NAME →

ROLL NO. →

TEST NO. → 01

SUBJECT NAME → GEOTECHNICAL ENGG AND EVIRONMENTAL ENGG.

BRANCH → CIVIL ENGINEERING.

1 (a)

Let coefficient of permeability = K

and Capillary head = h_c

For first stage test,

$$\frac{x_2^2 - x_1^2}{t_2 - t_1} = \frac{2K}{Sn}(h_{a_1} + h_c)$$

where $x_2 = 7\,cm$

$x_1 = 1.5\,cm$

$t_2 - t_1 = 7$ minutes

n = porosity = 0.35

S = degree of Saturation = 0.85

h_{a_1} = head = 60 cm.

$$\therefore \frac{7^2 - 1.5^2}{7} = \frac{2K}{0.85 \times 0.35}(h_{a_1} + h_c)$$

$$\Rightarrow 0.9934 = K(60 + h_c) \quad \text{— (1)}$$

For second stage test.

$$\frac{x_2^2 - x_1^2}{t_2 - t_1} = \frac{2K}{Sn}(h_{a_2} + h_c)$$

$$\Rightarrow \frac{18.5^2 - 7^2}{24} = \frac{2K}{0.85 \times 0.35}(180 + h_c)$$

$$\Rightarrow 1.8175 = K(180 + h_c) \quad \text{— (2)}$$

$$\frac{eq^n (1)}{eq^n (2)} \Rightarrow \frac{0.9934}{1.8175} = \frac{60 + h_c}{180 + h_c} \Rightarrow \boxed{h_c = 84.65\,cm}$$

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from equation ①

$$0.9934 = K(60+84.65)$$

$$\Rightarrow K = 6.868 \times 10^{-3} \text{ cm/min}$$

1 (b)

when saturated

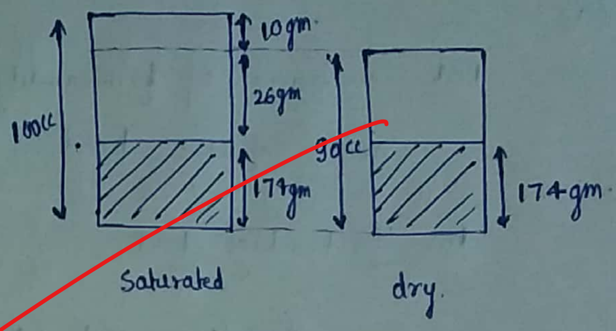
Volume of sample $V = 100$ cc

Mass of sample $M = 210$ gm.

when dried

Volume of Sample $V_d = 90$ cc

Mass of Sample $M_d = 174$ gm.



i) Initial water content $\omega = \frac{M_w}{M_s} = \frac{210-174}{174} = 0.2069 = 20.69\%$

ii) Volume of solid $V_s = 100 - (10+26) = 64$ cc

$\therefore$ density of solid $= \frac{M_s}{V_s} = \frac{174}{64}$ gm/cc $= 2.72$ gm/cc.

$\therefore$ specific gravity of ~~soil~~ solid $= \frac{2.72 \text{ g/cc}}{1 \text{ gm/cc}} = 2.72 \Rightarrow \boxed{G = 2.72}$

For specific gravity of soil, when dried

$$\text{specific gravity} = \frac{\frac{174 \text{ gm}}{90 \text{ cc}}}{1 \text{ gm/cc}} = 1.933$$

iii) $$\text{shrinkage limit} = \frac{\text{mass of water at shrinkage limit}}{\text{mass of solid}}$$

$$= \frac{26}{174}$$

$$= 0.1494$$

$$\boxed{\omega_s = 14.94\%}$$

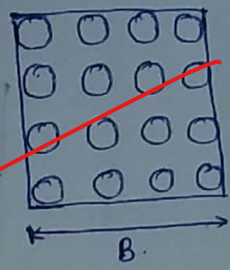
1(c)(i)

Unconfined Compressive strength $= q_u = 60\ kN/m^2$

$\therefore$ Undrained Cohesion $= C_u = \frac{q_u}{2} = \frac{60}{2} = 30\ kN/m^2$

diameter of pile $= D = 30cm = 0.3m$

spacing of pile $= S = 90cm = 0.9m$.



$\therefore B = 3S + D = 3 \times 0.9 + 0.3 = 3m$.

Depth of fill $= L = 3m$.

Adhesion factor $\alpha = 0.6$.

$\gamma = 15\ kN/m^3$

$\therefore$ Negetive frictional Load $= max^m \begin{cases} \text{Negative skin friction on pile} \times \text{no. of piles} \\ \text{Negative skin friction on block + weigh to soil block.} \end{cases}$

Negetive skin friction on pile $= \alpha C_u L (\pi D)$

$= 0.6 \times 30 \times 3 \times \pi \times 0.3$

$= 50.89\ KN$

Negative skin friction on block $= C_u L\ (4B)$

$= 30 \times 3 \times (4 \times 3)$

$= 1080\ KN$

Weight of soil block $= \gamma L B^2$

$= 15 \times 3 \times 3^2 = 405\ KN$.

$\therefore$ Negative frictional Load $= max^m \begin{cases} 50.89 \times 16 = 814.24\ KN \\ 1080 + 405 = 1485\ KN \end{cases}$

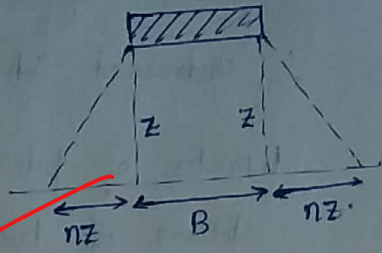
$= 1485\ KN$.

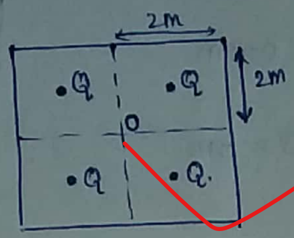
1 (c) (ii)

Raft square footing.

B = width of square footing = 4m.

Z = depth below central area = 4m





Q = Load on each square = 200 kN/m^2 × $2 \times 2\ m^2$ = 800 kN.

Using Bossinesque Equation.

