DETAILS EXPLANATIONS

CE : Paper-1 (Paper-8) [Full Syllabus]

[PART : A]

1. Ultimate-stress is the maximum value of stress before failure.

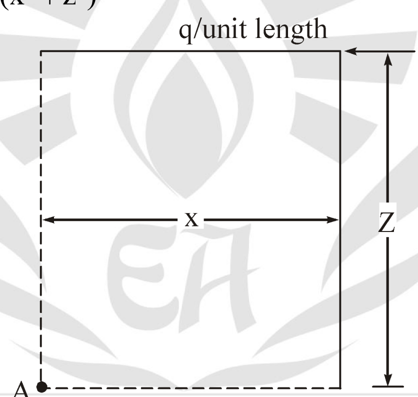
2. It is the stress on a plane on which there are no tangential stresses.

3.
- Open-Excavation
- Borings
- Sub-Surface sounding
- Geo-physical method.

4. The proctor compaction test is a laboratory method of experimentally determining the optimal moieture content at which a given soil type will become most dense.

5. ***Boussinesg's Equation :***

$$\sigma_z = \frac{2qxz^2}{\pi(x^2+z^2)^2}$$



6. It is an important tool in slope investigations.

7. It is a water tight reationing structure used, for example, to work on the foundations of a bridge pier.

8. Load factor = Factor of Safety × Shape factor

$$= \frac{300}{200} \times 1.5$$

Load-factor = 1.5 × 1.5 = 2.25

9.
$$f = \frac{M}{I}.y = \frac{M}{z}$$

$$= \frac{60 \times 10^6 \times 6}{300 \times 400^2} = \frac{360}{4 \times 4 \times 3}$$

$$= 7.5 \text{ N/mm}^2$$

10. Pore pressure :

$$U = \gamma_w.h = 10 \times 3 = 30 \text{ N/mm}^2$$

11. $\because$ S.e. = w G

$$\Rightarrow \quad e = \frac{1.20 \times 2.78}{1} = 3.348$$

12. It is the point at which bending moment diagram changes it's sign.

13. Degree of consolidation

$$U = \frac{\Delta h}{\Delta H} \times 100\%$$

$$\Rightarrow \quad 0.60 = \frac{200}{\Delta H}$$

$$\Rightarrow \quad \Delta H = 333.33 \text{ mm}$$

14. It is the property by virtue of which any material can be deformed by application of load and by removing the load it regains it's original shape.

15. Loss of Stress :

$$p_L = mf_c$$

where, m = Modular ratio

f_c = Stress in concrete at steel level.

16. $$\delta = \frac{5wl^4}{384EI}$$

17. It is just the number of unknown reactions or force which can't be calculated using equilibrium equations only.

18. Ratio of modulus of elasticity of steel to that of concrete.

19. $$\left.\begin{array}{c} l.l.D. \\ 16\phi_m \\ 48\phi_l \\ 300\,mm \end{array}\right\} \min$$

20. As per principal of virtual work, the internal work done by plastic moment is equal to the external work done by loads.

[PART : B]

21.
- The size of the beam required is lesser in case of Pre-stress than that in RCC.
- External loads are counter balanced uto a desired degree.
- The work progress is faster.

22. Effective length (l) = l_0 + w or l_0 + d} Lesser

$$l = 4 + 0.200 = 4.20 \text{ m}$$

or $= 4 + 0.180 = 4.18$ m

So, $l = 4.18$ m

Factored moment

$$\Rightarrow \quad M_u = \frac{w_u l^2}{8} = \frac{1.5\times 20\times 4.18^2}{8}$$

$$M_u = 62.52 \text{ kN-m}$$

23. • Steel = 0.15

Concrete = 1.5

•

D.L.	L.L.	EQL / wL
1.5	–	1.5
1.5	1.5	–
1.2	1.2	1.2
0.9	1.4	–

24.

$$\tau = \frac{T}{\tau}.r$$

$$100 = \frac{16T}{\pi D^3}$$

$$100 = \frac{16\times T}{\pi D^3}$$

$$T = \frac{\pi\times(250)^3\times 16\times 100\times 10^{-6}}{\pi}\text{ kN-m}$$

25. ***Assumptions made in simple beinding :***

- Plain section before bending remains plain after bending.
- Strain diagram is linear.
- Hook's law is valid, so for isotropic materials the stress diagram is linear.
- Material is homogeneous and isotropic.

