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Detailed Solutions

ESE-2024
Mains Test Series

Civil Engineering
Test No : 1

Section A : Geo-technical & Foundation Engineering

Q.1 (a) Solution:

Compaction for a given type of soil is a function of the following factors:

(i) Water content (ii) Compactive effort (iii) Method of compaction

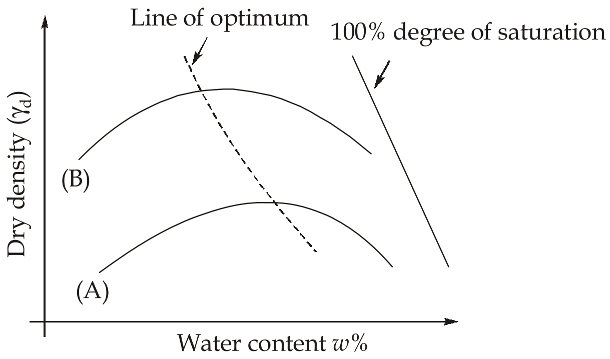
(i) **Water content** : Moisture content and dry unit weight relationship is explained with the help of lubrication theory. At low water contents, the soil is stiff and the soil grains offer more resistance to compaction. As the water content increases, the particles develop larger and larger water films around them which tend to lubricate the particles and make them easier to be worked around, to move closer into a denser configuration, resulting in a higher dry unit weight and lower air voids. The dry unit weight continues to increase till the optimum moisture content is reached, a stage when the lubrication effect is maximum. With further increase in moisture content, however the water starts to replace the soil particles and since unit weight of water is very less than that of soil solid, the dry unit weight starts decreasing.

- Another concept was given by Lambe which uses soil structure and the electrical double layer theory to explain the effect of water content on dry unit weight of soil. In case of cohesive soils, there is an attractive force namely the Van-der Waal's forces which act between two soil particles and a repulsive force which is due to double layers of adsorbed water tending to come into contact with each other. While the attractive forces remain same in magnitude, the repulsive forces is directly related to the size of double layers. If the net force between the particles is attractive, flocculated structure is the result. If it is repulsive, the particles tend to move away-

'disperse'. At low water contents, attractive forces are predominant which makes it difficult for the particles to move about when compactive effort is applied. A low dry unit weight is the consequence. As the water content increases, the double layer expands and inter particle repulsive forces increase. The particles easily slide over one another and get packed more closely, resulting in higher dry unit weight.

- The maximum expansion of the double layer is at the OMC. Beyond that, the addition of water does not add any further to the expansion of double layer but the water tends to occupy space which otherwise would have been occupied by the soil particles resulting in a decrease in unit weight.

(ii) Compactive Effort: For a given type of compaction, higher is the compactive effort, the higher will be the maximum dry unit weight and lower will be the optimum moisture content.



As shown in the graph, compaction curve B corresponds to the higher compactive effort in the modified Proctor test. Comparing it with the compaction curve A for the standard Proctor test, it can be seen that the compaction curve shifts to the top and to the left when the compactive effort is increased. However, as the moulding water content increases, the influence of compactive effort on dry unit weight tends to diminish.

(iii) Method of compaction: Ideally, the laboratory test must resemble a given field compaction procedure, because the mode of compaction does influence somewhat the shape and the position of the dry unit weight γ_d versus water content w plot. Since the field compaction is essentially a kneading type compaction or rolling type compaction and the laboratory tests use the dynamic-impact type compaction, it is expected to get some divergence in the OMC and $\gamma_{d(max)}$ values in the two cases.

Q.1 (b) Solution:

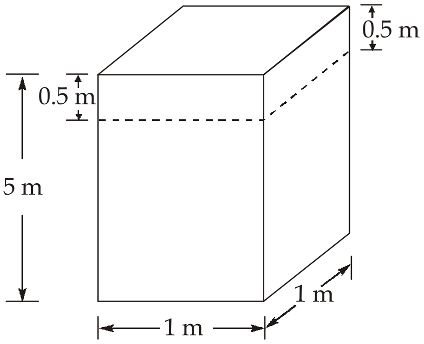
Given, $\gamma_{d\,(\text{in-situ})} = 14 \text{ kN/m}^3$

$\gamma_{d\,(\text{min})} = 10.8 \text{ kN/m}^3$

$$\text{Relative density, } (D_r) = 0.85 \text{ or } 85\%$$

where 'γ_d' is dry unit weight of sand.

(i) Consider a prism of soil of 5 m thickness, 1 m width and 1 m length in the direction perpendicular to the plane of paper.



Now, $\gamma_d = \dfrac{W_s}{V} = 14 \text{ kN/m}^3$

where W_s and V are weight of soil solids and volume of soil respectively.

Now, $V = 5 \times 1 \times 1 = 5 \text{ m}^3$

So, $W_s = 14 \times 5 = 70 \text{ kN}$

After compaction,

$$\text{Volume, } V_C = 4.5 \times 1 \times 1 = 4.5 \text{ m}^3$$

$$\gamma_{d(c)} = \frac{W_s}{V_C} = \frac{70}{4.5} = 15.56 \text{ kN/m}^3$$

where $\gamma_{d(c)}$ is dry unit weight of compacted soil.

(ii) $$\text{Relative density, } D_r = \frac{\gamma_{d\max}}{\gamma_d} \times \frac{\gamma_{d(c)} - (\gamma_d)_{\min}}{(\gamma_d)_{\max} - (\gamma_d)_{\min}}$$

$$\Rightarrow \quad 0.85 = \frac{\gamma_{d\max}}{15.56} \times \frac{(15.56 - 10.8)}{\gamma_{d\max} - 10.8}$$

$$\Rightarrow \quad \gamma_{d\max} = 16.85 \text{ kN/m}^3$$

(iii) An ideal compaction would produce a dry unit weight equal to $(\gamma_d)_{\max}$. i.e., 16.85 kN/m^3.

Now, $\gamma_{d\max} = \dfrac{W_s}{V_{\min}}$

where $V_{\min}$ is the minium volume of soil after compaction.

$$\Rightarrow \quad V_{\min} = \frac{70}{16.85} = 4.154 \text{ m}^3,$$

$$\therefore \quad \text{Thickness of soil, } H' = \frac{4.154}{1 \times 1} = 4.154 \text{ m}$$

Hence, maximum possible settlement of surface = 5 – 4.154 = 0.846 m

Q.1 (c) Solution:

For the layer of 5 m thickness,

$$\text{Degree of consolidation, } U = 50\%$$

$$\text{Time taken, } t = 1 \text{ year}$$

$$\text{Settlement in clay, } (S_c)_1 = 8 \text{ cm}$$

Now, $$\text{Final settlement, } (S_f)_2 = \frac{8}{0.5} = 16 \text{ cm}$$

Hence for the 25 m thick layer, $(S_f)_2 = 16 \times \dfrac{25}{5} = 80$ cm

From the equation, $T_v = \dfrac{C_v t}{H^2}$

For the same clay layer and same degree of consolidation,

$$\frac{t_1}{H_1^2} = \frac{t_2}{H_2^2}$$

For the 5 m thick layer, $t_{50} = 1$ year

Hence, for the 25 m thick layer, $t_{50} = 1 \times \left(\dfrac{25}{5}\right)^2 = 25$ years

No, for the 25 m thick layer, $\dfrac{(T_v)_1}{(T_v)_2} = \dfrac{t_1}{t_2}$

Also, since $$T_v = \frac{\pi}{4} \times U^2 \text{ for } U < 60\%$$

For, $t_1 = 25$ years $\quad U_1 = 50\%$

For, $t_2 = 1$ year

$$U_2 = \sqrt{U_1^2 \frac{t_2}{t_1}} = \sqrt{0.5^2 \times \frac{1}{25}} = \sqrt{0.01} = 0.1$$

Now, settlement corresponding of U_2

$$(S_c)_2 = U \times (S_f)_2 = 0.1 \times 80 = 8 \text{ cm}$$

Similarly, for, $t_3 = 4$ years

$$U_3 = \sqrt{U_1^2 \frac{t_3}{t_1}} = \sqrt{0.5^2 \times \frac{4}{25}} = \sqrt{0.04} = 0.2$$

Now, settlement corresponding of U_3

$$(S_c)_3 = 0.2 \times (S_f)_2$$

$$= 0.2 \times 80 = 16 \text{ cm}$$

Q.1 (d) Solution:

(i)

$$\text{Area ratio, } A_r = \frac{OD^2 - ID^2}{ID^2} \times 100$$

1. $$\text{Core cutter, } A_r = \frac{160^2 - 145^2}{145^2} \times 100\% = 21.759\%$$

As area ratio is slightly more than 20%, so reasonably good soil samples can be obtained.

2. $$\text{Split barrel, } A_r = \frac{50^2 - 34^2}{34^2} \times 100\% = 116.26\% >> 20\%$$

As the area ratio is much higher than 20%, so the soil samples will be disturbed in nature and are unacceptable.

3. $$\text{Seamles tube, } A_r = \frac{55^2 - 52^2}{52^2} \times 100 = 11.87\% < 20\%$$

As the area ratio$_r$ is less than 20%, soil samples will be undisturbed. However, in soft clays, area ratio must preferably be less than 10% for really undisturbed soil sample.

(ii)

$$\text{Saturated unit weight, } \gamma_{sat} = \left(\frac{G+e}{1+e}\right) \times \gamma_w$$

$$= \frac{2.65+0.75}{1+0.75} \times 9.81 = 19.06\,\text{kN/m}^3$$

$$\text{Submerged unit weight, } \gamma' = \gamma_{sat} - \gamma_w = 19.06 - 9.81 = 9.25\ \text{kN/m}^3$$

Now, $$\text{factor of safety, } F_c = \frac{C_u}{S_n \gamma' H} = \frac{18}{0.08 \times 9.25 \times 6} = 4.054$$

Q.1 (e) Solution:

Three different types of failure mechanism, based on the pattern of the shearing zones, have been identified as (i) general shear failure (ii) punching shear failure (iii) local shear failure.

(i) **General shear failure** mode occurs in soils possessing brittle type stress-strain behavior. It is characterized by a well-defined failure pattern, a sudden, catastrophic failure accompanied by tilting of foundation and a bulging of ground surface adjacent to the foundation.

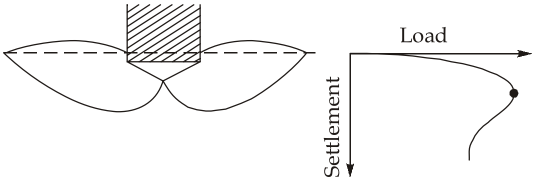


Fig. General shear failure

The load-settlement curve indicates that failure is abrupt in this mode. This type of failure pattern in generally observed is dense sand.

(ii) **Punching shear failure** occurs in soil possessing the stress-strain characteristics of a very plastic soil. Typical features of this mode are, poorly defined shear planes, soil zones beyond the loaded area being little affected and significant penetration of a wedge-shaped soil zone beneath the foundation, accompanied by vertical shear beneath the edges of the foundation.

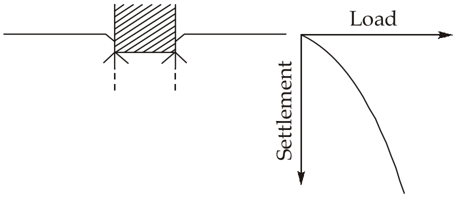


Fig. Punching shear failure

The load-settlement curve indicates a continuous increase in settlement with increasing load. Ultimate load cannot be clearly recognized.

(iii) **Local shear failure** pattern has some of the characteristics of both the general shear and the punching shear modes. Main features of local shear failures are, a well-defined wedge and slip surfaces only beneath the foundation, slip surfaces not visible beyond the edges of the foundation, slight bulging of the ground surface adjacent to the foundation and significant compression of the soil directly beneath the foundation.

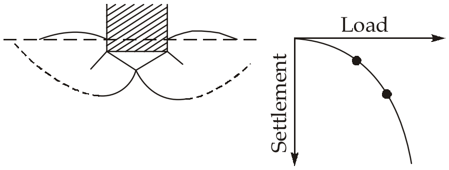


Fig. Local shear failure

The load-settlement curve does not indicate the ultimate load clearly. Soil possessing somewhat plastic stress-strain characteristics, fail in local shear mode. e.g. Loose sand.

Q.2 (a) Solution:

(i)

1. $$\sigma_{1f} = \sigma_{3f}\left(\frac{1+\sin\phi_u}{1-\sin\phi_u}\right) \qquad \text{since } C_u = 0$$

Where σ_{1f} and σ_{3f} are major and minor principal stress at failure respectively.