σ = stress below O due to one Q.

$$= \frac{3Q}{2\pi Z^2}\left[\frac{1}{1+\frac{r^2}{Z^2}}\right]^{5/2}$$

$$= \frac{3 \times 800}{2\pi \times 4^2}\left[\frac{1}{1+\left(\frac{\sqrt{2}}{4}\right)^2}\right]^{5/2}$$

$= 17.78\ kN/m^2$

∴ stress due to all four Q at O is $= 4 \times \sigma$

$= 4 \times 17.78$

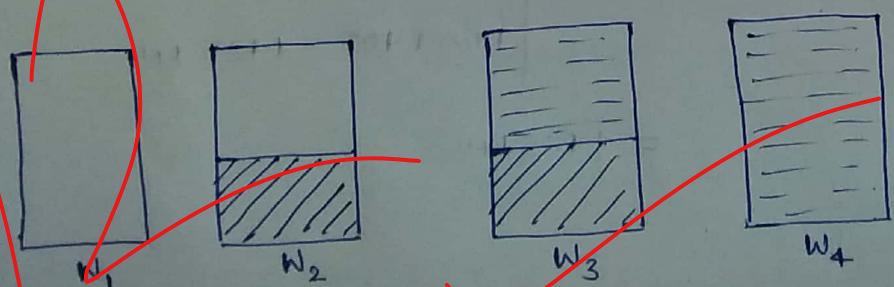
$= 71.14\ kN/m^2$

∴ Increament in stress = 71.14 − 200

$= -128.86\ kN/m^2$

−ve sign indicates decreament in stress.

1(d)



Let W_1 = weight of pycnometer.

W_2 = weight of pycnometer + weight of soil (dry)

W_3 = weight of pycnometer + soil + water.

W_4 = weight of pycnometer + Water.

W_s = weight of solid $= W_2 - W_1$

$$\therefore \; W_4 = W_3 - W_s + \frac{W_s}{G\gamma_w} \times \gamma_w .$$ where G = specific gravity of solid.

$$\Rightarrow W_4 = W_3 - W_s + \frac{W_s}{G}$$

$$\Rightarrow W_3 - W_4 = W_s - \frac{W_s}{G}.$$

$$\Rightarrow W_s = (W_3 - W_4)\,\frac{G}{G-1}$$

Weight of wet soil $= W_2 - W_1$.

$\therefore$ Weight of water in soil = weight of wet soil − weight of soil solid

$$= (W_2 - W_1) - (W_3 - W_4)\,\frac{G}{G-1}$$

$$\therefore \text{ water content} = \frac{\text{weight of water}}{\text{weight of solid}}$$

$$= \frac{(W_2 - W_1) - (W_3 - W_4)\frac{G}{G-1}}{(W_3 - W_4)\frac{G}{G-1}}$$

$$w = \left[\frac{W_2 - W_1}{W_3 - W_4} \cdot \left(\frac{G-1}{G}\right) - 1\right]$$

$$\therefore \; \boxed{w = \left[\frac{(W_2 - W_1)}{(W_3 - W_4)}\left(\frac{G-1}{G}\right) - 1\right] \times 100\%}$$

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1 (c)

Compaction specification in field –

γ_d = 95% of 1.94 gm/cc = 1.843 gm/cc

OMC = 13.5% ± 2% = 11.5% to 15.5%

At Location ①.

Volume of soil = $\frac{39.61 \, gm}{1.87 \, gm/cc}$ = 20.86 cc

$\therefore$ dry density = $\frac{38.46 \, gm}{20.86 \, cc}$ = 1.843 gm/cc = γ_d

Bulk density $\gamma = \frac{42.86 \, gm}{20.86 \, cc}$ = 2.055 gm/cc

$\therefore \gamma_d = \frac{\gamma}{1+\omega} \Rightarrow 1.843 = \frac{2.055}{1+\omega} \Rightarrow \omega = 0.115 = 11.5\%$

$\left.\begin{array}{l}\gamma_d = 1.843 \text{ gm/cc} \\ \text{OMC} = \omega = 11.5\%\end{array}\right\}$ so, satisfied. as per specification

At location ②

Volume of soil = $\frac{32.19 \, gm}{1.87 \, gm/cc}$ = 17.21 cc

$\therefore$ dry density $\gamma_d = \frac{32.12 \, gm}{17.21 \, cc}$ = 1.866 gm/cc

Bulk density $\gamma = \frac{37.18 \, gm}{17.21 \, cc}$ = 2.16 gm/cc

$\therefore \gamma_d = \frac{\gamma}{1+\omega} \Rightarrow 1.866 = \frac{2.16}{1+\omega} \Rightarrow \omega = 0.1578 = 15.78\%$

$\therefore \left.\begin{array}{l}\gamma_d = 1.866 \text{ gm/cc} \\ \text{OMC} = 15.78\% > 15.5\%\end{array}\right\}$ Not satisfied as per specification.

3

2(a).

Given :- Square footing.

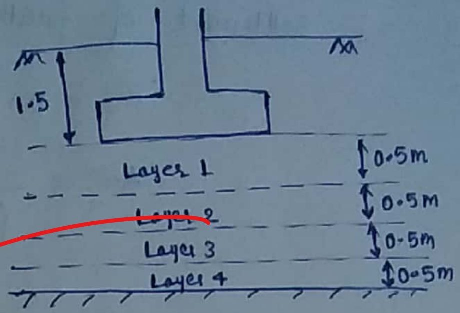
Width of footing $B = 1.5\ m$

Depth of footing $D_f = 1.5\ m$

$\gamma = 20\ kN/m^3$

$e = 0.8$

$C_c = 0.07$



Settlement of fist layer →

$$\bar{\sigma}_0 = 20 \times (1.5 + 0.25) = 35\ kN/m^2$$

$$\Delta\bar{\sigma} = \frac{225}{(1.5 + 0.5 + 0.5)^2} = 36\ kN/m^2$$

$$\therefore\ \Delta H_1 = C_c \frac{H_{01}}{1+e_0} \log\left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}\right)$$

$$= 0.07 \times \frac{0.5}{1+0.8} \log\left(\frac{35+36}{35}\right)$$

$$= 5.97\ mm$$

Settlement of second layer →

$$\bar{\sigma}_0 = 20\,(1.5 + 0.5 + 0.25) = 45\ kN/m^2$$

$$\Delta\bar{\sigma} = \frac{225}{(1.5 + 1.5 + 1.5)^2} = 11.11\ kN/m^2$$

$$\therefore\ \Delta H_2 = C_c \frac{H_{02}}{1+e_0} \log\left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}\right)$$

$$= 0.07 \times \frac{0.5}{1+0.8} \log\left(\frac{45 + 11.11}{45}\right)$$

$$= 2.05\ mm$$

Settlement of third layer →

$$\bar{\sigma}_0 = 20 \times (1.5 + 1.25) = 55\ kN/m^2$$

$$\Delta\bar{\sigma} = \frac{225}{(1.5 + 2.5 + 2.5)^2} = 5.33\ kN/m^2$$

$$\therefore\ \Delta H_3 = C_c \frac{H_{03}}{1+e_0} \log\left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}\right) = 0.07 \times \frac{0.5}{1+0.8} \log\left(\frac{55+5.33}{55}\right)$$

$$= 0.78\ mm$$

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Settlement of fourth layer →

$$\overline{\sigma_0} = 20 \times (1.5 + 1.75) = 65 \text{ kN/m}^2$$

$$\Delta\overline{\sigma} = \frac{225}{(1.5 + 3.5 + 3.5)^2} = 3.114 \text{ kN/m}^2$$

$$\therefore \Delta H_4 = C_c \frac{H_{04}}{1+e_0} \log\left(\frac{\overline{\sigma_0} + \Delta\overline{\sigma}}{\overline{\sigma_0}}\right)$$

$$= 0.07 \times \frac{0.5}{1+0.8} \log\left(\frac{65+3.114}{65}\right)$$

$$= 0.40 \text{ mm}.$$

$\therefore$ Total settlement $\Delta H = \Delta H_1 + \Delta H_2 + \Delta H_3 + \Delta H_4$.

