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Leading Institute for ESE, GATE & PSUs

Detailed Solutions

ESE-2026
Mains Test Series

Civil Engineering
Test No : 1

Section A : Geo-technical & Foundation Engineering [All Topics]

1. (a) Solution:

Given,

Thickness of clay, $H = 9 \text{ m}$

Ultimate settlement, $\Delta H = 308 \text{ mm}$

Case-1: After 2 years, settlement

$\Delta h = 108 \text{ mm}$

\therefore Degree of consolidation, $U = \frac{\Delta h}{\Delta H} \times 100 = \frac{108}{308} \times 100 = 35.06\%$

Since $U < 60\%$

\therefore Time factor, $T_V = \frac{\pi \left(\frac{U}{100} \right)^2}{4} = \frac{\pi \left(\frac{35.06}{100} \right)^2}{4} = 0.0965$

Case-2: For 220 mm settlement, degree of consolidation

$U = \frac{\Delta h}{\Delta H} \times 100 = \frac{220}{308} \times 100 = 71.43\%$

Since, $U > 60\%$

\therefore Time factor,
 $T_V = 1.781 - 0.933 \log_{10} (100 - U)$
 $= 1.781 - 0.933 \log_{10} (100 - 71.43)$
 $= 0.423$

$$\therefore T_V = \frac{C_v t}{d^2}$$

$$\therefore \frac{(T_V)_1}{(T_V)_2} = \frac{t_1}{t_2} \quad (\because C_v \text{ and } d \text{ are same})$$

$$\Rightarrow t_2 = t_1 \times \frac{T_{V2}}{T_{V1}} = 2 \times \frac{0.423}{0.0965} = 8.77 \text{ years}$$

1. (b) (i) Solution:

Given:

$$\text{Cohesion, } c = 50 \text{ kN/m}^2$$

$$\text{Angle of internal friction, } \phi = 0^\circ$$

$$\text{Unit weight of soil, } \gamma = 19 \text{ kN/m}^3$$

$$\text{Total weight of failure wedge, } W = 1050 \text{ kN}$$

$$\text{Radius of failure arc, } r = 10.2 \text{ m}$$

$$\text{Central angle, } \theta = 95^\circ$$

$$\text{Horizontal distance to centroid, } x = 4 \text{ m}$$

Length of Failure Arc

$$L = \frac{\pi r \theta}{180}$$

$$\Rightarrow L = \frac{\pi \times 10.2 \times 95}{180}$$

$$\Rightarrow L = 16.91 \text{ m}$$

Resisting Moment

$$M_r = CLr$$

$$\Rightarrow M_r = 50 \times 16.91 \times 10.2$$

$$\Rightarrow M_r = 8624.1 \text{ kN-m}$$

Driving Moment

$$M_d = W \times x$$

$$\Rightarrow M_d = 1050 \times 4$$

$$\Rightarrow M_d = 4200 \text{ kN-m}$$

Factor of Safety

$$FOS = \frac{M_r}{M_d}$$

$$\Rightarrow FOS = \frac{8624.1}{4200}$$

$$\Rightarrow FOS = 2.05$$

1. (b) (ii) Solution:

Aspect	Soil Liquefaction	Quick Sand Condition
Definition	Liquefaction is the phenomenon in which saturated loose cohesionless soil loses its shear strength due to increase in pore water pressure during dynamic loading.	Boiling occurs when upward seepage force equals the submerged weight of soil particles, causing soil to behave like a liquid.
Cause	Mainly caused by earthquakes, vibrations, or cyclic loading.	Caused by upward seepage of water due to hydraulic gradient.
Soil Type	Occurs mostly in loose saturated sands and silty sands	Occurs generally in cohesionless soils like sand under seepage conditions
Pore Water Pressure	Rapid increase in pore water pressure reduces effective stress nearly to zero.	Occurs when effective stress becomes zero due to seepage pressure.
Condition	Associated with cyclic or dynamic loading conditions.	Occurs under steady seepage conditions.
Effect	Soil temporarily behaves like a viscous fluid causing settlement or failure of structures.	Soil particles are lifted and flow with water causing piping or sand boiling
Prevention	Densification, drainage, vibro-compaction, grouting	Reduce hydraulic gradient using filters, cutoffs, or deeper foundations.

1. (c) Solution:

Given Data

$$\gamma_d = 1.65 \text{ g/cc}$$

$$w = 14.5\% = 0.145$$

$$G_s = 2.70$$

$$\gamma_w = 1 \text{ g/cc}$$

Degree of Saturation at OMC

Relation between dry density and void ratio:

$$\rho_d = \frac{G_s \rho_w}{1 + e}$$

$$\Rightarrow 1.65 = \frac{2.70 \times 1}{1 + e}$$

$$\Rightarrow 1 + e = \frac{2.70}{1.65}$$

$$\Rightarrow e = 0.636$$

Now,

$$S = \frac{wG_s}{e} = \frac{0.145 \times 2.70}{0.636}$$

$$\Rightarrow S = 0.616 = 61.6\%$$

Percentage Air voids:
$$\rho_d = \frac{(1 - n_a)G\rho_w}{1 + wG}$$

$$\Rightarrow 1.65 = \frac{(1 - n_a) \times 2.70 \times 1}{1 + 0.145 \times 2.70}$$

$$\Rightarrow n_a = 14.96\%$$

Theoretical Dry Density: The theoretical max dry density occurs when there are no air voids after compaction.

For zero air voids,

$$\therefore e = wG_s$$

$$\Rightarrow e = 0.145 \times 2.70$$

$$\Rightarrow e = 0.392$$

Now,

$$\rho_{d(ZAV)} = \frac{G_s \rho_w}{1 + e}$$

$$\Rightarrow \rho_{d(ZAV)} = \frac{2.70 \times 1}{1 + 0.392} = 1.94 \text{ g/cc}$$

1. (d) Solution:

Conversion of Footing Loads to Equivalent Point Loads

Footing A

$$P_A = 300 \text{ kPa} \times (2.5 \text{ m} \times 2.5 \text{ m})$$

$$\Rightarrow P_A = 1875 \text{ kN}$$

Footing B

$$P_B = 300 \text{ kPa} \times (3 \text{ m} \times 3 \text{ m})$$

$$\Rightarrow P_B = 2700 \text{ kN}$$

Application of Boussinesq Formula

The vertical stress at a depth z below the ground surface due to a point load P is given by:

$$\sigma_x = \frac{3P}{2\pi z^2} \left[\frac{1}{1 + (r/z)^2} \right]^{5/2}$$

(i) Total Vertical Stress at 3 m below the Center of Footing A

At this point:

- Radial distance from Footing A: $r_A = 0$
- Radial distance from Footing B: $r_B = 6$ m

Stress due to Footing A

$$\sigma_{zA} = \frac{3 \times 1875}{2\pi \times 3^2} \left[\frac{1}{1 + (0/3)^2} \right]^{5/2} = \frac{5625}{56.548} \times [1]^{2.5}$$

$$\Rightarrow \sigma_{zA} = 99.47 \text{ kPa}$$

Stress due to Footing B

$$\sigma_{zB} = \frac{3 \times 2700}{2\pi \times 3^2} \left[\frac{1}{1 + (6/3)^2} \right]^{5/2}$$

$$\Rightarrow \sigma_{zB} = \frac{8100}{56.548} \times \left[\frac{1}{1 + 4} \right]^{2.5} = 143.24 \times 0.01789$$

$$\Rightarrow \sigma_{zB} = 2.56 \text{ kPa}$$

Total Vertical Stress

$$\sigma_{z(i)} = 99.47 + 2.56 = 102.03 \text{ kPa}$$

(ii) Total Vertical Stress at 3 m below the Center of Footing B

At this point:

- Radial distance from Footing B: $r_B = 0$
- Radial distance from Footing A: $r_A = 6$ m

Stress due to Footing B

$$\sigma_{zB} = \frac{3 \times 2700}{2\pi \times 3^2} \left[\frac{1}{1 + (0/3)^2} \right]^{5/2}$$

$$\Rightarrow \sigma_{zB} = \frac{8100}{56.548} \times [1]^{2.5}$$

$$\Rightarrow \sigma_{zB} = 143.24 \text{ kPa}$$

Stress due to Footing A

$$\sigma_{zA} = \frac{3 \times 1875}{2\pi \times 3^2} \left[\frac{1}{1 + (6/3)^2} \right]^{5/2}$$

$$\Rightarrow \sigma_{zA} = \frac{5625}{56.548} \times \left[\frac{1}{1 + 4} \right]^{2.5}$$

$$\Rightarrow \sigma_{zA} = 1.78 \text{ kPa}$$

Total Vertical Stress

$$\sigma_{z(ii)} = 143.24 + 1.78 = 145.02 \text{ kPa}$$

(iii) Total Vertical Stress at 3 m below the Midpoint Between Footings

At the midpoint:

