Sixth semester B.E. Degree Examination, June/July 2018

DESIGN OF STEEL STRUCTURAL ELEMNTS

VTU-SOLUTION

MODULE 1

1. What are the advantages and disadvantages of steel structures?

Advantages of steel structure:

1. Less self-weight but able to resist heavy loads.
2. Gives sufficient warning before failure - because of its ductile property
3. Because of ductile property it enable to yield locally at the point of high stress concentration.
4. prevents premature failure.
5. Being light, steel members are conventionally handled and transported. for this reason, prefabricated members can be frequently provided.
6. It has longer life span, if it is maintained properly.
7. The properties of steel mostly do not change with time.
8. They can be erected @ faster rate.
9. Steel has highest scrap value amongst all building material.
10. Steel is a recycable material.

Disadvantage of steel structure:

1. The steel structure, when placed in exposed condition, are subjected to corrosion.
2. Steel structure need fire proof treatment which increases the cost.
3. Fatigue of steel is one of the major drawback. Fatigue includes a reduction in strength when steel is subjected to large number of stress reversals and even to a larger number of variations of tensile stress.
4. At the places of stress concentration in steel sections under certain conditions, the steel may loose its ductility.

1 b. what are the rolled steel sections? Mention any six shapes used as structural elements with sketch.

Rolled Steel Section:

Many of the cross-sectional area (or) the structural members are designed based on requirement and also there are of standard dimension and are ready for the usage, the cross-section and shapes are widely available rather than produced according to shape.

The largest categories are of standard shape and are produced by hot rolling. The hot steel passes through a series of rollers which squeeze the material into desired cross-sectional shapes. Steel sections are usually designated by their cross-sectional shapes.

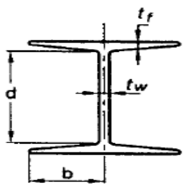
Types of steel shapes rolled are described as follows:

1. Rolled Steel I-Section. — Fig 1
2. Rolled Steel Channel Section. — Fig 2
3. Rolled Steel angle section. — Fig 3
4. Rolled Steel T-Section. — Fig 4
5. Rolled Steel tube Section. — Fig 5
6. Rolled Steel bars
7. Rolled Steel Flats.
8. Rolled Steel plates.
9. Rolled Steel Sheets.
10. Rolled Steel ship.

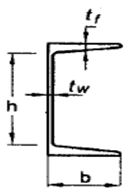
NOTE:

1. All standard I-beam and channels having a slope of 16 2/3 % on inside face of the flange.
2. ISLB and ISMB are the only I-Section being produced (or) rolled in India — These are suitable only for beams because of their sectional properties.

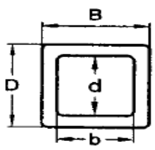
For columns - ISHB, but since this is not rolled ISMB is used for



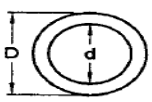
ROLLED BEAMS AND COLUMNS



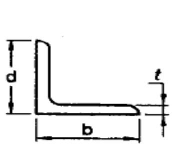
ROLLED CHANNELS

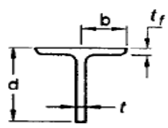


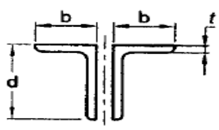
RECTANGULAR HOLLOW SECTIONS



CIRCULAR HOLLOW SECTIONS

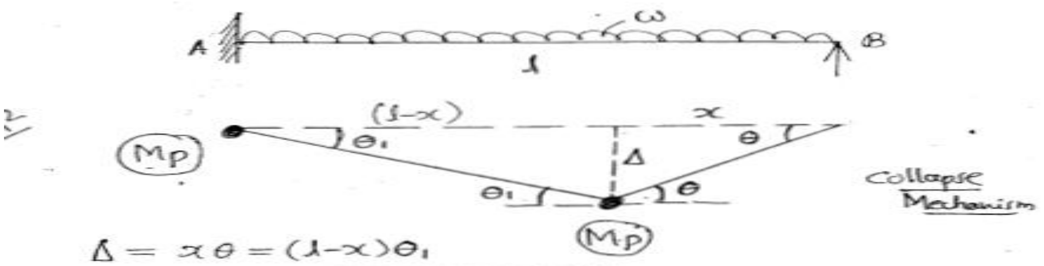






Or

2. a) Identify the plastic the hinge its distance 'x' is l from the simple support of a propped cantilever beam supporting a UDL of w kN/m over the entire span.



$\Delta = x\theta = (l-x)\theta_1$

$\therefore \boxed{\theta = \left(\frac{l-x}{x}\right)\theta_1}$

EWD = (Load)(Area Below the load)

$= \omega\left(\frac{1}{2} \times l \times \Delta\right) = \frac{\omega l}{2} \times (l-x)\theta_1$ —(i)

IWD = (Mp)(Rotation)

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

$= M_p\left(\frac{l-x}{x}\right)\theta_1 + 2M_p\theta_1$

$= M_p\theta_1\left[\frac{l-x}{x} + 2\right] = M_p\theta_1\left[\frac{l+x}{x}\right]$ —(ii)

EWD = IWD.

