DETAILS EXPLANATIONS

CE : Paper-1 (Paper-7) [Full Syllabus]

[PART : A]

1. It is the ratio of unconfined compressive strength of undisturbed soil to the unconfined compressive strength of remoulded soil.

2. It is the most common mode of failure of finite slope occuring in case of steep slopes when the soil mass is homogenous above and below the toe.

3. Ulimtate bearing capacity :

$q_u = CN_cS_cd_ci_c + qN_qS_qd_qi_q + 0.5B_rN_rS_rd_ri_r$

4. Compaction is the instantaneous decrease in volume of soil whether consolidation is the time taking process.

5. As per IS 800 : 2007

$\frac{d}{t_w} > 67 \in$ for an unstiffened web.

$\frac{d}{t_w} > 67 \in \sqrt{\frac{K_v}{5.35}}$ for a stiffened web.

6. It should have sufficient stiffness so that buckling of the beam occures in between the the braces.

7. Due to compactness of the plate girders, vibration and impact are not serious problems.

8. *The distribution factor at joint B :*

Joint	Member	Relative Stiffness	Total Relative Stiffness	Distribution Factors
B	BA	$\frac{I}{3} = \frac{4I}{12}$	–	$\frac{4}{7}$
	BC	$\frac{I}{4} = \frac{3I}{12}$	$\frac{7I}{12}$	$\frac{3}{7}$

9. Number of roller support = 1

Number of single reaction hinged suppon = 01

Number of double reaction higed suppon = 1

Number of joints = j = 8

Number of members = m = 15 = Number of unknown forces

Number of unknown reaction = 2 + 2 + 1 = 5

Total unknowns = 15 + 5 = 20

Total number of equations = 2j = 2 × 8 = 16

∴ Degree of redundancy = 20 – 16 = 4

10. A beam which is stiffened by a strut and tie rods is calledtrugged beam.

11. This method is most useful in the determination of stiffness and carryover factors for non prismatic members.

12. The nominal diameter of a bar is the diameter calculated from the volume of unit length of bar assuming it to be perfactly cylinder.

13. Effective width of flange of T-Beam :

$$b_f < \frac{l_0}{6} + b_w + 6D_f$$

$< b_w$ + half the sum of clear distances to the adjacent beams on either side.

14. Minimum reinforcement = $\frac{0.85}{f_y} \times B \times d$

$$A_{st_{min}} = \frac{0.85}{415} \times 300 \times 500 = 307.23 \text{ mm}^2$$

15. According to this 'the work done by external forces on a truss is equal to the internal work done. This is pricipal of virtual work.'

16.
- Freyssinet system
- Magnel blaton system
- Gifford udall
- P.S.C mono-wire system.
- Lee-McCall system

17. The temperature at which creep is uncontrollable.

18. Second moment of area ≡ Moment of inertia

$$I_{xx} = \frac{BD^3}{3} = \frac{400 \times 300^3}{3} = 36 \times 10^8 \text{ mm}^4$$

19. Load intensity

$$w = \frac{dF_x}{dx} = 9x^2 + 4x$$

At $x = 5$ m ; $w = 9(5)^2 + 4(5) = 245$ kN/m

20. Ductility is a measure of a metal's ability to withstand tensile stresshaving plastic strain greater than 5%.

[PART : B]

21. ***Liquifaction :***

In loose saturated sands, due to seismic disturbance (earthquake) or dynamic loading volume of soil decreases hence pore pressure change is positive. Due to built up of high pore pressure, sudden decrease in effective stress and decrease in shear strength is recorded, consequently large settlement of foundation suddenly occur along with vertical upward flow of muddy water.

22. Effective stress = Total pressure – Pore Pressure

$$\bar{\sigma} = \sigma_T - u$$

$$\bar{\sigma} = \{(10 \times 2) + (20 \times 5)\} - (10 \times 7)$$

$$\bar{\sigma} = 50 \text{ kN/m}^2$$

Comment : By increasing water level the effective stress will not be changed as total pressure and pore-pressure increase simultaneously.

23.
- ***Compactive Energy :*** By increasing compactive energy the compaction curve shifts up in left side i.e., by increasing compactive energy the dry density increases.
- ***Water Content :*** By increasing water content on dry side of O.M.C., dry density increases and Vice-Verca.
- ***Type of Soil :*** Granular soils give maximum dry density at O.M.C.
- ***Method of Compaction :*** For sandy soils compaction is achieved by vibration while for clay rooler is used.