Deviator stress, $$\sigma_{df} = (\sigma_1 - \sigma_3)_f = \sigma_{3f}\frac{(1+\sin\phi_u)}{(1-\sin\phi_u)} - \sigma_{3f}$$

$$= \frac{120(1+\sin 20°)}{(1-\sin 20°)} - 120$$

$$= 124.75 \text{ kN/m}^2$$

2. $$\sin\phi_u = \frac{(\sigma_1 - \sigma_3)_f}{(\sigma_1 + \sigma_3)_f}$$

Also, $$\sin\phi' = \frac{(\bar{\sigma}_1 - \bar{\sigma}_3)_f}{(\bar{\sigma}_1 + \bar{\sigma}_3)_f} = \frac{(\sigma_1 - \sigma_3)_f}{(\sigma_1 + \sigma_3)_f - 2u_f}$$

where u_f is pore water pressure at failure.

$$\therefore \quad \frac{\sin\phi_u}{\sin\phi'} = \frac{(\sigma_1 + \sigma_3)_f - 2u_f}{(\sigma_1 + \sigma_3)_f} = 1 - \frac{2u_f}{(\sigma_1 - \sigma_3)_f + 2\sigma_3}$$

$$\Rightarrow \quad \frac{\sin 20°}{\sin 45°} = 1 - \frac{2u_f}{124.75 + 2 \times 120}$$

$$\Rightarrow \quad u_f = 94.16 \text{ kN/m}^2$$

3. $$\bar{\sigma}_{3f} = \sigma_{3f} - u_f$$

$$= 120 - 94.16$$

$$= 25.84 \text{ kN/m}^2$$

4. $$\bar{\sigma}_{1f} = (\bar{\sigma}_1 - \bar{\sigma}_3)_f + \bar{\sigma}_{3f}$$

$$= 124.75 + 25.84$$

$$= 150.59 \text{ kN/m}^2$$

(ii)

Advantages of triaxial test:

- Failure occurs along the weakest plane which is not predetermined. Specimen can fail on any plane which is weakest.

- There is a complete control over drainage conditions.
- There is a mechanism to measure pore pressure.
- The stress distribution on the failure plane is uniform unlike in direct shear test.
- It is most verstile and accurate of all the other shear testing methods. It can be performed on all type of soils.
- Precise measurement of volume change during the test is possible.

Limitations of triaxial test:

- Triaxial test is more complicated than the direct shear test.
- Consolidation and drainage of cohesive soil samples in a triaxial test take much longer time as compared to that in a direct shear test.
- Samples of cohesionless soil such as sands are difficult to prepare, which are more conveniently tested in the direct shear test.

Q.2 (b) Solution:

(i) If the wall is restrained against yielding, the lateral pressure that would develop against the wall would be earth pressure at rest.

Coefficient of earth pressure at rest k_0 can be calculated as

$$k_0 = 1 - \sin\phi' = 1 - \sin 30° = 0.5$$

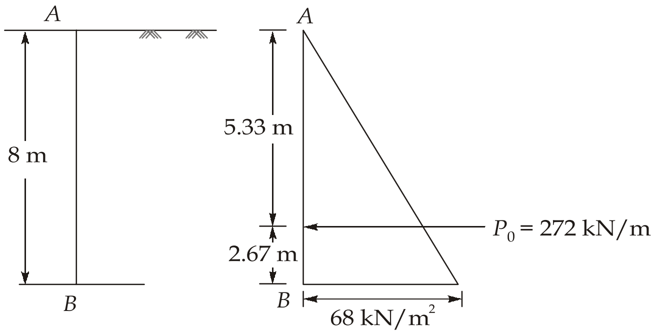
$$\gamma_b = 17 \text{ kN/m}^3$$

Earth pressure at rest,

Now, $p_0 = k_0 \gamma Z$

At point A, (Z = 0) $p_A = 0$

At pont B, (Z = 8 m) $p_B = 0.5 \times 17 \times 8 = 68 \text{ kN/m}^2$



Pressure distribution diagram

Total thrust per meter length of the wall, P_0

$$P_0 = \frac{1}{2} \times 68 \times 8 = 272 \text{ kN/m length of wall}$$

(ii) For this case, $\gamma = 17\ \text{kN/m}^3$ and the lateral pressure is the active earth pressure

$$\phi' = 30°$$

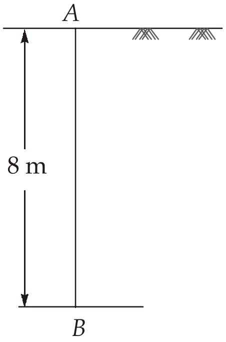
Now, active earth pressure coefficient,

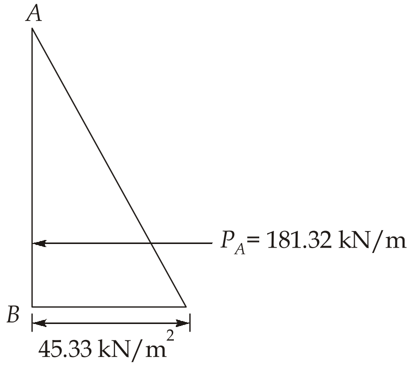
$$k_a = \frac{1-\sin\phi'}{1+\sin\phi'} = \frac{1+\sin 30°}{1+\sin 30°} = \frac{1}{3}$$

Active earth pressure, $p_A = k_A \gamma Z$

At point A (Z = 0) $\quad p_A = 0$

At point B (Z = 8 m) $\quad p_B = \frac{1}{3} \times 17 \times 8 = 45.33\ \text{kN/m}^2$





Total active thrust per meter length of wall,

$$P_A = \frac{1}{2} \times 45.33 \times 8 = 181.32\ \text{kN/m length of the wall.}$$

(iii) In this case, density of soil above water table, $\gamma = 17\ \text{kN/m}^3$

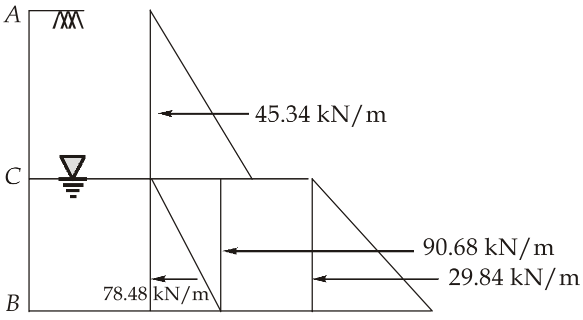
Density of soil below the water table, $\gamma_{sub} = 21 - 9.81 = 11.19\ \text{kN/m}^3$

1. Top soil *AC*

$$\text{Active earth pressure, } p_{a1} = k_a \gamma z = \frac{1}{3} \times 17 \times 4 = 22.67\ \text{kN/m}^2$$

$$\text{Active thrust , } P_{a1} = \frac{1}{2} \times 22.67 \times 4 = 45.34\ \text{kN/m}$$

$$\text{Point of application of } P_{a1}, \bar{H}_1 = \left(4 + \frac{4}{3}\right) = 5.33\ \text{m from base.}$$



2. Lower soil CB

- Effect of upper soil present in layer AC

$$\text{Surcharge, } q = \gamma_d H_1 = 17 \times 4 = 68 \text{ kN/m}^2$$

$$\text{Active thrust, } P_{a2} = \frac{1}{3} \times 68 \times 4 = 90.67 \text{ kN/m}$$

$$\text{Point of application of P}_{a2}\text{, } \bar{H}_2 = \frac{4}{2} = 2 \text{ m from base.}$$

- Effect of soil in *CB*

$$\text{Active earth pressure, } p_{a3} = k_a \gamma_{sub} Z$$

$$\text{At point } C, (Z = 0)\ p_{a3} = 0$$

$$\text{At point } B, (Z = 4 \text{ m})\ p_{a3} = \frac{1}{3} \times 11.19 \times 4 = 14.92 \text{ kN/m}^2$$

$$\text{Active thrust, } P_{a3} = \frac{1}{2} \times 14.92 \times 4 = 29.84 \text{ kN/m}$$

$$\bar{H}_3 = \frac{4}{3} = 1.33 \text{ m from base}$$

- $$\text{Water pressure, } P_{a4} = \frac{1}{2}(9.81 \times 4) \times 4 = 78.48 \text{ kN/m}$$

$$\bar{H}_4 = \frac{4}{3} = 1.33 \text{ m}$$

So, total thrust on wall, $P_a = P_{a1} + P_{a2} + P_{a3} + P_{a4}$

$$= 45.34 + 90.68 + 29.84 + 78.48$$

$$= 244.34 \text{ kN/m}$$

Line of action of total thrust,

$$\bar{H} = \frac{P_{a1}\bar{H}_1 + P_{a2}\bar{H}_2 + P_{a3}\bar{H}_3 + P_{a4}\bar{H}_4}{P_a}$$

$$= \frac{45.34 \times 5.33 + 90.68 \times 2 + 29.84 \times 1.33 + 78.48 \times 1.33}{244.34}$$

$$= 2.32 \text{ m from base.}$$

Q.2 (c) Solution:

(i)

The permeability of a soil is a property which describes quantitatively, the ease with which a liquid flows through that soil.

Factors affecting the permeability of a soil are as follows:

- **Effect of grain size**: The coefficient of permeability of a soil is proportional to the square of a representative particle size.
- **Effect of the permanent, its viscosity and temperature**: The coefficient of permeability depends on the kind and physical state of the pore fluid since both the viscosity and unit weight of a permeant vary with temperature since as the temperature increases viscosity decreases and hence permeability increases.
- **Void ratio of the soil**: As the void ratio of soil increases, the permeability of soil increases. Permeability (k) follows the following relationship with the void ratio (e) of the soil:

$$k \propto \frac{e^3}{1+e}$$

- **Presence of entrapped gases and organic matter** in soil decreases the permeability.
- **Dispersed or Flocculated structure of soil particles**: Two soils at the same void ratio, one having a dispersed structure and the other having a flocculated structure may exhibit a large difference in permeability values. Dispersion effect causes the permeability to decrease.

(ii)

There are two ways to measuring coefficient of permeability in laboratory viz.:

1. Constant head test 2. Falling head test

1. Constant head test: Constant head test is suitable for testing the pervious, coarse grained soils since adequate and measurable discharge is needed for the accurate determination of permeability by this method.

This test is based on the measurement of the quantity of water that flows under a given hydraulic gradient through a soil sample of known length and cross-sectional area in a given time.

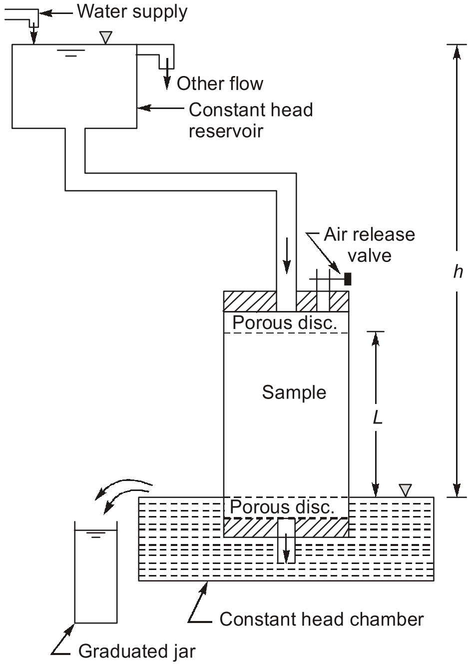


Fig. Constant head permeameter

A typical setup of this kind of permeameter is shown in the figure. The soil sample is compacted into the permeameter mould usually cylindrical at required desnty and water content. At the side of the cylinder, a number of manometer connection points are provided to enable pairs of pressure heads readings to be taken. Water is allowed to flow through the sample from a reservoir designed to keep the water level constant by overflow. The quantity of water flowing out of the soil or discharge 'Q' during a given time 't' is collected in a vessel and weighed.

Determination of k

$$\therefore \quad q = kiA$$

$$\Rightarrow \quad k = \frac{q}{iA} = \frac{VL}{Aht} \qquad \left[i = \frac{h}{L} \text{ and } q = \frac{V}{t}\right]$$

where, V is volume of permeant collected in time t.

A is cross-sectional area of the soil sample.

h is the difference in manometer levels.

L is the distance between manometer tapping points.

(b) **Falling head test**: This method is used to determine the permeability of soils such as fine sands and silts. In such soils, the permeability is too small to enable accurate measurements of discharge using a constant head permeameter. An arrangement of a falling head permeameter is shown in the figure. A cylinder containing the soil sample is placed on a base (perforated disc) fitted with a fine gauze. A graduated standpipe of known diameter is connected by filling the standpipe with de-aired water and allowing flow to take place through the sample. During the test, the water level will continuously drop and the height of water in the standpipe is recorded at several time intervals during the test. Any one pair of measurement, namely, the time taken for the head to fall from h_1 to h_2, will yield one value of k.