$\frac{\omega l}{2}(l-x)\cancel{\theta_1} = M_p\cancel{\theta_1}\left(\frac{l+x}{x}\right)$

$M_p = \frac{\omega l(l-x)x}{2(l+x)}$ $\quad \boxed{M_p = \frac{\omega l}{2} \times \frac{(lx - x^2)}{(l+x)}} \rightarrow$ (I)

For max. moment $\rightarrow \frac{\partial M_p}{\partial x} = 0$

$\frac{\partial M_p}{\partial x} = 0 = \frac{\omega l}{2}\left[\frac{(l+x)(l-2x) - (lx-x^2)(1)}{(l+x)^2}\right]$

$0 = l^2 - 2lx + \cancel{lx} - 2x^2 - \cancel{lx} + x^2$

$l^2 - 2lx - x^2 = 0$ or $x^2 + 2lx - l^2 = 0$

$\therefore x = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a}$

$x = \frac{-2l + \sqrt{(+2l)^2 - 4 \times 1 \times (-l^2)}}{2 \times 1} = \frac{-2l + \sqrt{8l^2}}{2}$

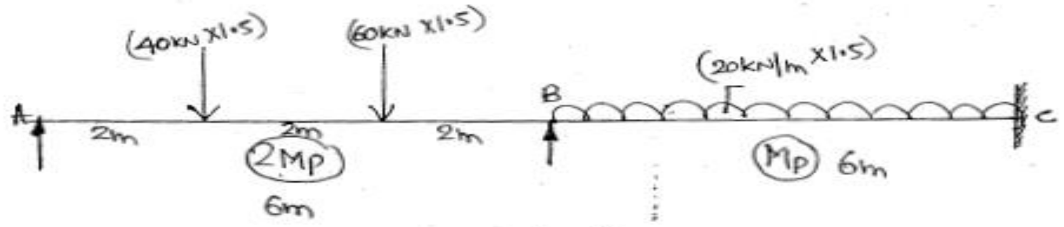
$x = \frac{-2l + 2\sqrt{2}l}{2}$ $\quad \therefore \boxed{x = 0.414l}$

From (I) $M_p = \frac{\omega l}{2} \frac{(l \times 0.414l - (0.414l)^2)}{(l + 0.414l)}$

$M_p = \frac{\omega \cancel{l}}{2} \times \frac{l^2(0.414 - 0.414^2)}{\cancel{l}(1+0.414)} = \frac{0.1715\,\omega l^2}{2}$

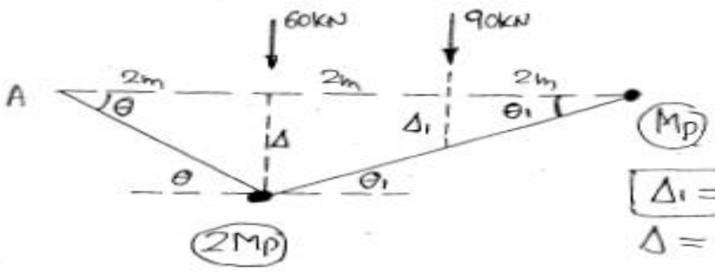
2.b)

Eg:— 6] Determine the plastic Moments capacity of the beam. Take load factor = 1·5.



Sol^n (a) consider span "AB" :

(i) Collapse Mechanism under 40kN Load.



$\boxed{\Delta_1 = 2\theta_1}$

$\Delta = 2\theta = 4\theta_1$

$\boxed{\theta = 2\theta_1}$

$EWD = 60 \times \Delta + 90 \times \Delta_1 = 60(4\theta_1) + 90(2\theta_1)$

$= (420)\theta_1 \rightarrow$

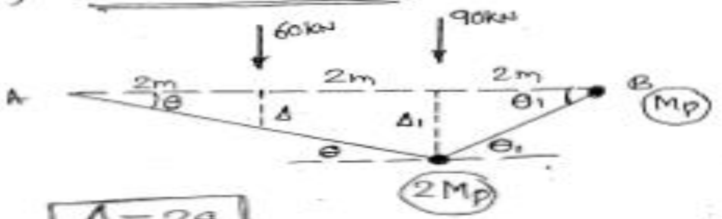
$IWD = 2M_p(\theta + \theta_1) + M_p\theta_1$

$= 2M_p\theta + 3M_p\theta_1 = 2M_p(2\theta_1) + 3M_p\theta_1$

$= 7M_p\theta_1 \rightarrow$

Equating, $7M_p\cancel{\theta_1} = 420\cancel{\theta_1}$ $\boxed{M_p = 60\,kN\text{-}m}$

(ii) Under 60kN Load : (90kN)



$\boxed{\Delta = 2\theta}$

$\Delta_1 = 2\theta_1 = 4\theta$ $\therefore \boxed{\theta_1 = 2\theta}$

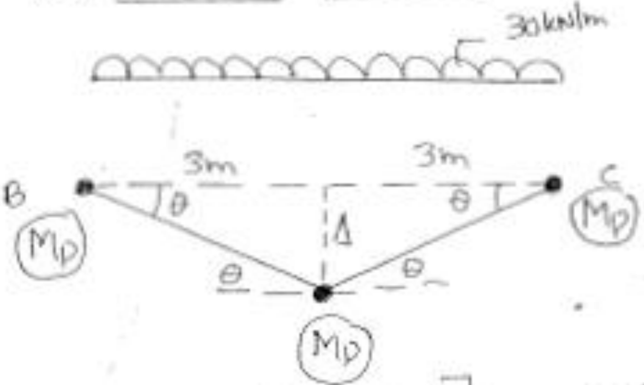
$EWD = 60\,\Delta + 90\,\Delta_1 = 60(2\theta) + 90(4\theta) = (480)\theta \rightarrow$

$IWD = 2M_p(\theta + \theta_1) + M_p(\theta) = 3M_p\theta + 2M_p\theta_1$

$= 3M_p\theta + 2M_p(2\theta) = 7M_p\theta \rightarrow$

Equating, $7M_p\cancel{\theta} = 480\cancel{\theta}$ $\boxed{M_p = 68.57\,kN\text{-}m}$

(b) Consider Span BC :



$\Delta = 3\theta$

$EWD = 30\left[\frac{1}{2} \times 6 \times \Delta\right] = 90(3\theta) = 270\theta \rightarrow$

$IWD = 4M_p\theta \rightarrow$

Equate $4M_p\cancel{\theta} = 270\cancel{\theta}$ $\boxed{M_p = 67.5\ kN\text{-}m}$

$\therefore$ Finally Plastic Moment $= \boxed{M_p = 68.57\ kN\text{-}m}$

MODULE 2

3. a) What is a HSFG bolt? What are the advantages of HSFG bolt?