$$= 5.97 + 2.05 + 0.78 + 0.40$$

$$= 9.2 \text{ mm}.$$

2(b)

Cell pressure $= \sigma_3$

Additional stress at failure = deviatoric stress $= \sigma_d$.

$$\therefore \sigma_1 = \sigma_3 + \sigma_d$$

2

(1)

For test (1)

$$\sigma_3 = 1000 \text{ kPa}.$$

$$\sigma_1 = 1000 + 1600 = 2600 \text{ kPa}.$$

$\therefore \sigma_1 = \sigma_3 N_\phi + 2c\sqrt{N_\phi}$ where $N_\phi = \tan^2\left(45 + \frac{\phi}{2}\right)$

$$2600 = 1000 N_\phi + 2c\sqrt{N_\phi} \quad \text{— (i)}$$

For test (2)

$$\sigma_3 = 1800 \text{ kPa}$$

$$\sigma_1 = 1800 + 2200 = 4000 \text{ kPa}.$$

$$\therefore \sigma_1 = \sigma_3 N_\phi + 2c\sqrt{N_\phi}$$

$$4000 = 1800 N_\phi + 2c\sqrt{N_\phi} \quad \text{— (ii)}$$

from (I) and (II)

$N_\phi = 1.75 \Rightarrow \tan^2\left(45+\frac{\phi}{2}\right) = 1.75 \Rightarrow \phi = 15.83^\circ$

$2c\sqrt{N_\phi} = 850 \Rightarrow c = 321.27$ kPa.

$\therefore$ Total stress Parameters are $c = 321.27$ kPa
$\phi = 15.83^\circ$

(II) For test (I)

$\bar{\sigma}_3 = 1000 - 400 = 600$ kPa.

$\bar{\sigma}_1 = 1000 + 1600 - 400 = 2200$ kPa.

$\therefore \bar{\sigma}_1 = \bar{\sigma}_3 N_{\phi'} + 2c'\sqrt{N_{\phi'}}$

$\Rightarrow 2200 = 600 N_{\phi'} + 2c'\sqrt{N_{\phi'}}$ — (III)

For test (2)

$\bar{\sigma}_3 = 1800 - 800 = 1000$ kPa.

$\bar{\sigma}_1 = 1800 + 2200 - 800 = 3200$ kPa.

$\therefore 3200 = 1000 N_{\phi'} + 2c'\sqrt{N_{\phi'}}$ — (IV)

from (III) and (IV)

$N_{\phi'} = 2.5 \Rightarrow \tan^2\left(45+\frac{\phi'}{2}\right) = 2.5 \Rightarrow \phi' = 25.38^\circ$

$2c'\sqrt{N_{\phi'}} = 700 \Rightarrow c' = 221.34$ kPa.

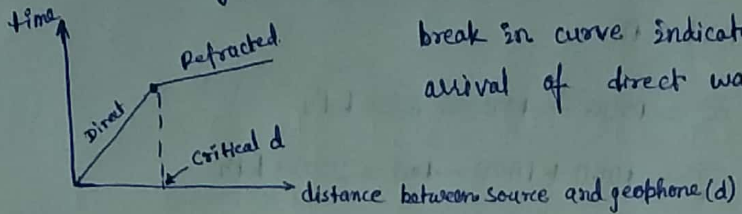
$\therefore$ Effective stress parameters are $c' = 221.34$ kPa.
$\phi' = 25.38^\circ$

2(c) (i)

Methods of Soil exploration (Geophysical)

i) Seismic wave refration method.

It is based on the principle that ~~diff~~ seismic wave have different velocity in different soil types. It consists of source and geophone. Source emit the seismic wave and geophone records the direct wave and refracted. When the wave crosses the boundary between different soil, the wave get refracted.



break in curve indicates the simultaneous arrival of direct wave and refracted wave.

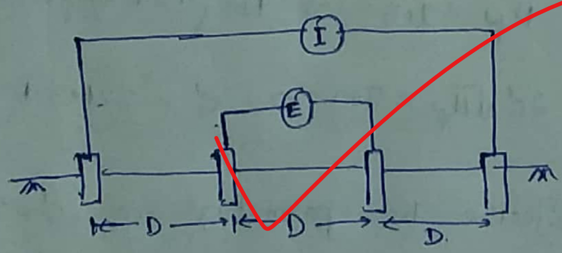
The velocity of wave become faster when the soil is denser. It is a quick and reliable method of establishing the soil profile of different strata.

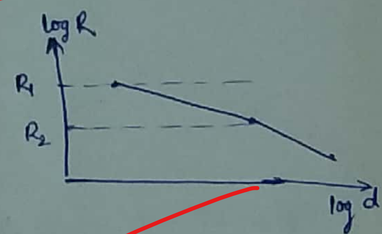
$$\text{Depth of Strata } D = \frac{d}{2}\sqrt{\frac{V_2-V_1}{V_2+V_1}}$$

$d \rightarrow$ distance between source and geophone.
$V_2 \rightarrow$ wave velocity in lower strata.
$V_1 \rightarrow$ wave velocity in upper strata.

ii) Electrical Resistivity Method.

It is based on the principle that different soil has different resistivity and so has different resistance to flow of electric current. It can be used below or above the water table.





If the upper strata has low resistivity and low resistance then current flow close to the ground and hence a large potential drop occurs. In this method, the distance between increased or decreased to find the break in log-log curve of resistance. Apparant resistance is obtained by recording potential difference and current for different values of D. Break in resistance curve show change in resistance and hence change in soil profile.

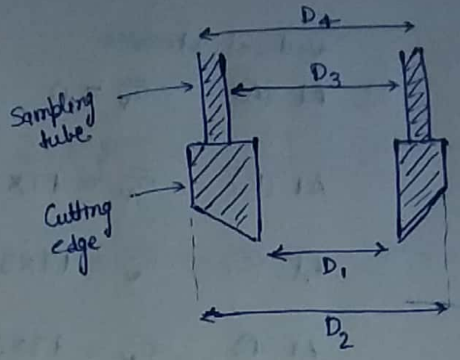
2(c) (II)

Given :- $D_1 = 69$ mm.

$D_2 = 73$ mm

$D_3 = 70$ mm

$D_4 = 72$ mm.