26. Let p_1 and p_2 be the principal stresses at a point in strained material: The principal stresses are given by

$$e_1 = \frac{p_1}{E} - \frac{p_2}{mE}$$

and $$e_2 = \frac{p_2}{E} - \frac{p_1}{mE}$$

Hence strain energy per unit volume :

$$= \frac{1}{2}p_1 e_1 + \frac{1}{2}p_2 e_2$$

$$= \frac{1}{2}p_1\left(\frac{p_1}{E} - \frac{p_2}{mE}\right) + \frac{1}{2}p_2\left(\frac{p_2}{E} - \frac{p_1}{mE}\right)$$

$$U = \frac{1}{2E}\left\{\frac{p_1^2 + p_2^2 + p_3^2 - 2p_1p_2 + 2p_2p_3 + 2p_1p_3}{m}\right\}$$

27. Time Factor :

$$T_v = \frac{\pi}{4}(u)^2 \qquad | \text{ For } U \leq 60\%$$

$$T_v = \frac{C_v.t}{d^2}$$

For same soil and same drainage-condtions.

$$T_v \alpha t \qquad (\because T_{v1} = \frac{\pi}{4}(0.5)^2 = 0.196;$$

$$\frac{T_{v1}}{T_{v2}} = \frac{t_1}{t_2} \qquad T_{v2} = \frac{\pi}{4}(0.4)^2 = 0.1256)$$

$$\Rightarrow \frac{0.196}{0.1256} = \frac{3\text{year}}{t_2}$$

$$t_2 = \frac{3 \times 0.1256}{0.196} = 1.92 \text{ years}$$

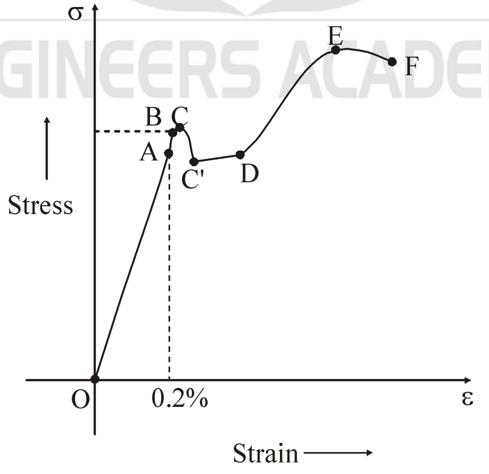
28. *Wave and Current Loads :*

Such loads have to be considered for offshore structures. These are random loads and one has to consult specialist literature to determine then (Radhakrishnan at al.1979 and subramanian and pranesh 1982) (for fef.). The force exerted by the water current should be considered in the design of bridge piers, abutments, and other water front structures.

29. *Block Shear Failure :*

Angle, gusset plate and coped beams connections amy fail as a result of blaock shear failure occures in shear at a row of bolt holes parellel to the applied loads, accuompined by tensile rupture along a perpendicular face. This type of failure results in a block of material being tern out by the applied shear force.

30. *Mild Steel Stress-Strain Curve :*



A = Proportional limit
B = Elastic limit
C = Upper yield point
D = Lower yield point
DE = Strain hardening
EF = Necking
F = Failure

31. ***Liquifaction*** :

In loose saturated sands, due to seismic loads or dynamic loading, volume 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. Such a phenomenon is called liquifaction of sand. It is generally observed near the vivers and sea during seismic effects.

32. Minimum size of weld for a 8 mm thick section = 3 mm

Maximum size of weld = 8 – 1.5 = 6.5 mm

Choose the size of weld as 6 mm

Effective throat thickness = 0.7 × 6 = 4.2 mm

$$\text{Strength of weld} = \frac{4.2\times 410}{\sqrt{3}\times 1.5} = 662.7 \text{ N/mm}$$

Assuming that there are only longitudinal (side) welds,

$$\Rightarrow \text{Required length of weld} = \frac{120\times 10^3}{662.7} = 181 \text{ mm}$$

⇒ Length to be provide on each side

$$= \frac{181}{2} = 90.5 \text{ mm} > 75 \text{ mm}$$

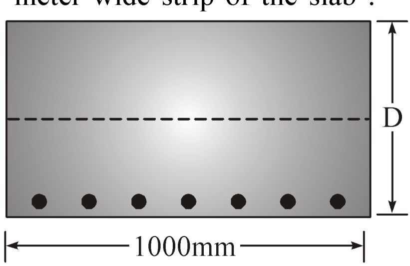
Hence, provide 90.5 mm held on each side with an end return of 2 × 6 = 12 mm

Therefore, the overall length of weld provided

= 2 × (90.5 + 2 × 6) = 205 mm

[PART : C]