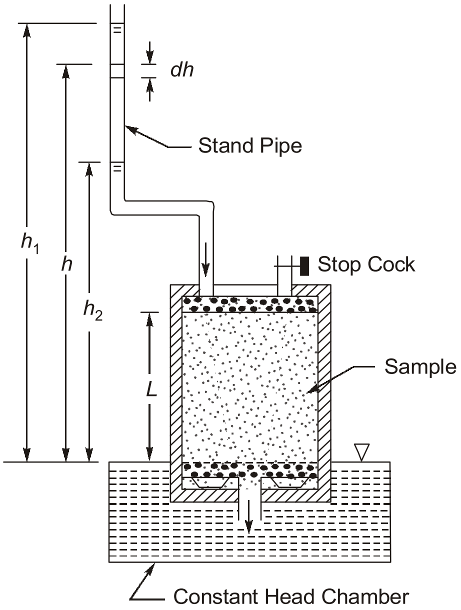


Fig. Falling head permeameter

As per Darcy's law, $q = kiA = k\dfrac{h}{L}A$

where h is the flow head at an intermediate time t between t_1 and t_2 corresponding to heads h_1 and h_2 respectively.

If the level of water in standpipe falls by dh in time dt, the flow in unit time through the sample = area of stand pipe × velocity of the fall, i.e. $-a\dfrac{dh}{dt}$. Here negative sign indicates that the head decreasesing with time.

$$\therefore \quad -a\frac{dh}{dt} = k\frac{h}{L}A$$

$$\Rightarrow \quad -a\int_{h_1}^{h_2}\frac{dh}{h} = k\cdot\frac{A}{L}\int_{t_1}^{t_2}dt$$

$$\therefore \quad k = \frac{aL}{A(t_2-t_1)}\log_e\frac{h_1}{h_2}$$

$$\Rightarrow \qquad k = 2.3\frac{aL}{At}\log_e\frac{h_1}{h_2}$$

Q.3 (a) Solution:

- Skin friction capacity Q_{sf},

(i) Loose sand stratum:

$$\delta = \frac{3}{4}\phi = \frac{3}{4}\times 25° = 18.75°$$

$$\frac{L}{D} \text{ in this stratum} = \frac{2}{0.5} = 4 < 15 \text{; therefore } \bar{\sigma} \text{ varies linearly with depth}$$

$$\bar{\sigma}_{avg} = \frac{17\times 2}{2} = 17 \text{ kN/m}^2$$

$$q_{s1} = \bar{\sigma}_{avg}k\tan\delta$$

$$= 17 \times 1 \times \tan 18.75° = 5.77 \text{ kN/m}^2 < 100 \text{ kN/m}^2$$

$\therefore$ Skin friction resistance, $Q_{s1} = q_{s1}A_s$

$$= 5.77 \times \pi \times 0.5 \times 2 = 18.13 \text{ kN}$$

(ii) Soft clay stratum:

$$C_u = 25 \text{ kN/m}^2, \alpha = 0.8$$

Skin friction resistance, $Q_{s2} = \alpha C_u \times A_s = 0.8 \times 25 \times \pi \times 0.5 \times 5$

$$= 157.08 \text{ kN}$$

(iii) Dense sand stratum:

$$\frac{L}{D} = \frac{4}{0.5} = 8 < 15 \text{, hence } \bar{\sigma} \text{ varies linearly with depth}$$

$\bar{\sigma}$ at the top of sand stratum $= 17 \times 2 + (19 - 9.81) \times 5 = 79.95 \text{ kN/m}^2$

$\bar{\sigma}$ at the bottom of sand stratum $= 79.95 + (20 - 9.81) \times 4 = 120.71 \text{ kN/m}^2$

$$(\bar{\sigma}_v)_{avg} = \frac{79.95+120.71}{2} = 100.33 \text{ kN/m}^2$$

From table $\quad \delta = \frac{3}{4}\phi = \frac{3}{4}\times 30 = 22.5°$ and $k = 2$

$\therefore \quad q_{s3} = 100.33 \times \tan 22.5° \times 2 = 83.116 \text{ kN/m}^2 < 100 \text{ kN/m}^2$

$\therefore$ Skin friction resistance, $Q_{s3} = q_{s3}A_s = 83.116 \times \pi \times 0.5 \times 4 = 522.23 \text{ kN}$

- End bearing capacity, Q_{eb}

$$\bar{\sigma} \text{ at the bottom of pile} = 120.71 \text{ kN/m}^2$$

$$q_{eb} = \bar{\sigma} N_q = 120.71 \times 80 = 9656.8 \text{ kN/m}^2$$

$$\therefore \quad Q_{eb} = q_{eb} \times A_b$$

$$= 9656.8 \times \pi \times \frac{0.5^2}{4} = 1896.11 \text{ kN}$$

$\therefore$ Ultimate pile load capacity,

$$Q_u = Q_{eb} + Q_{sf}$$

$$= 1896.11 + 18.13 + 157.08 + 522.23$$

$$= 2593.55 \text{ kN}$$

Now, $\text{FOS} = 2.5$

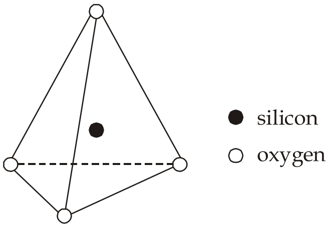
Allowable pile load capacity, $Q_a = \dfrac{Q_u}{2.5} = \dfrac{2593.55}{2.5} = 1037.42 \text{ kN}$

Q.3 (b) Solution:

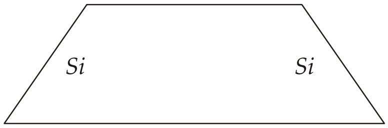
(i)

The atomic structures of clay minerals are built of two fundamental crystal sheets, the tetrahedral or silica sheet and the octahedral or alumina sheet. It is only the mode of stacking of these sheets, the nature of bonding forces and the different metallic ions in the crystal lattice that result in different clay minerals.

(a) **Tetrahedral sheet**: The tetrahedral sheet is a result of combining silica tetrahedral units which consist of four oxygen atoms placed at the tips of a tetrahedron enclosing a silicon atom as shown in the figure.



(a) Single tetrahedral unit



(b) Symbolic representation of the silica sheet

Each of the ‘O’ions at the base is common to two adjacent units. The sharing of charges leaves three negative charges at the base per tetrahedral unit. This along with the two negative charges at the apex makes a total of five negative charges to four positive charges of the silicon ion. Thus the tetrahedral unit contains a net negative charge of –1 per unit. Figure (b) is a symbolic representation of the silica sheet.

(b) Octahedral sheets: The octahedral sheet is a combination of octahedral units. An octahedral unit has six hydroxyl ions at the tips of an octahedron enclosing an aluminium or magnesium or some other metallic atom as shown in the figure.

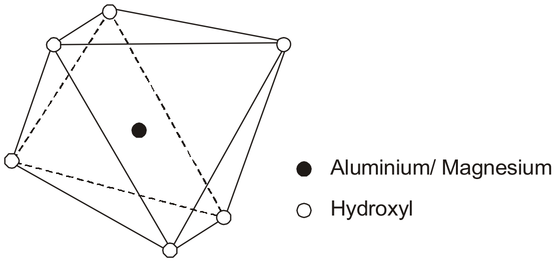
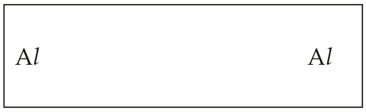


Fig. Single octahedral unit



(b) Symbolic representation of the octahedral sheet

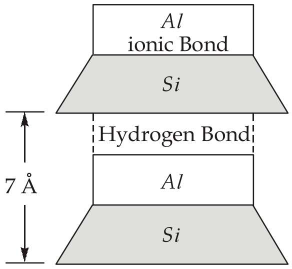
The rows of hydroxyl ions in the sheet are in two planes with each hydroxyl ion common to three octahedral units. Each hydroxyl ion divides its –1 charge with two other units, leaving a total of two negative charges, $\left(\frac{1}{3}\times 6\right)$ per unit. Thus the net charge of an octahedral unit with aluminium ion (+3 charge) at the centre is +1.

(ii)

Different clay minerals simply consist of the two basic components viz. the silica sheet and the octahedral sheet stacked together in certain unique fashions with certain metallic ions present in these sheets. The three main structures are:

(a) Kaolinite (b) Montmorillonite (c) Illite

(a) Kaolinite: The kaolinite structural unit consists of alternating layers of silica tetrahedra with the tips embedded in an alumina (gibbsite) octahedral unit as shown in the figure below.



Schematic diagram of kaolinite structure

This stacking of one layer of each of the two basic sheets produces what is sometimes called as 1 : 1 clay mineral. The resulting layer is about 7 Å thick and extends indefinitely in the other two dimensions. The structural units are held together by hydrogen bonds between the hydroxyls of the octahedral sheet and the oxygen atoms of the tetrahedral sheet. The bonding with hydrogen bond results in considerable strength and stability with little tendency in the interlayers to allow water and to swell. Kaolinite is thus, the 'least active' of the clay minerals.

(b) **Montmorillonite**: The structural unit of montmorillonite (often called as smeclite) is composed of two silica sheets and one alumina sheet, a 2 : 1 mineral. The octahedral sheet is sandwiched between two silica sheets with the tips of the tetrahedral combining with the hydroxyls of the octahedral sheet to form a single layer. The thickness of the layer is about 9.6 Å and the dimensions in the other two directions are indefinite.

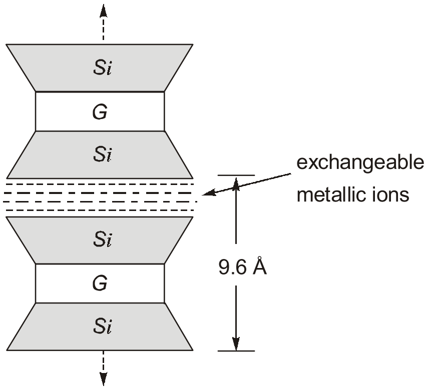


Fig. Schematic diagram of montmorillonite structure

The interlayer bonding between the tops of silica sheets is mainly due to Van-der Waal forces and is thus, very weak compared to hydrogen or other ion bonding. Montmorillonite has the largest specific surface area among major clay minerals. A large amount of water and other exchangeable ions can easily enter between the layers causing the layer to be separated. Because of this affinity for water, clay soils containing montmorillonite minerals are susceptible to substantial volume changes.

(c) **Illite**: The Illite mineral is also a 2 : 1 mineral similar to montmorillonite. Illite too has a substantial amount of isomorphous substitution of silicon ions by aluminium in the silica sheet. But in illite, potassium ions occupy positions between the adjacent 'O' base planes. Due to this phenomenon, Illite lattice is much less susceptible to cleavage and therefore does not swell as much in the presence of water as montmorillonite, but it does more than kaolinite.

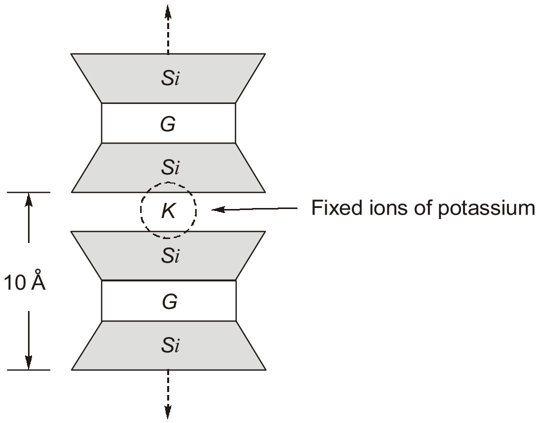


Fig. Schematic diagram of Illite structure

Q.3 (c) Solution:

(i)

Percentage of soil retained on 75 μ sieve = (100 – 10)% = 90%

It is greater than 50%. Hence, it is a coarse grained soil. Since gravel fraction (i.e. 4%) is less than 50%, large portion of the coarse grained soil is sand.

Now, fineness = % fraction which passes through 75μ sieve

i.e. $10\% < 12\%$

Now, $I_p = w_L - w_p = 27 - 18 = 9\%$

Which is more than 7%.

It means, silt fraction < clay fraction.

Also,

Coefficient of uniformity, $C_u = \frac{D_{60}}{D_{30}} = \frac{0.23}{0.20} = 1.15 < 6$

and coefficient of curvature,

$$C_c = \frac{D_{30}^2}{D_{60} \times D_{10}} = \frac{(0.2)^2}{0.23 \times 0.17} = 1.023$$

For well graded sand

$$C_u > 6 \text{ and } 1 < C_c < 3$$

Hence, the given soil is classified as SP – SC i.e. poorly graded clayey sand.

