = 1.5 × 150 = 225 kN

(a) Assume $f_{cd} = 100\ N/mm^2$

$$Area_{req} = \frac{Load}{f_{cd}} = \frac{225 \times 10^3}{100} = 2250 mm^2 \simeq 22.50 cm^2$$

From steel table try ISA 100×100×12mm. (area = 22.59 cm²)

$r_{xx} = 3.03cm$

$r_{yy} = 3.03cm$

$r_{uu} = 3.82cm$

$r_{vv} = 1.94cm$

$r_{min} = 19.4mm$.

Effective length $- l_e = 0.85l$ → Since planning to provide more than 2 bolts.

$= 0.85 \times 2.5$

$= 2.125\ m$

$$\lambda = \frac{l_e}{r_{min}} = \frac{2125}{19.4} = 109.54$$

For single angle Buckling class — 'c' Table 10 Pg. 44.

$f_{cd} = 94.6\ N/mm^2$ → from table 9 (c)

∴ Design compressive strength } = $P = A_e f_{cd}$.

$P = 2259 \times 94.6$

$= 213.70$ KN. $<$ 225 KN.

∴ Unsafe

Hence revise the Section

Now try ISA 125 x 95 x 12mm.

area = 2498 ⟶ Pg-8 of Steel table

$r_{min} = 2.01cm = 20.1mm$

$\lambda = \frac{l_e}{r_{min}} = \frac{2125}{20.1} = 105.7$

$f_{cd} = 99.93\ N/mm^2$

$P = A_e f_{cd}$

$= 2498 \times 99.93$

$= 249.62$ KN $>$ 225 KN. (Safe)

b) Connection: M_{20} property class 5.6

(1) In shear ⟶ Assume tread is touching shear plane.

$V_{nsb} = \frac{500}{\sqrt{3}}(1 \times 0.78 \times \pi/4 \times 20^2 + 0)$

$= 70.74$ KN.

$V_{dsb} = \frac{70.74}{1.25} = 56.59$ KN.

(ii) In bearing:

$K_b =$ take $e = 1.7 \times 22 = 40mm$

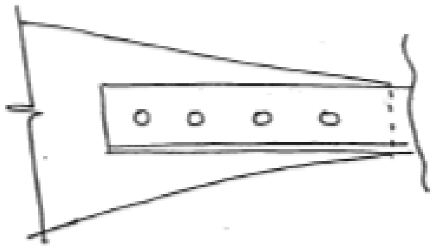
$p = 2.5 \times 20 = 50mm$

∴ $K_b = 0.507$

$V_{npb} = 2.5 \times 0.507 \times 20 \times 12 \times 410$

$= 124.72$ KN

∴ $V_{dpb} = \frac{124.72}{1.25} = 99.78$ KN.



Bolt value = 56.59 KN.

∴ No. of bolts $= \frac{225}{56.59}$ ≈ 4no.

Module 4

7a)

▶ **Tension yielding : gross sectional yielding**

Tension yielding : (IS 800:2007 pg 32)

This failure mode looks at yielding on the gross cross sectional area, A_g.

Consequently, the critical area is located away from the connection as shown.

Strength of the section Tdg= the *gross area*, A_g, times the *minimum yield stress*, F_y, of the member.

$T_{dg} = f_y A_g / g_{m0}$

▶ **Tensile rupture : Net sectional rupture**

Tensile rupture : Net section rupture

In this case we have two potential failure paths that see the full force of the member.

It is common to have multiple potential failure paths.

Tensile rupture is complicated by the need to get the forces out of the flanges, through the web, and into the bolts.

This means that we need to account for the stress concentrated in and around the bolts.

The capacity of each failure path = the *effective net area*, A_n, times the *tensile stress*, F_u, of the member.

▶ **Block shear failure:**

Block shear occurs when a "block" of the member is "torn" out.

Block shear is characterized by a failure that includes both tension (i.e. normal to the force) and shear (i.e. parallel to the force) failure planes.

Like tensile rupture, there are frequently multiple valid failure paths that must be investigated.

Each tension area capacity = the tension area (either gross or net) times a tensile stress (yield or ultimate).

Each shear area capacity = the shear area (either gross or net) times a shear stress (yield or ultimate).