High- Strength Bolts :

Washers used in were serving a purpose.

• To distribute the clamping pressure to a larger area of the softer metal of the fastened parts, and to prevent the nut (or) bolt head from damaging the component (or) member

• To prevent the threaded ~~position~~ portion of bolt from bearing on connected members.

High - Strength bolts are commonly used in steel construction.

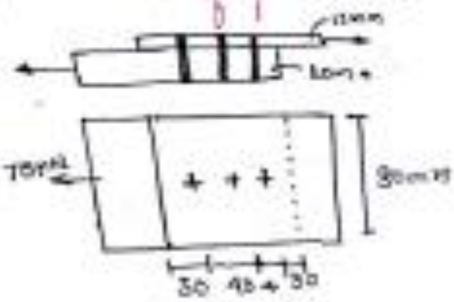
- These high- Strength bolts are made of bars of medium carbon steel.
- Bolts are 8.8 to 10.9 property class used in construction.
- Bolts of size M16, M20, M24 & M30 are widely used in

2. Advantages of HSFG Bolt

1. HSFG bolts do not allow any slip between the elements connected, especially in close tolerance holes, thus providing rigid connections.
2. Due to clamping action, load is transmitted by friction only and the bolts are not subjected to shear and bearing.
3. Due to the smaller number of bolts, the gusset plate size are reduced.
4. Deformation is minimized.

Solved problems

Q. Design a lap joint b/w two plates as shown in fig. So as to transmit a factored load of 70KN using M16 bolts of grade 4.6 and grade of plates as 410.



Strength calculation.

- Nominal dia of bolt = $d = 16mm$
- Hole dia $= 16 + 2 = 18mm$
- Bolts are in single shear and hence shear capacity of bolt

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

for single shear $n_s = 0$

$$\therefore \quad V_{nsb} = \frac{400}{\sqrt{3}} \left(1 \times 0.78 \times \frac{\pi}{4} \times 16^2\right)$$

$$= \frac{400}{\sqrt{3}} (157) = 36.25\ KN$$

$$V_{dsb} = \frac{V_{nsb}}{\lambda_{m}} = \frac{36.25}{1.25} = 29.00\ KN.$$

Since the top plate is only 12mm, it is assumed the shear plane is through the threaded portion and hence $= n_s = 0$.

Bearing capacity of thinner plate $= 2.5 \frac{K_b d t f_u}{\lambda_{mb}} = 77.15KN$

$$K_b = \text{Smaller of } \frac{e}{3d_0},\ \frac{p}{3d_0} - 0.25,\ \frac{f_{ub}}{f_u} \text{ and } 1.0$$

$$= \frac{30}{3 \times 18},\ \frac{40}{3 \times 18} - 0.25,\ \frac{400}{410} \text{ and } 1.0$$

$$= 0.55,\ 0.74 - 0.25,\ 0.97\ \&\ 1.0$$

$$= 0.55,\ 0.49,\ 0.97\ \&\ 1.0$$

$$\therefore K_b = 0.49$$

Bolt value = 29 KN.

Required No. of bolt = $\frac{\text{load}}{\text{Bolt Value}} = \frac{70}{29} = 2.41 \simeq 3$ bolt.

Detailing :

Minimum pitch = 2.5 x d = 40 mm.

Minimum edge distance = 29 mm — (Table 5.6)

Provide 3 bolts as shown in fig.

Or

4. a) What are the advantages and Disadvantages of welded connection?

Advantages of welding:

1. wt. of joint is minimized as there is no requirement of Gusset plate, Splice plate and other connections.

2. In tension members, efficiency will be increased by avoiding the bolt hole - which reduced the gross area.

3. Involves less fabrication and reduced cost of connection.

4. It offers air tight and water tight joining and hence is ideal for oil Storage tank, Ships etc.

5. welded Structures also have a neat appearance and enable the connection of complicated Shapes.

6. welded Structures are more rigid compared to structures with rivited and bolted connection.

7. welded joint are stronger than base metal, there by placing no restriction to joints.

8. Stress concentration effect is also considerably less than welded connection.

Disadvantages of welding.

1. It requires skilled manpower for welding as well as inspection.

2. Non-destructive evaluation may be carried out to detect defective weld.

3. welding in field may be difficult due to the location

4. welded joints are highly prone to cracking under fatigue

4. **b)18mm thick plate is joined to a 16mm thick plate by 200mm butt weld. Determine the strength of joint if, (i) A double V-butt weld is used (ii) A single V Butt weld is used. Take $f_u = 410\ N/mm^2$.**

a) Single V- groove butt weld: (There will be incomplete penetration)

∴ $t = 5/8 \times$ thickness of thinner plate $= 5/8 \times 12 = 7.5$ mm.

$L_w = 150$ mm.

Design Strength of weld $= \dfrac{(L_w\, t\, f_u)}{\sqrt{3}\ \gamma_{mw}} = \dfrac{150 \times 7.5 \times 410}{\sqrt{3} \times 1.25} = 213.044 < 300$ kN.

Hence the joint is not safe.

(b) Double V- groove butt weld: (There will be complete penetration)

$t = t_p = 12$ mm, $L_w = 150$ mm, (where throat thickness $t = t_p$ (thickness of thinner plate)

Design Strength $= \dfrac{L_w \times t \times f_u}{\sqrt{3}\ \gamma_{mw}} = \dfrac{150 \times 12 \times 410}{\sqrt{3} \times 1.25}$

$= 340.87$ kN > 300 kN

Hence the joint is safe.