(I) Inside clearence $= \dfrac{D_3 - D_1}{D_1} \times 100$

$= \dfrac{70 - 69}{69} \times 100$

$= 1.45\%$

(II) Outside clearence $= \dfrac{D_2 - D_4}{D_4} \times 100$

$= \dfrac{73 - 72}{72} \times 100$

$= 1.39\%$

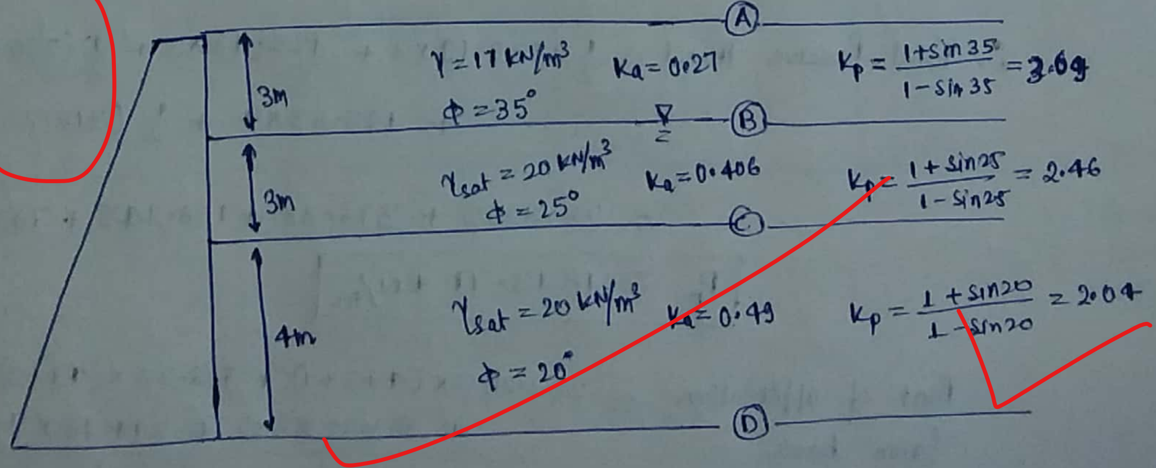
(III) Area ratio $= \dfrac{D_2^2 - D_1^2}{D_1^2} \times 100$

$= \dfrac{73^2 - 69^2}{69^2} \times 100$

$= 11.93\%$

As per IS Code Area ratio for soft sensitive soil should be less than 10%.

3(a)



Vertical stresses

At (A) $\bar{\sigma}_v = 0$

At (B) $\bar{\sigma}_v = 17 \times 3 = 51 \text{ kN/m}^2$

At (C) $\bar{\sigma}_v = 17 \times 3 + (20 - 9.81) \times 3 = 81.57 \text{ kN/m}^2$

At (D) $\bar{\sigma}_v = 17 \times 3 + (20 - 9.81) \times 3 + (20 - 9.81) \times 4 = 122.33 \text{ kN/m}^2$

Passive earth pressure.

At (A) $p_p = 0$

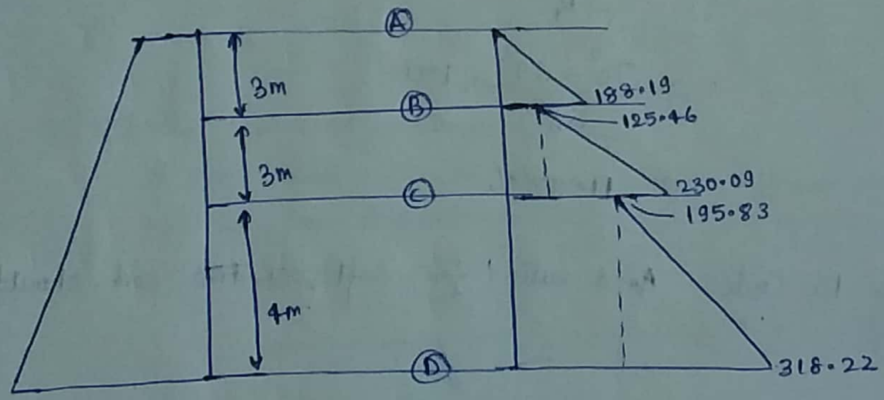
At (B) just above (B) $p_p = K_p \bar{\sigma}_v = 3.69 \times 51 = 188.19 \text{ kN/m}^2$

At just below (B) $p_p = 2.46 \times 51 = 125.46 \text{ kN/m}^2$

At just above (C) $p_p = 2.46 \times 81.57 + 9.81 \times 3 = 230.09 \text{ kN/m}^2$

At just below (C) $p_p = 2.04 \times 81.57 + 9.81 \times 3 = 195.83 \text{ kN/m}^2$

At (D) $p_p = 2.04 \times 122.33 + 9.81 \times 7 = 318.22 \text{ kN/m}^2$



$\therefore$ Total Passive Thrust $= \frac{1}{2} \times 188.19 \times 3 + 125.46 \times 3 + \frac{1}{2}(230.09 - 125.46) \times 3 + 195.83 \times 4 + \frac{1}{2}(318.22 - 195.83) \times 4$

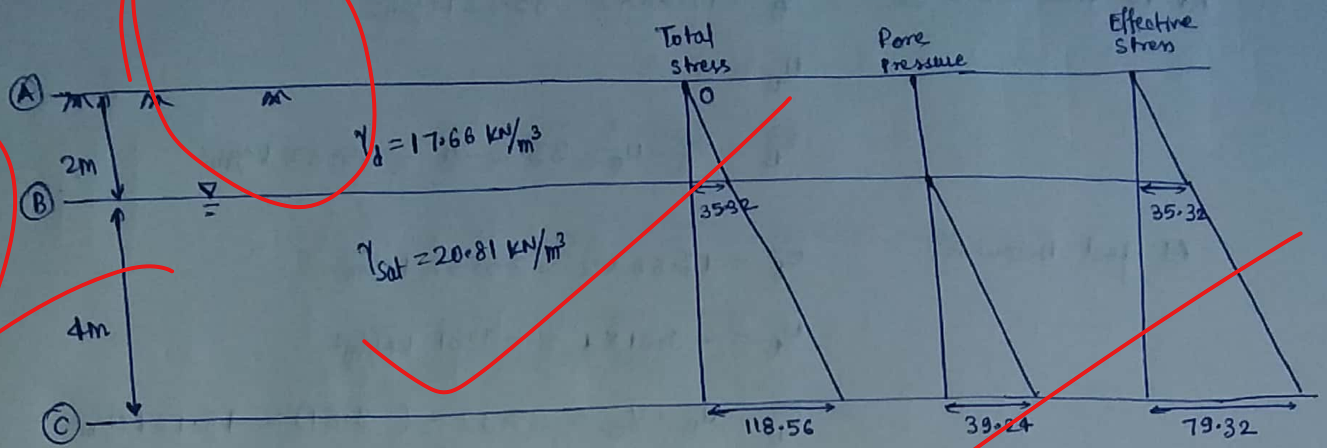
$= 282.285 + 376.38 + 156.945 + 783.32 + 244.78$

$$\boxed{P_p = 1843.71 \text{ kN/m}}$$

Point of application from base $= \dfrac{282.285 \times (4+3+1) + 376.38 \times (4+1.5) + 156.945 \times (4+1) + 783.32 \times (2) + 244.78 \times 4/3}{1843.71}$

$\boxed{z = 3.8 \text{ m}}$ above base

3(b).