- Radial distance from both footings: $r = 3 \text{ m}$

Stress due to Footing A

$$\sigma_{zA} = \frac{3 \times 1875}{2\pi \times 3^2} \left[\frac{1}{1 + (3/3)^2} \right]^{5/2}$$

$$\Rightarrow \sigma_{zA} = 99.47 \times \left[\frac{1}{1 + 1} \right]^{2.5} = 99.47 \times (0.5)^{2.5}$$

$$\Rightarrow \sigma_{zA} = 17.58 \text{ kPa}$$

Stress due to Footing B

$$\sigma_{zB} = \frac{3 \times 2700}{2\pi \times 3^2} \left[\frac{1}{1 + (3/3)^2} \right]^{5/2}$$

$$\Rightarrow \sigma_{zB} = 143.24 \times (0.5)^{2.5}$$

$$\Rightarrow \sigma_{zB} = 25.32 \text{ kPa}$$

Total Vertical Stress

$$\sigma_{z(iii)} = 17.58 + 25.32 = 42.90 \text{ kPa}$$

1. (e) Solution:**Borrow area**

$$w = 16\%$$

$$e = 0.55$$

$$G = 2.7$$

Embankment

$$V = 60,000 \text{ m}^3$$

$$w = 19\%$$

$$\rho_d = 18.0 \text{ kN/m}^3$$

1. Properties of Undisturbed Borrow Material

Given:

$$w = 0.16, e = 0.55, G_s = 2.7, \gamma_w = 10 \text{ kN/m}^3$$

Dry Unit Weight (γ_d)

$$\gamma_d = \frac{G_s \gamma_w}{1 + e} = \frac{2.7 \times 10}{1 + 0.55} = 17.42 \text{ kN/m}^3$$

Bulk Unit Weight (γ)

$$\gamma = \gamma_d(1 + w) = 17.42(1 + 0.16) = 20.21 \text{ kN/m}^3$$

Degree of Saturation (S)

$$S = \frac{wG_s}{e} = \frac{0.16 \times 2.7}{0.55} = 0.785 = 78.5\%$$

2. Number of Truck Loads Required

Total Dry Weight Required for Embankment

$$W_{d,\text{total}} = \text{Volume} \times \gamma_{d,\text{compacted}} = 60,000 \times 18.0 = 1,080,000 \text{ kN}$$

Dry Weight of Soil per Truck

Given: each truck carries 70 kN of soil at $w = 16\%$

$$W_{d,\text{truck}} = \frac{W_{\text{bulk}}}{1 + w} = \frac{70}{1.16} = 60.3448 \text{ kN}$$

Number of Truck Loads

$$N = \frac{W_{d,\text{total}}}{W_{d,\text{truck}}} = \frac{1,080,000}{60.3448} \approx 17898 \text{ loads}$$

3. Water to be Added per Truck Load

Increase in water content:

$$\Delta w = 19\% - 16\% = 0.03$$

Weight of Water to be Added

$$W_w = \Delta w \times W_{d,\text{truck}} = 0.03 \times 60.34 = 1.81 \text{ kN}$$

Volume of Water Added

$$V_w = \frac{1.81}{10} \times 1000 = 181 \text{ liters per truck}$$

2. (a) Solution:

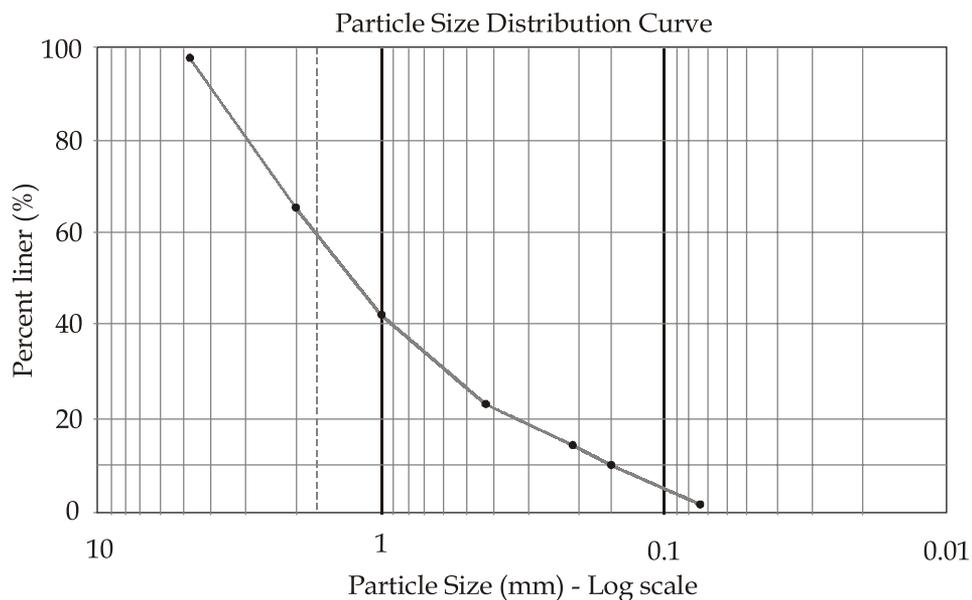
Sieve Analysis of Soil Sample

Total mass of sample, $W = 500$ g

Sieve Size (mm)	Mass Retained (g)	Cumulative Mass Retained (g)	Cumulative % Retained = $\frac{\text{Cumulative mass}}{500} \times 100$	% Finer = $100 - \text{Cumulative \% retained}$
4.75	12	12	2.4	97.6
2.00	160	172	34.4	65.6
1.00	115	287	57.4	42.6
0.425	95	382	76.4	23.6
0.212	45	427	85.4	14.6
0.150	25	452	90.4	9.6
0.075	40	492	98.4	1.6
Pan	8	500	100.0	0.0

Soil FractionsGravel (greater than 4.75 mm): Gravel = $100 - 97.6 = 2.4\%$ Coarse Sand (4.75 – 2.00 mm): Coarse Sand = $97.6 - 65.6 = 32.0\%$ Medium Sand (2.00 – 0.425 mm): Medium Sand = $65.6 - 23.6 = 42.0\%$ Fine Sand (0.425 – 0.075 mm): Fine Sand = $23.6 - 1.6 = 22.0\%$ Silt-Clay Fraction (less than 0.075 mm) Fines = 1.6%

Since sand > 50% → soil is primarily sand.



Effective Sizes (From Semi-Log Plot)

$$\log_{10}(D_{60}) = \log_{10}(1) + \frac{\log_{10}(2) - \log_{10}(1)}{65.6 - 42.6}(60 - 42.6)$$

$$D_{60} = 1.69 \text{ mm}$$

$$\log_{10}(D_{30}) = \log_{10}(0.425) + \frac{\log_{10}(1) - \log_{10}(0.425)}{42.6 - 23.6}(30 - 23.6)$$

$$D_{30} = 0.57 \text{ mm}$$

$$\log_{10}(D_{10}) = \log_{10}(0.150) + \frac{\log(0.212) - \log(0.150)}{14.6 - 9.6}(10 - 9.6)$$

$$D_{10} = 0.15 \text{ mm}$$

Coefficients of Gradation

Uniformity Coefficient

$$C_u = \frac{D_{60}}{D_{10}} = \frac{1.69}{0.15} = 11.27$$

Coefficient of Curvature

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

$$\Rightarrow C_c = \frac{(0.57)^2}{1.69 \times 0.15} = \frac{0.3249}{0.2535}$$

$$\Rightarrow C_c = 1.28$$

Gradation

For well-graded sand:

$$C_u > 6 : \text{Condition ok}$$

$$1 < C_c < 3 \text{ Condition Ok}$$

Soil Classification = Well-Graded Sand (SW)

2. (b) Solution:

Given Data

Two triaxial test specimens were tested under different confining pressures, deviator stresses, and pore pressures.

- **Specimen 1**

Cell pressure, $\sigma_3 = 100 \text{ kPa}$

Deviator stress, $\Delta\sigma_d = 160 \text{ kPa}$

Pore pressure, $u = 40$ kPa

- **Specimen 2**

Cell pressure, $\sigma_3 = 300$ kPa

Deviator stress, $\Delta\sigma_d = 320$ kPa

Pore pressure, $u = 120$ kPa

1. Calculation of Principal Stresses

The major principal stress is calculated as:

$$\sigma_1 = \sigma_3 + \Delta\sigma_d$$

The effective stresses are obtained by subtracting pore pressure from the total stresses.