(ii)

Dry unit weight of soil in layer *AB*,

$$\gamma_d = \frac{G\gamma_w}{1+e} = \frac{2.65 \times 9.81}{1+0.5} = 17.331 \text{ kN/m}^3$$

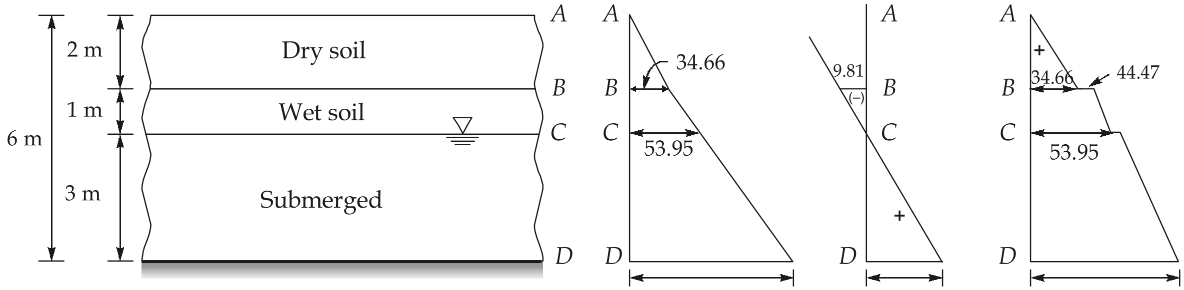
$$\simeq 17.33 \text{ kN/m}^3$$

Saturated unit weight of soil in layer CD, $\gamma_{sat} = \frac{(G+e)\gamma_w}{1+e}$

$$= \left(\frac{2.65+0.5}{1+0.5}\right)\times 9.81 = 20.6 \text{ kN/m}^3$$

Bulk unit weight of soil in the capillary zone BC, $\gamma_{\text{bulk}} = \dfrac{(G+eS)\gamma_w}{1+e}$

$$= \frac{(2.65+0.6\times 0.5)\times 9.81}{1+0.5} = 19.29 \text{ kN/m}^3$$



(a) Conditions after lowering the water table by 1 m

Total stress

Pore water pressure

Effective stress

(Values in kN/m^2)

Fig : Soil profile and pressure distribution diagrams

Total stress (σ)

At A, $\sigma = 0$

At B, $\sigma = \gamma_d \times z = 17.33 \times 2 = 34.66 \text{ kN/m}^2$

At C, $\sigma = 17.33 \times 2 + 1 \times 19.29 = 53.95 \text{ kN/m}^2$

At D, $\sigma = 17.33 \times 2 + 1 \times 19.29 + 20.6 \times 3 = 115.75 \text{ kN/m}^2$

Pore water pressure (u)

At A, $u = 0$

At B in AB, $u = 0$

At B in BC, $u = -H_C\gamma_w = -1 \times 9.81 = -9.81 \text{ kN/m}^2$

At D, $u = 3 \times 9.81 = 29.43 \text{ kN/m}^2$

Effective stress ($\bar{\sigma} = \sigma - u$)

At A, $\bar{\sigma} = 0$

At B in AB, $\bar{\sigma} = 34.66 - 0 = 34.66 \text{ kN/m}^2$

At B in BC, $\bar{\sigma} = 34.66 - (-9.81) = 44.47 \text{ kN/m}^2$

At C, $\bar{\sigma} = 53.95 - 0 = 53.95 \text{ kN/m}^2$

At D, $\bar{\sigma} = 115.75 - 29.43 = 86.32 \text{ kN/m}^2$

Q.4 (a) Solution:

(i)

Assumptions of Boussinesq's theory:

(a) Soil is homogeneous, isotropic, semi-infinite and elastic.

(b) Hooke's law is valid.

(c) Self weight of soil is neglected.

(d) Soil is initially unstressed.

(e) Any change in volume of the soil due to the application of the load is neglected.

(f) The top surface of the soil is free from any shear stress and is subjected to only point load.

(g) Distribution of stresses is symmetrical along the vertical axis.

(h) Continuity of stress exists in the medium.

Assumptions of Westergaard's theory:

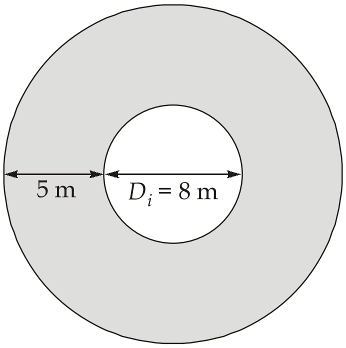
(a) Medium is assumed to be homogeneous, semi-infinite, stratified, elastic and non-isotropic.

(b) Medium is assumed to be laterally reinforced by numerous, closely spaced, horizontal sheets of negligible thickness but infinite rigidity.

This assumption signifies that there is no lateral deformation but only vertical deformations which are taking place due to loads.

- Natural deposits of soil are seldom isotropic. Water deposited sedimentary soils, which are quite common in occurrence, are formed by deposition, alternately, of horizontal layers of silts and clays. For this condition, Westergaard's solution is better suited. However, in the range where stresses are significant, results obtained by Boussinesq's theory are more conservative than that of Westergaard's theory. Hence, Boussinesq's theory is practically used for the computation of vertical stresses in soil mass.

(ii)



$$\text{Internal diameter, } D_i = 8 \text{ m}$$

$$\text{External diameter, } D_0 = 8 + 5 + 5 = 18 \text{ m}$$

$$\therefore \quad r_i = 4 \text{ m}$$

$$r_0 = 9 \text{ m}$$

$$q = 60 \text{ kN/m}^2$$

$$z = 10 \text{ m}$$

In this case entire area is not loaded ant thus to find vertical stress, assume whole area of radius 9 m is loaded by 60 kN/m^2, whereas the circular area of radius 4 m is also loaded with same load intensity but opposite in nature.

$$\therefore \qquad \sigma_z = q\left[1-\left(\frac{1}{1+\left(\frac{r_0}{z}\right)^2}\right)^{3/2}\right]-q\left[1-\left(\frac{1}{1+\left(\frac{r_i}{z}\right)^2}\right)^{3/2}\right]$$

$$= 60\left[1-\left(\frac{1}{1+\left(\frac{9}{10}\right)^2}\right)^{3/2}\right]-60\left[1-\left(\frac{1}{1+\left(\frac{4}{10}\right)^2}\right)^{3/2}\right]$$

$$= 60-60\left(\frac{1}{1+\left(\frac{9}{10}\right)^2}\right)^{3/2}-60+60\left(\frac{1}{1+\left(\frac{4}{10}\right)^2}\right)^{3/2}$$

$$= 60\left\{\left(\frac{1}{1+0.4^2}\right)^{3/2}-\left(\frac{1}{1+0.9^2}\right)^{3/2}\right\}$$

$$= 60\left[\left(\frac{1}{1.16}\right)^{3/2}-\left(\frac{1}{1.81}\right)^{3/2}\right] = 23.39\text{ kN/m}^2$$

Q.4 (b) Solution:

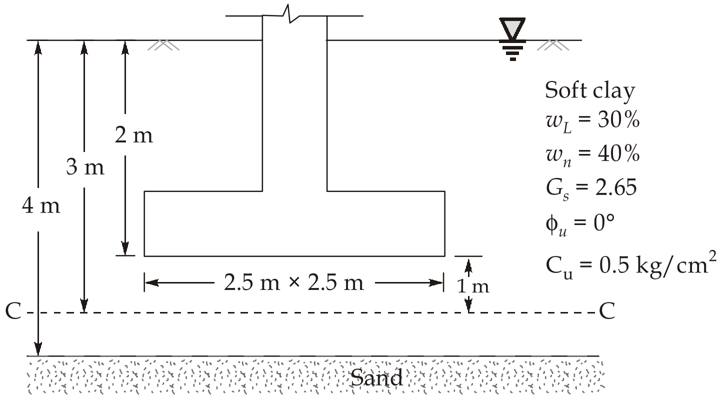
Depth of the footing, $D_f = 2\text{ m}$

Width of the footing, $B_f = 2.5\text{ m}$

$$\frac{D_f}{B_f} = \frac{2}{2.5} = 0.8 < 2.5$$

Hence for square footing, $N_c = 6\left(1+\frac{0.2D_f}{B_f}\right) \not> 9.0$

$$\therefore \qquad N_c = 6\times\left(1+0.2\times\frac{2}{2.5}\right) = 6.96$$



Net ultimate bearing capacity is given by

$$q_{nu} = C_u N_c$$

where $C_u = 0.5\text{ kg/cm}^2 = 5\text{ t/m}^2$

$\therefore$ $q_{nu} = 5 \times 6.96 = 34.8\text{ t/m}^2$

Net safe bearing capacity, $q_{ns} = \dfrac{34.8}{3} = 11.6\text{ t/m}^2$

Saturated unit weight, $\gamma_{sat} = \left(\dfrac{G_s + e}{1+e}\right) \times \gamma_w$

where, $e_0 = w_n \times G_s = 0.4 \times 2.65 = 1.06$

$\therefore$ $\gamma_{sat} = \left(\dfrac{2.65+1.06}{1+1.06}\right) \times 1 = 1.8\text{ t/m}^3$

Submerged unit weight, $\gamma' = 1.8 - 1 = 0.8\text{ t/m}^3$

For a normally consolidated clay,

$$\text{Settlement, } S = \frac{HC_c}{1+e_0} \times \log_{10} \frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}$$

where $C_c = 0.009(w_L - 10)$ for normally consolidated clay.

$= 0.009 \times (30 - 10) = 0.18$

Effective overburden pressure at (c-c),

$$\bar{\sigma}_0 = 0.8 \times 3 = 2.4\text{ t/m}^2$$

$$\text{Change in pressure, } \Delta\sigma = \frac{11.6 \times (2.5)^2}{(2.5 + 2 \times 0.5)^2} = 5.92\text{ t/m}^2$$

$$\therefore \quad \text{Settlement, } S = 2000 \times \frac{0.18}{1+1.06} \times \log\left(\frac{2.4+5.92}{2.4}\right) = 94.35\text{ mm}$$

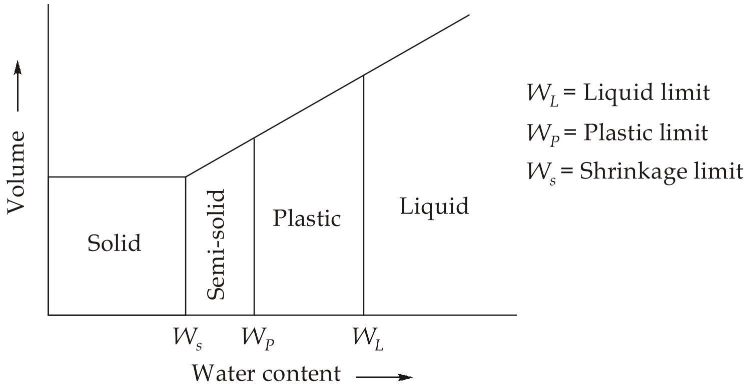
Q.4 (c) Solution:

(i)

Consistency is a term which is used to describe the degree of firmness of a soil in a qualitative manner by using descriptions such as soft, medium, firm, stiff or hard. It indicates the relative ease with which a soil can be deformed. In practice, the property of consistency is associated only with fine grained soils, especially clays.

The physical properties of clays are considerably influenced by the amount of water present in them. Depending upon the water content, the following four states of consistency are used to describe the consistency of clays: (i) the liquid state, (ii) the plastic state (iii) the semi-solid state and (iv) the solid state.

The boundary water contents at which the soil undergoes a change from one state to another are called “consistency limits” which are also known as Atterberg limits.

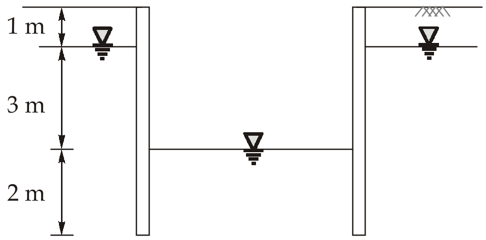


Atterberg limits

When a fine-grained soil is mixed thoroughly with a large quantity of water, the resulting suspension is in a liquid state, and offers practically no resistance to flow. If the water content of the suspension is gradually reduced keeping the consistency of the sample uniform, a stage comes when it just begins offering resistance to flow. This is the stage when the sample changes from possessing no shear strength to having an infinitesimal shear strength and changes from the liquid to plastic state. The boundary water content between the liquid state and the plastic state is called the **liquid limit**. In the plastic state, the soil can be moulded to different shapes without rupturing it due to its plasticity. If the water content is further reduced, the clay sample changes from the plastic state to the semi-solid state at a boundary water content which is called as **plastic limit**. In the semi-solid state, the soil does not have plasticity and it becomes brittle. A further reduction in the water content brings about a state when with decrease in moisture, the volume of the soil mass does not decrease any further, the sample changes from the semi-solid to solid state. The boundary water content is called the shrinkage limit.

(ii)

As shown in the figure, the depth of the water table above the bottom of the trench = 3 m. The sheet is taken 2 m below the bottom of the trench to increase the seepage length.



$$\text{Critical gradient, } i_c = \left(\frac{G-1}{1+e}\right)$$

$$= \frac{2.64-1}{1+0.7} = 0.96$$

If the trench is to be stable, the hydraulic gradient, i, prevailing at the bottom should be less than i_c. The hydraulic gradient i is

$$i = \frac{h}{L}$$

There will be no quick condition if,

$$i < i_c$$

$$\Rightarrow \quad \frac{h}{L} < 0.96$$

$$\text{Now,} \quad \frac{h}{L} = \frac{3}{2} = 1.5 > 0.96$$

Hence, there will be quicksand condition.