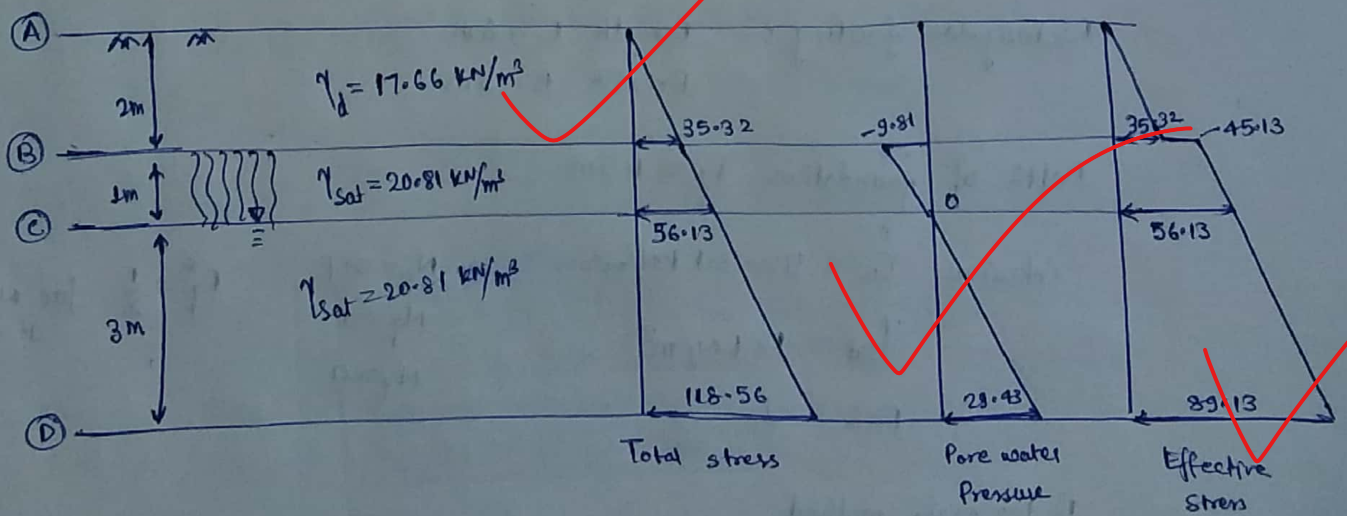
$$\gamma_d = \gamma_w \frac{G}{1+e} \Rightarrow 17.66 = 9.81 \times \frac{2.65}{1+e} \Rightarrow e = 0.472$$

$$\therefore \gamma_{sat} = \gamma_w \frac{G+e}{1+e} = 9.81 \times \frac{2.65+0.472}{1+0.472} = 20.81\ kN/m^3$$

At (A) $\quad \sigma_A = 0 \quad u_A = 0 \quad \bar{\sigma}_A = \sigma_A - u_A = 0$

At (B) $\quad \sigma_B = 17.66 \times 2 = 35.32\ kN/m^2 \quad u_B = 0 = 0 \quad \bar{\sigma}_B = \sigma_B - u_B = 35.32\ kN/m^2$

At (C) $\quad \sigma_C = 17.66 \times 2 + 20.81 \times 4 = 118.56\ kN/m^2 \quad u_C = 9.81 \times 4 = 39.24\ kN/m^2 \quad \bar{\sigma}_C = \sigma_C - u_C = 118.56 - 39.24 = 79.32\ kN/m^2$



At (A) $\quad \sigma_A = 0$

$u_A = 0$

$\bar{\sigma}_A = \sigma_A - u_A = 0.$

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At (B) just above (B) $\sigma_B = 17.66 \times 2 = 35.32\ kN/m^2$

$u_B = 0,$

$\bar{\sigma}_B = \sigma_B - u_B = 35.32 - 0 = 35.32\ kN/m^2$

At just below (B) $\sigma_B = 17.66 \times 2 = 35.32\ kN/m^2$

$u_B = -9.81 \times 1 = -9.81\ kN/m^2$

$\bar{\sigma}_B = \sigma_B - u_B = 35.32 - (-9.81) = 45.13\ kN/m^2$

At (C) $\sigma_C = 17.66 \times 2 + 20.81 \times 1 = 56.13\ kN/m^2$

$u_C = 0$

$\bar{\sigma}_C = \sigma_C - u_C = 56.13 - 0 = 56.13\ kN/m^2$

At (D) $\sigma_D = 17.66 \times 2 + 20.81 \times 4 = 118.56\ kN/m^2$

$u_D = 9.81 \times 3 = 29.43\ kN/m^2$

$\bar{\sigma}_D = \sigma_D - u_D = 118.56 - 29.43 = 89.13\ kN/m^2$

So, due to drop of water level there is no increase or effect on total stress but the effective stress get increased and cause settlement.

3 (c) (i)

12

• Rectangular footing :- Length $L = 6\ m$

Breadth $B = 3\ m$

Depth of foundation $D_f = 1.5\ m$.

Cohesion $C_u = 52.34\ kN/m^2$ $N_c = 5.7$ $R_q^* = \frac{1}{2}$ for water table at GL.

$\gamma_{sat} = 18\ kN/m^3$. $N_q = 1$

$N_\gamma = 0$

FOS = 3.

i) Terzaghi method.

$$q_u = \left(1 + 0.3\frac{B}{L}\right) C N_c + R_q^* \gamma D_f N_q + R_\gamma^* \frac{1}{2}\left(1 - 0.2\frac{B}{L}\right)\gamma B N_\gamma$$

$$= \left(1 + 0.3 \times \frac{3}{6}\right) \times 52.34 \times 5.7 + R_q^* \times 18 \times 1.5 \times 1 + 0$$

$$= 343.09 + \frac{1}{2} \times 27$$

$$= 356.59\ kN/m^2$$

$\therefore q_{nu} = q_u - \gamma' D_f$.

$= 356.59 - (18-9.81) \times 1.5$

$= 344.31 \ kN/m^2$

$\therefore q_{ns} = \frac{q_{nu}}{Fos} = \frac{344.31}{3} = 114.77 \ kN/m^2$

$$\therefore \boxed{\begin{aligned} q_u &= 356.59 \ kN/m^2 \\ q_{na} &= 114.77 \ kN/m^2 \end{aligned}}$$

ii) Skempton Method

$q_{nu} = c N_c$.