33. Consider a 1 meter wide strip of the slab :



live load = 20 kN/m^2

Moment due to live load,

$$m_1 = \frac{20\times12^2}{8} = 360 \text{ kN/m}$$

(i) ***Section Modulus Required :***

$$z = \frac{m_1}{f_c} = \frac{360\times10^6}{14} = 25714286 \text{ mm}^3$$

But $$z = \frac{BD^2}{6}$$

$$\Rightarrow \quad D^2 = \frac{25714286\times6}{1000}$$

$$\Rightarrow \quad D = 393 \text{ mm} \approx 400 \text{ mm}$$

(ii) ***Prestressing Force :***

$$P = \frac{Af_c}{2} = \frac{1000\times400\times14}{2} = 2800 \text{ kN}$$

(iii) ***Bending Moment due to dead load :***

Dead load = 0.4 × 1 × 1 × 25 = 10 kN/m^2

$$m_2 = \frac{10\times12^2}{8} = 180 \text{ kN-m}$$

(iv) ***Eccentricity :***

$$e = \frac{2m_2+m_1}{2p} = \frac{(2\times180\times360)\times10^6}{2\times2800\times10^3}$$

$$e = 128.57 \text{ mm}$$

(v) ***Area of Steal :***

Strength of 1 cable of 12 wires of 5 mm ϕ = 225 kN

$$\text{Number of cables} = \frac{2800}{225} = 12.4 \approx 13$$

$$\therefore \quad A_{st} = 13\times12\times\frac{\pi}{4}\times(5)^2 = 3063.05 \text{ mm}^2$$

$$\text{Spacing of cables} = \frac{1000}{13} = 76.92 \approx 80 \text{ mm}$$

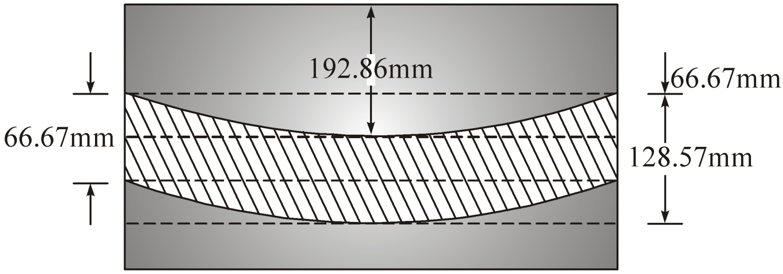
Hence probide 13 cables @ 80 mm c/c

Now, kern distances

$$k_b = k_t = \frac{z}{A} = \frac{D}{6} = \frac{400}{6} = 66.67 \text{ mm}$$

Distance from the top where the cable may lie

$$\frac{m_2+m_1}{p} = \frac{(180\times360)\times10^6}{2800\times10^3} = 192.86$$



Shaded portion is the zone in which resultant cable must lie.

34. (i) $\frac{\text{Effective length}}{\text{Least lateral dimension}} = \frac{3600}{460} = 7.83 < 12$

Hnece the column will be designed as short column.

(ii) ***Adopting*** $e_{min} = 20$ mm

But e_{min} shall not exceed 0.05 B and 0.05 D

$\therefore \quad 20 \leq 0.05 \times 460$ and $20 \leq 0.05 \times 600$

$20 \leq 23$ and $20 \leq 30$ hence OK

Hense, the equation

$p_u = 0.4\ f_{ok}A_c + 0.67\ f_yA_{sc}$ can be used

(iii) ***Service Load*** = 2500 kN

Factored load, $p_u = 1.5 \times 2500 = 3750$ kN

$f_{ok} = 20$ N/mm^2; $A_c = A - A_{sc}$

where, $A = 460 \times 600 = 276000$ mm^2

$f_y = 415$ N/mm^2

$\therefore \quad P_u = 0.4\ f_{ok}A_c + 0.67 f_y.A_{sc}$

$\Rightarrow 3750\times10^3 = 0.4\times20\times(276000 - A_{sc}) + 0.67 \times 415 \times A_{sc}$