Remedy:

1. Increase seepage length to at least 4 m below the bottom of trench so that h/L = 0.75 which gives a margin of safety.
2. Keep the water table outside the trench at a low level by pumping out water. This reduces the head 'h'.
3. Do not pump water up to the bottom level of the trench. Arrange the work in such a way that the work may be carried out with some water in trench.

Section B : Environmental Engineering

Q.5 (a) Solution:

Geometrical increase method:

Year	Population	Increment	Growth rate per decade
1961	42,560		
1971	47,820	5260	0.124
1981	63,500	15680	0.328
1991	86,452	22952	0.361
2001	1,11,230	24778	0.287
2011	1,39,886	28656	0.258
2021	1,82,800	42914	0.307

$$\text{Geometric mean, } r_g = \sqrt[n]{r_1 \times r_2 \times \ldots\ldots \times r_n}$$

$$= \sqrt[6]{0.124 \times 0.328 \times 0.361 \times 0.287 \times 0.258 \times 0.307}$$

$$= 0.263$$

Population after 30 years of commencement of water supply i.e. in 2056,

$$P_{2056} = P_{2021} \times (1 + r_g)^{3.5}$$

$$= 1,82,800 \times (1 + 0.263)^{3.5} = 4,13,982.84$$

$$\simeq 413893$$

(b) Incremental increase method:

Year	Population	Increase (x)	Incremental increase (y)
1961	42,560		
1971	47,820	5260	
1981	63,500	15680	10420
1991	86,452	22952	7272
2001	1,11,230	24778	1826
2011	1,39,886	28656	3878
2021	1,82,800	42914	14258
	Total	1,40,240	37654

$$\overline{x} = \frac{140240}{6} = 23,373$$

$$\overline{y} = \frac{37654}{5} = 7531$$

$$\therefore \quad P_n = P_0 + n\overline{x} + \frac{n(n+1)}{2}\overline{y}$$

$$\therefore \quad P_{2056} = P_{2021} + 3.5 \times 23373 + \frac{3.5 \times (3.5+1)}{2} \times 7531$$

$$= 182800 + 81805.5 + 59306.625 \simeq 323912$$

Q.5 (b) Solution:

When a discrete particle settles down in water, its downward settlement is opposed by the drag force offered by the water. The effective weight of the particle causes the particle to accelerate in the beginning, till it attains a terminal velocity (V_s) at which the drag force becomes equal to the effective weight of the particle. After attaining that velocity (V_s), the particle falls down with the constant velocity.

Now, we know that

$$\text{Effective weight of the particle} = \text{Total weight} - \text{Buoyancy}$$

$$= \frac{4}{3}\pi r^3 \cdot \gamma_s - \frac{4}{3}\pi r^3 \gamma_w$$

where, r is radius of particle, γ_s is unit weight of particle and γ_w is unit weight of water.

Also, $$\text{Drag force} = C_D \times A \times \rho_w \times \frac{V^2}{2}$$

where C_D is coefficient of drag, A is area of particle, and V is velocity of fall.

Now, when V becomes equal to V_s, the drag force becomes equal to the effective weight of the particle.

$$\therefore \quad C_D \cdot A \rho_w \cdot \frac{V_s^2}{2} = \frac{4}{3}\pi r^3 (\gamma_s - \gamma_w)$$

$$\Rightarrow \quad C_D \pi r^2 \cdot \rho_w \frac{V_s^2}{2} = \frac{4}{3}\pi r^3 (\gamma_s - \gamma_w) \qquad [\because A = \pi r^2]$$

$$\Rightarrow \quad V_s^2 = \frac{4}{3} \times \frac{2r(\gamma_s - \gamma_w)}{\rho_w \cdot C_D}$$

$$\Rightarrow \quad V_s^2 = \frac{4}{3} \times \frac{dg(\rho_s - \rho_w)}{\rho_w \cdot C_D}$$

$$\Rightarrow \quad V_s^2 = \frac{4}{3} \times d\rho_w \cdot g\left[\frac{\rho_s}{\rho_w} - 1\right] \times \frac{1}{\rho_w \cdot C_D}$$

$$= \frac{4}{3} gd(G_s - 1) \times \frac{1}{C_D} \qquad \text{...(i)}$$

where, d is the diameter of particle and G_s is the specific gravity of the particle.

Now, for laminar flows and for small particles, coefficient of drag, $C_D = \frac{24}{\text{Re}}$, where Re is particle's Reynolds number.

Now, $$\text{Re} = \frac{V_s d}{\nu}$$ where ν is kinematic viscosity of fluid

Putting value of C_D in (i), we get

$$\therefore \quad V_s^2 = \frac{4}{3} gd(G_s - 1) \times \frac{V_s d}{\nu \times 24}$$

$$\Rightarrow \qquad V_s = \frac{g}{18}\frac{(G_s - 1)d^2}{\nu}$$

Q.5 (c) Solution:

(i) Cyclone collectors: It consists of a cylindrical shell, conical-base, dust hopper and an inlet where the dust-laden gas enters tangentially.

- This is a closed chamber, in which the inlet velocity of the gas (smoke) is transformed into a spinning vortex, which helps to throw out the particles under the generated centrifugal force. The particles then slide down the chamber walls into the hopper from where they come out.
- The operating or separating efficiency of a cyclone depends on the magnitude of the centrifugal force exerted on the particles. The greater the centrifugal force, greater is the separating efficiency.
- Large-diameter cyclone collectors have good collection-efficiencies for particles 40 μm to 50 μm in diameter. High-efficiency cyclones with diameters of 23 cm or less have good efficiencies for particles from 15 μm to 20 μm.
- The cleaning efficiency for units may be as high as 90 percent for particulates in 5 μm to 10 μm range of particle size.
- They are relatively inexpensive to construct and operate, and they can handle-large volumes of gases at temperatures upto 980°C. Pressure drops across these units are generally low and range from 2.5 cm to 20 cm of water.
- The cyclones have been used successfully at feed and grain mills, cement plants, fertilizer plants, petroleum refineries, asphalt mixing plants and other applications involving large quantities of gas containing relatively large particles.

(ii) Electrostatic precipitators: In these precipitators, the emitted gas (flue gas) is passed through a highly ionised atmosphere (high-voltage field) and there in that zone, particulates get electrically charged with the result that they get separated out from the gaseous stream with the help of electrostatic forces.

NOTE: They are extensively used in thermal power plants, pulp and paper industries, mining and metallurgical industries, iron and steel plants, chemical industries, etc.

There are two types of electrostatic precipitates available, viz.:

1. **Low voltage two stage units:** These units operate at 6000 to 12000 V and are commonly used to collect liquid particles. They have a design capacity of 10 m³/sec for gases passing with a velocity of 0.5 m/s.
2. **High voltage single stage units:** These units work at 30000 to 100000 V. They are recommended for use in large industrial plants.

There are many advantages associated with electrostatic precipitators as listed below:

1. Very small particles also, wet or dry can be easily trapped (size about 1 micrometer).
2. More than 99% efficiency can be achieved in their functioning (usual range of efficiency is 95 to 99%).
3. In this if corrosive and adhesive particles are absent from the gasses, they require a nominal maintenance.
4. It has only few moving parts and thus the requirements of repair, etc. is minimal.
5. They can be operated at high temperature (upto 300 to 450°C).
6. Pressure drops and power requirement are not very high.

(iii) Fabric Filters: In this system, the particulate laden gas stream passes through a woven fabric which filters out particulate matter allowing a gas to pass through it.

- Small particles are initially retained on the fabric by direct interception, inertial impaction, diffusion, electrostatic attraction and gravitational settling.
- After a dust mat has formed on the fabric, more efficient collection of submicron particles (99+ percent) is accomplished by sieving.
- Filter bags, usually tubular or envelope shaped, are capable of removing most of particles as small as 0.5 μm and will remove substantial quantities of particles as small as 0.1 μm.

Advantages

- Fabric filter has a high collection efficiency over a broad range of particle sizes.
- Externally flexible in design.
- Ability to handle large volumes of gases at relatively high speeds.
- Reasonable operating pressure drops the power requirements and the ability to handle a diversity of solid materials.
- They are particularly useful in many high volume operations such as cement kiln foundries, steel furnaces and grain handling plants.

Q.5 (d) Solution:

Number of people = 250

Water supplied = 135 lt/person/day

Detention period = 24 hours

Tank is cleaned once in a year.

Rate of deposition of sludge = 42 lt/person/year

Depth of tank = 2 m

Free board = 0.3 m

Length to breadth ratio = 3.5 : 1

Now,the quantity of water supplied = Water supplied per capita × population

$= 135 \times 250 = 33750$ lt/day

Given that 80% of water supplied becomes waste water.

The quantity of sewage produced $= 33750 \times 0.80 = 27000$ lt/day

For detention period of 24 hours,

Quantity of sewage produced during detention period $= 27000 \times \frac{24}{24}$

$= 27000$ lt.

Rate of sludge deposition is 42 litre/capita/year

Period of cleaning is 1 year, we have,

Volume of sludge deposited per year $= 42 \times 250 \times 1 = 10500$ lt.

Total capacity of tank required = Capacity of sewage + capacity of sludge

$= 27000 + 10500 = 37500 \text{ lt} = 37.5 \text{ m}^3$

For 2 m depth of septic tank,

$$\text{Surface area of tank} = \frac{37.5}{2} = 18.75 \text{ m}^2$$

The raio of length of width is to be kept as 3.5 : 1

$$\therefore \quad L \times B = 18.75 \text{ m}^2$$

$$\Rightarrow \quad 3.5 \times B \times B = 18.75$$

$$\Rightarrow \quad B = \sqrt{\frac{18.75}{3.5}} = 2.3 \text{ m}$$

$$\therefore \quad \text{Length of tank} = 3.5 \times B$$

$$= 3.5 \times 2.3 = 8.05 \simeq 8.1 \text{ m}$$

∴ Dimensions of septic tank are 8.1 m × 2.3 m × (2 + 0.3)m

= 8.1 m × 2.3 m × 2.3 m

Q.5 (e) Solution:

Given: $(BOD)_5$ @ 20°C = 150 mg/lt

Deoxygenation constant, $k_D = 0.1$ per day

Now, $(BOD)_5 = L_0 \times [1 - 10^{-k_D t}]$ where L_o is ultimate BOD

$$\Rightarrow \quad 150 = L_0\,[1 - 10^{-0.1 \times 5}]$$

$$\Rightarrow \quad L_0 = \frac{150}{0.684}$$

$$= 219.3 \text{ mg/lt}$$

Now, $(k_D)_{T^\circ} = (k_D)_{20^\circ}[1.056]^{T - 20^\circ}$ $[\because \quad \theta = 1.056 \text{ for } T < 20°\text{C}]$

$$\therefore \quad (k_D)_{15^\circ} = 0.1 \times [1.056]^{15 - 20}$$

$$\Rightarrow \quad (k_D)_{15^\circ} = 0.076 \text{ day}^{-1}$$

Now, $(\text{BOD})_8 \text{ at } 15°\text{C} = L_0[1 - 10^{-k_D t}]$

$$= 219.3 \times [1 - 10^{-0.076 \times 8}] = 165.22 \text{ mg/lt}$$

Q.6 (a) Solution:

(i) Water required per day = 4 MLD

As 4% of filtered water is required for washing of the filter everyday,

$$\text{Total filtered water required per day} = \frac{4}{0.96} = 4.167 \text{ MLD}$$

Given that 0.5 hour is lost everyday in washing the filter.

$$\therefore \text{ Filtered water required per hour} = \frac{4.167}{23.5} = 0.177 \text{ Million litres per hour}$$

Now, the rate of filtration is 5000 lt/hr/m^2.

$$\therefore \quad \text{Area of filter required} = \frac{0.177 \times 10^6}{5000} = 35.4 \text{ m}^2$$

Number of units (filter bed) are given as,

$$N = 1.22\sqrt{Q}$$

$$= 1.22\sqrt{4.167} = 2.49 \simeq 3$$

Thus, provide 4 filter units (3 operational + 1 standby)

$$\therefore \quad \text{Area of each filter unit} = \frac{35.4}{3} = 11.8 \text{ m}^2 \simeq 12 \text{ m}^2$$

Now, assuming the length of the filter bed as 1.5 times the width of the filter bed.

Now, $L \times B = 12 \text{ m}^2$

$$\Rightarrow \quad 1.5 \times B \times B = 12 \text{ m}^2$$

$$\Rightarrow \quad B = 2.83 \text{ m} \simeq 2.9 \text{ m (say)}$$

$$\therefore \quad L = 1.5 \times B$$

$$= 1.5 \times 2.9 = 4.35 \text{ m} \simeq 4.4 \text{ m (say)}$$

∴ Adopt 3 filter units each of dimensions 2.9 m × 443 m.