24. Cofficient of compression

$$C_c = \frac{\Delta e}{\log_{10}(\bar{\sigma}_2/\bar{\sigma}_1)} = \frac{0.36 - 0.21}{\log_{10}(100/10)} = 0.15$$

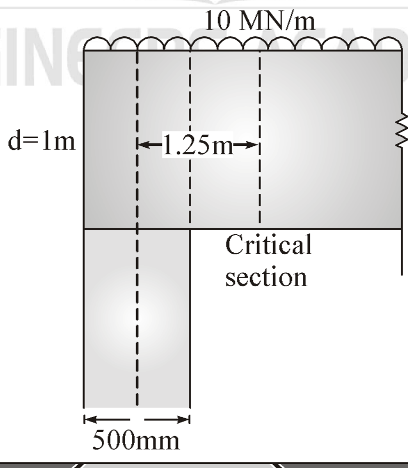
Cofficient of compressibility

$$a_v = \frac{\Delta e}{\Delta\bar{\sigma}} = \frac{0.15}{(100/10)} = 0.015$$

Cofficient of volume compressibility :

$$\Rightarrow \quad m_v = \frac{a_v}{1+e_0} = \frac{0.015}{1+0.36} = \frac{0.015}{1.36} = 0.011$$

25. Reaction $= \frac{wl}{2} = \frac{10 \times 10}{2} = 50$ MN



Design shear force = 50 MN

Critical section is at 'd' = (1 m)

From face of column i.e., 1.25 m from centre of support

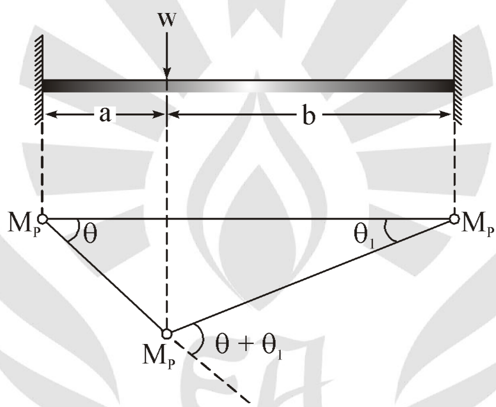
Design shear force $V_u = 50 - (1.25 \times 10) = 37.5$ MN

26. The additional assumptions are : (As per IS : 456 - 2000)

As per IS 456 : 2000,

- The maximum compressive - strain in concrete in axial compression is taken as 0.002.
- The maximum compressive starin at highly stressed extreme fibre in concrete subjected to axial compression and bending and when there is no tension on the section shall be 0.0035 minus 0.75 times the strain at the least compressed extreme fibre.

27. Kinematic Method :



Work done by plastic hinge :

$$= M_p.\theta + M_P\theta_1 + M_P(\theta + \theta_1)$$

$$= 2M_p\theta + 2M_P\theta_1$$

$$\theta_1 = \frac{a}{b}\theta = 2M_P\theta\left(\frac{a+b}{b}\right)$$

$$= \frac{2M_P L\theta}{b}$$

External work done :

$$= w_u.a\theta$$

$$\therefore \quad w_u a\theta = 2M_P\theta\left(\frac{a+b}{b}\right)$$

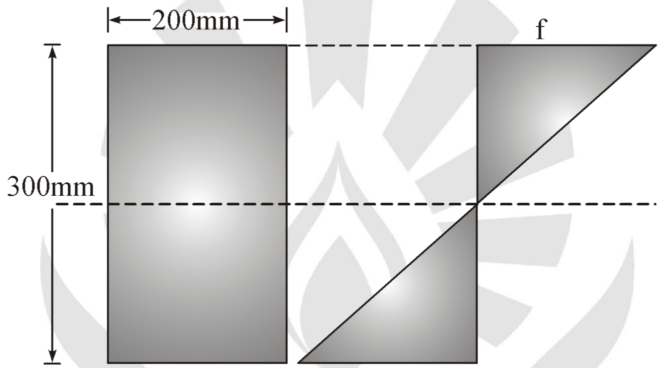
$$w_u = \frac{2M_P.L}{b}$$

28. ***Stud-Welding (SW) :***

It permits rapid attachment of studs (shear connectors used in composite construction) and other fastners to steel beam without piercing the structure metal. The power source may be DC-constant-current supply or a capacitor-discharge supply. The SW-process begins by inserting a stud into the stud gun, placing a ceramic cup or ferrule on the end of the stud, and positioning the gun perpendicular to the surface of the structure.

29. Maximum bending stress :

$$f = \frac{M}{I} \cdot y = \frac{120 \times 10^6 \times 6}{200 \times 300^2} = \frac{720}{18} = 40 \text{ N/mm}^2$$



Load withheld safely by upper half portion :

$$F = \frac{1}{2} \times f \times \left(\frac{D}{2}\right) \times B$$

$$F = \frac{1}{2} \times 40 \times \left(\frac{300}{2}\right) \times 200 \times 10^{-3}$$

$$F = 20 \times 150 \times 200 \times 10^{-3}$$

$$F = 600 \text{ kN}$$

30. ***Plastic Moment Capacity :***

$$M_p = \frac{A}{2}(\bar{y}_1 + \bar{y}_2) \cdot f_y$$

$$M_p = f_y \cdot z_p$$

Where, z_p = Plastic section modulous.