$\Rightarrow \quad A_{sc} = \frac{3750\times10^3 - 2208000}{270.05} = 5710$ mm^2

Adoping 36 mm f bars, we getnumber of bars

$= \frac{5710}{(\pi/4)\times(36)^2} = 5.61 \approx 6$ bars

(iv) ***Diameter of Lateral ties :***

(a) $\frac{1}{4}\times36 = 9$ mm

(b) 0.6 mm

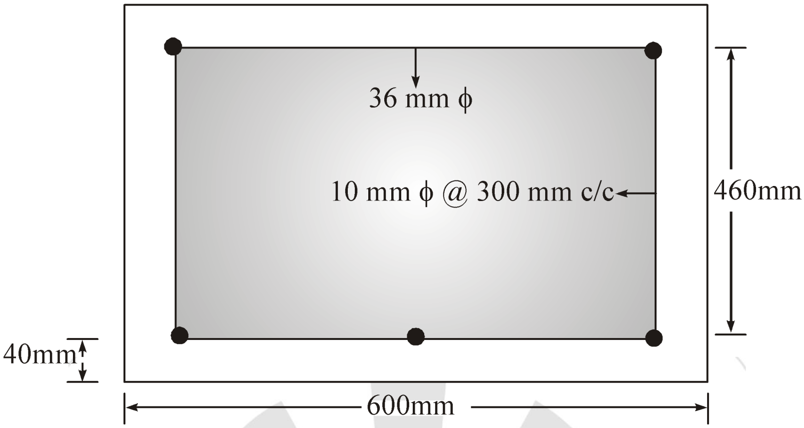
$\therefore$ Provide 0 mm bars as lateral ties.

(v) ***Spacing of lateral ties :***

(a) Least lateral limension = B = 460 mm

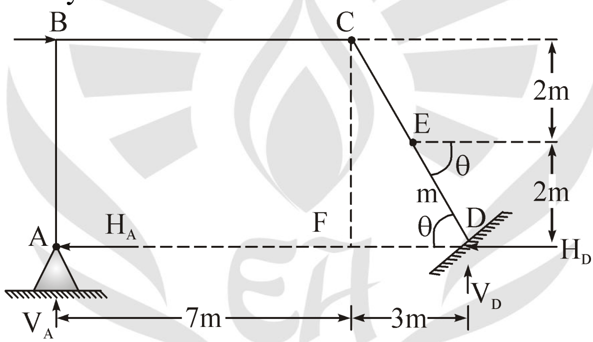
(b) $16\ \phi = 16 \times 36 = 576$ mm

(c) 300 mm



Lesser of the above three will be provided as the spacing. Hence 10 mm ϕ bars will be provided at 300 mm c/c.

35. Let vertical reactions at A and D be $V_A(\uparrow)$ and $V_D(\uparrow)$ respectively. Also let horizontal reactions at A and D be $H_A(\leftarrow)$ $H_D(\leftarrow)$ respectively. Also let the reactive moment at D be $m(\rightarrow)$.



Let $\angle CDA = \theta$

$$\tan\theta = \frac{4}{3}$$

Equilibrium Conditions :

$\Sigma F_y = 0;$ $V_A + V_D = 0$ ...(1)

$\Sigma F_y = 0;$ $H_A + H_D = 12$ ...(2)

Now taking moments about A, we have

$$\Sigma m_A = 0$$

$\Rightarrow (V_D \times 10) - (12 \times 4) - m = 0$

$\Rightarrow 10V_D - m = 48$ ...(3)

Since C is hinge, therefore moments about C either from right or left will be zero.

$$m_c \text{ for Left} = 0$$

$\Rightarrow V_A \times 7 + H_A \times 4 = 0$

$\Rightarrow 4H_A + 7V_A = 0$ ...(4)

Taking moments about E from right, we have m_E from right = 0

$$\Rightarrow V_D \times \frac{2}{\tan\theta} - H_D \times 2 - m = 0$$

$$\Rightarrow V_D \times \frac{2}{4/3} - 2H_D - m = 0$$

$$\Rightarrow 1.5V_D - 2H_D - m = 0 \qquad ...(5)$$

Substituting value of $V_D.H_D$ and m from (1), (2) and (3) respectively in (5) we get

$$1.5(-V_A) - 2(12 - H_A) - (10\ V_D - 48) = 0$$

$$\Rightarrow -1.5V_A - 24 + 2H_A - 10V_D + 48 = 0$$

$$\Rightarrow -1.5V_A - 10(-V_A) + 2H_A + 24 = 0$$

$$\Rightarrow 8.5V_A + 2H_A = -24 \qquad ...(6)$$

Solveing (4) and (6) equation, we get

$$H_A = 8.4 \text{ kN}$$

$$C_A = -4.8 \text{ kN}$$

$$V_D = -V_A = -(-4.8) = 4.8 \text{ kN}$$

and $H_D = 12 - H_A = 12 - 8.4 = 3.6$ kN

and $m = 10V_D - 48$

$$= 10 \times 4.8 - 48 = 0 \text{ kN-m}$$

Thus the reactions will be

$$H_A = 8.4 \text{ kN } (\leftarrow)$$

$$V_A = 4.8 \text{ kN } (\downarrow)$$

$$H_D = 3.6 \text{ kN } (\leftarrow)$$

$$V_D = 4.8 \text{ kN } (\uparrow)$$

$$m = 0 \text{ kN-m}$$

36. The force can be resolved into horizontal and vertical components as

$$p_h = P \cos 60^o = 0.5 \text{ p}$$

$$p_v = p \sin 60^o = 0.866 \text{ p}$$

Balt 5 will have the maximum stress (this can be verified by calculating the forces in various bolts in a tabuylar from as shown in example) Force in bolt 5 in y-y direction.