For $\frac{D_f}{B} = \frac{1.5}{3} = 0.5$

$N_c = 5 \left[1 + 0.2 \frac{B}{L}\right] \left[1 + 0.2 \frac{D_f}{B}\right]$

$= 5 \left[1 + 0.2 \times \frac{3}{6}\right] \left[1 + 0.2 \times \frac{1.5}{3}\right]$

$= 6.05$

$\therefore q_{nu} = 52.34 \times 6.05 = 316.66 \ kN/m^2$

$\therefore q_u = q_{nu} + \gamma' D_f = 316.66 + (18-9.81) \times 1.5$

$= 328.95 \ kN/m^2$

$$\boxed{q_u = 328.95 \ kN/m^2}$$

$q_{ns} = \frac{q_{nu}}{Fos} = \frac{316.66}{3} = 105.55 \ kN/m^2$

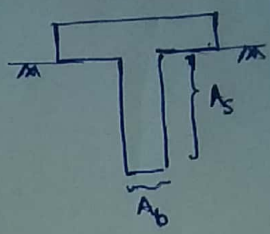
$$\boxed{q_{na} = 105.55 \ kN/m^2}$$

3(c) (ii) Methods of estimation of pile load capacity -

i) Static pile load capacity formula –

For granular soil $Q_s = K \, \bar{\sigma}_{avg} \cdot L A_s$

For cohesion soil $Q_s = C_u A_b N_c + \alpha C_u A_s$



Negative skin friction will also be considered if there is recent loose fill or lowering of water table.

ii) Pile load test →

It is the most reliable method because the capacity is determined for insitu pile. It is easy out to determine pile load by two way -

a) Initial test → Load is applied on test pile and pile load with settlement is obtained.

b) Routine test → Load is working load that is applied on working pile and the settlement value is obtained for the working loads.

iii) Pile load capacity by dynamic formula.

It is based on conservation of energy principle. Energy given to the pile is equal to energy lost in settlement or driving into the soil.

a) Engineer's New formula. $Q_p = \dfrac{WH}{FOS\,(S+C)}$

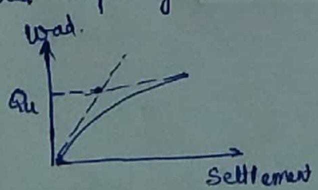
W → wt of hammer
H → height of fall.
S → settlement for last 30 cm blow

b) Hilly Formula $Q_p = \dfrac{Wh\,\eta}{\left(S + \frac{C}{2}\right)}$

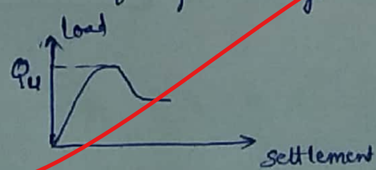
η → efficiency of blow.

iv) Correlation between pile load and settlement.

Load settlement curve is obtained from test and from this curve load capacity is calculated. Mainly two types of Load-settlement curve.



No peak



Peak

$\therefore$ Pile load capacity = minimum $\begin{cases} 50\% \text{ of Load at which settlement} = \frac{1}{10} \times \text{dia of pile} \\ \frac{2}{3} \text{ of load at which settlement} = 12 \text{ mm} \end{cases}$

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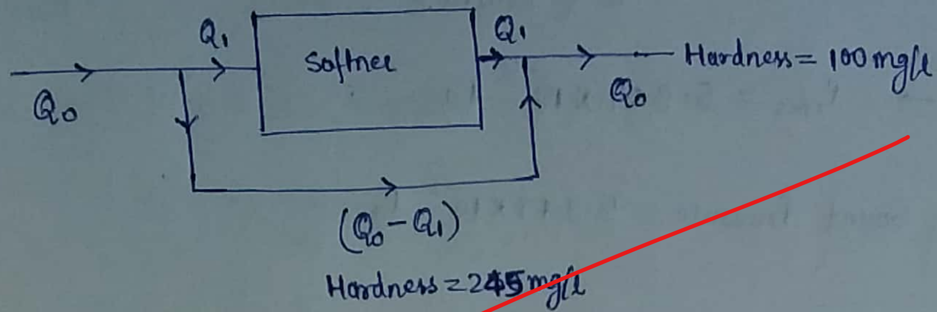
5(a)



6

$Q_0 = 1500$ lit/day.

Let Q_1 = flow of water through Softner.

$$\therefore \quad 100 = \frac{Q_1 \times 0 + (Q_0 - Q_1) \times 245}{Q_0}$$

$$\Rightarrow \quad 100\, Q_0 = 245\, Q_0 - 245\, Q_1$$

$$\Rightarrow \quad Q_1 = \frac{145\, Q_0}{245} = \frac{145}{245} \times 1500 = 887.76 \text{ lit/day.}$$

$\therefore$ Total hardness removed $= 887.76$ lit/day $\times$ 245 mg/l.

$$= 0.2175 \text{ kg/day.}$$

$\therefore$ Time between regeneration cycle $= \dfrac{46 \times 0.07 \text{ kg}}{0.2175 \text{ kg/day}} = 14.8$ days.

5(b)

4

Sound Power $W = 0.001$ Watt $= 10^{-3}$ Watt

$\therefore$ Sound Intensity $I = \dfrac{W}{A} = \dfrac{10^{-3}}{\pi \times 6^2}$ Watt/m^2 $= 8.842 \times 10^{-6}$ Watt/m^2

Sound Intensity level $L = 10 \log\left(\dfrac{I}{I_0}\right) = 10 \log\left(\dfrac{8.842 \times 10^{-6}}{10^{-12}}\right)$

$$= 69.47 \text{ dB.}$$

$\therefore$ Sound Pressure level $L = 69.47$ dB.

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$$\therefore L_p = 20 \log \left(\frac{P_{rms}}{20\mu Pa} \right)$$

$$\Rightarrow 69.47 = 20 \log \left(\frac{P_{rms}}{20\mu Pa} \right)$$

$$\Rightarrow P_{rms} = 5.947 \times 10^{-2} \text{ Pa}$$

$\therefore$ Sound Pressure $= 5.947 \times 10^{-2}$ Pa.

Also, Sound Level $L_w = 10 \log_{10} \left(\frac{W}{10^{-12}} \right)$

$$= 10 \log_{10} \left(\frac{10^{-3}}{10^{-12}} \right)$$

$$= 90 \text{ dB}.$$

5(C) (1)

Year	Population	Arithmatic Increase
1930	25000	
		+ 3000
1940	28000	
		+ 6000
1950	34000	
		+ 8000
1960	42000	
		+ 5000
1970	47000	

$\therefore$ Arithmatic mean $= \frac{3000 + 6000 + 8000 + 5000}{4} = 5500$

$P_{1980} = 47000 + 5500 = 52500$

$P_{1990} = 47000 + 2 \times 5500 = 58000$

$P_{2000} = 47000 + 3 \times 5500 = 63500$

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5(c) (ii)

Year	Population
1930	25000
1970	47000

Let r = rate of increase of population

$$P_n = P_0\left(1+\frac{r}{100}\right)^n$$

$$\Rightarrow P_{1970} = P_{1930}\left(1+\frac{r}{100}\right)^4$$

$$\Rightarrow 47000 = 25000\left(1+\frac{r}{100}\right)^4$$

$$\Rightarrow r = 17.1\ \%$$

$$\therefore P_{2000} = P_{1970}\left(1+\frac{r}{100}\right)^n$$

$$\Rightarrow P_{2000} = 47000\left(1+\frac{17.1}{100}\right)^3$$

$$\boxed{P_{2000} = 75469}$$

$$P_{2006} = P_{2000}\left(1+\frac{r}{100}\right)^{0.6}$$

$$\Rightarrow P_{2006} = 75469\left(1+\frac{17.1}{100}\right)^{0.6}$$

$$\boxed{P_{2006} = 82967}$$

5(d).