Strength of weld: 10.5.7.1

= Throat Area × Allowable Shear Stress in weld

= Throat thickness × length × Allowable Shear Stress in weld.

$\boxed{\text{Strength of weld/mm} = \dfrac{0.707\, D \times f_u}{\sqrt{3}\ \gamma_{mw}}}$

MODULE -3

5. a) Explain the Laced and Battened column with sketches.

7.6 Laced Columns

7.6.1 *General*

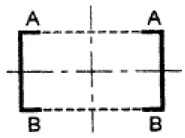
7.6.1.1 Members comprising two main components laced and tied, should where practicable, have a radius of gyration about the axis perpendicular to the plane of lacing not less than the radius of gyration about the axis parallel to the plane of lacing (*see* Fig. 10A and 10B).

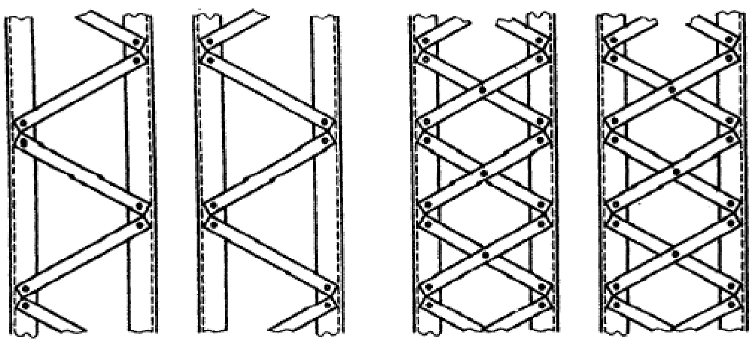
7.6.1.2 As far as practicable, the lacing system shall be uniform throughout the length of the column.

7.6.1.3 Except for tie plates as specified in **7.7**, double laced systems (*see* Fig. 10B) and single laced systems (*see* Fig. 10A) on opposite sides of the main components shall not be combined with cross members (ties) perpendicular to the longitudinal axis of the strut (*see* Fig. 10C), unless all forces resulting from deformation of the strut members are calculated and provided for in the design of lacing and its fastenings.

7.6.1.4 Single laced systems, on opposite faces of the components being laced together shall preferably be in the same direction so that one is the shadow of the other, instead of being mutually opposed in direction.

7.6.1.5 The effective slenderness ratio, $(KL/r)_e$, of laced columns shall be taken as 1.05 times the $(KL/r)_0$, the actual maximum slenderness ratio, in order to account for shear deformation effects.





LACING ON FACE A | LACING ON FACE B | LACING ON FACE A | LACING ON FACE B

PREFFERED LACING ARRANGEMENT

10A Single Laced System

PREFFERED LACING ARRANGEMENT

10B Double Laced System

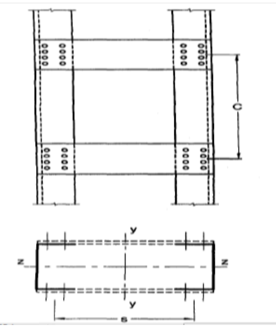
7.7 Battened Columns

7.7.1 *General*

7.7.1.1 Compression members composed of two main components battened should preferably have the individual members of the same cross-section and symmetrically disposed about their major axis. Where practicable, the compression members should have a radius of gyration about the axis perpendicular to the plane of the batten not less than the radius of gyration about the axis parallel to the plane of the batten (*see* Fig. 11).

7.7.1.2 Battened compression members, not complying with the requirements specified in this section or those subjected to eccentricity of loading, applied moments or lateral forces in the plane of the battens (*see* Fig. 11), shall be designed according to the exact theory of elastic stability or empirically, based on verification by tests.

NOTE — If the column section is subjected to eccentricity or other moments about an axis perpendicular to battens, the battens and the column section should be specially designed for such moments and shears.



5. b)

1. Design a column section using single rolled steel beam ~~using~~ along with cover plates to carry a factored load of 2000kN. The column both ends are fixed. Take $l = 6m$.

(a) Assume $f_{cd} = 220\,N/mm^2$

$$Area = \frac{load}{f_{cd}} = \frac{2000 \times 10^3}{220} = 9090.9\,mm^2 \simeq 90.90\,cm^2$$

From steel table → Rolled steel beams with cover plate.

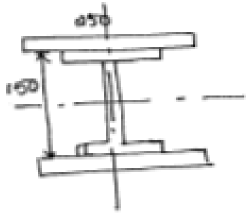
Let us try ISHB @ 150 @ 34.6 kg/m & cover plate 250×12mm.

area = $104.8 \times 100\,mm^2$

$r_x = r_z = 7.32\,cm$.

$r_y = 5.90\,cm$.

$\therefore r_{min} = 59mm$.



Supported ends given as

$KL = 0.65\,l_e$ → Given both ends fixed.

$= 0.65 \times 6000$

$= 3900mm$.

$$\therefore \lambda = \frac{l_e}{r_{min}} = \frac{3900}{59} = \boxed{66.10 = \lambda}$$

Buckling class → (c) ($\because$ Built up section)

$\therefore$ Table 9 (c)

→ $f_{cd} = 158.24\,N/mm^2$

$\therefore$ Design compressive Strength $= P = A_e f_{cd}$

$= (104.08 \times 100)(158.24)$

$= 1647.0\,kN < 2000\,kN$. [UNSAFE]

Since unsafe "Revise the section."

Let us change the section & try ISHB – 150 @ 30.6 kg/m. & 250×20mm cover plate.

area = $138.98 \times 100\,mm^2$

$r_x = r_z = 7.96\,cm$.