Specimen 1

Total major principal stress: $\sigma_1 = 100 + 160 = 260$ kPa

Effective minor principal stress: $\sigma'_3 = 100 - 40 = 60$ kPa

Effective major principal stress: $\sigma'_1 = 260 - 40 = 220$ kPa

Specimen 2

Total major principal stress: $\sigma_1 = 300 + 320 = 620$ kPa

Effective minor principal stress: $\sigma'_3 = 300 - 120 = 180$ kPa

Effective major principal stress: $\sigma'_1 = 620 - 120 = 500$ kPa

2. Total Stress Shear Strength Parameters (c, ϕ)

The Mohr-Coulomb failure criterion in terms of total stress is written as:

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

where

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

Substituting the values for the two specimens:

- For Specimen 1:

$$260 = 100 N_\phi + 2c\sqrt{N_\phi} \quad \dots(i)$$

- For Specimen 2:

$$620 = 300 N_\phi + 2c\sqrt{N_\phi} \quad \dots(ii)$$

Subtracting the equation (i) from the (ii) gives:

$$360 = 200 N_\phi$$

$$N_{\phi} = 1.8$$

Using the relationship between N_{ϕ} and ϕ :

$$\tan^2\left(45^\circ + \frac{\phi}{2}\right) = 1.8$$

$$\phi = 16.6^\circ$$

Substituting

$N_{\phi} = 1.8$ back into the first equation to find cohesion:

$$260 = 100(1.8) + 2c\sqrt{1.8}$$

$$c = 29.83 \text{ kPa}$$

3. Effective Stress Shear Strength Parameters (c' , ϕ')

Using effective stresses, the failure criterion becomes:

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

Substituting effective stresses:

- For Specimen 1:

$$220 = 60N_{\phi'} + 2c'\sqrt{N_{\phi'}} \quad \dots(i)$$

- For Specimen 2:

$$500 = 180N_{\phi'} + 2c'\sqrt{N_{\phi'}} \quad \dots(ii)$$

Subtracting the equations (iii) from (iv) gives:

$$280 = 120 N_{\phi'}$$

$$N_{\phi'} = 2.33$$

Solving for the effective friction angle:

$$\tan^2\left(45^\circ + \frac{\phi'}{2}\right) = 2.33$$

$$\Rightarrow \phi' = 23.578^\circ$$

Substituting back to obtain effective cohesion:

$$220 = 60(2.33) + 2c'\sqrt{2.33}$$

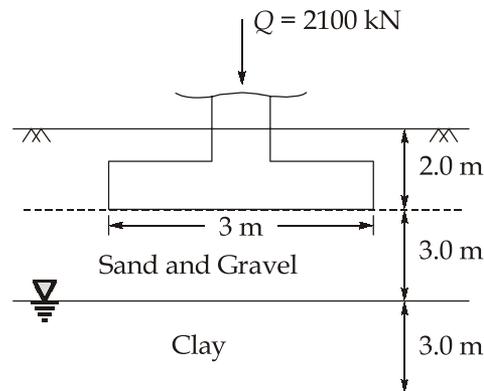
$$\Rightarrow c' = 26.27 \text{ kPa}$$

Final Results

- Total stress parameters:
 - Cohesion, $c = 29.8 \text{ kPa}$
 - Friction angle, $\phi = 16.6^\circ$

- Effective stress parameters:
 - Cohesion, $c' = 26.3 \text{ kPa}$
 - Friction angle, $\phi' = 23.6^\circ$

2. (c) (i) Solution:



Soil Properties (Clay Layer)

Saturated Unit Weight

$$\gamma_{\text{sat}} = \frac{(G_s + e_0)\gamma_w}{1 + e_0}$$

$$\gamma_{\text{sat}} = \frac{(2.72 + 1.1) \times 9.81}{1 + 1.1} = \frac{37.47}{2.1} = 17.84 \text{ kN/m}^3$$

Submerged Unit Weight

$$\gamma' = \gamma_{\text{sat}} - \gamma_w$$

$$\gamma' = 17.84 - 9.81 = 8.03 \text{ kN/m}^3$$

Initial Effective Stress (σ'_0)

Depth to center of clay layer:

$$5.0 + 1.5 = 6.5 \text{ m}$$

Stress from Sand (Above Water Table)

$$\sigma'_{\text{sand}} = 5.0 \times 21.0 = 105.0 \text{ kPa}$$

Stress from Clay (at center)

$$\sigma'_{\text{clay}} = 1.5 \times 8.03 = 12.05 \text{ kPa}$$

Total Initial Effective Stress

$$\sigma'_0 = 105.0 + 12.05 = 117.05 \text{ kPa}$$

Stress Increase ($\Delta\sigma$)

Depth below footing base:

$$z = 6.5 - 2.0 = 4.5 \text{ m}$$

Using 2:1 distribution method:

$$\Delta\sigma = \frac{P}{(B+z)^2}$$

$$\Rightarrow \Delta\sigma = \frac{2100}{(3.0+4.5)^2} = \frac{2100}{56.25} = 37.33 \text{ kPa}$$

Consolidation Settlement (S_c)

$$S_c = \frac{C_c H}{1+e_0} \log_{10} \left(\frac{\sigma'_0 + \Delta\sigma}{\sigma'_0} \right)$$

$$\Rightarrow S_c = \frac{0.6 \times 3.0}{1+1.1} \log_{10} \left(\frac{117.05 + 37.33}{117.05} \right)$$

$$\Rightarrow S_c = 0.8571 \times \log_{10}(1.3189)$$

$$\Rightarrow S_c = 0.8571 \times 0.1202 = 0.103 \text{ m}$$

Total Expected Consolidation Settlement

$$S_c = 0.103 \text{ m } (\approx 103 \text{ mm})$$

2. (c) (ii) Solution:

Protective filters are provided in earth dams to prevent erosion and piping of base soil while allowing free drainage of seepage water. The design of filters is mainly based on grain size distribution criteria proposed by Karl von Terzaghi.

Filter Design Criteria

The filter material should satisfy the following conditions:

1. Retention Criterion (Prevention of Soil Migration)

The filter openings must be small enough to retain the base soil particles.

$$\frac{D_{15(\text{filter})}}{D_{85(\text{protected soil})}} < 5$$

Where

D_{15} = particle size for which 15% of soil is finer

D_{85} = particle size for which 85% of soil is finer

This criterion ensures no significant invasion of base soil particles into the filter.

2. Permeability Criterion (Adequate Drainage)

The filter should be sufficiently permeable to allow water to pass without excessive head loss.

$$4 < \frac{D_{15(\text{filter})}}{D_{15(\text{protected soil})}} < 20$$

This ensures that seepage water flows freely through the filter without build-up of pore pressure.

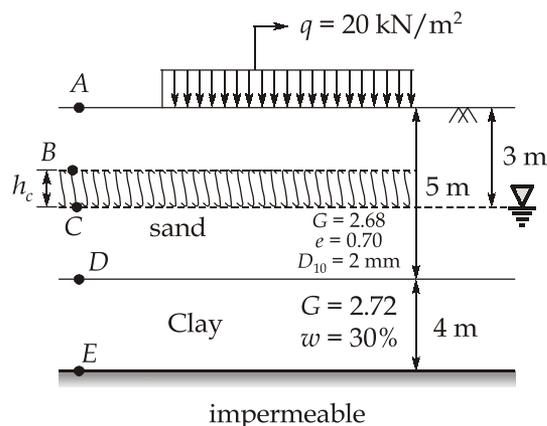
3. Gradation Criterion

The filter material should not be excessively coarse relative to the base soil.

$$\frac{D_{50(\text{filter})}}{D_{50(\text{protected soil})}} < 25$$

This criterion ensures proper compatibility between filter and protected soil gradation.

3. (a) (i) Solution:



Capillary Rise

$$h_c = \frac{C}{eD_{10}}$$

$$\Rightarrow h_c = \frac{0.5}{0.70 \times 0.02} = 35.71 \text{ cm} = 0.36 \text{ m}$$

Top of capillary zone:

$$z = 3.0 - 0.36 = 2.64 \text{ m}$$

Unit Weights

$$\gamma_d(\text{sand}) = \frac{G\gamma_w}{1+e} = \frac{2.66 \times 9.81}{1.70} = 15.35 \text{ kN/m}^3$$

$$\gamma_{\text{sat}}(\text{sand}) = \frac{(G + e)\gamma_w}{1 + e} = \frac{(2.66 + 0.70) \times 9.81}{1.70} = 19.39 \text{ kN/m}^3$$

For clay,

$$e = wG = 0.30 \times 2.72 = 0.816$$

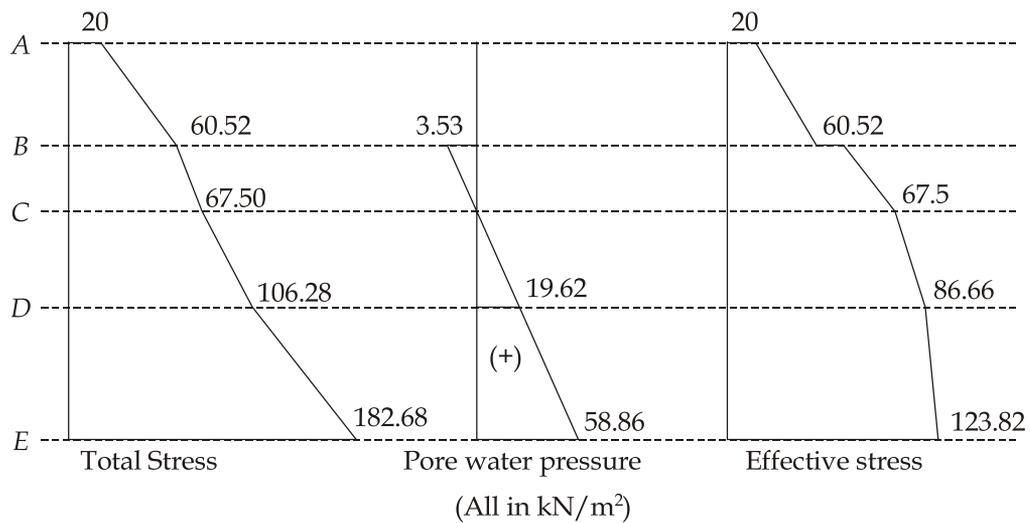
$$\gamma_{\text{sat}}(\text{clay}) = \frac{(G + e)\gamma_w}{1 + e} = \frac{(2.72 + 0.816) \times 9.81}{1.816} = 19.10 \text{ kN/m}^3$$