7b)

Missing things from previous questions: Size of the section, no and dia of bolt,

Bolted connection:

axial load= 400kN

T_{dg} =Factored load =1.5*400=900kN

$T_{dg} = f_y \times A_g / ƛ_{m0}$

$A_g = T_{dg} * ƛ_{m0} / f_y = 90*1000*1.1/250 = 396mm^2 = 3.96cm^2$

Try ISA 65*45*5 a=5.26=526mm²

r_{min}= 0.96cm = 9.6mm

Check for slenderness ratio: P-20 table 3 of IS 800:2007

¥= $\frac{L_{eff}}{r_{min}}$ =$\frac{1560}{9.6}$= *162.5 less than 180 hence good to proceed*

1. **Design Yielding Strength T_{dg}- (6.2)**

T_{dg} = $f_y \times A_g / \gamma_{m0}$ (γ_{m0} = 1.10 from table 5)

= 250×526/1.1*1000= 119.5 kN is greater than 90kN hence safe

Connection details:

Assume 16mm dia bolt

- Strength shear (*assume **shank** interfere the shear plane of bolts*)

$$V_{dsb} = \frac{f_{ub}}{\sqrt{3}}(n_n A_{nb} + n_s A_{sb}) / \gamma_{mb}$$

$$= \frac{400}{\sqrt{3}}(0 + 1 * (pi * 16 * 16/4) /1.25 = 37.14\text{kN}$$

- Bolts in Bearing

$V_{nsb} = (2.5 K_b\, d\, t\, f_u) / \gamma_{mb}$

K_b is least of the following $e/3d_o$, $p/3d_o$- 0.25, f_{ub}/f_u , 1

Assuming p=2.5*d=50mm and e=1.7*do= 40mm

40/ 3*18= 0.74

50/3*18-0.25= 0.68

400/410= 0.98

1

V_{nsb} = (2.5**0.68*16*5*400)/1.25*1000= 43.52kN*

No. of bolts = load /bolt value = 90/37.14= 2.42= 3No's

2. **Design Rupture Strength of Net Area T_{dn}- (6.3.3)** *since it is an **angle (since it is affected by shear lag)***

T_{dn} = $0.9 \times A_{nc} \times f_u / \gamma_{m1} + \beta \times A_{go} \times f_y / \gamma_{m0}$ (γ_{m1} = 1.25)

*A_{nc} = Net c/s area of the **connected** leg = ((60-5/2-18) *5) =222.5mm²*

*A_{go} = Gross c/s area of the **unconnected** leg= (45-5/2) *5= 212.5mm²*

$\beta = 1.4 - 0.076\left(\frac{W}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L_c}\right) \leq \left(\frac{f_u}{f_y}\right)\left(\frac{\lambda m0}{\lambda m1}\right) \geq 0.7$ ………. pg. 33

$\beta = 1.4 - 0.076\left(\frac{45}{5}\right)\left(\frac{250}{410}\right)\left(\frac{75}{150}\right) \leq \left(\frac{410}{250}\right)\left(\frac{1.1}{1.25}\right) \geq 0.7$

$=1.19 \leq 1.44 \geq 0.7$

$b_s = w + w_1 - t$ = 45+35-5= 75mm

L_c= 3*50= 150mm

T_{dn} = 0.9 ×222.5 × 410 / 1.25 +1.19×212×250 / 1.1=123.08kN

3. **Design Block Shear Strength T_{db} (6.4.1)**

T_{db1} =[A_{vg} × (f_y/√3) / λ_{m0} + 0.9 × A_{tn} × f_u / λ_{m1}]...............................pg 33, 6.4.1

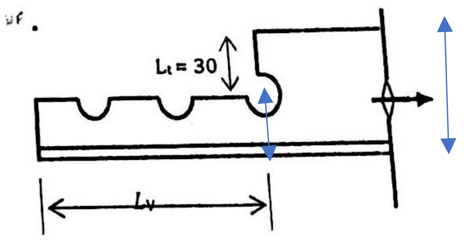
=700 × (250/√3) / 1.1 + 0.9 ×105 ×410/ 1.25

= 122.85kN

T_{db2} = [0.9× A_{vn}× (f_u/√3) / λ_{m1} + A_{tg} × f_y / λ_{m0}]...............................pg 33, 6.4.1