$r_y = 6.39\,cm \simeq r_{min} = 63.9mm$.

$$\therefore \lambda = \frac{KL}{r_{min}} = \frac{0.65 \times 6000}{63.9}$$

$\boxed{\lambda = 61.0}$

$\therefore\ f_{cd} = 166.4\,N/mm^2$

$P_d = A_e f_{cd}$

$= 13898 \times 166.4$

$= 2312.6\,kN > 2000\,kN$

6. a)

Q.3 Design a compression member using double channel section "face to face" to carry a factored load of 1600 kN. The length of column is 5m with one end fixed & one end hinged. Also design "single lacing system"

Design of column

$l_e = KL$

$= 0.8 \times 5000$

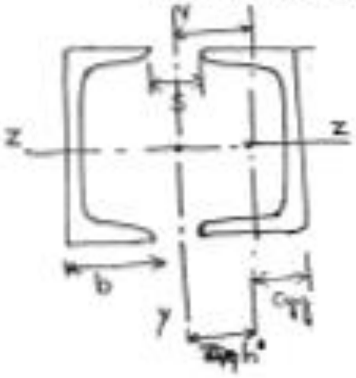
$l_e = 4000mm$

Assume $f_{cd} = 200 N/mm^2$

$Area = \frac{load}{Stress} = \frac{1600 \times 10^3}{200} = 8000mm^2 = 80.0cm^2$

area of one channel $= \frac{80}{2} = 40cm^2$

From steel table Try ISLC -300 @ 33.1 kg/m



Properties of one channel

area = 4211 mm^2

$I_{xx} = 6047.9 \times 10^4 mm^4$

$I_{yy} = 346.0 \times 10^4 mm^4$

$C_{yy} = 25.5mm$

b = 100 mm

To make any structure safe $I_{xx} = I_{yy}$

$\therefore I_{xx} = I_{xx} + ah^2$

$= 2[6047.9 \times 10^4 + 4211 \times 0^2]$

$I_{xx} = 120.958 \times 10^6 mm^4$

$I_{yy} = 2\left[346.0 \times 10^4 + 4211 \times \left(b - C_{yy} + \frac{S}{2}\right)^2\right]$

$= 2\left[346.0 \times 10^4 + 4211 \times \left(100 - 25.5 + \frac{S}{2}\right)^2\right]$

$\therefore$ on equating $I_{xx} = I_{yy}$

$\therefore \quad 120.958 \times 10^6 = 2\left[346.0 \times 10^4 + 4211 \times 100 - 25.5 + \frac{S}{2}\right]$

$S = 83.73 \simeq 80mm$

$I_{min} = 180.95 \times 10^4\ mm^4.$

$$r_{min} = \sqrt{\frac{I_{min}}{A}} = \sqrt{\frac{120.95 \times 10^4}{2 \times 4211}} = \sqrt{\frac{120.95 \times 10^4}{2 \times 4211}}$$

$r_{min} = 119.84mm.$

$\therefore$ from table 9(c) = Buckling class ⓒ

$f_{cd} = 206.61\ N/mm^2$

Compression load = $P_d = A_e f_{cd}$

$= 2 \times 4211 \times 206.61$

$= 1740.0 \times 10^3\ N > 1600 \times 10^3\ N.$

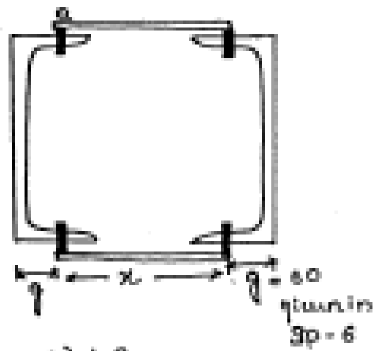
Hence SAFE

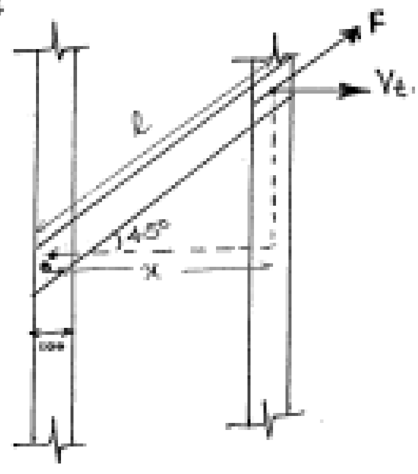
★ Design of lacing: Pg No - 48, 49, 50.

(i) Transverse shear = 2.5% of column load. → 7.6.6.1

$$V_t = \frac{2.5}{100} \times 1600 = 40KN.$$

(ii) lacing inclination = $\theta = 45°$ → 7.6.4





$x = 2(b - g) + S$

$= 2(100 - 60) + 80$

$= 160mm$

$\cos 45° = \frac{x}{l} = \frac{160}{l}$

$\therefore\ l = \frac{160}{\cos 45°} = 226.27mm$

$\therefore$ length of lacing = 226.27 mm

For single lacing = $l_e = l = 226.27$ mm

For Double lacing = $l_e = 0.7l = 0.7 \times 226$

(iv) Lacing dimension: (b & t)

- Width = 3 x dia of bolt. [Clause No. 7.6.2]

  = 3x18. (Assuming 18 mm)

  $\boxed{b = 54mm}$ ≃ 55 mm.

- Thickness: $t = \frac{l_e}{40}$ → for single lacing. [Clause No: 7.6.3]

  $t = \frac{l_e}{60}$ → for double lacing.

∴ for single lacing. $t = \frac{l_e}{40} = \frac{226.7}{40} = 5.66 \simeq 6mm$.