$$\Sigma(x^2 + y^2) = 4(50^2 + 50^2) = 20{,}000 \text{ mm}^4$$

$$R_y = \frac{0.866 \text{ p}}{5} + [(0.866 \text{ p} \times 400 - 0.5 \text{ p} \times 100) \times \frac{50}{20000}] = 0.9142 \text{ p}$$

$$R_x = \frac{0.5 \text{ p}}{5} + [(0.866 \text{ p} \times 400 - 0.5\text{p} \times 100) \times \frac{50}{20000}] = 0.841 \text{ p}$$

Resultant force in the bolt

$$= p\sqrt{0.9142^2 + 0.841^2} = 1.242\ p$$

- Joint is considered as slip joint strength of a 20 mm diameter bolt (grade 8.8) from table.

  Bearing $= 410 \times 10 \times \dfrac{20}{1000} = 82$ kN

  Shear $= 370 \times \dfrac{245}{1000} = 90.65$

  Therefore, Bolt strength = 82 kN.

  Equating bolt strength with the maximum force, we get,

  $$1.242\ p = 82 \text{ kN}$$

  or $$p = \frac{82}{1.242} = 66 \text{ kN}$$

  Thus, $$p = 42.35 \text{ kN}$$

  Hence, if the joint is a non slip joint, the load carrying capcity of the bolt graup, is reduced by 36% (from 66 kN to 42.35 kN) for the same grade and diameter of the boil.

37. Fixed End moments

$$\bar{m}_{ab} = \bar{m}_{ba} = \bar{m}_{cd} = \bar{m}_{dc} = 0$$

$$\bar{m}_{bc} = -\frac{60}{3^2}(1\times 2^2 + 2\times 1^2) = -40 \text{ kNm};$$

$$\bar{m}_{cb} = +40 \text{ kNm}$$

Note $$i_a = i_d = 0$$

Member AB : $$m_{ab} = 0 + \frac{2EI}{2}\left(0 + i_b - \frac{3\delta}{2}\right) = EI\,i_b - \frac{3}{2}EI\delta$$

$$m_{bd} = 0 + \frac{2EI}{2}\left(2i_b + 0 - \frac{3\delta}{2}\right) = 2EI\,i_b - \frac{3}{2}EI\delta$$

Member BC : $$m_{bc} = -40 + \frac{2EI}{3}(2i_b + i_c)$$

$$= -40 + \frac{4}{3}EI\ i_b + \frac{2}{3}EI\ i_c$$

$$m_{cb} = +40 + \frac{2EI}{3}(2i_c + i_b)$$

$$= +40 + \frac{2}{3}EI\ i_b + \frac{4}{3}EI\ i_c$$

Member CD : $$m_{cd} = 0 + \frac{2EI}{4.5}\left(2i_c + 0 - \frac{3\delta}{4.5}\right)$$

$$= \frac{8}{9}EI\,i_c - \frac{8}{27}EI\delta$$

$$m_{dc} = 0 + \frac{2EI}{4.5}\left(0 + i_c - \frac{3\delta}{4.5}\right)$$

$$= \frac{4}{9}EI\,i_c - \frac{8}{27}EI\delta$$

Quilibrium condition at B, $m_{ba} + m_{bc} = 0$

$$2EI\ i_b - \frac{3}{2}\ EI\ \delta - 40 + \frac{4}{3}\ EI\ i_b + \frac{2}{3}\ EI\ i_c = 0$$

$$\frac{10}{3}EI\ i_b + \frac{2}{3}\ EI\ i_c - \frac{3}{2}\ EI\ \delta = 40$$

$$20EI\ i_b + 4EI\ i_c - 9EI\ \delta = 240$$

Equilibrium condition at $m + m_{cd} = 0$

$$40 + \frac{2}{3}EI\ i_b + \frac{4}{3}EI\ i_c + \frac{8}{9}EI\ i_c - \frac{8}{27}EI\ \delta = 0$$

$$\frac{2}{3}EI\ i_b + \frac{20}{9}EI\ i_c - \frac{8}{27}\ EI\ \delta = 40$$

$$\therefore\ 9EI\ i_b + 30\ EI\ i_c - 4EI\ \delta = -540$$

For horizontal equilibrium, $H_a + H_d = 0$

$$\frac{m_{ab} + m_{ba}}{2} + \frac{m_{cd} + m_{dc}}{4.5} = 0 \Rightarrow 9m_{ab} + 9m_{ba} + 4m_{cd} + 4m_{dc} = 0$$