$\Rightarrow$ For rectangular section $z_p = \dfrac{BD^2}{4}$

$$z_p = \frac{400 \times 250^2}{4} = 6250000 \text{ mm}^3$$

$$M_p = 250 \times 6250000 \times 10^{-6} \text{ kN-m}$$

$$M_p = 1562.5 \text{ kN-m}$$

31. Fixed end moments :

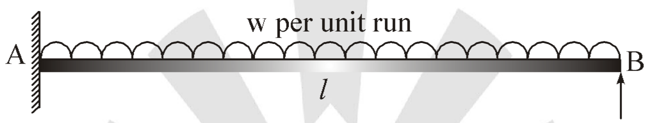
$$M_{FAB} = -\frac{60\times 5^2}{12} = -125 \text{ kN-m}$$

$$M_{FBA} = +\frac{60\times 5^2}{12} = +125 \text{ kN-m}$$

$$N_{FBC} = -\frac{90\times 1\times 2^2}{3^2} = -40 \text{ kN-m}$$

$$M_{FCB} = +\frac{90\times 1^2\times 2}{3^2} = +20 \text{ kN-m}$$

32. Let R be the reaction at support B.



Bending moment at any section distance 'x'.

From B = M = $R.x - \frac{wx^2}{2}$

Strain energy stored by the beam

$$\Rightarrow \quad w_1 = \int \frac{M^2 \cdot dx}{2EI}$$

$$\Rightarrow \quad w_1 = \int_0^l \left(Rx - \frac{wx^2}{2}\right)^2 \frac{dx}{2EI}$$

By the second theorem of casting liano reaction 'R' shall have such a value that stored strain energy is minimum. So,

$$\frac{\partial w_i}{\partial R} = 0$$

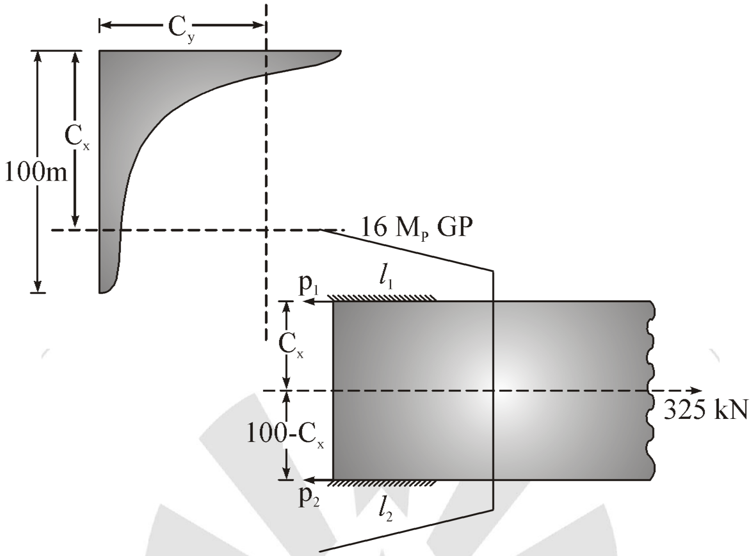
$$\Rightarrow \int_0^l 2\left[Rx - \frac{wx^2}{2}\right] x \frac{dx}{2EI} = 0$$

$$\Rightarrow \frac{R}{EI}\int_0^l x^2 dx = \frac{w}{2EI}\int_0^l x^3 dx$$

$$\Rightarrow \quad R = \frac{3}{8} w.l$$

[PART : C]

33. The edge support is conisted of two angles i.e., the load taken by them is 650 kN. Thus each angle will take $\frac{650}{2}$ = 325 kN.



Assuming the thickness of angles as 8 mm, hence minimum size will be 5 cm, because the minimum thickness of thicker part governs the minimum size of weld.

From the above figure, we have

$$p_1 + p_2 = 325$$

Taking moment about the line of action of load of 325 kN.

$\therefore \quad p_1 \times C_x - p_2 \times (100 - C_x) = 0$

$\Rightarrow p_1 \times 30.9 - p_2 \times (100 - 30.9) = 0$

$\Rightarrow \qquad p_1 = \dfrac{69.1}{30.9} p_2 = 2.24\ p_2$

Substituting (2) in (1), we get

$$p_2 = \frac{325}{3.24} = 100.425 \text{ kN}$$

and $\qquad p_1 = 325 - 100.425 = 224.575$ kN

For l_1 lenth of weld,

$p_1 = l_1 \times$ Thorat thickness $\times$ Permissible stress in weld

$\Rightarrow \qquad 224.575 \times 10^3 = 1 \times \dfrac{5}{\sqrt{2}} \times 76$

$$l_1 = 835.78 \text{ mm} \approx 836 \text{ mm}$$

For l_2 length fo weld,

$p_2 = l_2 \times$ throat thickness $\times$ Permissible stress in weld

$$\Rightarrow \quad 100.425 \times 10^3 = l_2 \times \frac{5}{\sqrt{2}} \times 76$$