∴ $\boxed{\text{Lacing } b \times t = 55mm \times 6mm}$.

(v) Check for Slenderness ratio: [Clause No. 7.6.6.3]

$\lambda = \frac{l_e\sqrt{12}}{t} \ngtr 145$

$= \frac{226.7\sqrt{12}}{6}$

= 130 is not greater than 145 → Safe.

Using λ = 130, calculating from table 9c

$\boxed{f_{cd} = 71.3 N/mm^2}$

(vi) Forces in lacing:

$\cos 45 = \frac{V_t}{F}$

∴ $F = \frac{V_t}{n \cdot \cos 45}$

n = no. of planes of lacing

$F = \frac{40 \times 10^3}{2 \times \cos 45°}$

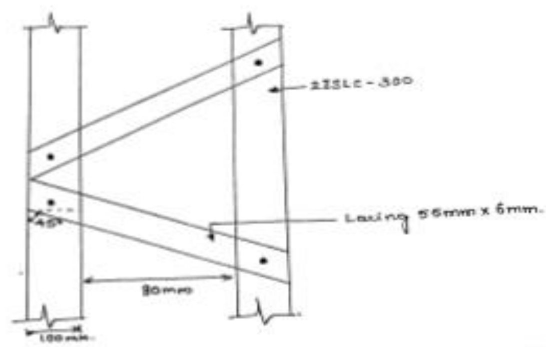
$\boxed{F = 28.28 kN}$

(vii) Check for Strength:

- Compression Strength = Area x Stress
  $= b \times t \times f_{cd}$
  $= 55 \times 6 \times 71.3$
  $= 23.5 \times 10^3 N$.

- Tensile Strength $= (b - d_0) t \times f_{cd}$
  $= (55 - 20) 6 \times 71.3$
  $= 14.97 \times 10^3 N$.

- Provide single bolt @ each end.

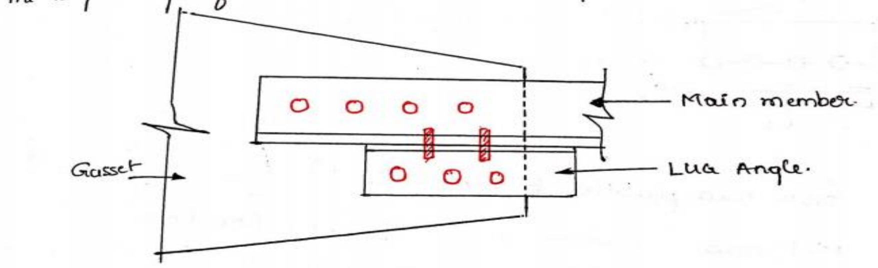


MODULE 4

7. a) What is lug angle? Explain with neat sketch.

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 member 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.

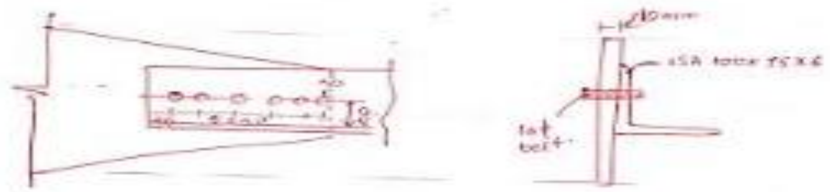


7.b)

Q.2 A single unequal angle 100 × 75 × 6 is connected to a 10mm thick gusset plate at the ends with 6 - 16mm ϕ bolts to transfer tension as shown in fig. Determine tensile strength of angle assuming the $f_y = 250$, $f_u = 410$

(i) If gusset plate connected to 100mm leg → g = 60 mm

(ii) — " — to 75 mm leg → g = 40mm.



(i)

$A_{nc} = (100 - 6/2 - 18) \times 6 = 474 \text{ mm}^2$

$(b - t/2 \times d_0) \times t$

$A_{go} = (75 - 6/2) \times 6 = 1010 \text{ mm}^2$

(a) Strength governed by yielding of gross section.

$$T_{dg} = \frac{A_g f_y}{\gamma_{m0}} = \frac{1010 \times 250}{1.10} \times 10^{-3} = 229.5 \text{ KN}.$$

(b) Strength governed by rupture of critical section.

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

$$\geq \frac{f_u \gamma_{m0}}{f_y \gamma_{m1}}$$

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

$$= 1.4 - 0.076\left(\frac{75}{6}\right)\left(\frac{250}{410}\right)\left(\frac{75 + (100 - 40) - 6}{5 \times 40}\right) \leq 1.44 \left(\frac{410}{250}\right)\left(\frac{1.1}{1.25}\right)$$

$\underline{\beta = 1.03}$

$$T_{dn} = \frac{0.9 \times 410 \times 474}{1.25} + 1.03 \times \frac{432}{1.10} \times 250$$

$T_{dn} = 641.34 \text{ KN}.$

(c) 5 Block Shear:

$A_{vg} = 6 \times (5 \times 40 + 40) = 1440\,mm^2$ $\quad A_{vg} = (L_v \times t)$

$A_{vn} = 6 \times [(5 \times 40 + 40) - 5.5 \times 18]$ $\quad A_{vn} = (L_v \times t) - (n d_h \times t)$

$A_{tg} = 6 \times 40 = 240$ $\quad A_{tg} = (L_t \times t)$

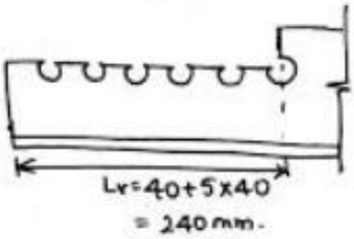
$A_{tn} = 6 \times (40 - 0.5 \times 18) =$ $\quad A_{tn} = (L_t \times t) - (n d_h \times t)$