Stress Calculations

$$\sigma' = \sigma - u$$

Stress Computation Table

Point	Depth (m)	Total Stress σ (kN/m ²)	Pore Pressure u (kN/m ²)	Effective Stress σ' (kN/m ²)
A	0.00	$\sigma = 20$	$u = 0$	$\sigma' = 20 - 0 = 20.00$
B	2.64	$\sigma = 20 + (2.64 \times 15.35)$ $= 20 + 40.52 = 60.52$	$u = -0.36 \times 9.81$ $= -3.53$	$\sigma' = 60.52 - (-3.53) = 64.05$
C	3.00	$\sigma = 60.52 + (0.36 \times 19.39)$ $= 60.52 + 6.98 = 67.50$	$u = 0$	$\sigma' = 67.50 - 0 = 67.50$
D	5.00	$\sigma = 67.50 + (2.0 \times 19.39)$ $= 67.50 + 38.78$ $= 106.28$	$u = 2.0 \times 9.81 = 19.62$	$\sigma' = 106.28 - 19.62 = 86.66$
E	9.00	$\sigma = 106.28 + (4.0 \times 19.10)$ $= 106.28 + 76.40$ $= 182.68$	$u = 6.0 \times 9.81$ $= 58.86$	$\sigma' = 182.68 - 58.86$ $= 123.82$



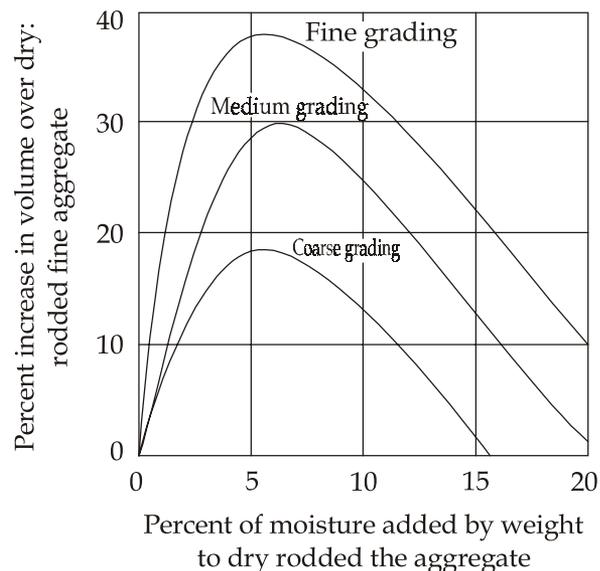
3. (a) (ii) Solution:

Bulking of Sand

Bulking of sand refers to the increase in volume of sand when it contains a small amount of moisture. This phenomenon occurs because a thin film of water forms around sand particles, creating surface tension that pushes the particles slightly apart. As a result, the apparent volume of sand increases without any actual increase in the quantity of solid particles.

Bulking is most significant in fine sands and occurs at low moisture contents (usually 4–8%). With further addition of water, the water films break down and the sand particles rearrange into a denser state, reducing the volume back toward its original condition. In fully saturated sand, bulking disappears.

In construction, bulking is important in concrete and mortar preparation. If moist sand is measured by volume without correcting for bulking, less sand than required will be used, affecting mix proportions and strength. Therefore, field tests (such as the simple bulking test using a measuring cylinder) are performed to determine the percentage bulking and apply necessary corrections.



3. (b) (i) Solution:

Given:

$$\text{Thickness of sand layer} = 22 \text{ m}$$

$$\text{Water table depth} = 3.5 \text{ m}$$

$$\text{Initial saturated thickness: } H = 22 - 3.5 = 18.5 \text{ m}$$

$$\text{Discharge: } Q = 25 \text{ lit/sec} = 0.025 \text{ m}^3/\text{s}$$

Observation wells:

$$r_1 = 10 \text{ m}, r_2 = 25 \text{ m}$$

$$h_1 = 18.5 - 2.15 = 16.35 \text{ m}$$

$$h_2 = 18.5 - 1.60 = 16.90 \text{ m}$$

(i) Coefficient of Permeability (k)

For an unconfined aquifer:

$$k = \frac{Q \log_{10} (r_2 / r_1)}{\pi (h_2^2 - h_1^2)} = \frac{0.025 \times \log_e \left(\frac{25}{10} \right)}{\pi \times (16.90^2 - 16.35^2)}$$

$$\Rightarrow k = 3.98 \times 10^{-4} \text{ m/s}$$

$$\Rightarrow k = 34.39 \text{ m/day}$$

Radius of Influence (R)

$$\log_e \left(\frac{R}{r_1} \right) = \frac{k\pi (H^2 - h_1^2)}{Q}$$

$$\Rightarrow \log_e \left(\frac{R}{10} \right) = \frac{3.98 \times 10^{-4} \times \pi \times (18.5^2 - 16.35^2)}{0.025}$$

$$\Rightarrow R = 424.12 \text{ m}$$

(iii) Effective Size (Allen Hazen's Formula)

$$\text{Hazen's relation: } k = CD_{10}^2$$

$$\text{Convert } k \text{ to cm/s: } k = 3.98 \times 10^{-4} \text{ m/s} = 0.0398 \text{ cm/s}$$

$$0.0398 = 110 D_{10}^2$$

$$\Rightarrow D_{10} = \sqrt{\frac{0.0398}{110}}$$

$$\Rightarrow D_{10} = 0.019 \text{ cm} = 0.19 \text{ mm}$$

3. (b) (ii) Solution:

Expansive soils (such as black cotton soil) undergo significant volume change with variation in moisture content. They swell during wet seasons and shrink during dry periods, causing heaving, cracking, and differential settlement. Therefore, foundations in expansive soils must either avoid the active zone, resist movement, or isolate the structure from soil volume changes.

The commonly adopted foundation types are discussed below.

1. Under-Reamed Pile Foundation

This is the most suitable and widely recommended foundation in expansive soils (as per IS 2911 Part III).

Under-reamed piles have one or more enlarged bulbs along the shaft.

- The pile is extended below the zone of seasonal moisture variation.
- The bulbs provide anchorage against uplift due to swelling pressure.
- Suitable for low- to medium-rise buildings.

These foundations effectively resist heave and provide good stability.

2. Pier (Drilled Shaft) Foundation

Bored cast-in-situ concrete piers extended below the active zone are used.

- The shaft passes through expansive soil and rests in stable strata.
- Sometimes a belled base is provided to resist uplift.
- Used for heavier structures.

3. Raft (Mat) Foundation

A rigid raft spreads the load over a large area and helps in reducing differential settlement.

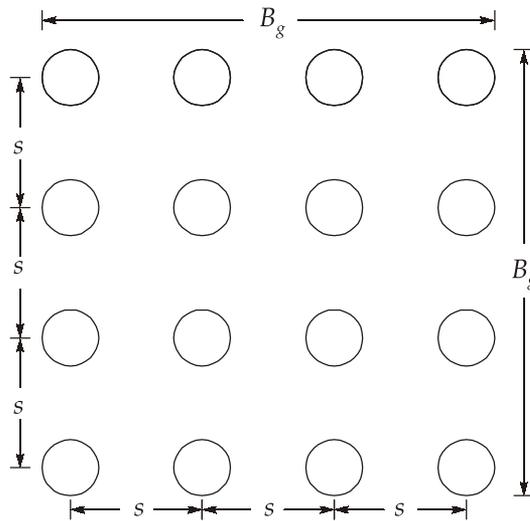
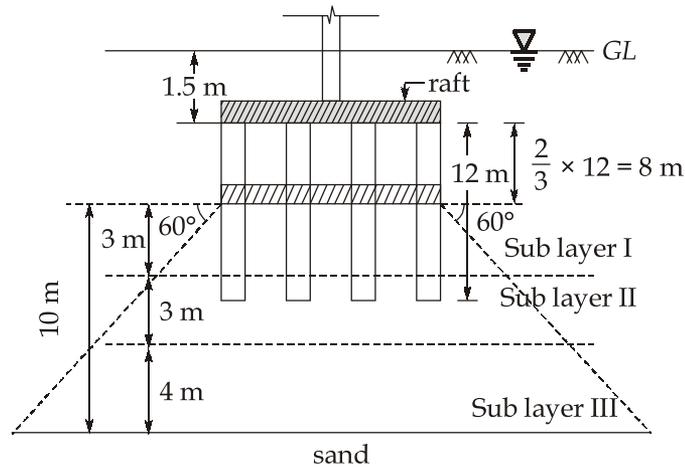
- Suitable where swelling is moderate.
- Often combined with sand cushioning or void formers below the raft.
- Reinforcement is designed to resist bending due to differential heave.

4. Pile Foundation (Friction or End Bearing)

Straight piles taken below the expansive layer into firm strata (such as dense sand or rock) are used.

- The pile transfers load to stable soil.
- Uplift forces due to swelling must be considered in design.

3. (c) Solution:



Plan

Soil Property Calculations

First, we determine the compression characteristics and the initial state of the clay stratum.