(ii) Design of manifold lateral under drainage system:

To design this system, let us assume that the total area of the perforations in all laterals is 0.2% of filter area.

$$\therefore \quad \text{Total area of perforations} = 0.2\% \times \text{filter area}$$

$$= \frac{0.2}{100} \times 2.9 \times 4.4 = 0.025 \text{ m}^2$$

Now, assuming the area of each lateral is four times the area of perforations in it.

$$\therefore \quad \text{Total area of laterals} = 4 \times 0.025 = 0.1 \text{ m}^2$$

Now, assuming the area of manifold to be twice the area of laterals,

$$\text{Area of manifold} = 2 \times 0.1 = 0.2 \text{ m}^2$$

∴ Diameter of manifold (d) is given by,

$$\frac{\pi}{4} \times d^2 = 0.2$$

$$\Rightarrow \quad d = 0.5 \text{ m} = 50 \text{ cm}$$

Hence, use a 50 cm diameter manifold pipe laid lengthwise along the centre of the filter bottom.

Laterals running perpendicular to the manifold and emanating from the manifold may be laid at a spacing of say 15 cm.

$$\text{Number of laterals on each side} = \frac{\text{Length of filter bed}}{\text{Spacing between laterals}}$$

$$= \frac{4.4 \times 100}{15} = 29.33 \simeq 30$$

Hence, use 60 laterals in all in each filter unit. The diameter of the laterals is adopted as 6 mm.

$$\text{Now, length of each lateral} = \frac{\text{Width of filter}}{2} - \frac{\text{Diameter of manifold}}{2}$$

$$= \frac{2.9}{2} - \frac{0.5}{2} = 1.2 \text{ m}$$

Now, total number of perforations in all 60 laterals × area of each lateral = Total area of perforations

$$x \times \frac{\pi}{4} \times \left(\frac{6}{1000}\right)^2 = 0.025$$ where x is number of perforations in all 60 laterals

$$\Rightarrow \quad x = 884.19 \simeq 885$$

Number of perforations in each lateral $= \dfrac{885}{60} = 14.75 \simeq 15$

Area of perforations per lateral $= 15 \times \dfrac{\pi}{4} \times 0.6^2 = 4.24 \text{ cm}^2$

Now area of each lateral = 4 × area of perforation in it

$= 4 \times 4.24 = 16.96 \text{ cm}^2$

$\therefore$ Diameter of each lateral $= \sqrt{\dfrac{16.96 \times 4}{\pi}} = 4.65$ cm

Hence, use 60 laterals each of 4.65 cm dia. @ 15 cm c/c, each having 15 perforations of 6 mm size with 50 cm diameter manifold.

Check: $\dfrac{\text{Length of each lateral}}{\text{Diameter of lateral}} = \dfrac{1.2 \times 100}{4.65} = 25.8 < 60$ [Hence OK]

(ii) Wash water discharge:

Given rate of washing of filter= 60 cm rise/minute

Wash water discharge $= \dfrac{0.6 \times 2.9 \times 4.4}{60} = 0.1276 \text{ m}^3/\text{sec.}$

Velocity of flow in the lateral for wash water $= \dfrac{0.1276}{60 \times \dfrac{\pi}{4} \times \left(\dfrac{4.65}{100}\right)^2} = 1.25$ m/sec.

Similarly, velocity of flow in the manifold $= \dfrac{\text{Discharge}}{\text{Area}}$

$= \dfrac{0.1276}{\dfrac{\pi}{4} \times 0.5^2} = 0.65$ m/sec.

$\therefore$ Velocity of flow is less than 1.8 to 2.4 m/sec. which is OK.

Q.6 (b) Solution:

(i)

- **Break-Point Chlorination**: Break point chlorination is a term which gives us an idea of the extent of chlorine added to water. Infact, it represents that much dose of chlorination, beyond which any further additional chlorine will appear as free residual chlorine.

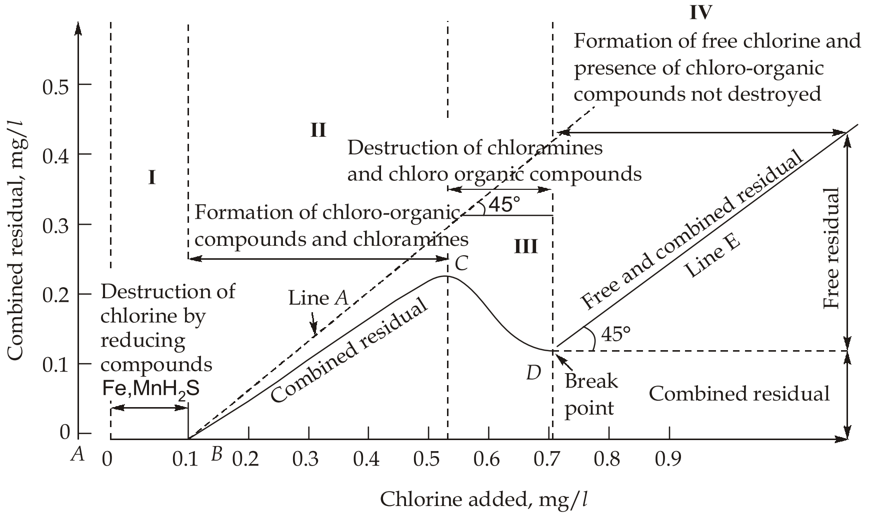


Fig. Chlorine Reaction in Water

NOTE: When chlorine is added to pure water which has no chlorine demand, a curve, such as line A in figure is obtained between applied chlorine and residual chlorine relationship.

1. As raw water has a chlorine demand and therefore curve A is generally not obtained between residual chlorine and applied chlorine. During the disinfection process, the amount of residual chlorine will be very less in beginning (marked by stage I in figure), during which various chemicals such as ions of ferrous, sulphides or nitrites present in water, will be oxidised.
2. Stage-II reflects the formation of combined residuals as the ammonia or amines reacts with HOCI and chloramines that has formed. During state II, as the demand for disinfection is satisfied, combined residual chlorine goes on increasing till a point C is reached where amount of combined residual is maximum. The stage of point C is accompanied of bad taste and odour indicating the comencement of stage III.
3. During state-II, oxidation of organic matter present in water starts. Therefore, after point C, with increase in applied chlorine, residual chlorine goes on decreasing as shown by curve CD in figure. In this stage, free chlorine breaks down chloramines into nitrogen compounds at point D and bad smells suddenly disappear showing that oxidation of organic matter is completed. At point D, residual chlorine has minimum value.
4. Further addition of applied chlorine results in increase in residual chlorine as represented by line E the slope of which is 45° as shown in figure during stage IV.

   Actually, upto point con curve shown in figure, chloramines have been recorded as residual chlorine while at point D (break point), true residual free chlorine appears.

Thus, break point in chlorination of water may be defined as point on applied residual chlorine curve at which all, or nearly, all of residual chlorine is free chlorine. Free chlorine residual is that part of total residual remaining in water (after a specified contact period) that will react chemically or biologically as hypochlorous acid or hypochlorite ion.

Break point chlorination has following advantages:

(i) It will remove taste and odour.

(ii) It will have adequate chlorine residual.

(iii) It will leave a desired chlorine residual.

(iv) It will lead to complete oxidation of ammonia and other compounds.

(v) It will remove colour due to organic matter by about 30%.

(vi) It will remove manganese.

Generally, the break point lies between 3 to 7 ppm of chlorine dose, though this is greatly influenced by quantity of free ammonia present in water.

(ii)

Testing of chlorine residuals: The amount of residual chlorine left in the chlorinated water after the required contact period can be experimentally determined by using any of the following tests:

(a) Orthotolidine test (b) Starch iodide test

(c) Chlorotex and (d) D.P.D. test

(a) Orthotolidine test: The main test used for residual chlorine for many years has been that using orthotolidine. One of the reasons of it is that orthotolidine is not recognized as being among those chemical substances capable of causing cancer.

In this test, 10 ml of chlorinated water sample is taken in a test tube after the required contact period. To this, 0.1 ml of orthotolidine solution is added.

The colour formed is noted after 5 minutes. The presence of yellow colour indicates the presence of chlorine (free or combined) in water. By comparing this colour with a colour of known concentration, the amount of chlorine residue is obtained. If free and combined chlorine are to be found out separately, then colour formed in 5 seconds (R1) and colour formed in 5 minutes (R2) is noted. (R1) will correspond to free chlorine and (R2) will correspond to free and combined chlorine both.

In orthotolidine test, presence of iron, manganese, nitrite etc. will give false yellow colour thereby indicating increased chlorine residue which is wrong. For such water, orthotolidine arsenite test is performed. In this case, sodium arsenite is added to chlorinated water. This will dechlorinate the sample.

To this dechlorinated sample, orthotolidine solution is added and colour formed R1 is noted. Now R1 will correspond to quantity of iron, manganese and nitrite. After that, another sample of chlorinated water is taken and colour formed in 5 seconds (R2) and colour formed in 5 minutes (R3) is noted. Now, (R2) will correspond to combination of free chlorine and interfering ions such as iron , manganese and nitrite while (R3) will correspond to combination of free and combined chlorine and interfering ions such as iron , manganese and nitrite.

NOTE: When chloramine is used as disinfectant, ordinary orthotolidine test is done and colour should be noted in 15 minutes instead of 5 minutes because reaction is slow with chloramines.

(b) Starch iodide test (Iodometric Test): In this test, one litre of water sample is collected in a heat proof earthenware vessel known as casserole. To this, 10 ml of potassium iodide solution is added. The contents are thoroughly mixed, and about 5 ml of starch solution is added, which produces blue colour. This blue colour is removed by titrating this water sample against sodium thiosulphate. solution of normality N/100. The amount of chlorine can then be easily ascertained by using the simplified titration equation.

It is used when there is presence of nitrite, nitrite, manganese compounds which make orthotolidene test unsuitable. This test is more precise than orthotolidine test particularly when residual chlorine is more than 1 ppm.

(c) Chlorotex test: It is based on colour matching technique.

- In this case, BDH (British Drug House) Reagent i.e. chlorotex reagent is used.
- 50 ml of water is taken with 5 ml of chlorotex reagent which develops a colour that is matched with that known concentration.
- Corresponding to 0.2 mg/*l* residue, colour formed is pink colour.
- No comparator is used in this method.

(d) DPD Test (Diethyl-Paraphenylene Diamine): Due to problem with orthotolidene, the use of orthotolidene for water testing has now been superceded by the plain test using DPD.

- The necessary reagents are supplied as tablets with detailed instruction.
- It is also based on colour matching technique.
- The reagent used is Pallian's DPD reagent.
- The test is generally carried out in a BDH comparator.

Q.6 (c) Solution:

(i) At maximum flow, the $\frac{d}{D}$ ratio is 0.7.

$$\therefore \quad \frac{d}{D} = 0.7$$

and v = Velocity at the time of maximum flow.
= 0.9 m/sec.

$$\text{Now,} \quad \frac{d}{D} = \frac{1}{2}\left[1-\cos\frac{\alpha}{2}\right]$$

$$\Rightarrow \quad 0.7 = \frac{1}{2}\left[1-\cos\frac{\alpha}{2}\right]$$

$$\Rightarrow \quad \alpha = 227.156°$$

$$\text{Now,} \quad \frac{v}{V} = \left[\frac{\frac{\alpha}{360°}-\frac{\sin\alpha}{2\pi}}{\frac{\alpha}{360°}}\right]^{2/3}$$

$$= \left[\frac{\frac{227.156°}{360°}-\frac{\sin 227.156°}{2\pi}}{\frac{227.156°}{360°}}\right]^{2/3} = 1.12$$

$$\therefore \quad \frac{0.9}{V} = 1.12$$

$$\Rightarrow \quad V = \frac{0.9}{1.12} = 0.8 \text{ m/sec.}$$

(ii) Now, at maximum flow, $\frac{q_{\max}}{q_{avg.}} = 3$

$$\text{Now,} \quad \frac{q_{\max.}}{Q} = \left[\frac{\alpha}{360°}-\frac{\sin\alpha}{2\pi}\right]\left[\frac{\frac{\alpha}{360°}-\frac{\sin\alpha}{2\pi}}{\frac{\alpha}{360°}}\right]^{2/3}$$

$$= \left[\frac{227.156°}{360°}-\frac{\sin(227.156°)}{2\pi}\right]\left[\frac{\frac{227.156°}{360°}-\frac{\sin(227.156°)}{2\pi}}{\frac{227.156°}{360°}}\right]^{2/3}$$

$$= 0.837$$

$\therefore \quad q_{max.} = 0.837Q$

At average flow, $q_{avg.} = \frac{1}{3} q_{max.}$

$$= \frac{1}{3} \times 0.837 \times Q = 0.279\text{Q}$$

Now, $\frac{q_{avg.}}{Q} = 0.279$

$$\Rightarrow \left[\frac{\alpha}{360^\circ} - \frac{\sin\alpha}{2\pi}\right] \left[\frac{\frac{\alpha}{360^\circ} - \frac{\sin\alpha}{2\pi}}{\frac{\alpha}{360^\circ}}\right]^{2/3} = 0.279$$

Solving by trial and error.