$$9\left(EI\,i_b - \frac{2}{3}EI\delta\right) + 9\left(2EI\,i_b - \frac{3}{2}EI\delta\right) + 4\left(\frac{8}{9}EI\,i_c - \frac{8}{27}EI\delta\right)$$

$$+4\left(\frac{4}{9}EI\,i_c - \frac{8}{27}EI\delta\right) = 0$$

$$27EI\ i_b + \frac{16}{3}EI\ i_c - \frac{793}{27}EI\ \delta = 0$$

$$\therefore\ 729EI\ i_b + 144EI\ i_c - 793EI\ \delta = 0$$

Thus, we have the following equations :

$$20EI\ i_b + 4EI\ i_c - 9EI\ \delta = 240$$

$$9EI\ i_b + 30EI\ i_c - 4EI\ \delta = -540$$

$$729EI\ i_b + 144EI\ i_c - 793EI\ \delta = 0$$

Solving, we get

$EI\ i_b = 25.1091$; $EI\ i_c = -23.0122$ and $EI\ \delta = 18.9038$

Substituting for $EI\ i_b$, $EI\ i_c$ and $EI\ \delta$, the final moments are determined.

$$m_{ab} = 25.1091 - \frac{3}{2}(18.9038) = -3.2466 \text{ say } - 2.325 \text{ kNm}$$

$$m_{ba} = 2(25.1091) - \frac{3}{2}(18.9038) = 21.8625 \text{ say } + 21.86 \text{ kNm}$$

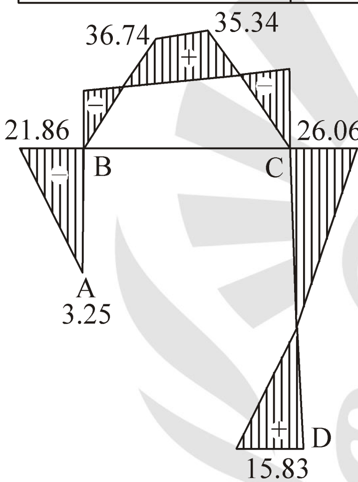
$$m_{bc} = -40 + \frac{4}{3}(25.1091) + \frac{2}{3}(-23.0122) = -21.8625 \text{ say } -21.86 \text{ kNm}$$

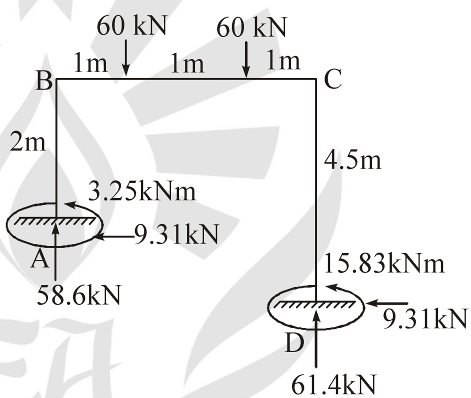
$$m_{cb} = +40 + \frac{2}{3}(25.1091) + \frac{4}{3}(-23.0122) = +26.056 \text{ say } + 26.06 \text{ kNm}$$

$$m_{cd} = \frac{8}{9}(-23.0122) - \frac{8}{27}(18.9038) = -26.056 \text{ say } -26.06 \text{ kNm}$$

$$m_{dc} = \frac{4}{9}(-23.0122) - \frac{8}{27}(18.9038) = -15.829 \text{ say } -15.83 \text{ kNm}$$

A		B		C	D
–3.25	+21.86	–21.86	+26.06	–26.06	–15.83





Reactions $\quad H_a = \dfrac{-3.25 + 21.86}{2} + 9.31 \text{ kN} \rightarrow$

$$H_d = \frac{-26.06 - 15.83}{4.5} = -9.31 \text{ kN} \leftarrow$$

$$V_d = \frac{-21.86 + 26.06(60 \times 1) + (60 \times 2)}{3}$$

$$= 6.14 \text{ kN} \uparrow$$

$$V_a = 120 - 6.14 = 58.6 \text{ kN} \uparrow$$

38. I.L. = 20 kN/m = 160 kN per skan

D.L. = 10 kN/m = 80 kN per skan

Total load = 30 kN/m = 240 kN per skan

(a) Using elastic analysis

The maximum bending moment at the centre support

$$= \frac{wL}{8} = 240 \times \frac{8}{8} = 240 \text{ kNm}$$

Shear = 1.25 × 240 = 300 kN

Unsing elastic design and assigning a stress of 0.66

$$f_y = 0.66 \times 250 = 165 \text{ MPa}$$

for Fe410 grade steel, the minimum section modulus required.

$$z_e = 240 \times \frac{10^6}{165} = 1454.5 \times 10^3 \text{ mm}^3$$

Hence IS MB 500 @ 86.9 kg/m is required with

$$z_e = 1810 \times 10^3 \text{ mm}^3$$
$$t_w = 10.2 \text{ mm}$$
$$D = 500 \text{ mm}$$

Shear capacity of the section

$$= (0.4 \times 250) \times 500 \times 10.2 \times 10^{-3}$$
$$= 510 \text{ kN}$$

This applied shear (300 kN) is less than 510 kN.