$$l_2 = 373.74 \text{ mm} \approx 374 \text{ mm}$$

Thus adopting l_1 = 836 mm and l_2 = 374 mm

34. With the usual natations when both loads are on the span, with w_2 at the section distant x from the left end,

$$_2m_x = \frac{l-x}{l}\{w_2x + w_1(x - d)\}$$

$$= \frac{5-x}{5}\{10x + 20x(x-3)\}$$

$$\therefore \quad _2m_x = 6(x - 2)(5 - x)$$

$$\therefore \ (_2m_x)_{max} \text{ occurs at } x = \frac{2+5}{2} = 3.5 \text{ m}$$

$$(_2m_x)_{max} = 6(3.5 - 2)(5 - 3.5) = 13.50 \text{ kN-m}$$

$$_1m_x = \frac{x}{1}\{w_1(1 - x) + w_2(1 - x - d)\}$$

$$= \frac{x}{5}\{20(5 - x) + 10(5 - x - 3)\}$$

$$= 6x(4 - x)$$

$$(_1m_x)_{max} \text{ occurs at } x = \frac{0+4}{2} = 2\text{m}$$

$$(_1m_x)_{max} = 6 \times 2(4 - 2) = 24 \text{ kNm}$$

$$_0m_x = \frac{w_1x(l-x)}{l} = \frac{20x(5-x)}{5}$$

$$\therefore \quad _0m_x = 4x(5 - x)$$

$$(0m_x)_{max} \text{ Will occur at } x = \frac{0+5}{2} = 2.5 \text{ m(centre of span)}$$

$$(_0m_x)_{max} = 4 \times 2.5(5 - 2.5) = 25 \text{ kNm}$$

But $(_1m_x)_{max}$ = 24 kNm at x = 2 m

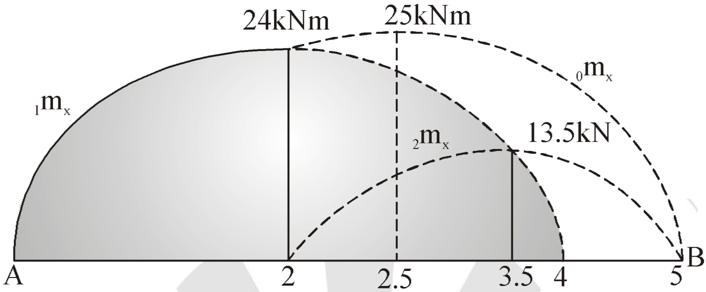
Let us find where the $(_0m_x)$ curve and the $(_1m_x)$ curve will intersect.

For this condition 4x(5 – x) = 6x(4 – x)

$\therefore$ x = 2m

Thus it should be clear that the curve $_1m_x$ is applicable only from x = 0 to x = 2 m. Beyond this range $_0m_x$ is valid since it is greater than $_2m_x$.

The absalutely maximum bending moment has occurred at the centre corresponding to the ${}_0m_x$ curve, and is equal to 25 kNm. Let 1 be the minimum span length below which the greatest bending moment anywhere in the girder is determined by ${}_0m_x$.