=0.9× 475× (410/√3) / 1.25 +150 × 250/ 1.1

=115.05 kN



A_{vg}= L_v*t=140*5=700 mm^2

A_{tn}= (L_t-nd_o)*t=(30-0.5*18)5=105mm^2

A_{vn}= (L_v-nd_o)*t=(140-2.5*18)*5=475 mm^2

A_{tg}= L_t*t =30*5= 150mm^2

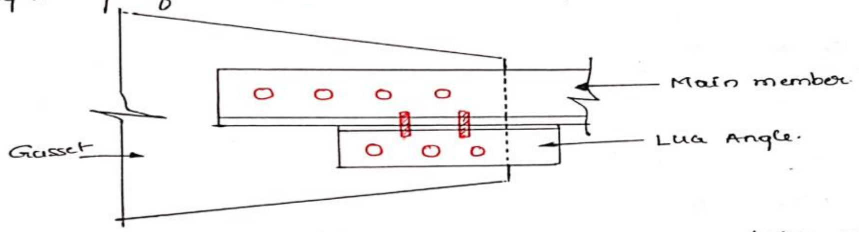
DESIGN STRENGTH= least of T_{dg}, T_{dn}, T_{db1}, and T_{db2}

Therefore, Design strength of the angle is 115.05kN is greater than 90kN hence safe to proceed with ISA 65*45*5 as tie member

8a)

Lug Angle:

Lug angles are short angles used to connect the gusset and outstanding leg of the main member as shown in fig below. The lug angles help to increase the efficiency of outstanding leg of angles or channels. They are normally provided when the tension members carries a very large load. Higher load results in a larger end connection which can be reduced by providing lug angles. It is ideal to place the lug angle at the beginning of connection than at any other position.



GUSSET PLATE:- is used to make connections at the place where more than one member is to be jointed eg. joints of truss, truss girder etc. The line of action of truss members meeting at a joint are assumed to coincide as shown above fig.

The size and shape of the gusset plates are usually decided from the direction of the members meeting at joint. The plate outlines are fixed so as to meet the specification of edge distance for the bolts used to connect the various members meeting at joint.

8b)

Solution:

Data:

Axial load= 700kN

Factored load = 1.5*700= 1050kN

Bearing strength of concrete =0.45 f_{ck}= 0.45*20= 9N/mm^2

Area of the base plate = $\frac{load}{bearing\ stregth\ of\ concrete}$= $\frac{1050*10^3}{9}$= 116.67*10^3 m^2

Properties of ISHB 225@46.8 kg/m

From the sp 6.

h= 225mm

b_f= 225mm

t_f= 9.1mm

overall dimension of the column is square: let is design a square base plate

Sides of the base plate = $\sqrt{116.67 * 10^3}$ = 341.56mm rounding up= 350mm

Since we are providing the cleat angles we shall increase the dimension by 10mm. then the size of the base plate = 360mm*360mm

Projection beyond the column = 360-225/2= 67.5mm

Net upward pressure 'w'= $\frac{load}{Area\ of\ the\ base\ plate} = \frac{1050*1000}{360*360} = 8.10\ M/mm2 < 9N/mm2\ hence\ safe\ to\ proceed\ with\ the\ section$

Thickness of the base plate: IS 800:2007, P 47 clas 7.4.31.

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

$t_s = \sqrt{\frac{2.5 * 8.1(67.5^2 - 0.3 * 67.8^2)1.1}{250}}$= 16.06mm so let us round it up to 18mm

Provide the base plate of dimension 360*360*18mm

<u>Design of bolt:</u>

No of bolt= $\frac{load}{bolt\ value}$ =

Bolt value = least of Vdsb and Vdpb …… from pag 75 of IS 800:2007

P= 50mm, e=45mm, d=20mm, assume the bolt's shear plane is intersecting the shank.

- <u>Strength shear (*assume shank interfere the shear plane of bolts*)</u>

$$V_{dsb} = \frac{f_{ub}}{\sqrt{3}}(n_n A_{nb} + n_s A_{sb})\ /\gamma_{mb}$$

$$= \frac{400}{\sqrt{3}}(0 + 1 * (pi * 20 * 20/4)\ /1.25 = 58.03\text{kN}$$

- <u>Bolts in Bearing</u>

$V_{nsb} = (2.5 K_b\ d\ t\ f_u)/\ \gamma_{mb}$

K_b *is least of the following* $e/3d_o$, $p/3d_o - 0.25$, f_{ub}/f_u , 1

Assuming p=50mm and e= 45mm

45/ 3*22= 0.68

50/3*22-0.25= 0.50

400/410= 0.97

1

V_{nsb} = (2.5*0.50*20*9.1*400)/1.25*1000= 72.8kN

No of bolt= $\frac{262.5}{58.03} = 4.52 = 5no's$ =

Total load on the column = 1050kN,

Therefore, on each cleat angle the load will be= 1050/4= 262.5kN

Cleat angle: Take cleat angle of ISA 100*100*10mm to secure the column with the base plate by 5 bolts of 20mm dia