$\Rightarrow \quad \alpha = 147.77^\circ$

Now, $\frac{d_{avg.}}{\text{D}} = \frac{1}{2}\left[1 - \cos\frac{\alpha}{2}\right]$

$$= \frac{1}{2}\left[1 - \cos\frac{147.77^\circ}{2}\right] = 0.36$$

and $\frac{V_{avg.}}{V} = \left[\frac{\frac{\alpha}{360^\circ} - \frac{\sin\alpha}{2\pi}}{\frac{\alpha}{360^\circ}}\right]^{2/3} = 0.86$

$\Rightarrow \quad V_{avg.} = 0.86\,V$

$$= 0.86 \times 0.8 = 0.69 \text{ m/sec.}$$

Now, at minimum flow, $\frac{q_{avg.}}{q_{min.}} = 2.5$

$$\Rightarrow \quad q_{min} = \frac{q_{avg.}}{2.5} = \frac{0.279Q}{2.5} = 0.1116Q$$

Now, $\left[\frac{\alpha}{360^\circ} - \frac{\sin\alpha}{2\pi}\right] \left[\frac{\frac{\alpha}{360^\circ} - \frac{\sin\alpha}{2\pi}}{\frac{\alpha}{360^\circ}}\right]^{2/3} = 0.1116$

Solving by trial nd error,

$$\alpha = 113.443°$$

Now, $$\frac{d}{D} = \frac{1}{2}\left[1-\cos\frac{\alpha}{2}\right]$$

$$= \frac{1}{2}\left[1-\cos\frac{113.443°}{2}\right] = 0.226$$

and $$\frac{v_{\text{min.}}}{V} = \left[\frac{\frac{\alpha}{360°}-\frac{\sin\alpha}{2\pi}}{\frac{\alpha}{360°}}\right]^{2/3}$$

$$= \left[\frac{\frac{113.443°}{360°}-\frac{\sin(113.443°)}{2\pi}}{\frac{113.443°}{360°}}\right]^{2/3} = 0.66$$

∴ $$v_{\text{min.}} = 0.66V = 0.66 \times 0.8 = 0.53 \text{ m/sec.}$$

Q.7 (a) Solution:

(i)

Quantity of sewage flowing into the filter per day = 4.5 MLD

BOD concentration in raw sewage = 250 mg/lt

Total BOD present in raw sewage = $4.5 \times 10^6 \times 250 \times 10^{-6} = 1125$ kg/day

BOD removal in primary tank= 30%

BOD left in the sewage entering per day in the filter unit

$$= 1125 \times (1 - 0.3) = 787.5 \text{ kg/day}$$

BOD concentration desired in final effluent = 30 mg/lt

Total BOD left in the effluent per day = $4.5 \times 10^6 \times 30 \times 10^{-6} = 135$ kg/day

BOD removal by the filter = 787.5 – 135 = 652.5 kg

Now, efficiency of the filter, $$\eta = \frac{\text{BOD removed}}{\text{Total BOD}}\times 100$$

$$= \frac{652.5}{787.5}\times 100 = 82.86\%$$

Now, $$\eta = \frac{100}{1+0.0044\sqrt{\frac{W}{V.F}}}$$

where $\eta = 82.86\%$

$$W = \text{Total BOD in kg/day}$$

$$= 787.5 \text{ kg/day}$$

$$V = \text{Volume of filter in ha-m}$$

$$F = \frac{1+R}{(1+0.1R)^2} = \frac{1+1.5}{(1+0.1\times1.5)^2} = \frac{2.5}{1.3225} = 1.89$$

$$\therefore \quad 82.86 = \frac{100}{1+0.0044\sqrt{\dfrac{787.5}{V\times1.89}}}$$

$$\Rightarrow \quad 1+0.0044\sqrt{\frac{416.67}{V}} = 1.2$$

$$\Rightarrow \quad V = 0.2 \text{ ha–m} = 2000 \text{ m}^3$$

$$\therefore \quad \text{Surface area required} = \frac{2000}{1.5}\text{m}^2 = 1333.3 \text{ m}^2$$

$$\therefore \quad \text{Diameter of circular filter required} = \sqrt{1333.3\times\frac{4}{\pi}} = 41.2 \text{ m}$$

(ii)

Zones of pollution in a river stream: A river/stream undergoing self purification can be divided into following four zones of pollution viz.

- **Zone of degradation**: This zone exists upto a certain length beyond the point just below the sewage out fall.

  Here in this zone, water becomes dark and turbid along with deposits of sludge at the bottom. Dissolved oxygen gets reduced to about 40% of the saturation value (~ 7.6 mg/lit at 30°C). Rate of de-oxygenation is much higher than re-oxygenation rate. Due to such un-favourable conditions, aquatic life such as algae etc, dies out.
- **Zone of active decomposition:** This is the highly polluted zone. Here water becomes greyish and much darker. Dissolved oxygen (DO) level falls down to almost zero. Anaerobic conditions get set in thereby resulting in evolution of gases like CH_4, CO_2, H_2S etc. Due to decomposition of organic matter, the upper part of this zone is dominated by anaerobic bacteria while the lower part is dominated by aerobic bacteria.
- **Zone of recovery**: Here the stream recovers from its degraded condition. Water becomes clear and algae reappears. Dissolved oxygen (DO) level rises and organic matter gets mineralized to form nitrates, sulphates, phosphates and carbonates.

- **Zone of cleaner water:** Here the river attains its original condition, DO level reaches to saturation value and usual aquatic life prevails.

Q.7 (b) Solution:

(i)

Average sewage flow = 20 MLD

Total suspended solids = 300 mg/lt.

So, mass of suspended solids in sewage per day

$$= \frac{300 \times 20 \times 10^6}{10^6} \text{kg/day} = 6000 \text{ kg/day}$$

Now, 65% solids are removed in primary settling tank,

Mass of solids removed in primary settling tank = 65% × 6000 kg/day

= 3900 kg/day

Fresh sludge has a moisture content of 95%, and therefore,

5 kg of dry solids will make = 100 kg of wet sludge

$\Rightarrow$ 3900 kg of dry solids will make = $\frac{100}{5} \times 3900 \text{ kg}$ of wet sludge per day.

= 78,000 kg of wet sludge per day

Density of wet sludge = 1.02 × 1000 = 1020 kg/m^3

Volume of raw sludge produced per day, $V_1 = \frac{78000}{1020} = 76.47 \text{ m}^3/\text{day}$

The volume of digested sludge (V_2) at 85% moisture content is given as,

$$V_2 = V_1\left[\frac{100-p_1}{100-p_2}\right]$$

$$= V_1\left[\frac{100-95}{100-85}\right] = V_1\left[\frac{5}{15}\right]$$

$$= \frac{1}{3}V_1 = \frac{1}{3} \times 76.47 = 25.49 \text{ m}^3/\text{day}$$

Now, for digestion period of 30 days, the capacity of the digestion tank required is,

$$\text{Capacity} = \left[V_1 - \frac{2}{3}[V_1 - V_2]\right] \times t$$

$$= \left[76.47 - \frac{2}{3} \times [76.47 - 25.49]\right] \times 30$$

$$= 1274.5 \simeq 1275 \text{ m}^3$$

Now, providing 6.0 m depth of cylindrical digestion tank, cross-sectional area of tank

$$= \frac{1275}{6} = 212.5 \text{ m}^2$$

$$\therefore \quad \text{Diameter of tank} = \sqrt{\frac{212.5}{\pi / 4}} = 16.45 \text{ m} \simeq 16.5 \text{ m}$$

(ii)

The productivity of a lake is defined as a measure of its ability to support a food chain.

- Since, the algae forms the base of this food chain, the higher algal growth leads to decreased water quality.
- Depending upon the increasing level of its productivity, the lakes may be classified as

(i) Oligotrophic lake (ii) Mesotrophic lake
(iii) Eutrophic lake (iv) Senescent lake

1. Oligotrophic lake

- Oligotrophic lakes have a low level of productivity due to severely limited supply of nutrients to support algal growth.
- The water of such a lake is, therefore, clear enough as to make its bottom visible upto a considerable depth.
- In such a case, the euphotic zone often extends into the hypolimnion zone, which is aerobic.
- Oligotrophic lake, supports cold water game fish.

2. Mesotrophic Lake

- The lakes having medium productivity levels, with medium growth of algae and turbidity, are usually classified as mesotrophic lakes.
- Hypolimnion zone remains aerobic although substantial depletion of oxygen occurs.

3. Eutrophic lake

- Eutrophic lakes do have a high level of productivity, because of an abundant supply of algal nutrients.

- Euphotic zone will partially extend into epilimnion.
- Hypolimnion zone is anaerobic.

4. **Senescent lake**
   - These are very old shallow lakes, having thick organic sediment deposits at their bottoms.
   - Rooted water plants abundantly grow in such shallow ponds, which ultimately become marshes i.e. swamps.

Q.7 (c) Solution:

Per capita BOD of the domestic sewage = 72 gm/day

= 72000 mg/day

Per capita sewage produced = 240 lt/day

$$\text{BOD per litre of the domestic sewage} = \frac{72\times1000}{240}\text{ mg/lt}$$

= 300 mg/lt.

Amount of domestic wastewater produced per day = 30000 × 240 = 7.2 million litres

Total wastewater discharge = 7.2 + 3 + 2.4 = 12.6 MLD = 145.8 lt/sec.

$$\text{Net BOD of all wastewaters} = \frac{7.2\times300+3\times1100+2.4\times1500}{7.2+3+2.4} = 719\text{ mg/lt.}$$

Total wastewater discharge with 10% expansion factor

$= 1.1 \times 145.8$ lt/sec $= 160.38 \simeq 160$ lt/sec (say)

Initial DO of saturated stream water = 9 mg/lt [saturation DO]

$$\text{DO of mixture (at } t = 0) = \frac{\text{DO of river}\times Q_R+\text{DO of sewage}\times Q_s}{Q_R+Q_S}$$

$$= \frac{9\times4500+0\times160}{4500+160} = 8.69\text{ mg/lt.}$$

∴ Initial DO deficit, D_0 = 9 – 8.69 = 0.31 mg/lt

Now, critical DO deficit, D_c = 9 – 4 = 5 mg/lt

Now, $$\left[\frac{L}{D_c \cdot f}\right]^{f-1} = f\left[1-(f-1)\frac{D_0}{L}\right]$$

where $D_c = 5$ mg/lt

$D_0 = 0.31$ mg/lt

$k_D = 0.1\ \text{day}^{-1}$

$k_R = 0.3\ \text{day}^{-1}$

Self purification constant, $$f = \frac{k_R}{k_D} = \frac{0.3}{0.1} = 3$$

$\therefore$ $$\left[\frac{L}{5\times 3}\right]^{2} = 3\times\left[1-\frac{2\times 0.31}{L}\right]$$

$\Rightarrow$ $L = 25.67$ mg/lt

Maximum permissible 5 day BOD of mix

$$(\text{BOD})_{\text{mix}} = L[1 - 10^{-0.1\times 5}]$$

$$= 0.684 \times L$$

$$= 0.684 \times 25.67 = 17.56 \text{ mg/lt.}$$

Now, $$(\text{BOD})_{\text{mix}} = \frac{C_sQ_s + C_RQ_R}{Q_s + Q_R}$$

where C_s and C_R are BOD of sewage and river respectively

$\Rightarrow$ $$17.56 = \frac{C_s\times 160 + 0\times 4500}{160+4500}$$

$\Rightarrow$ $C_s = 511.44$ mg/lt

Maximum permissible BOD_5 of waster water, $C_s = 510.85$ mg/lt

Initial BOD of city wastewaters = 719 mg/lt

Degree of treatment required $$= \frac{719-511.44}{719}\times 100 = 28.9\%$$

Q.8 (a) Solution:

(i)

The manner in which the emitted plumes behave are mentioned below.

(a) Looping plume

- Looping plume has wavy character and occurs in super adiabatic environments which give rise to highly unstable atmosphere, because of rapid mixing.
- High turbulence helps in rapid dispersion of the plume, and results in high concentration near the ground.

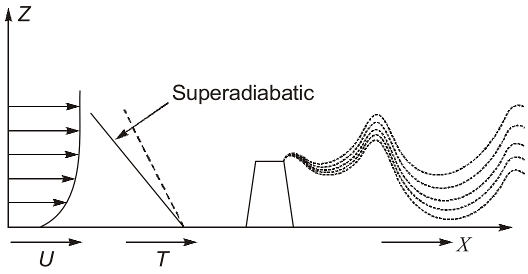


Fig. Looping Plume

(b) Neutral plume

- Neutral plume is the upward vertical rise of the plume from the stack, which occurs when the ELR is equal to ALR.
- The upward lifting of the plume will continue till it reaches an air of density similar to that of the plume itself.
- The neutral plumes tend to cone, when the wind velocity is greater than 32 km/h and when cloud cover blocks the solar radiation by day and terrestrial radiation by night.