Hence the beam is safe.

(b) Using plastic analysis :

The maximum bending moment (by considering each span as a propped cantileve)

$$m_p = \frac{w_a L^2}{11.66}$$

The load factor to be considered in plastic analysis = 1.7 Less table 2.2

$$\text{Required } m_p = (30 \times 1.7) \times \frac{8^2}{11.66} = 279.93 \text{ kNm}$$

$$\text{Required } z_p = 279.93 \times \frac{10^6}{250} = 1{,}119{,}720 \text{ mm}^3$$

ISMB 400 @ 61.6 kg/m is required with

$$z_p = 1176.18 \times 10^3 \text{ mm}^3,$$
$$B = 140 \text{ mm}, A = 7840 \text{ mm}^2,$$

and $l_y = 622 \times 10^4 \text{ mm}^4$

$$\text{Shear capacity} = \frac{250}{\sqrt{3}\times} \times 400 \times 8.9 \times 10^{-3}$$
$$= 513.84 \text{ kN}$$

Maximum shear force at support

$$= \frac{279.93}{8} + \frac{(240 \times 17)}{2}$$
$$= 238.97 \text{ kN}$$

This is less than 0.6 times the shear capacity (0.6 × 5.13.84 = 308.3 kN) of the section, and hence there is no need to reduce the moment capacity of the section for shear.

Moment capacity = $250 \times \dfrac{1176.18}{10^{-3}}$ = 294.045

Both flanges must be restrained at hinge positions and at a point located at a maximum distance L_m from the hinge position (clause 4.5.2.1 of the code)

$$L_m \geq \frac{38r_y}{[(f_c/130)+(f_y/250)^2(x_1/40)^2]^{0.5}}$$

$$x_t = 1.132\left(\frac{AI_w}{I_yI_t}\right)^{0.5}$$

$$h = 400 - 16 = 384 \text{ mm}$$

$$I_w = \frac{I_f h^2}{2} = \frac{t_f B^3 h^2}{24}$$

$$= 16 \times 140^3 \times \frac{384^2}{24}$$

$$= 2.697 \text{ m}^6$$

$$I_t = \left(\frac{1}{3}\right)[2Bt_f^3 + ht_w^3]$$

$$= \left(\frac{1}{3}\right)[2\times140\times16^3 + 384\times8.9^3]$$

$$x_t = 1.132 \times \frac{7840 \times 2.697 \times 10^{11}}{622\times10^4\times472529^{0.5}}$$

$$= 30.362$$

$$L_m = \frac{38\times28.2}{\left[\left(\frac{0}{130}\right)+\left(\frac{250}{250}\right)^2\left(\frac{30.362}{40}\right)^2\right]^{0.5}}$$

$$= 1411.76 \text{ mm}$$

Also as per caluse 4.5.2.2 of the code stiffeners must be provided at the supports if the shear is greater than 100% of the shear capacity of the section, since in this case, the shear is greater than 10% of the shear capacity, stiffeners must be provided at the centre support. Although the weight of ISMB 400 (61.6 kg/m) as per plastic design is about 40.6% (103.7 kg/m) less than the weight as per elastic design, the additional cost of providing stiffeners and restraints in the plastic design may reduce the margin gained due to weight.