For this critical condtion $({}_1m_x)_{max} = ({}_0m_x)_{max}$

But $${}_1m_x = \frac{x}{1}\{w_1(1-x) + w_2(1-x-d)\}$$

$$= \frac{x}{1}\{20(1-x) - 10(l-x-3)\}$$

$$= \frac{x}{1}\{30(l-1) - 30x\}$$

$$= \frac{30}{1}x\{(l-1) - x\}$$

This is maximum at $x = \frac{0+(1-1)}{2} = \frac{1-1}{2}$

∴ $$({}_1m_x)_{max} = \frac{30}{l}.\frac{1-1}{2}.\frac{1-1}{2}$$

∴ $$({}_1m_x)_{max} = \frac{15}{21}(l-1)^2$$

$${}_0m_x = \frac{w_1x(1-x)}{1}$$

$({}_0m_x)_{max}$ will occur at $x = \frac{1}{2}$

∴ $$({}_0m_x)_{max}\frac{w_1 l}{4}\frac{20\times l}{40} = 5l$$

Since $(m_x)_{max} = ({}_1m_x)_{max}$

$$51 = \frac{15}{21}(1-1)^2$$

∴ $$101^2 = 15(l-1)^2$$

$$1^2 - 6l + 3 = 0$$

∴ $$1 = \frac{6\pm\sqrt{36-12}}{8}$$

$$\Rightarrow \qquad l = \frac{6 \pm 2\sqrt{6}}{2}$$

$$l = 3 \pm \sqrt{6}$$

Rejecting the negative value,

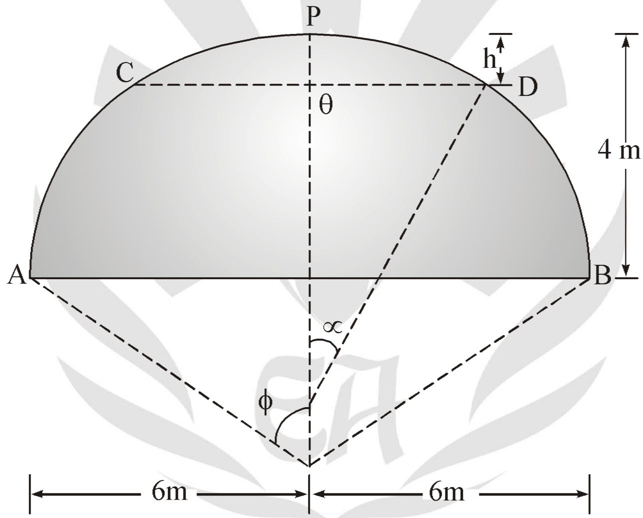
$$l = 3 \pm \sqrt{6} = 5.45 \text{ m}$$

For spans above 5.45 m the maximum bending moment anywhere is given by $_1m_x$.

For spans less than 5.45 m the maximum bending moment anywhere is given by $_0m_x$.

35. (i) Geometry of the dome :

Figure shows the dome let the radius of the dome be r. The diameter of base = AB = Diameter of room = 12 m.



Rise PS = 4 m

$$(2r - PS)PS = 6 \times 6$$

or $$(2r - 4)4 = 36$$

From which $r = AO = 6.5$ m

Again CD = Diameter of pening = 1.6 m

PQ = Rise at opening = n

$\therefore \qquad h = (2 \times 6.5 - h) = CQ^2 = (0.8)^2$

From which $h = 0.05$ m

$$\sin \propto = \frac{0.8}{6.5} = 0.1231 \; ; \; \therefore \; \alpha = 70^o ; \; \cos \propto = 0.9924$$

$$\sin \phi = \frac{6}{6.5} = 0.9231 \; ; \; \therefore \; \phi = 67^o23' \; ; \; \cos \phi = 0.3840$$

(ii) ***Loading :*** The various formulae derived earlier are valid when there is no apening at the crown. In our case, there is opening of 1.6 m diameter. However, for calculation purpose. We can assume that there is no apening, and the weight of the extra portion of the dome shell CPD can be accounted for by reducing the load of the lantern and taking into consideration only the effective weight of the extra portion of the dome shell CPD can accounted for by reducing the load of the lantern and taking into consideration only the effective weight of lentern. That is :

effective wt. of lentern = Actual wt of lantern – wt of dome shell CPD

Let the thickness of the dome be 100 mm. The uniformly distributed load per sq. m of surface area are:

(1) Self-weight of done shell

$$= 0.1 \times 25000 = 2500 \text{ N/m}^2$$

$$= 1500 \text{ N/m}^2$$

$$\text{Total} = 4000 \text{ N/m}^2 = 4 \text{ kN/m}^2 = w$$

Weight of dome shell

$$\text{CPD} = w \times 2\pi rh$$

$$= 4 \times 2\pi \times 6.5 \times 0.05 = 8.17 \text{ kN}$$

∴ Effective weight of lantern

$$w = 22 - 8.17 = 13.83 \text{ kN}$$

3. Calculation of stresses due to combined load the stress at any horizontal blane will be equal to the algebric sum of stresses due to the two loading and the dome will be designed for the maximum of these stresses. Total meridionel stress

$$= \frac{wr(1-\cos\theta)}{t\sin^2\theta} + \frac{w}{2\pi rt\sin^2\theta}$$

$$= \frac{4\times 6.5(1-\cos\theta)}{\sin^2\theta} + \frac{13.83}{2\pi\times 6.5\times 0.1\sin^2\theta}\text{kN/m}^2$$

$$= \left[260\frac{1-\cos\theta}{\sin^2\theta} + \frac{3.39}{\sin^2\theta}\right]\times 10^{-3}$$

$$= 0.26\frac{1-\cos\theta}{\sin^2\theta} + \frac{0.00339}{\sin^2\theta} \text{ N/mm}^2 \quad ...(1)$$

$$\text{Stress} = \frac{wr}{t}\left[\frac{\cos^2\theta+\cos\theta-1}{1+\cos\theta}\right]-\frac{w}{2\pi rt}\frac{1}{\sin^2\theta}$$

$$= \frac{4\times6.5}{0.1}\left[\frac{\cos^2\theta+\cos\theta-1}{1+\cos\theta}\right]-\frac{13.83}{2\pi\times6.5\times0.1}\times\frac{1}{\sin^2\theta}\ \text{N/mm}^2$$

$$= \left[260\left(\frac{\cos^2\theta+\cos\theta-1}{1+\cos\theta}\right)\right]-\frac{3.39}{\sin^2\theta}\times10^{-3}\ \text{N/mm}^2$$

$$= 0.26\frac{\cos^2\theta+\cos\theta-1}{1+\cos\theta}-\frac{0.00339}{\sin^2\theta}\ \text{N/mm}^2 \qquad ...(2)$$

The values of meridional stress and hoop stress for various values of θ are tabulated in table.