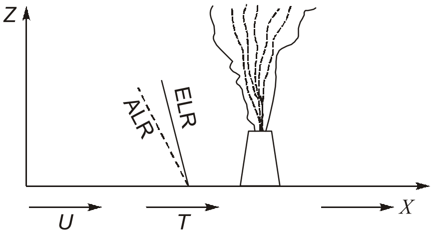


Fig. Neutral Plume

(c) Coning plume

- Conning plume occurs under sub-adiabatic conditions.
- The plume dispersion is known as coning, because the plume makes a cone like shape about the plume line.

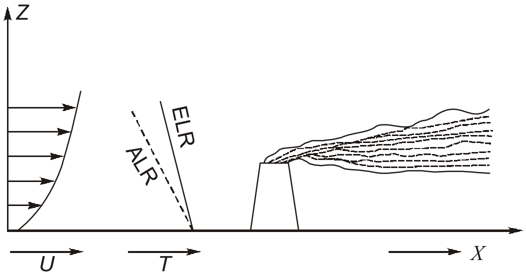


Fig. Conning plume

(d) Fanning plume

- Under extreme inversion conditions in the presence of light wind, fanning plume occurs.
- There will be no vertical mixing and the plume will extend horizontally over large distances. Such a plume pattern is called as fanning plume.

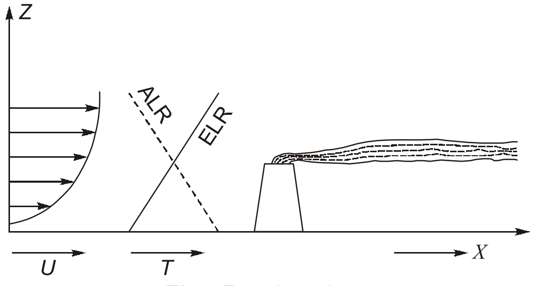


Fig. Fanning plume

(e) Lofting plume

- Where there exists a strong super adiabatic lapse rate above a surface inversion, then the plume is said to be lofting.
- Such a plume has minimum downward mixing as its downward motion is prevented by inversion, but the upward mixing will be quite turbulent and rapid. The dispersion of pollutants will therefore, be rapid and no concentration will reach the ground. Hence, this would be the most ideal case for dispersion of emission.

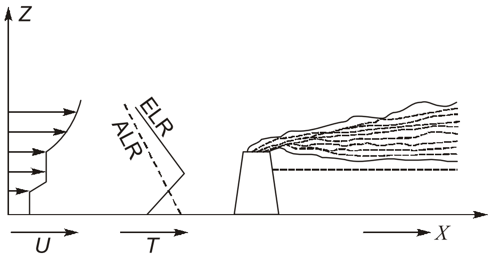


Fig. Lofting plume

- Lofting is the most favourable plume type as far as ground level concentrations is one of the major goals of tall-stack operation.

(f) Fumigating Plume

- When an inversion layer occurs at a short distance above the top of the stack and super adiabatic conditions prevail below the stack, then the plume is said to be fumigating.
- The conditions for fumigation are just the inverse of lofting plume.

- The pollutants cannot escape above the top of the stack because of inversion layer and they will be brought down near the ground due to turbulence in the region above the ground and below the inversion, caused by strong lapse rate.
- This represents worst case of atmospheric conditions for dispersion.

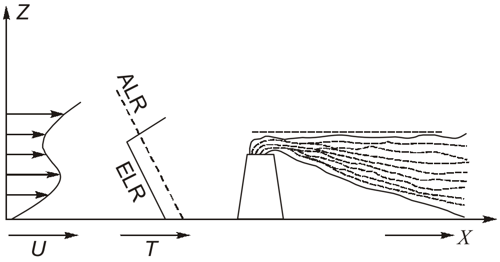


Fig. Fumigating plume

(g) Trapping Plume

- When inversion layers occurs above the emission source, as well as below the source, then naturally, the emitted plume will neither go up, nor will it go down and would remain confined between the two inversions.
- Such a plume is called a trapping plume and is considered a bad condition for dispersion, as the dispersion cannot go above a certain height.

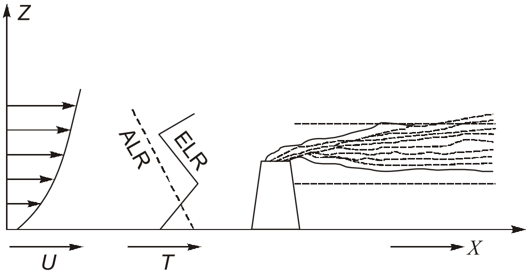


Fig. Trapping plume

(ii)

The given data is symbolised as below:

$$h = 180 \text{ m}$$

$$D = 0.95 \text{ m}$$

$$u = 2.75 \text{ m/sec.}$$

$$T_a = 20°\text{C} = (20 + 273)\text{K} = 293 \text{ k}$$

$$P = 1000 \text{ millibars}$$

$$v_s = 11.12 \text{ m/sec.}$$

$$T_s = 160°\text{C} = (160 + 273)\text{K} = 433 \text{ K}$$

Now,

$$\Delta h = \frac{V_s D}{v}\left[1.5 + 2.68 \times 10^{-3} PD\left(\frac{T_s - T_a}{T_s}\right)\right]$$

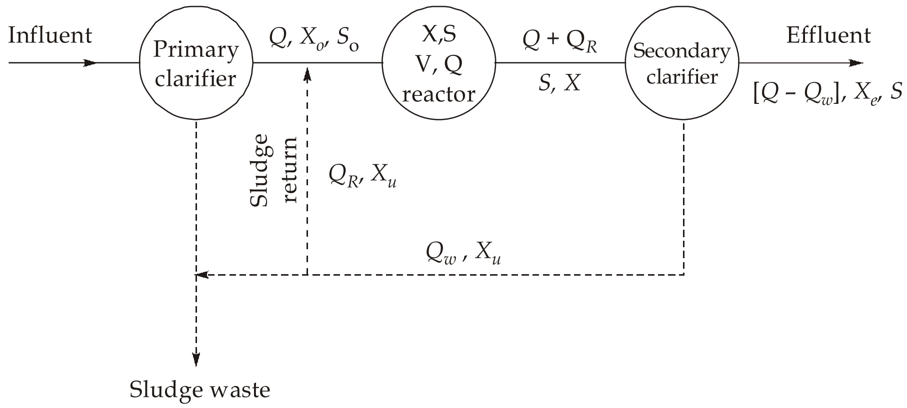
$$= \frac{11.12 \times 0.95}{2.75} \times \left[1.5 + 2.68 \times 10^{-3} \times 1000 \times 0.95 \times \left(\frac{433 - 293}{433}\right)\right]$$

$$= \frac{11.12 \times 0.95}{2.75} \times \left[1.5 + \frac{2.68 \times 0.95 \times 140}{433}\right] = 8.92 \text{ m}$$

Now, effective height of stack $H = h + \Delta h$

$$= 180 + 8.92 = 188.92 \text{ m}$$

Q.8 (b) Solution:



1. $$\frac{1}{\theta_c} = \frac{Q(S_0 - S) \times y}{VX} - k_d$$

where V is volume in m^3.

$$\frac{1}{10} d^{-1} = \frac{10000 \, m^3/d \times 0.5 \times \left(0.15 \, kg/m^3 - 0.005 kg/m^3\right)}{V \times 3.0 \, kg/m^3} - 0.05 \, d^{-1}$$

$$\Rightarrow \quad 0.1 d^{-1} = \frac{241.67 \, m^3/d}{V} - 0.05 d^{-1}$$

$$\Rightarrow \quad 0.15 d^{-1} = \frac{241.67 \, m^3/d}{V}$$

$$\Rightarrow \quad V = 1611 \, m^3$$

2. At equilibrium condition,

$$\theta_c = \frac{\text{Mass of solids in reactor}}{\text{Mass of solids wasted}} = \frac{VX}{Q_w X_u}$$

$$\Rightarrow \quad Q_w \cdot X_u = \frac{VX}{\theta_c} = \frac{1611 m^3 \times 3 kg/m^3}{10 \, d} = 483.3 \text{ kg/d}$$

Given the concentration of solids in the underflow is 10000 mg/lt i.e.

$$X_u = 10000 \text{ mg/lt} = 10 \text{ kg/m}^3$$

$$\therefore \quad Q_w = \frac{483.3 \text{ kg/d}}{10 \text{ kg/m}^3} = 48.3 \text{ m}^3\text{/d}$$

3. A mass balance equation around the secondary clarifier can be written as follows:

$$(Q + Q_R)X = (Q + Q_R - Q_w)X_e + (Q_R + Q_w)X_u$$

Assuming that the solids in the effluent are negligible compared to the influent and underflow

$$\therefore \quad QX + Q_R X = Q_R \cdot X_u + Q_w X_u$$

$$\Rightarrow \quad Q_R[X_u - X] = QX - Q_w X_u$$

$$\Rightarrow \quad Q_R = \frac{QX - Q_w X_u}{X_u - X}$$

$$Q_R = \frac{10000 \text{ m}^3\text{/d} \times 3.0 \text{ kg/m}^3 - 483.3 \text{ kg/d}}{10 \text{ kg/m}^3 - 3 \text{ kg/m}^3}$$

$$= 4216.7 \text{ m}^3\text{/d} \simeq 4217 \text{ m}^3\text{/d}$$

The recirculation ratio is, $\dfrac{Q_R}{Q} = \dfrac{4217}{10000} = 0.42$

Q.8 (c) Solution:

(i)

We know that all gases at 0°C and atmospheric pressure occupies 22.4 lt/mole.

$$V_1 = 22.4 \text{ litre}$$

Let Volume at 20°C = V_2

Now, $$\frac{P_1 V_1}{T_1} = \frac{P_2 V_2}{T_2}$$

But pressure is always atmospheric.

$$\therefore \quad V_2 = \frac{V_1 T_2}{T1} = \frac{V_1[273 + T]}{273} = 22.4 \times \frac{(273+20)}{273}$$

$$= 24.04 \text{ lt/mol}$$

Now, $$1 \text{ ppm of } SO_2 = \frac{1 \text{ m}^3 \text{ of } SO_2}{10^6 \text{ m}^3 \text{ of air}}$$

Molecular weight of SO_2 = 64 gm

Given mass of SO_2 in 1 m^3 of air = 150 mg

$\therefore$ Mass of SO_2 in 10^6 m^3 of air $= 150 \times 10^{-3} \times 10^6$

$= 150 \times 10^3$ gm

Now, volume occupied by 64 gm of SO_2 = 24.04 lt

$$\therefore \text{ Volume occupied by } 150 \times 10^3 \text{ gm of } SO_2 = \frac{24.04 \times 10^{-3}}{64} \times 150 \times 10^3 = 56.34 \text{ m}^3$$

So, 150 mg/m^3 of SO_2 consentration at 20°C = 56.34 ppm SO_2 at 20°C.

(ii)

Since the aquifer is unconfined and thus S_c is equal to specific yield.

$$\text{Now,} \quad u = \frac{r^2 \cdot S_c}{4Tt}$$

where T is transmissivity of aquifer and is given as

$$T = kH = 50 \times 20 = 1000 \text{ m}^2/\text{day}$$

$$\therefore \quad \mu = \frac{25^2 \times 0.22}{4 \times 1000 \times 1} = 0.034 \qquad \text{(Dimensionless)}$$

$$\text{Well function, } W(u) = -0.5772 - \ln u + u$$

$$\Rightarrow \quad W(u) = -0.5772 - \ln(0.034) + 0.034$$

[Neglecting higher order terms of u]

$$= 2.838$$

$$\text{Now,} \quad S = \frac{Q}{4\pi T} \times W(u)$$

$$\Rightarrow \quad Q = \frac{S(4\pi T)}{W(u)} = \frac{2.5 \times 4\pi \times 1000}{2.838} = 11069.74 \text{ m}^3/\text{day}$$

where, S = Drawdown in the observation well,

r = distance from the pumped well

After 1 year of pumping,

$$u = \frac{r^2 s}{4T \cdot t} = \frac{25^2 \times 0.22}{4 \times 1000 \times 365} = 9.418 \times 10^{-5}$$

$$\therefore \quad W(u) = -0.5772 - \ln(9.418 \times 10^{-5}) + 9.418 \times 10^{-5}$$

$$= 8.693$$

$$\therefore \quad S = \frac{Q\,W(u)}{4\pi T}$$

$$= \frac{11069.74 \times 8.693}{4\pi \times 1000} = 7.66 \text{ m}$$

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