Radius of well $= r_w = \frac{0.6}{2} = 0.3m$

Permeability of aquifer $k = 50\ m/day$

storage Coefficient $S_c = 0.22$

$b = 20m$.

$\therefore$ Transmissibility $T = kb = 50\times 20\ m^2/day = 1000\ m^2/day$.

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drawdown $S = \frac{Q}{4\pi T}\left[\ln \frac{4Tt}{r^2 S} - 0.5772\right]$

$$\Rightarrow \quad 2.5 = \frac{Q}{4\pi\, 1000}\left[\ln \frac{4 \times 1000 \times 1}{25^2 \times 0.22} - 0.5772\right]$$

$$\Rightarrow \quad Q = 11247.19\ m^3/day. \quad = 130.2\ \text{litre/sec}.$$

For Q = Constant. If $t = 1\,year = 365\,days$.

$$S = \frac{Q}{4\pi T}\left[\ln \frac{4Tt}{r^2 S} - 0.5772\right]$$

$$= \frac{11247.19}{4\pi \times 1000}\left[\ln \frac{4 \times 1000 \times 365}{25^2 \times 0.22} - 0.5772\right]$$

$$= 7.78\ \text{metres}.$$

5(e)

Methods of defluoridation →

i) Prashanti technque.

- Activated alumina or activated carbon is added to water that absorb fluoride from the water.
- It is carried out under acidic medium.

ii) Nalconda Technique

- Alum is added to water to remove fluoride from the water.

$$\text{Water} + CaO/Na_2CO_3 + \text{Alum} \longrightarrow \text{Sediment presipitate}$$

- After defluoridisation, the water require filteration to settle the sediments. and it take approximately 1 hour to settle.
- It is carried out under alkaline medium.

iii) Ion adsorption method

- Zeolite is used in the form of chloride to remove floride.
- Fluoride is used by ion-exchange between floride and chlorine ion. and discharge a fluoride free water.

iv) Reverse Osmosis method -

- A semi-permeable membrane is used to remove highly concentrated fluoride water to pure water.
- This method also removes other salts along with fluoride.

7 (a)

$$FeSO_4 \cdot 7H_2O + Ca(HCO_3)_2 \longrightarrow Fe(HCO_3)_2 + CaCO_3 + 7H_2O$$

$$Fe(HCO_3)_2 + 2Ca(OH)_2 \longrightarrow Fe(OH)_2 + 2CaCO_3 + 2H_2O$$

$$4Fe(OH)_2 + O_2 + 2H_2O \longrightarrow 4Fe(OH)_3 \downarrow$$

(a) 1 mole of $FeSO_4 \cdot 7H_2O$ reacts with 1 mole of $Ca(HCO_3)_2$ and 1 mole of $Ca(HCO_3)_2$ is equivalent to 1 mole of $CaCO_3$.

$\therefore$ 1 mole of $FeSO_4 \cdot 7H_2O$ require 1 mole $CaCO_3$

$\therefore$ 278 gm of $FeSO_4 \cdot 7H_2O$ require 100 gm $CaCO_3$

$\therefore$ 30 gm/m³ of $FeSO_4 \cdot 7H_2O$ require $\frac{100}{278} \times 30$ gm/m³ as $CaCO_3$

$= 10.79 \frac{gm}{m^3}$ as $CaCO_3$

$$\boxed{\therefore \text{Alkalinity} = 10.79 \frac{gm}{m^3} \text{ as } CaCO_3}$$

(b) 1 mole of $FeSO_4 \cdot 7H_2O$ require 2 moles of $Ca(OH)_2$ and 2 mole of $Ca(OH)_2$ is equivalent to 2 moles of CaO.

$\therefore$ 1 mole of $FeSO_4 \cdot 7H_2O$ require 2 mole CaO

278 gm of $FeSO_4 \cdot 7H_2O$ require 2×56 gm CaO

$\therefore$ 30 $\frac{gm}{m^3}$ of $FeSO_4 \cdot 7H_2O$ require $\frac{2 \times 56}{278} \times 30 \frac{gm}{m^3}$ CaO

$= 12.086 \frac{gm}{m^3}$ CaO

$\therefore$ Lime required $= \frac{12.086}{0.7}$ gm/m³ $= 17.266$ gm/m³

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(c) 1 mole $FeSO_4 \cdot 7H_2O$ require $\frac{1}{4}$ mole of O_2

$\therefore$ 278 gm $FeSO_4 \cdot 7H_2O$ require $\frac{1}{4} \times 32$ gm of O_2

$\therefore$ 30 gm/m^3 $FeSO_4 \cdot 7H_2O$ require $\frac{\frac{1}{4} \times 32}{278} \times 30 \ gm/m^3$ of O_2

$= 0.863 \ gm/m^3$ of O_2

$\therefore \boxed{O_2 \text{ required} = 0.863 \ gm/m^3}$

7(b)

Rate of flow $= Q = 4.5$ MLD (Assuming average flow)

Assuming 5% for backwash in 30 minutes.

Design discharge $Q_0 = (1 + 0.05) \times 1.8 \times 4.5$ MLD

$= 8.505 \text{ MLD} = \frac{8.505 \times 10^6}{23.5 \times 60}$ lit/min $= 6032$ Lit/min

Assuming the rate of flow $= 150$ Litre/m^2/min

$\therefore$ Surface area of rapid filter $= \frac{6032}{150} \ m^2 = 40.21 \ m^2$

Number of filter required $= 1.22\sqrt{Q_0} = 1.22\sqrt{8.505}$

$= 3.56$

$\therefore$ Provide 4 unit + 1 unit (stand by)

$\therefore$ Surface area of each Rapid Sand filter $= \frac{40.21}{4} \ m^2$

$= 10.05 \ m^2$

Assuming $\frac{L}{B} = 2$.

$\Rightarrow L \times B = 10.05$.

$\Rightarrow 2B^2 = 10.05$.