θ	Meridional Stress (N/mm^2)			Hope Stress (N/mm^2)		
	Due to w	Due to w	Total	Due to w	Due to w	Total
7°4'	0.1305	0.224	0.354	0.127	–0.224	–0.097
10°	0.131	0.112	0.243	0.125	–0.112	+0.013
30°	0.140	0.014	0.154	0.086	–0.014	+0.072
60°	0.174	0.004	0.178	–0.044	–0.004	–0.048
67°23'	0.186	0.004	0.190	–0.088	–0.004	–0.092

36. In case of cantilever retaining wall design becomes uneconomical. In that case counterfort retaining walls are used. Design principles for various component parts are discussed below in brief.

- ***Design of Stem :*** Unlike the stem of cantilever retaining wall, the stem of a counterfort retaining wall acts as a continous slab supported on counterfort. Due to the varijing earth pressure over the height of stem, the stem slab deflects outwards, and hence main reinforcement is provided along the length of the retaining wall, at the outer face of the stem between the counterforts and at the inner face near the counterforts. The reaction of the stem is taken by the counterforts to which It is firmly anchored. The maximum bending moment occurs at B, where the uniformly distributed earth pressure load is calucated for unit height. If w is the load on the stem, slab, at B, per unit length.

$$w = P_a \times 1 \times 1 = K_a.H$$

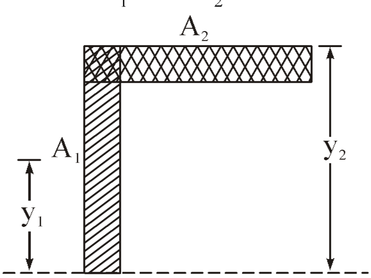
- ***Design of Heel Slab :*** The action of heel slab is similar that of stem. The heel slab is subjected to the downward load due to weight of soil and self weight and load is found to act downwards.

The maximum net pressure is found to act on a strip of unit width near edge C, since the wpward soil reaction is minimum there. If P is the net downward pressure per unit area, the maximum bending moment (negative) near counterforts will be (p l^2)/12 and positive bending movement will be (p l^2)/16. The total load (reaction) from the heel slab is transferred to the counterforts. The heel slab is firmly attached to the counter forts by means of vertical ties (two legged stirrups).

- ***Design of Toe Slab :*** The to slab bends as a contilever due to upward soil reaction as in the case of cantilever retaining wall. Due to bending of the toe a cantilever, clockwise moments are induced at E, which are shared by both vertical stem and heel slab. The distribution of this moment between the stem and heel slab are not known. These transfered edge moments will case bending of stem as well as heel slab, in a direction normal to their usual bending. It countreforts are also provided over the toe slab, upto the height of soil, their behaviour become certain. In such a case, the reaction is transferred directly to the counterforts, without affecting the bending behavirour of stem and heel slab.
- ***Design of Counterforts :*** The counterforts take reactions both from the stem as well as the heel slab. Since the active pressure on stem acts outwards and net pressure heel slab acts downwards, the couterforts are sujected to tensile stresses along the outer face AC of the counterforts, the angle ABC between stem and hell slab has a tendency to increase from 90°, and this tendency is ressisted by counterforts. Thus the counterfort may be considered to bend as a cantilever, fixed at BC. The maximum depth of this T-beam is at the junction B. The depth is measured perpendicular to the sloping face AB, i.e., depth d_1 = BB, at B. The width b_1 of the countefort is kept constant throughout its height, main reinforcement is provided parrallel to AC.

37. Plastic section modulus about major axis.

To determine the plastic section modulus about the z-z axis, divide the section into areas A_1 and A_2 as shown in figure.



$$A_1 = \left(\frac{D}{2}\right)t = \frac{300}{2}\times 7.6 = 1140 \text{ mm}^2$$

$$A_2 = (B - t)T = (90 - 7.6) \times 13.6$$
$$= 1120.64 \text{ mm}^2$$

$$\bar{y}_1 = \frac{D}{4} = \frac{300}{4} = 75 \text{ mm}$$

$$\bar{y}_2 = \frac{D}{2} - \frac{T}{2} = \frac{300}{2} - \frac{13.6}{2} = 143.2 \text{ mm}$$

The plastic section modulus

$$z_{pz} = 2(A_1\bar{y}_1 + A_2\bar{y}_2)$$
$$= (1140 \times 75 + 1120.64 \times 143.2)$$
$$= 491.95 \times 10^3 \text{ mm}^3$$

$$I_{zz} = (300-2\times13.6)^3\times\frac{7.6}{12} + 2\times90\times\frac{13.6^3}{12} + 2\times90\times13.6\times\left(150 - \frac{13.6}{2}\right)^2$$

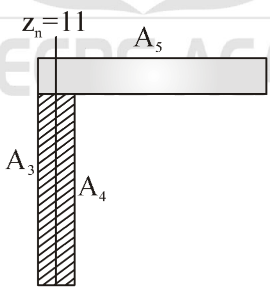
$$I_{zz} = 63094.77 \times 10^3 \text{ mm}^4$$

$$z_{ez} = 63094.77 \times \frac{10^3}{150} = 420.631 \times 10^3 \text{ mm}^3$$

Shape factor about the z-z axis

$$= \frac{z_{pz}}{z_{ez}} = \frac{491.95\times10^3}{(420.631\times10^3)} = 1.169$$

Plastic section modulus about minor axis similarly, to determine the plastic section moduuls of the y-y axis, divide the section is into areas A_3, A_4 and A_5 as shown in figure.