Compression Index (C_c):

Using the empirical formula for normally consolidated clay:

$$C_c = 0.009 (w_L - 10) = 0.009(50 - 10) = 0.360$$

Initial Void Ratio (e_0):

Assuming the soil is fully saturated ($S = 100\%$) because the water table is at the ground level:

$$e_0 = \frac{w \cdot G}{S} = \frac{0.32 \times 2.67}{1} = 0.8544$$

Submerged Unit Weight (γ'):

$$\gamma' = \frac{(G-1) \cdot \gamma_w}{1+e_0} = \frac{(2.67-1) \cdot 9.81}{1.8544} = 8.834 \text{ kN/m}^3$$

Equivalent Raft Geometry and Loading

We treat the pile group as an equivalent raft to simplify the stress distribution analysis.

Width of Pile Group (B_g):

For a 4×4 group with 1.2 m spacing and 0.4 m diameter:

$$B_g = (3 \times 1.2) + 0.4 = 4.0 \text{ m}$$

Depth of Equivalent Raft (D_{eq}):

Located at $2/3$ of the pile length from the top of the piles:

$$\begin{aligned} D_{eq} &= 1.5 \text{ m (founding depth)} + \frac{2}{3} (12 \text{ m}) \\ &= 9.5 \text{ m below G.L.} \end{aligned}$$

Total Load (Q):

$$Q = 350 \text{ t} \times 9.81 = 3433.5 \text{ kN}$$

Load Dispersion Angle:

A 60° angle with the horizontal implies a 30° angle with the vertical.

Width at depth z :

$$B_z = B_g + 2z \tan(30^\circ) = 4.0 + 1.155z$$

Sublayer Stress and Settlement Calculations

The 10 m clay layer below the equivalent raft is divided into three sublayers.

Sublayer 1 ($H = 3 \text{ m}$)

Midpoint Depth (z):

1.5 m below the equivalent raft (11.0 m below G.L.).

Effective Overburden (σ'_0):

$$\sigma'_0 = (9.5 + 1.5) \times 8.834 = 97.17 \text{ kN/m}^2$$

Stress Increase ($\Delta\sigma$):

$$\begin{aligned} B_z &= 4.0 + 2(1.5)\tan(30^\circ) = 5.732 \text{ m} \\ \Delta\sigma &= \frac{Q}{(5.732)^2} = \frac{3433.5}{(5.732)^2} = 104.50 \text{ kN/m}^2 \end{aligned}$$

Settlement (S_{C1}):

$$S_{C1} = \frac{C_e H_e}{1 + e_o} \log_{10} \left(\frac{\sigma'_0 + \Delta\sigma'}{\sigma'_0} \right)$$

$$\Rightarrow S_{C1} = \frac{0.360 \times 3}{1.8544} \log_{10} \left(\frac{97.17 + 104.50}{97.17} \right)$$

$$\Rightarrow S_{C1} = 0.1847 \text{ m} = 184.7 \text{ mm}$$

Sublayer 2 ($H = 3 \text{ m}$)

Midpoint Depth (z):

4.5 m below the equivalent raft (14.0 m below G.L.).

Effective Overburden (σ'_0):

$$\sigma'_0 = (9.5 + 3 + 1.5) \times 8.834 = 123.68 \text{ kN/m}^2$$

Stress Increase ($\Delta\sigma$):

$$B_z = 4.0 + 2(4.5)\tan(30^\circ) = 9.196 \text{ m}$$

$$\Rightarrow \Delta\sigma = \frac{3433.5}{(9.196)^2} = 40.60 \text{ kN/m}^2$$

Settlement (S_{C2}):

$$S_{C2} = \frac{0.360 \times 3}{1.8544} \log_{10} \left(\frac{123.68 + 40.60}{123.68} \right)$$

$$\Rightarrow S_{C2} = 0.0718 \text{ m} = 71.8 \text{ mm}$$

Sublayer 3 ($H = 4 \text{ m}$)

Midpoint Depth (z):

8.0 m below the equivalent raft (17.5 m below G.L.).

Effective Overburden (σ'_0):

$$\sigma'_0 = (9.5 + 3 + 3 + 2) \times 8.834 = 154.60 \text{ kN/m}^2$$

Stress Increase ($\Delta\sigma$):

$$B_z = 4.0 + 2(8.0)\tan(30^\circ) = 13.238 \text{ m}$$

$$\Delta\sigma = \frac{3433.5}{(13.238)^2} = 19.59 \text{ kN/m}^2$$

Settlement (S_{C3}):

$$S_{C3} = \frac{0.360 \times 4}{1.8544} \log_{10} \left(\frac{154.60 + 19.59}{154.60} \right)$$

$$\Rightarrow S_{c3} = 40.2 \text{ mm}$$

Total Settlement

Summing the individual sublayers:

$$S_{\text{total}} = S_{c1} + S_{c2} + S_{c3}$$

$$\Rightarrow S_{\text{total}} = 184.7 + 71.8 + 40.2 = 296.7 \text{ mm}$$

The total probable consolidation settlement is 296.7 mm.

4. (a) (i) Solution:

Layer 1: Clayey Sand (Top 3 m)

$$\gamma_1 = 1.85 \text{ t/m}^3$$

$$\phi_1 = 24^\circ$$

$$c = 1 \text{ t/m}^2$$

Active pressure coefficient:

$$K_{a1} = \frac{1 - \sin(24^\circ)}{1 + \sin(24^\circ)} \approx 0.422$$

Layer 2: Dense Sand (Bottom 1.5 m)

$$\gamma_2 = 1.95 \text{ t/m}^3$$

$$\phi_2 = 36^\circ$$

$$c = 0$$

Active pressure coefficient:

$$k_{a2} = \frac{1 - \sin(36^\circ)}{1 + \sin(36^\circ)} \approx 0.26$$

2. Calculation of Lateral Earth Pressures (σ_a)

The active pressure is given by

$$\sigma_a = \gamma z K_a - 2c\sqrt{K_a}$$

At $z = 0$ (Surface):

$$\sigma_a = 0 - 2(1)\sqrt{0.422} = -1.299 \text{ t/m}^2$$

At $z = 3$ m (Bottom of Layer 1):

$$\sigma_a = (1.85 \times 3 \times 0.422) - 2 \times 1 \times \sqrt{0.422}$$

$$\Rightarrow \sigma_a = 2.342 - 1.299 = 1.043 \text{ t/m}^2$$

At $z = 3$ m (Top of Layer 2):

Overburden surcharge from Layer 1:

$$q = \gamma_1 z_1 = 1.85 \times 3 = 5.55 \text{ t/m}^2$$

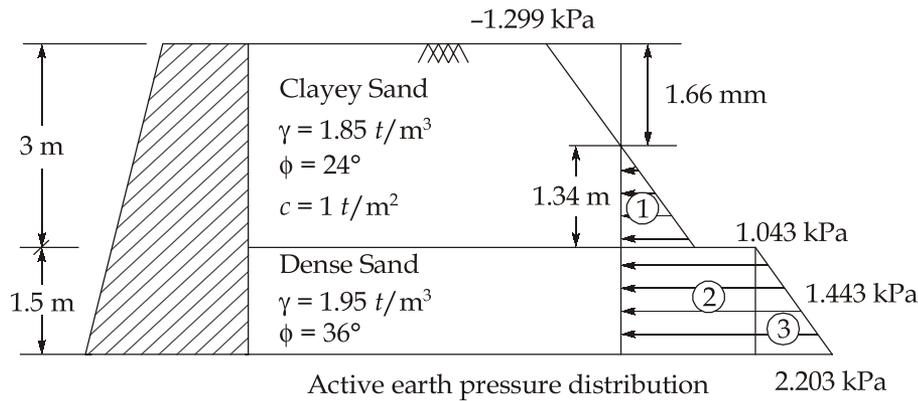
$$\sigma_a = qK_{a2} = 5.55 \times 0.26 = 1.443 \text{ t/m}^2$$

At $z = 4.5 \text{ m}$ (Base of Wall):

$$\sigma_a = 1.443 + (\gamma_2 \times 1.5 K_{a2})$$

$$\Rightarrow \sigma_a = 1.443 + (1.95 \times 1.5 \times 0.26)$$

$$\Rightarrow \sigma_a = 1.443 + 0.76 = 2.203 \text{ t/m}^2$$



3. Total Active Thrust (P_a)

Depth of zero pressure in Layer 1:

$$z_0 = \frac{2c}{\gamma\sqrt{K_a}} = \frac{2(1)}{1.85\sqrt{0.422}} \approx 1.66 \text{ m}$$

Only positive pressure area is considered.