$A_{vg} = 240 \times 6 = 1440\,mm^2$

$A_{vn} = (240 \times 6) - (5.5 \times 18 \times 6) = 846\,mm^2$

$A_{tg} = (22 \times 6) = 132\,mm^2$

$A_{tn} = (22 \times 6) - (0.5 \times 18 \times 6) = 78\,mm^2$



$$T_{db_1} = \left[\frac{A_{vg} f_y}{\sqrt{3}\,\gamma_{m0}} + \frac{0.9 A_{tn} f_u}{\gamma_{m1}}\right]$$

$$= \left[\frac{846 \times 250}{\sqrt{3} \times 1.10} + \frac{0.9 \times 78 \times 410}{1.25}\right] = \cancel{134.03}\ 211.97\ kN.$$

$$T_{db_2} = \left[\frac{0.9 A_{vn} f_u}{\sqrt{3}\,\gamma_{m1}} + \frac{A_{tg} f_y}{\gamma_{m0}}\right] =$$

$$= \left[\frac{0.9 \times 846 \times 410}{\sqrt{3} \times 1.25} + \frac{132 \times 250}{1.1}\right]$$

$$= \underline{174.18\ kN}$$

8. a) Briefly explain the type of column base.

Types of column bases

(1) Transmitting direct load only.

(2) Carrying appreciable bending moment in addition to direct load.

The two prevalent column bases are

(1) Slab base
(2) Gusset base. } These can also be bolted & welded.

★ Slab base:

★ When a column is subjected to only direct loads, the base can be designed by only slab base.

★ Guesset Base:

★ when load from column is very high and bending moment is present along with axial load. A part of load is transmitted through the gusset plate to the base slab. Gussets and Stiffeners brace the base slab against bending. So a thin base plate is sufficient.

★ Grillage foundation: when the load is extremely high. → tall buildings.
when the soil bearing capacity is poor.
It uses beam tiers to support base slab.

8. b)

Problem:

Q.1. Design a slab base plate for a steel column ISHB 225 @ 46.8 kg/m carrying a total load of 700 kN. M20 grade concrete is used for foundation. Take SBC as 180 kN/m².

Solution

Axial load = 700 kN.

Factored load = 1.5 × 700 = 1050 kN.

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

$$\text{Area of base plate} = \frac{\text{Factored load}}{\text{Bearing strength of concrete}} = \frac{1050 \times 10^3}{9} = 116.67 \times 10^3 m^2$$

Properties of ISHB 225 @ 46.8 kg/m. (IS 808 : 1989, Pg no : 5)

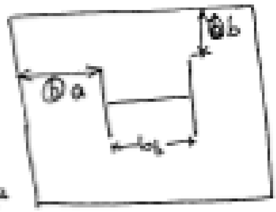
$D = h = 225mm$, $b_f = 225mm$, $t_f = 9.1mm$

∴ Overall dimension of the column is square, let us design a square base plate.

Side of square plate = $\sqrt{116.67 \times 10^3} = 341.56mm \simeq 350mm$.

Note : Keeping in view the cleat angle to be provided, fix the size of base plate. Size of base plate = 360mm × 360mm.

Projection beyond the column $a = b = \frac{360 - 225}{2} = 67.5mm$ 67.5mm



$$\text{Net upward pressure } (w) = \frac{P_u}{\text{Area of base plate}}$$

$$= \frac{1050 \times 10^3}{360 \times 360} = 8.10\ N/mm^2$$

$8.10 < 9\ N/mm^2$

★ Thickness of base plate : (t_s) [P-41, cl-no - 7.4.3.1]

$$t_s = \sqrt{\frac{2.5 w (a^2 - 0.3 b^2) \gamma_{m0}}{f_y}} \nless t_f.$$

$$t_s = \sqrt{\frac{2.5 \times 8.1 (67.5^2 - 0.3 \times 67.5)^2 \times 1.1}{250}} = 16.86 \nless t_f\ (9.1mm) \quad \text{Say } 18mm.$$

∴ provide base plate of thickness 18mm i.e 360 × 360 × 18mm.

Design of bolt:

$$\text{No. of bolt} = \frac{\text{Force}}{\text{Bolt Value}}$$

Force / Side = 25% of column load = $0.25 \times 1050 = 262.5$ KN.

let us assume 22mm dia bolt

∴ bolt hole dia = $22 + 2 = 24$ mm

Bolt value : least of following.

1. Strength of one bolt in shear.
2. Strength of one bolt in bearing.

1. strength of one bolt in single shear (P-75, cl: 10.3.3)

$$V_{dsb} = \left(\frac{f_u}{\sqrt{3}}\right) \times \left(\frac{n_n A_{nb} + n_s A_{sb}}{\lambda_{mb}}\right)$$

Assuming shank is interfering the shear plane.

$n_n = 0$, $n_s = 1$, $\lambda_{mb} = 1.25$

$$A_{sb} = \frac{\pi}{4} d^2 = \frac{\pi}{4} \times 22^2 = 379.94 \text{ mm}^2$$

$$V_{dsb} = \frac{400}{\sqrt{3}} \times \left(\frac{0 + 1 \times 379.94}{1.25}\right)$$

$V_{dsb} = 70.23$ KN.

2. strength of bolt in bearing $V_{dpb} = \dfrac{2.5 \times K_b \times d \times t \times f_u}{\lambda_{mb}}$

K_b = least of following.