Since the neutral axis divides the total area into two equal parts.

$$z_n = \frac{(A_1 + A_2)}{D} = \frac{(1140 + 1120.64)}{300}$$
$$= 7.535 \text{ mm}$$

Now the areas :

$$A_3 = \left(\frac{D}{2}\right)z_n = \left(\frac{300}{2}\right)7.535$$

$$A_3 = 1130.25 \text{ mm}^2$$

$$A_4 = \left(\frac{D}{2} - T\right)(T - z_n)$$

$$A_4 = (300 - 13.6)(7.6 - 7.535)$$

$$A_4 = 18.616 \text{ mm}^2$$

$$A_5 = (B - z_n)T = (90 - 7.535) \times 13.6$$

$$A_5 = 1121.524 \text{ mm}^2$$

The distances :

$$\bar{z}_3 = \frac{z_n}{2} = \frac{7.535}{2} = 3.7675 \text{ mm}$$

$$\bar{z}_4 = \frac{(t - z_n)}{2} = \frac{(7.6 - 7.535)}{2} = 0.0325 \text{ mm}$$

$$\bar{z}_5 = \frac{(B - z_n)}{2} = \frac{(90 - 7.535)}{2} = 41.2325 \text{ mm}$$

The plastic section modulus about the y-y axis.

$$z_{py} = 2\left(A_3\bar{z}_3 + A_4\bar{z}_4 + A_5\bar{z}_5\right)$$

$$z_{py} = 2(1130.25 \times 3.7675 + 18.616 \times 0.0325 + 1121.524 \times 41.2325)$$

$$z_{py} = 101.004 \text{ mm}^3$$

Calculation of z_{pe}

Distance of the netural axis from the web,

$$\bar{z} = \frac{\dfrac{[2 \times 90 \times 13.6 \times 45 + (300 - 2 \times 13.6) \times 7.6 \times 7.6]}{2}}{[90 \times 13.6 \times 2 + (300 - 2 \times 13.6)7.6]} = 26.1 \text{ mm}$$

$$I_{yy} = 2 \times 90^3 \times \frac{13.6}{12} + 2 \times 90 \times 13.6 \times (45 - 26.1)^2 + (300 - 2 \times 13.6) \times \frac{7.6^3}{12}$$

$$+ (300 - 2 \times 13.6) \times 7.6 \times \left(26.1 - \frac{7.6}{2}\right)^2$$

$$I_{yy} = 3567.85 \times 10^3 \text{ mm}^4$$

$$z_{ey} = \frac{3567.85 \times 10^3}{(90 - 26.1)} = 55834.9 \text{ mm}^3$$

Hence the shape factor about the y-y axis

$= 101,004155,834.9 = 1.809$

and the plastic moment capacity of the section

$z_{pfy} = 101,004 \times 250 \times 10^{-6}$

$= 25.251$ kNm

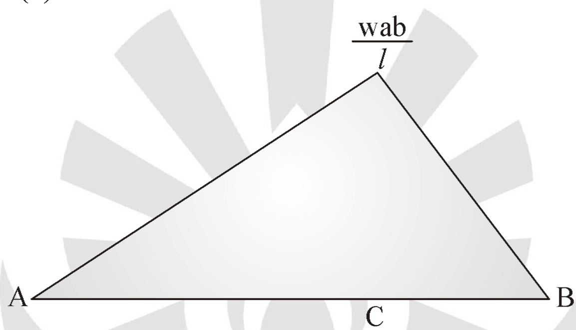
38. Figure (a) shows the beam AB simply supported at A and B. Let the load w act at C.

Let AB = l

AC = a

and BC = b

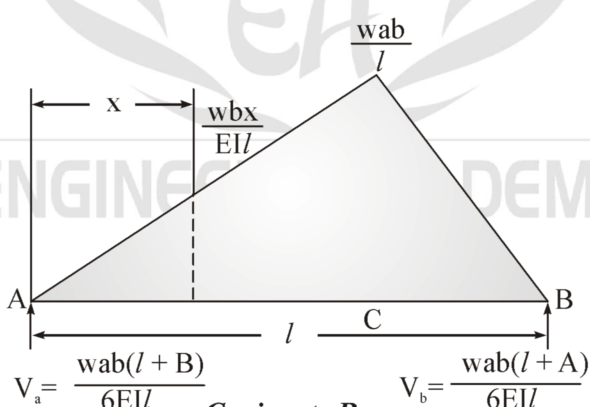
Figure (b) showns



B.M. Diagram

The B.M. diagram for the beam.