Force from triangular portion in Layer 1:

$$P_1 = \frac{1}{2}(3 - 1.66)(1.043) = 0.699 \text{ t/m} = 0.70 \text{ t/m}$$

Force from rectangular portion in Layer 2:

$$P_2 = 1.443 \times 1.5 = 2.16 \text{ t/m}$$

Force from triangular portion in Layer 2:

$$P_3 = \frac{1}{2} \times 1.5 \times (2.203 - 1.443) = 0.57 \text{ t/m}$$

Total active thrust:

$$P_a = 0.70 + 2.16 + 0.57 \approx 3.43 \text{ t/m}$$

4. Point of Application (\bar{y})

Distances from base:

$$y_1 = 1.5 + \frac{1}{3}(3 - 1.66) = 1.5 + 0.447 = 1.947 \text{ m}$$

$$y_2 = \frac{1.5}{2} = 0.75 \text{ m}$$

$$y_3 = \frac{1.5}{3} = 0.50 \text{ m}$$

Moment equilibrium about base:

$$\bar{y} = \frac{(0.70 \times 1.947) + (2.16 \times 0.75) + (0.57 \times 0.50)}{3.43}$$

$$\Rightarrow \bar{y} = 0.95 \text{ m above the base}$$

4. (a) (ii) Solution:

Given Data

Initial volume of saturated soil, $V_1 = 9.6 \text{ cm}^3$ Initial weight of saturated soil, $W_1 = 17.46 \text{ g}$ Dry weight of soil, $W_d = 11.58 \text{ g}$ Dry volume of soil, $V_d = 5.22 \text{ cm}^3$ Density of water, $\rho_w = 1.0 \text{ g/cm}^3$ 1. Shrinkage Limit (w_s)

The shrinkage limit is given by:

$$w_s = \left[\frac{W_1 - W_d}{W_d} - \frac{(V_1 - V_d)\rho_w}{W_d} \right] \times 100$$

$$\frac{W_1 - W_d}{W_d} = \frac{17.46 - 11.58}{11.58} = 0.5078$$

$$\frac{(V_1 - V_d)\rho_w}{W_d} = \frac{(9.6 - 5.22) \times 1.0}{11.58} = 0.3782$$

$$w_s = (0.5078 - 0.3782) \times 100 = 12.96\%$$

2. Shrinkage Ratio (SR)

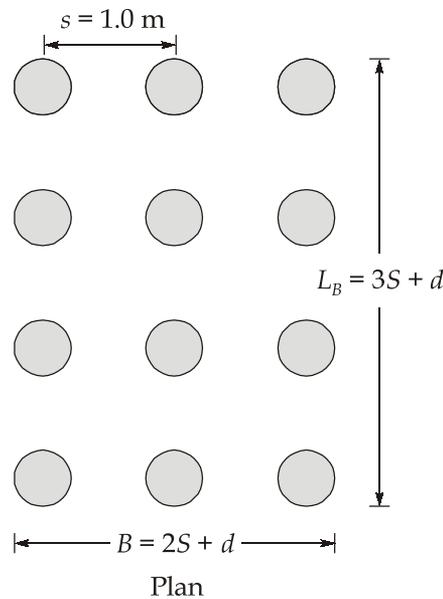
Shrinkage ratio is defined as:

$$SR = \frac{\rho_d}{\rho_w} = \frac{W_d}{V_d \times \rho_w}$$

$$\Rightarrow SR = \frac{11.58}{5.22 \times 1.0} = 2.22$$

4. (b) (i) Solution:

Identify Geometric and Soil Parameters



Pile Configuration:

$$n = 12 \text{ piles } (3 \times 4 \text{ formation})$$

Pile Details:

Diameter, $d = 0.4 \text{ m}$

Length, $L = 12 \text{ m}$

Soil Properties:

Undrained cohesion, $c = 4.5 \text{ t/m}^2$

Unit weight, $\gamma = 1.8 \text{ t/m}^3$

Adhesion factor, $\alpha = 0.9$

Spacing: $s = 1.0 \text{ m}$

Calculate Individual Pile Capacity

The ultimate capacity is the sum of the capacities developed by each of the 12 piles.

Skin Friction (Q_s)

$$Q_s = \alpha c(\pi dL)$$

⇒

$$Q_s = 0.9 \times 4.5 \times (\pi \times 0.4 \times 12) = 61.07 \text{ t}$$

End Bearing (Q_b)

$$Q_b = cN_c A_b$$

⇒

$$Q_b = 4.5 \times 9 \times \left(\frac{\pi}{4} \times 0.4^2 \right) = 5.09 \text{ t}$$

Total Individual Action (Q_{ug1})

$$Q_{ug1} = 12 \times (61.07 + 5.09) = 793.92 \text{ t}$$

Net Block Capacity

In this mode, the entire pile group is treated as a single solid rectangular block. Block

Dimensions

Width (B):

$$B = (2 \times 1.0) + 0.4 = 2.4 \text{ m}$$

Length (L_B):

$$L_B = (3 \times 1.0) + 0.4 = 3.4 \text{ m}$$

Gross Block Resistance ($Q_{ug2-gross}$)

Skin Friction:

$$Q_{s \text{ group}} = \alpha c (\text{perimeter}) L = 1 \times 4.5 [2(2.4 + 3.4)] \times 12 = 626.40 \text{ t}$$

Base Bearing:

$$Q_{s \text{ group}} = CN_c A_{base} = (2.4 \times 3.4) \times 9 \times 4.5 = 330.48 \text{ t}$$

Total Gross Resistance:

$$Q_{ug-gross} = 626.40 + 330.48 = 956.88 \text{ t}$$

Deduction of Block Weight (W_{block})

$$W_{block} = (B \cdot L_B \cdot L) \cdot \gamma$$

⇒

$$W_{block} = (2.4 \cdot 3.4 \cdot 12) \cdot 1.8 = 176.26 \text{ t}$$

Net Block Capacity ($Q_{ug2-net}$)

$$Q_{ug2-net} = 956.88 - 176.26 = 780.62 \text{ t}$$

Determine Governing Allowable Capacity

Net ultimate Capacity of Pile Group:

$$Q_{ug} = \min (793.92, 780.62) = 780.62 t$$

Allowable Net Capacity (FOS = 2):

$$Q_{\text{allowable}} = \frac{780.62}{2} = 390.31 t$$

Allowable net capacity of the pile group = 390.31 t

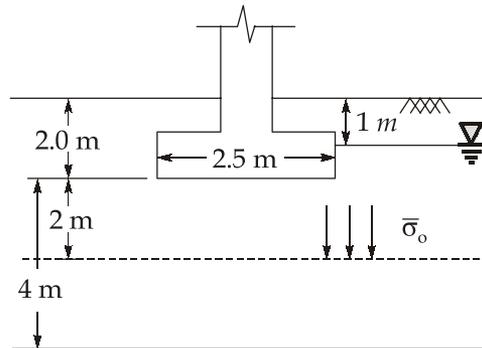
4. (b) (ii) Solution:

When fixing the depth of foundation, the following factors must be kept in mind to ensure safety, serviceability, and durability of the structure:

1. **Safe Bearing Capacity of Soil:** The foundation should rest at a depth where soil has adequate bearing capacity. Weak topsoil, filled-up ground, or organic layers must be avoided.
2. **Settlement Considerations:** Depth influences stress distribution and consolidation. The foundation should be placed such that total and differential settlements remain within permissible limits.
3. **Seasonal Moisture Variation:** In expansive soils, the depth must be below the zone of seasonal moisture fluctuation to prevent shrink-swell problems.
4. **Frost Depth (Cold Regions):** Foundation must be taken below the frost line to prevent frost heave.
5. **Groundwater Table:** High groundwater reduces effective stress and bearing capacity. Possible seasonal fluctuations should also be considered.
6. **Scour Depth (For Bridge Foundations):** In river or coastal areas, foundation depth must be below the maximum expected scour depth.
7. **Adjacent Structures:** Depth should ensure that the new foundation does not adversely affect nearby existing foundations.
8. **Type and Magnitude of Loads:** Heavy loads may require deeper foundations to mobilize higher bearing resistance and reduce settlement.
9. **Minimum Depth as per Codes:** As per IS recommendations, minimum depth of foundation in soil (other than rock) should generally not be less than 0.5 m to ensure lateral stability and protection against surface effects.
10. **Erosion and Surface Conditions:** In sloping or erosion-prone areas, sufficient embedment is required to prevent exposure of foundation.
11. **Underground Utilities:** Depth must account for existing services such as pipelines, drains, and cables.

12. Structural Stability Requirements: Adequate embedment is needed to resist sliding, overturning, and uplift forces (especially in retaining walls, towers, and waterfront structures).

4. (c) Solution:



Given:

Footing Dimensions

$$B = 2.5\text{ m}, L = 2.5\text{ m}$$

Depth of Foundation

$$D_f = 2.0\text{ m}$$

Soil Properties

$$c = 3.5\text{ t/m}^2, \phi = 0^\circ, \gamma = 1.75\text{ t/m}^3$$

Settlement Parameters

$$C_c = 0.280, e_0 = 0.90, S_c = 0.05\text{ m}$$

Shear Failure Criterion (Skempton's Method)

For a square footing resting on saturated clay with $\phi = 0^\circ$, the net ultimate bearing capacity is given by:

$$q_{mu} = c \times N_c$$

Determination of Bearing Capacity Factor N_c

According to Skempton:

$$N_c = 6.0 \left[1 + 0.2 \left(\frac{D_f}{B} \right) \right]$$

(Valid for $D_f/B \leq 2.5$)

$$\frac{D_f}{B} = \frac{2.0}{2.5} = 0.8$$

$$N_c = 6.0[1 + 0.2(0.8)]$$

$$N_c = 6.96$$

Net Ultimate Bearing Capacity

$$q_{nu} = c \times N_c = 3.5 \times 6.96$$

$$q_{nu} = 24.36 \text{ t/m}^2$$

Net Safe Bearing Capacity (Factor of Safety = 3)

$$q_{ns} = \frac{q_{nu}}{3}$$

$$q_{ns} = 8.12 \text{ t/m}^2$$

Settlement Criterion

The allowable net pressure corresponding to the maximum permissible consolidation settlement of 0.05 m is determined.