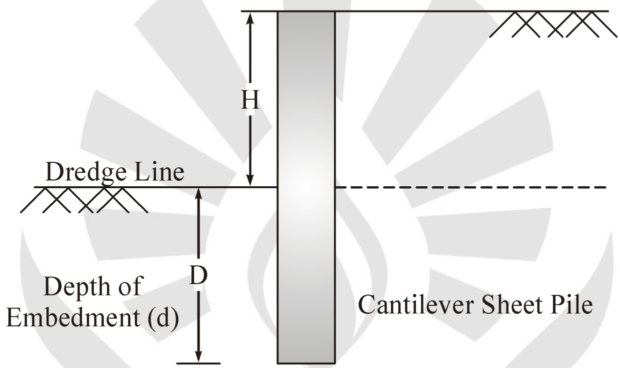
39. ***Sheet Pile Wall :***

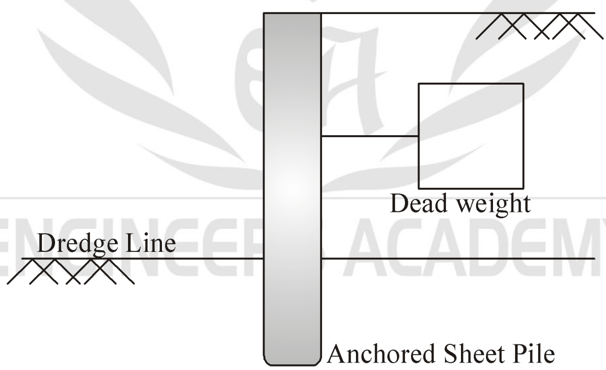
Sheet pile walls consist of no of sheet piles driven side by side to form a continuous vertical wall into the medium. Which is used to retain earth sheet pile walls are generally used in water from construction temperary construction, river training works to prevent the pipeing failure below the dams and to prevent walls of excavation from failure.

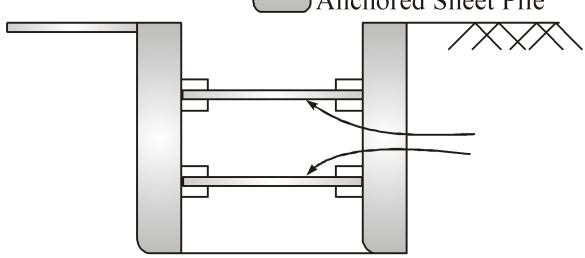
Material used for construction of sheet pile should be strong light in weight and thin in section.

Sheet pile must have sufficient depth embedment in the ground to prevent failure against overturning.

Sheet pile may be of following types :



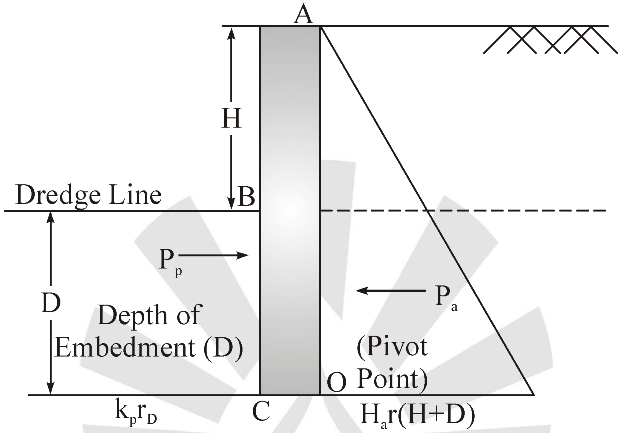




Braced Sheeting

Cantilever Sheet Pile :

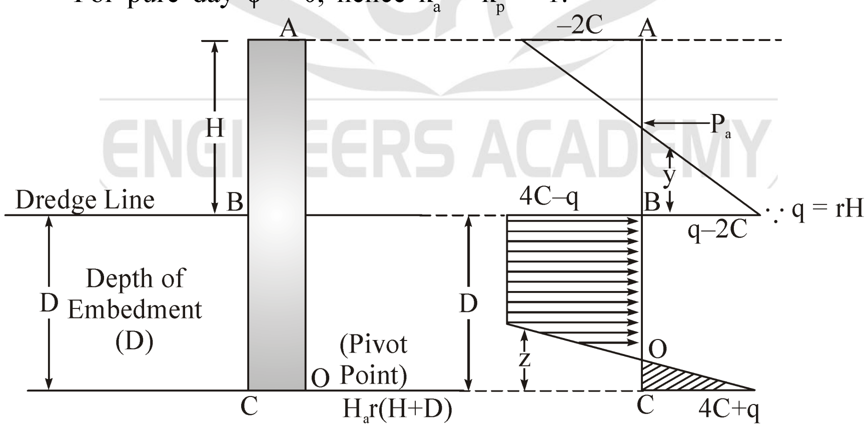
Case-(A) : For overturning of sheet pile in actual taken place above the pivot point '0' but for simplicity in computation, it is assumed to overtum about base point C, hence in such case, it is subjected to active pressure on its right for ABC and passive pressure on left foce CB below the dredge line.



Case (B) : For cohensive soil

Since overturning of sheet pile wall. taken place about the pirot point '0' sheet pile wall is subjected active pressure along the section ABO and passive pressure along the section OC on its right face and it is also subjected on its right face and it is also subjected to passive pressure along section BO and active pressure along OC on its left face.

For pure day ϕ = 0, hence $k_a = k_p = 1$.



Suppose : Total earth pressure above the dredge line is 'Pa' which will act at $\bar{y}$ above the drege line.

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