Concrete bed block:

Axial load= 700kN

Self weight of the concrete= 10% of the load= 0.1*700= 70kN

Therefore the total load on the concrete bed block= 770kN

Area = load/ stress= 770/180= 4.28m^2

Side of the concrete block = $\sqrt{4.28}$ = = 2.06m = 2.1m

Thickness of the block

Depth of the concrete block= projection of concrete beyond base plate =

=2100-360/2= 870mm = 900mm

Now the size of the concrete bed block= 2.1*2.1*0.9m

Module 5

9a)

Factors affecting lateral stability

Type of C/S - The lateral buckling strength can be improved by choosing an appropriate c/s where IYY is large. Box sections satisfies this and also has large torsional rigidity as it is a closed section. Open sections like I sections have low torsional rigidity and are more susceptible to lateral instability. Cl 8.2.2 pp - 54 mentions that hollow sections need not be checked for lateral buckling strength.

Support conditions - The lateral restraint provided depends on the restraint provided by the supports. The effect of various support conditions is taken into account using the concept of effective length as given in Table 15 - pp 58 for simply supported beams and Table 16 - pp 61 for cantilever beams.

Effective length - This concept incorporates the various types of restraints to the flanges and for simply supported beams Table 15 - pp 58 can be used. The same information for cantilever beams is given in Table 16 - pp 61.

Beams without proper restraint of the compression flange undergoes lateral buckling resulting in lesser load carrying capacity.

Design strength of laterally supported and unsupported beams

These are analysis problems where the strength of the beam is required. The design strength will be based on flexural or bending strength and shear strength.
Bending strength of laterally supported beams are calculated using the provisions given 8.2.1.2 (pp -53) by knowing the plastic section modulus Zpz
Bending strength of laterally unsupported beams are calculated using the provisions given 8.2.2 and 8.2.2.1 (pp -54) by knowing the plastic section modulus Zpz
The shear strength of the c/s is obtained from cl. 8.4(pp -59).

$a = (h - 2t_f)$

★ Factors affecting lateral stability:

★ Type of c/s — The lateral buckling strength can be improved by choosing an appropriate c/s where I_{yy} is large. Box sections satisfies this and also has large torsional rigidity as it is a closed section. Open sections like I-section have low torsional rigidity and are more susceptible to lateral instability, cl-8.2.2 pp-54 mentions that hollow sections need not be checked for lateral buckling strength.

★ Support conditions: The lateral restraint provided depends on the restraint provided by the support. The effect of various support conditions is taken into account using the concept of effective length as given in table 15-pp-58 for simply supported beams and table 16-pp 61 for cantilever beam.

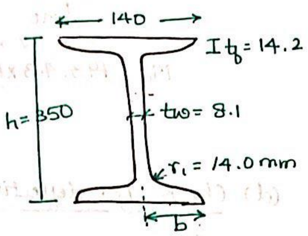
★ Effective length: This concept incorporates the various types of restraints to flanges and for simply supported beam, Table 15 PNo-58 can be used. The same information for cantilever beams is given in Table 16, PNo-61.

Beams without proper restraint of the compression flange undergoes lateral buckling resulting in lesser load carrying capacity.

PROBLEMS:

Q.1. Simply supported Beam ISMB 350 @ 52.4 Kg/m. is used over a span of 5m. The beam carries an UDL live load 20 KN/m. & DL 15 KN/m. The beam is laterally supported throughout "Check the safety of beam"

This is not a design problem → its just a check



Properties of ISMB 350 @ 52.4 Kg/m.