Initial Effective Overburden Pressure (p_0)

Stress is calculated at the center of the compressible clay layer.

Depth of compressible layer, $H = 4.0 \text{ m}$

Mid-depth below ground surface:

$$D_f + \frac{H}{2} = 2.0 + 2.0 = 4.0 \text{ m}$$

Effective stress:

$$\bar{\sigma}_0 = (1.0 \times \gamma) + (3.0 \times \gamma_{\text{sub}})$$

$$\Rightarrow \bar{\sigma}_0 = (1.0 \times 1.75) + (3.0 \times 0.75)$$

$$\Rightarrow \bar{\sigma}_0 = 4.0 \text{ t/m}^2$$

Vertical Stress Increase (Δp)

Using the consolidation settlement equation:

$$S_c = \frac{C_c \cdot H}{1 + e_0} \log_{10} \left(\frac{\bar{\sigma}_0 + \Delta \bar{\sigma}}{\bar{\sigma}_0} \right),$$

$$\Rightarrow 0.05 = \frac{0.280 \times 4}{1 + 0.90} \log_{10} \left(\frac{4.0 + \Delta \bar{\sigma}}{4.0} \right)$$

$$0.05 = 0.5895 \log_{10} \left(\frac{4.0 + \Delta p}{4.0} \right)$$

$$\Rightarrow \log_{10} \left(\frac{4.0 + \Delta \bar{\sigma}_o}{4.0} \right) = 0.0848$$

$$\Rightarrow \frac{4.0 + \Delta \bar{\sigma}}{4.0} = 1.215$$

$$\Rightarrow \Delta \bar{\sigma} = 0.86 \text{ t/m}^2$$

Net Allowable Pressure at Base (q_{np})

Using the 2:1 load dispersion method at depth $z = 2.0$ m:

$$\Delta \bar{\sigma} = \frac{q_{np} \cdot B^2}{(B + z)^2}$$

$$\Rightarrow 0.86 = \frac{q_{np} \times 2.5^2}{(2.5 + 2.0)^2}$$

$$\Rightarrow 0.86 = \frac{6.25 q_{np}}{20.25}$$

$$\Rightarrow q_{np} = 2.78 \text{ t/m}^2$$

Final Determination of Allowable Net Bearing Capacity

- Based on shear failure: $q_{ns} = 8.12 \text{ t/m}^2$
- Based on settlement: $q_{np} = 2.78 \text{ t/m}^2$

Allowable net bearing capacity = 2.78 t/m^2

Section B : Environmental Engineering [All Topics]

5. (a) Solution:

(i) We have

$$P_0 = 40000 \quad t = 0$$

$$P_1 = 160000 \quad t_1 = 20 \text{ years}$$

$$P_2 = 400000 \quad t_2 = 40 \text{ years}$$

(a) Saturation population of the city

$$P_s = \frac{2P_0P_1P_2 - P_1^2(P_0 + P_2)}{P_0P_2 - P_1^2}$$

$$\Rightarrow P_s = \frac{2 \times 40000 \times 160000 \times 400000 - 160000^2 (40000 + 400000)}{40000 \times 400000 - 160000^2}$$

$$\Rightarrow P_s = 640000$$

(b) The expected population of the city in next 50 years, $t_3 = 40 + 50 = 90$ years

$$P = \frac{P_s}{1 + me^{-nt}}$$

Where

$$m = \frac{P_s - P_0}{P_0} = \frac{640000 - 40000}{40000} = 15$$

$$n = \frac{1}{t_1} \ln \left[\frac{P_0 (P_s - P_1)}{P_1 (P_s - P_0)} \right] = \frac{1}{20} \ln \left[\frac{40 \times (640 - 160)}{160 \times (640 - 40)} \right]$$

$$n = -0.0805$$

So,

$$P = \frac{640000}{1 + 15e^{-0.0805 \times 90}} = 633220$$

5. (a) (ii) Solution:

1. Specific Capacity (of a well)

Definition: Specific capacity is the rate of discharge of a well per unit drawdown.

$$\text{Unit} = \text{m}^3/\text{s}/\text{m}$$

$$\text{Specific Capacity} = \frac{Q}{s}$$

Where

- Q = discharge of the well (m^3/s or L/s)
- s = drawdown in the well (m)

2. Specific Yield (of an aquifer)

Definition: Specific yield is the ratio of the volume of water that drains from a saturated soil or rock by gravity to the total volume of the soil or rock.

$$\text{Specific Yield} = \frac{\text{Volume of water drained by gravity}}{\text{Total volume of soil}}$$

Units:

- Dimensionless (often expressed as a percentage)

5. (b) Solution:

(i) $Q = 900 \text{ l/min} = \frac{0.900}{60} \text{ m}^3/\text{s} = 0.015 \text{ m}^3/\text{sec}$

A_s = area of opening per m length of screen

$$\Rightarrow A_s = (\pi \cdot d) \times 1 \times P$$

$$\Rightarrow A_s = (\pi \times 0.15) \times \frac{13}{100} = 0.0612 \text{ m}^2/\text{m length of screen}$$

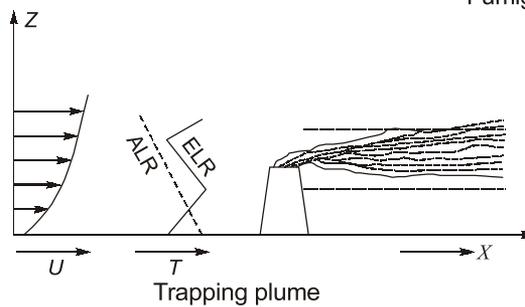
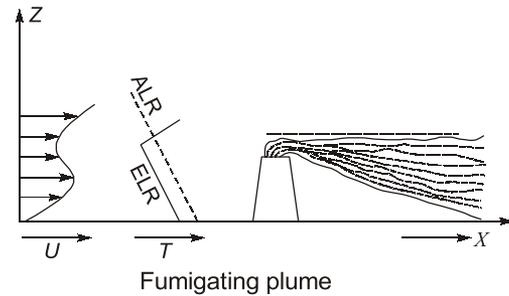
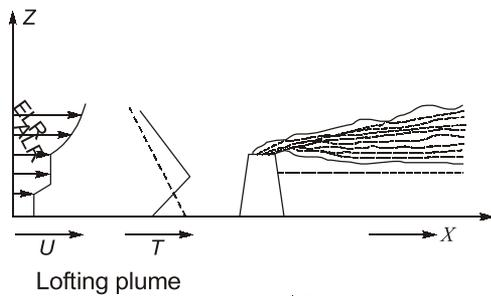
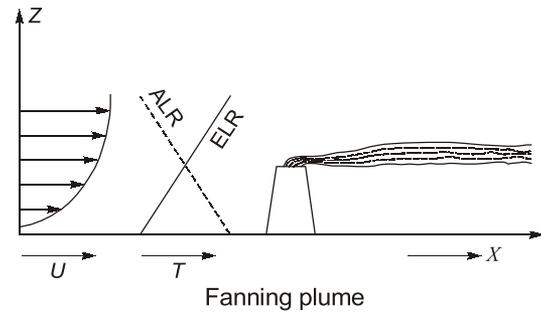
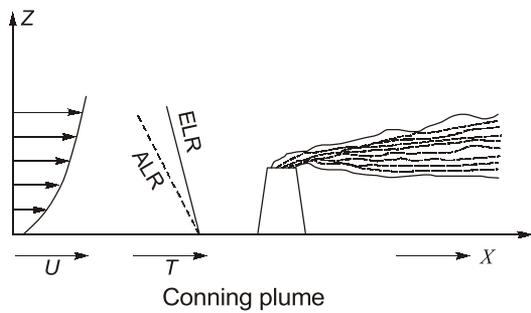
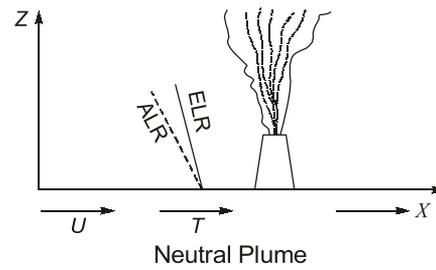
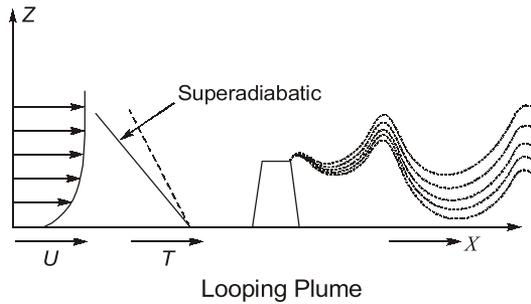
$$v_{\text{safe}} = 2 \text{ cm/sec}$$

So $Q = (v_{\text{safe}}) \times \text{area of opening per m length of screen} \times \text{length of strains}$

$$0.015 = 0.02 \times 0.0612 \times h$$

$$h = 12.25 \text{ m}$$

5. (b) (ii) Solution:



5. (c) Solution:

Given percent dilution = 10%

for determining the BOD, 300 mL bottle is used,

$$\therefore \text{Volume of river water sample} = 300 \times \frac{10}{100} = 30 \text{ ml}$$