Figure (c) shows



Conjugate Beam

Deflection at C for the given beam

= At C for the cinjugate beam

$$= \frac{wab(l+b)}{6EI.l}.a - \frac{1}{2}a\frac{wab}{EIl}.\frac{a}{3}$$

$$= \frac{wa^2b(l+b)}{6EIl} - \frac{wa^3b}{6EIl}$$

$$= \frac{wa^2b^2}{6EIl}$$

Maximum Deflection : This will occur at the section of zero slope of the given beam, i.e., at the section of zero shear of the conjugate beam.

Let at a distance x from A, the S-F for the conjugate beam be zero.

$$\therefore \quad \frac{wab(l+b)}{6EIl} - \frac{1}{2}.x.\frac{wbx}{EIl} = 0$$

$$\Rightarrow \quad x = \sqrt{\frac{l^2 - b^2}{3}}$$

B.M. at a distance x from A for the conjugate beam.

$$= \frac{wal(l+b)}{6EIl}.x - \frac{1}{2}x\frac{wbx}{EIl}.\frac{x}{3}$$

$$= \frac{xbx}{6EIl}(al + ab - x^2)$$

Figure (c) shows the conjugate beam subjected to $\frac{M}{EI}$ loading.

Let V_a and V_b be the reactions at the supports A and B of the conjugate beam.

By taking moments about A and B, we get

$$V_a = \frac{wab(l+a)}{6EIl}$$

$$V_b = \frac{wab(l+b)}{6EIl}$$

Height of $\frac{M}{EI}$-diagram at a distance x(x < a) from the end

$$'A' = \frac{wbx}{EIl}$$

Slope at A for the given bem = D.F. at A for the conjugate beam.

$$\frac{wab(l+b)}{6EIl} = \frac{wb}{6EIl}(l - b)(l + b)$$

$$= \frac{wb(l^2 - b^2)}{6EIl}$$

Slope at B for the given beam = S.F. at B for the conjugate

$$= \frac{-wab(l+a)}{6EIl} = \frac{-wa(l+a)(l-a)}{6EIl}$$

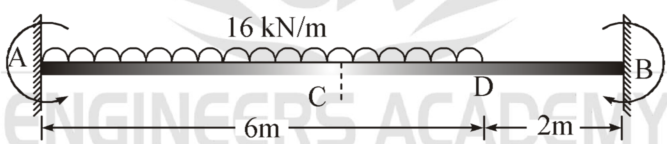
$$= \frac{-wa(l^2 - a^2)}{6EIl}$$

Put $x = \sqrt{\frac{l^2 - b^2}{3}}$

$$\text{Maximum deflection} = \frac{wb}{6EIl}\sqrt{\frac{l^2 - b^2}{3}}\left(al + ab - \frac{l^2 - b^2}{3}\right)$$

$$\text{Maximum delfection} = \frac{wb}{9\sqrt{3}\,EIl}(l^2 - b^2)^{3/2}$$

39. Note that the given loading is a uniformly distributed loading of 16 kN/m acting from 'C to D' fixing moments at 'A' and 'B' due to downward uniformly distributed load of 16 kN/m acting from A to D are given by



(A) $M_A = \frac{wa^2}{12l^2}(6l^2 - 8la + 3a^2)$

$$= \frac{16 \times 6^2}{12 \times 8^2}(6 \times 8^2 - 8 \times 8 \times 6 + 3 \times 6^2)$$

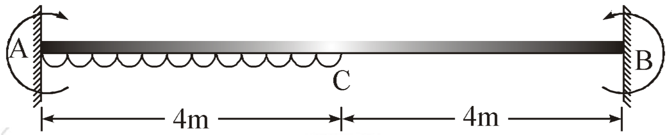
M_A = 81 kN-m(hogging)

(B) $M_B = = \frac{wa^3}{12l^2}(4l - 3a)$

$$= \frac{16\times6^3}{12\times8^2}(4 \times 8 - 3 \times 6)$$

$$M_B = 63 \text{ kN-m(hogging)}$$

Fixed end moments at A and B due to upward uniformly distributed load of 16 kN/m acting from A to C are given by



$$M'_A = \frac{16\times4^2}{12\times8^2}(6 \times 8^2 - 8 \times 8 \times 4 + 3 \times 4^2)$$

$$= 58.67 \text{ kN-m (Sagging)}$$

$$M'_B = \frac{16\times4^2}{12\times8^2}(4 \times 8 - 3 \times 4) = 26.67 \text{ kN-m (Sagging)}$$

∴ Net fixing moment at A = 81 – 58.67 = 22.33 kN-m (hogging)

and Net fixing moment at B = 63 – 26.67 = 36.33 kN-m (hogging)

Note : *In the above questin formula (A) and (B) will be derived.*

❍❍❍