(a) load calculation

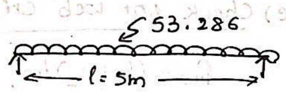
Self wt of beam = 0.524 KN/m

Dead load = 15 KN/m

Live load = 20 KN/m

35.524 KN/m.

∴ Ultimate load = 35.524 × 1.5 = 53.286 KN/m.



$$M_u = \frac{W_u l^2}{8} = \frac{53.286 \times 5^2}{8} = 166.5 \text{ KN-m}.$$

$$V_u = \frac{wl}{2} = \frac{53.286 \times 5}{2} = 133.21 \text{ KN}.$$

(b) Check for Shear: (Pg NO-59)

$$V_d = 0.6\left[\frac{f_y}{\sqrt{3}.\lambda_{mo}} \times A_v\right] > V_u$$

$A_v = h \times t_w$
$= 350 \times 8.1$
$= 2835 mm^2$.

$\lambda_{mo} = 1.10$.

$$\therefore V_d = 0.6\left[\frac{250}{\sqrt{3} \times 1.10} \times 2835\right]$$

$= 223.1$ KN $> V_u = 133.21$ ⟶ Hence SAFE

(c) Check for "Moment of Resistance" (Pg NO 53)

$$M_d = \frac{\beta_b Z_p . f_y}{\lambda_{mo}}$$

$$\left(\frac{b}{t_f}\right) = \frac{140/2}{14.2} = 4.92 < 9.4.$$

$$\left(\frac{d}{t_w}\right) = \left(\frac{h-2t_f}{t_w}\right) = \frac{350 - 2 \times 14.2}{8.1} = 39.7 < 84$$

Page 18 to decide which type of section.

∴ It is plastic section i.e $\beta = 1$

9b)

from Table 46 pg 138.

for ISMB - 350 $\longrightarrow Z_p = 851.11 cm^2$

$= 851.11 \times 10^3 mm^2$

$M_d = \frac{\beta Z_p \cdot f_y}{\gamma_{mo}} = \frac{1.0 \times \sout{851.11 \times 10^3} \times 250}{1.10}$ (corrected: 889.5)

$M_d = \sout{193.43 \times 10^6}$ 202.17×10^6 N-mm $> M_u = 166.5$ KN·m $\longrightarrow$ HENCE SAFE.

(d) Check for deflection:

$\frac{Span}{250}$ = permissible deflection = $\frac{5000}{250} = 20mm$ $\longrightarrow$ for Simply supported.

Actual deflection $\delta = \frac{5}{384} \frac{wL^4}{E_s I}$ → not ultimate load

$E_s = 2 \times 10^5 N/mm^2$

$I_{xx} = 13630.3 \times 10^4 mm^4 = I_{zz}$

$\delta = \frac{5}{384} \times \frac{35.524 \times 5^4}{2 \times 10^5 \times 13630.3 \times 10^4}$

$\delta = 10.60 < 20mm$ SAFE.

(e) Check for web crippling

b_1 = Bearing width (or) Support width.

∴ Assume $b_1 = 200$

$F_w = (b_1 + n_2) t_w \frac{f_{yw}}{\gamma_{mo}}$

∴ $n_2 = 2.5(t_f + r_1)$

$= 2.5(14.2 + 14)$

$= 70.5mm$

$F_w = (200 + 70.5) 8.1 \times \frac{250}{1.1}$

$F_w = 497.96$ KN $> V_u$ SAFE.

(f) Check for "web Buckling":

$F_{wb} = (b_1 + n_1) t_w \cdot f_c$

From table 9(c) $\rightarrow f_{cd} = 107 N/mm^2 = f_c$.

$n_1 = h/_2 = 350/_2 = 175mm$

$F_{wb} = (200 + 175) \times 8.1 \times 107$

$= 325.01$ KN. $> V_u$ SAFE.

$\lambda = 2.5 \frac{d}{t_w}$

$= 2.5 \times \frac{(h - 2t_f)}{t_w} = \frac{2.5(350 - 2 \times 14.2)}{8.1}$

$\lambda = 99.25$

∴ ISMB - 350 @ 52.4 kg/m is capable of taking load and satisfies all the other conditions of check

Q.2. Design a beam of effective span 6m. subjected to an UDL 10kN/m. along with 100kN load. The beam is laterally supported. The thickness of wall is 230mm.

(a) Load calculation.

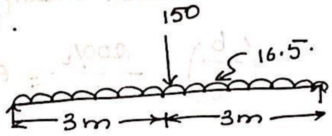
UDL on beam = 10 kN/m.

Assume self wt. = 1 kN/m.

11 kN/m.

ultimate load = UDL = 11 × 1.5 = 16.5 kN/m.