$\dfrac{e}{3d_0} = \dfrac{45}{3 \times 24} = 0.625.$

$e = 1.7 \times 24 = 40.8$ mm. ≈ 45mm

$P = 2.5 \times 22 = 55$ mm. ≈ 60mm

$\dfrac{P}{3d_0} - 0.25 = 0.58$

$\dfrac{f_{ub}}{f_u} = \dfrac{400}{410} = 0.98$

1

∴ $K_b = 0.58$.

t = The minimum of 1) Thickness of gusset plate (assumed) = 16mm
2) Thickness of ~~angle~~ base pl = ~~6mm~~. 9.1mm

$$\therefore \quad V_{dpb} = \frac{2.5 \times 0.58 \times 22 \times 9.1 \times 400}{1.25 \times 1000} = 92.90 \text{ KN}.$$

Bolt value = 70.23 KN.

No. of bolts = $\frac{262.5}{70.23}$ = 3.73 ≈ 4 No's

Cleat angle:

Provide cleat angle ISA 100×85×10 mm to secure the column with the base plate by 4 bolts of 22mm dia.

Concrete bed block:

Axial load = 700 KN.

Self wt of concrete = 10% of 700 i.e 0.1×700 = 70.

Total load = 770 KN.

$$\text{Area} = \frac{\text{Total load}}{\text{SBC of Soil}} = \frac{770}{180} = 4.28\ m^2$$

Side of Square block = $\sqrt{4.28}$ = 2.06 mm Say 2.1m

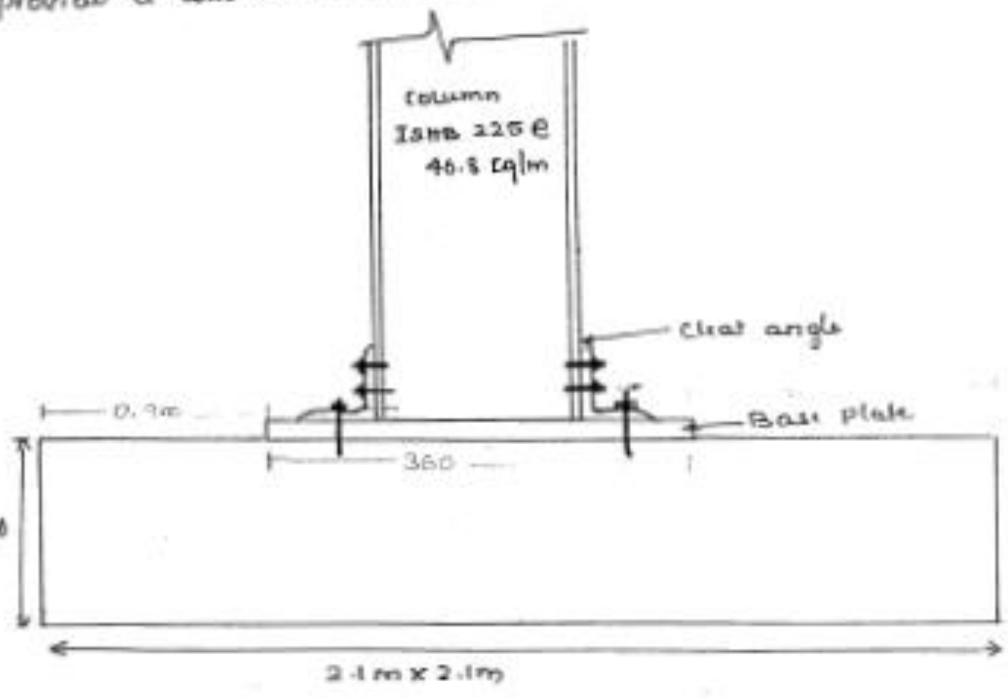
Thickness of block:

Assuming the dispersion angle of load as 45°

Depth of concrete bed = projection of concrete beyond base plate.

$$= \frac{2.1 - 0.36}{2} = 0.87m \quad \text{Say } 0.90m.$$

∴ provide a concrete bed block = 2.1×2.1×0.9 m.



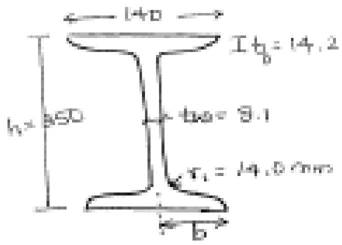
MODULE 5

10.

PROBLEMS:

Q1. 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 → it's 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)

$$V_d = 0.6\left[\frac{f_y}{\sqrt{3}.\lambda_{m0}} \times A_v\right] > V_u$$

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

$\lambda_{m0} = 1.10.$

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

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

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

$$M_d = \frac{\beta_b Z_p . f_y}{\lambda_{m0}}$$

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

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

Pag (18) to decide which type of Section.

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

From Table 46 pg 138.

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

$= 8511 \times 10^3 mm^2$

$$M_d = \frac{\beta Z_p \cdot f_y}{\lambda_{mo}} = \frac{1.0 \times 851.11 \times 10^3 \times 250}{1.10}$$

$M_d = 193.43 \times 10^6$ N-mm $> M_u = 166.5$ KN-m $\longrightarrow$ HENCE SAFE.

(d) Check for deflection:

$\frac{Span}{250}$ = permissible deflection = $\frac{5000}{250} = 20mm$

Actual deflection $\delta = \frac{5}{384} \frac{wl^4}{E_s I}$ $\xrightarrow{\text{not ultimate}}$ 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:

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

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

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

b_1 = Bearing width (or) Support width.

$\therefore$ Assume $b_1 = 200$

$\therefore n_2 = 2.5(t_f + r_1)$

$= 2.5(14.2 + 14)$

$= 70.5mm$

(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.

$$\left\{ \begin{aligned} \lambda &= 2.5 \frac{d}{t_w} \\ &= 2.5 \times \frac{(h - 2t_f)}{t_w} = \frac{2.5(350 - 2 \times 14.2)}{8.1} \\ \lambda &= 99.25 \end{aligned} \right.$$

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

HENCE SAFE.

End