$\therefore B = 2.24$ m

$\therefore$ size of filter Length $= 2 \times 2.25 \text{ m} = 4.5$ m

Breadth $= 2.25$ m

Depth of filter $= 3$m

No. of Units $= 4 + 1$

Design of drainage system –

Assuming diameter of perforation $= 1.3$ cm $= 13$ mm

Spacing of perforation $= S_p = 20$ cm c/c

Spacing of Laterals $= S_L = 30$ cm c/c

$\therefore$ Number of Laterals $= 2\left(\frac{L}{S_L}+1\right) = 2\left(\frac{4.5}{0.3}+1\right) = 32$

Cross sectional area of all perforation $= 0.2\%$ of filter area.

$= \frac{0.2}{100} \times 2.25 \times 4.5 \ m^2$

$= 202.5 \ cm^2$

Cross sectional area of one lateral $= 2 \times$ Cross sectional area of perforation in one lateral

$= 2 \times \frac{202.5}{32}$

$= 12.7 \ cm^2$

If $D_L =$ dia of Lateral

$\therefore \frac{\pi}{4} \times D_L^2 = 12.7 \Rightarrow D_L = 4$ cm

$\boxed{\text{Provide dia of lateral} = 4\text{cm}}$

Cross sectional area of manifold $= 2 \times$ cross sectional area of all laterals

$= 2 \times 32 \times \frac{\pi}{4} \times 4^2$

$= 804.2 \ cm^2$

If $D_M =$ diameter of manifold

$\therefore \frac{\pi}{4} D_M^2 = 804.2 \Rightarrow D_M = 32$ cm

$\boxed{\text{Provide diameter of manifold} = 32\text{cm}}$

No. of perforations $= \frac{202.5}{\frac{\pi}{4} \times 13^2} = 153$

$\therefore$ No. of perforations in each Lateral $= \frac{153}{32} = 4.77$

$\boxed{\text{Provide 5 no. of perforation in each lateral}}$

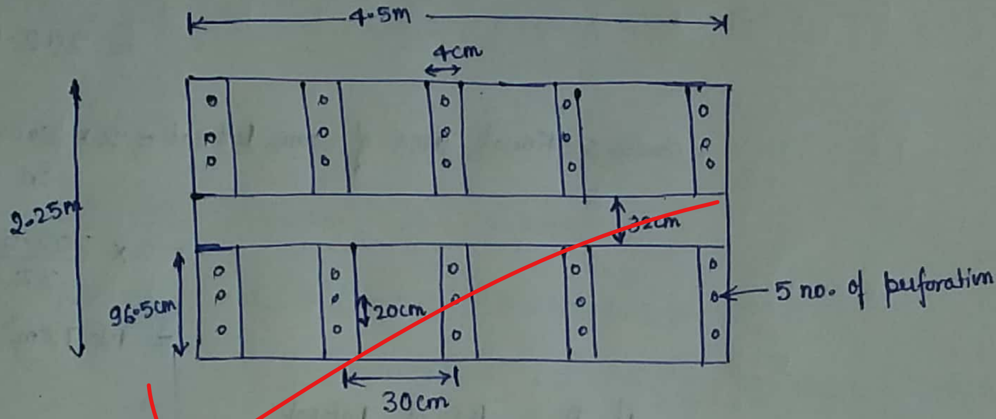
Check

Length of Lateral $L_L = \frac{B - D_M}{2} = \frac{2.25 - 0.32}{2} = 96.5\ cm$

$\boxed{\text{Provide length of each lateral} = 96.5\ cm}$

$\therefore \frac{L_L}{D_L} = \frac{96.5}{4} = 24.125 < 60$ ok

Since $\frac{L_L}{D_L} < 60$ so, design is fine.



Assuming 45 cm/min of backwash ~~discharge~~ velocity.

Backwash discharge $= \frac{0.45}{60} \times (4 \times 2.25 \times 4.5)$

$= 0.30375\ m^3/s.$

$= 303.75$ Litres/sec.

Provide Sand layer depth = 90 cm.

Gravel layer depth = 3 − 0.9 = 2.1 m.

Use sand of specific gravity 2.65.

and $\frac{D_{60}}{D_{10}} = 1.5.$

and $D_{10} = 0.5\ mm.$

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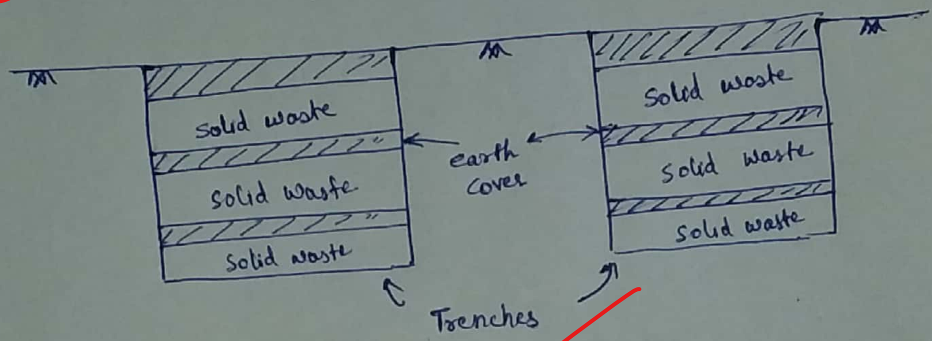
7(c).

Methods of Landfills and their operations.

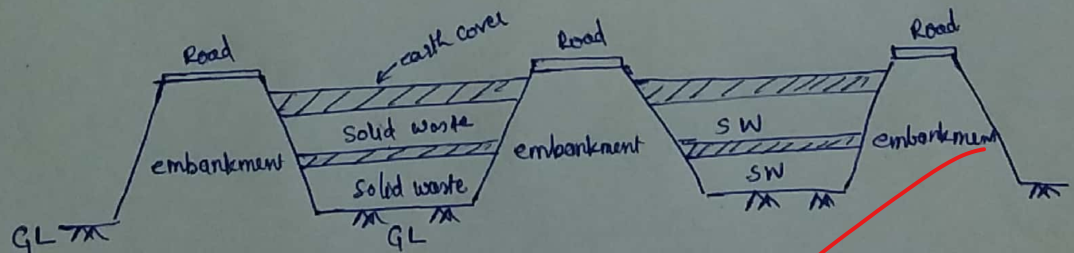
i) Trench method →

In this method a trench is excavated for landfilling. This method is adopted when water table is sufficiently low. The solid wastes are disposed in these trenches layer by layer and the layers are separated by earth covers. At top a thick layer of earth cover is provided. So, a sufficient amount (volume) of earth cover should also be available at landfilling sites.



ii) Area Method →

This method is adopted when water table is high and may affect the landfilling by trench method. Entire area of landfill sites are divided into a number of subareas called 'cells' separated by embankments or roads. The solid wastes are disposed in these cells layer by layer separated by earth covers.



iii) Depression method.

This method is adopted when a natural or artificial depression is available for landfilling such as cryons, borrow pits etc. The solid wastes are disposed in these depressions from head and ends at mouth of depression. to prevent the accumulation of water behind the landfill. The solid waste is pushed against the head end so, a high degree of compaction of landfill is achieved.