& ultimate point load = 1.5×100 = 150 kN.



$$V_u = \frac{16.5 \times 6}{2} + \frac{150}{2} = 124.5 \text{ kN}.$$

$$M_u = \frac{16.5 \times 6^2}{8} + \frac{150 \times 6}{4} = 299.2 \text{ kN-m}.$$

Plastic Modulus required $= Z_p = \frac{M_d \gamma_{mo}}{\beta \times f_y}$ mm². → pg 53.

$$Z_p = \frac{299.2 \times 10^6 \times 1.10}{1 \times 250} = 1316.5 \times 10^3 \text{ mm}^3 \text{ or } 1316.5 \text{ cm}^3$$

let us increase value by 20% approximately.

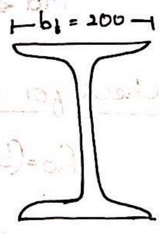
$= 1.20 \times 1316.5 = 1580 \text{ cm}^3$.

from IS-800 page 138 Try. ISWB -450 @ 79.4 kg/m.

$Z_p = 1760.59 \text{ cm}^3$

$Z_e = 1558.1 \text{ cm}^3 = Z_{xx}$.

$I_{xx} = 35057.6 \text{ cm}^4$ → SP 6 steel table.



(b) Check for deflection.

Permissible $= \frac{\text{Span}}{250} = \frac{6000}{250} = 24\text{mm}$.

$$\delta = \frac{5}{384}\frac{wl^4}{EI} + \frac{wl^3}{48EI}$$

$$= \frac{5}{384} \times \frac{11 \times 6000^4 \times 10^3}{2 \times 10^5 \times 3.505 \times 10^8} + \frac{100 \times 10^3 \times 6000}{48 \times 2 \times 10^5 \times 35057.6 \times 10^4}$$

$\delta = 9.06 < 24\text{mm}$ Safe.

(c) Check for Shear. → 59 pg No

$$V_d = 0.6\left[\frac{f_y}{\sqrt{3}\times \lambda_{mo}} \times A_v\right] > V_u.$$

$$= 0.6\left[\frac{250}{\sqrt{3}\times 1.1} \times 450\times 9.2\right] = 325.94 \text{ KN} > V_u \text{ Safe.}$$

(d) Check for Moment of Resistance:

Section classification based on table 2 @ page 18.

$$\left(\frac{b}{t_f}\right) = \frac{200/2}{15.4} = 6.49 < 9.4.$$

$$\left(\frac{d}{t_w}\right) = \frac{(h-2t_f)}{t_w} = \frac{450-2\times 15.4}{9.2} = 45.56 < 84.$$

Hence the Section is Plastic $\therefore \beta = 1$

$$M_d = \frac{\beta_b Z_p f_y}{\lambda_{mo}} > M_u.$$

$$= \frac{1\times 1760.59\times 10^3\times 250}{1.1}$$

$= 400.13$ KN-m $> M_u = 299.2$ SAFE

But $M_d < \frac{1.2 Z_e f_y}{\lambda_{mo}}$.

$$= \frac{1.2\times 1558.1\times 10^3\times 250}{1.1}$$

$M_d < 424.93$ kN-m SAFE.

(e) Check for ~~deflection~~ web crippling:

$$F_w = (b_1+n_2)t_w \frac{f_{yw}}{\lambda_{mo}}$$

$$= (\overset{230}{\cancel{200}} + 70.5)9.2 \times \frac{250}{1.1}$$

Assume $b_1 =$ ~~200~~ 230mm

$n_2 = 2.5(t_f + r_1)$

$= 2.5(14.2 + 14)$

$= 70.5$ mm

$F_w \qquad > V_u$

Hence safe.

(f) Check for web Buckling:

$F_{wb} = (b_1 + n_1)\, t_w\, f_c$

$\lambda = 2.5 \frac{d}{t_w}$

$= 2.5 \times \frac{(h - 2t_f)}{t_w}$

$= 2.5 \frac{(450 - 2 \times 15.4)}{9.2}$

$=$

So find fcd from table 9C for $\lambda =$

$\therefore f_{cd} =$

$F_{wd} = (200 + \quad) 9.2 \times$

$n_1 = h/2 = 450/2 =$

$= \quad > V_u$

Hence safe.

$\therefore$ The Section Decided is ISWB 450 @ 79.4 kg/m is safe to take loads coming on 6m Span Beam.