Volume of distilled water = 300 - 30 = 270 ml

Initial DO of river water = 2.5 mg/l

Initial DO of filled waste = 6 mg/l

$$\therefore \text{Initial DO}_{\text{mix}} = \frac{2.5 \times 30 + 6 \times 270}{300} = 5.65 \text{ mg/l}$$

Given final DO_{mix} = 1.2 mg/l

\therefore 5 day standard BOD of river water

$$\text{BOD}_{5, 20^\circ\text{C}} = \frac{(5.65 - 1.2) \times 300}{30} = 44.5 \text{ mg/l}$$

Ultimate BOD remains constant at all temperature.

$$(\text{BOD})_u = \frac{(\text{BOD})_{5/20^\circ\text{C}}}{(1 - e^{-kt})} = \frac{44.5}{(1 - e^{-0.23 \times 5})} = 65.12 \text{ mg/l}$$

$$K_{27^\circ\text{C}} = K_{20} (\theta)^{T-20} \\ = 0.23 \times (1.047)^{(27-20)} = 0.317 \text{ per day}$$

$$\therefore \text{BOD}_{3, 27^\circ\text{C}} = \text{BOD}_u (1 - e^{-k_{27} \times t}) \\ = 65.12 \times (1 - e^{-0.317 \times 3}) \\ = 39.960 \text{ mg/l}$$

5. (d) Solution:

Given, Dia of settling tank $d = 25 \text{ m}$

Side water depth $H = 2 \text{ m}$

Total water flow = 25000 m³/day

$$\text{For each tank } Q = \frac{25000}{2} = 12500 \text{ m}^3/\text{d}$$

(i) Surface area and volume:

For circular tank,

$$V = d^2 (0.011 d + 0.785 H)$$

$$= 25^2 (0.011 \times 25 + 0.785 \times 2)$$

$$= 1158.125 \text{ m}^3$$

Surface area, $A = \frac{\pi D^2}{4} = \frac{\pi \times 25^2}{4}$

$$= 490.87 \text{ m}^2$$

(ii) Overflow rate:

$$\text{Overflow rate} = \frac{Q}{\text{Surface area}}$$

$$\therefore \text{Overflow rate} = \frac{12500}{490.87} = 25.464 \text{ m}^3/\text{m}^2/\text{d}$$

(iii) Detention time:

$$t_d = \frac{V}{Q} = \frac{1153.125}{12500} \text{ day} = 0.0928 \text{ day}$$

$$t_d = 0.0928 \times 24 \text{ hrs} = 2.227 \text{ hrs}$$

(iv) Weir loading:

$$\text{Weir loading} = \frac{Q}{\text{length of weir}}$$

\therefore length of weir is along the periphery of tank

So length of weir = $\pi \times d = 25 \pi \text{ m}$

$$\therefore \text{Weir loading} = \frac{12500}{25\pi} = 159.155 \text{ m}^3/\text{m}.\text{day}$$

5. (e) Solution:

Given, Fuel consumption = 2 ml/month

$$= 2 \times 12 \text{ ml/year}$$

$$= 24 \text{ ml/year}$$

Emission data for each ml of fuel per year

$$PM = 2.5 \text{ t/year}$$

$$SO_2 = 15 \text{ t/year}$$

Minimum height of chimney required

$$H_{\text{minimum}} = \max \begin{cases} 14(Q_{\text{SO}_2})^{1/3} \\ 74(Q_{\text{SPM}})^{0.27} \\ 30 \text{ m} \end{cases}$$

When Q_{SO_2} is in kg/hr

Q_{SPM} is in t/hr

$$\therefore Q_{\text{SO}_2} = 15 \text{ t/year for 1 ml fuel}$$

$$\therefore Q_{\text{SO}_2} = 15 \times 24 \text{ t/year for 24 ml fuel}$$

$$= \frac{15 \times 24 \times 1000}{365 \times 24} \text{ kg/hr}$$

$$= 41.095 \text{ kg/hr}$$

$$\therefore Q_{\text{SPM}} = 2.5 \text{ t/year for 1 ml of fuel}$$

$$= 2.5 \times 24 \text{ t/year for 24 ml of fuel}$$

$$= \frac{2.5 \times 24}{365 \times 24} \text{ t/hr}$$

$$= 6.849 \times 10^{-3} \text{ t/hr}$$

$$\text{So, } H_{\text{min}} = \max \begin{cases} 14 \times (41.095)^{1/3} = 48.312 \text{ m} \\ 74 \times (6.849 \times 10^{-3})^{0.27} = 19.269 \text{ m} \\ 30 \text{ m} \end{cases}$$

$$= 48.312 \text{ m,}$$

6. (a) Solution:

Given, $Q = 4.5 \text{ MLD}$

$$SS_o = \text{Total suspended solids in waste water}$$

$$= 4.5 \times 220 = 990 \text{ kg/day}$$

Suspended solids removed in primary clarifier will become influent to digester

$$\therefore \text{Total solids in raw sludge} = \frac{65}{100} \times 990 = 643.5 \text{ kg/day}$$

Given moisture content of influent (raw) sludge as 95%

i.e. 5 kg of dry solids, will make total wet raw sludge of 100 kg.

$$\begin{aligned} \therefore 643.5 \text{ kg of solids will make total } & \frac{100}{5} \times 643.5 \\ & = 12870 \text{ kg of wet sludge} \end{aligned}$$

Given S.G of raw sludge = 1.03

$$\begin{aligned} \therefore \text{Volume of primary sludge produced} \\ & = \frac{12870}{1.03 \rho_w} \quad (\text{Taken } \rho_w = 1000 \text{ kg/m}^3) \\ & = \frac{12870}{1.03 \times 1000} \\ V_1 & = 12.495 \text{ m}^3 \end{aligned}$$

$$\text{Mass of volatile solids in raw sludge} = \frac{70}{100} \times 643.5 = 450.45 \text{ kg}$$

$$\begin{aligned} \text{Mass of non volatile solids in raw sludge} & = 643.5 - 450.45 \\ & = 193.05 \text{ kg} \end{aligned}$$

As given 35% of solids gets destroyed so mass of volatile solids in digested sludge

$$= 450.45 \times \frac{(100 - 35)}{100} = 292.792 \text{ kg}$$

Mass of non volatile solids in digested sludge will remain same as in raw sludge

So total mass of solids in digested sludge.

$$\begin{aligned} & = 292.792 + 193.05 \\ & = 485.842 \text{ kg} \end{aligned}$$

Given moisture content of digested sludge = 92%

$$\begin{aligned} \therefore \text{Total mass of wet digested sludge} & = \frac{100}{100 - 92} \times 485.842 \\ & = 6073.025 \text{ kg/d} \end{aligned}$$

Given S.G. of digested sludge = 1.04

$$\begin{aligned} \therefore \text{Volume of digest sludge, } V_2 & = \frac{6073.025}{1.04 \times 1000} \\ & = 5.839 \text{ m}^3 \end{aligned}$$

\therefore Volume of the digester required,

$$V_{\text{required}} = \left[V_1 - \frac{2}{3}(V_1 - V_2) \right] \times t_d$$

$$= \left[12.495 - \frac{2}{3}(12.495 - 5.839) \right] \times 20$$

$$= 161.153 \text{ m}^3$$

6. (b) Solution:

(i) Given design life = 35 years

No. of households = 5000

City is having an average population 4 person per household

∴ Total population = $5000 \times 4 = 20,000$

Given solid waste generation rate = 2.5 kg per capita per day

So amount of MSW generated in a day

$$= 2.5 \times 20,000$$

$$= 50,000 \text{ kg/day}$$

∴ Total amount of MSW to be generate in design life

$$= 50,000 \times 35 \times 365$$

$$= 638,750,000 \text{ kg}$$

Given density of compacted MSW = 900 kg/m^3

∴ Volume of MSW to be generated in design life

$$= \frac{638,750,000}{900} = 709722.222 \text{ m}^3$$

∴ The required volume of landfill site

$$= 7,09,722.22 \text{ m}^3 \times \frac{5}{4} = 887152.778 \text{ m}^3$$

Given, the ratio of solid waste to cover = 4 : 1

Volume of cover soil required during design life,

$$= 887152.778 \times \frac{1}{5}$$

$$= 177430.556 \text{ m}^3$$

On yearly basis

$$= \frac{1,77,430.55}{35} = 5069.44 \text{ m}^3$$

6. (b) (ii) Solution:

Landfilling methods are selected to minimise water infiltration, control odour, prevent windblown litter, and ensure stable waste placement. The commonly used landfilling methods and control measures for landfill gas and leachate are explained below.

1. **Area Method:** In dry and semi-arid areas, the area method of landfilling is commonly used, in which solid waste is spread in layers above the ground, compacted properly, and covered daily with soil. This method is suitable for flat terrain and regions where excavation is difficult. It provides good control of odour and wind-blown litter but requires a large land area.
2. **Trench Method:** The trench method is the most economical method in dry areas where the groundwater table is deep. Trenches are excavated, waste is placed and compacted inside them, and the excavated soil is used as daily cover. This method ensures good protection from wind and minimises visual nuisance but is not suitable in rocky areas or where groundwater is shallow.
3. **Depression method:** The depression method involves filling natural or artificial depressions such as abandoned quarries or pits with solid waste. The waste is compacted in layers and covered with soil. This method helps in reclaiming unusable land but requires careful drainage and lining to avoid groundwater contamination.

Control of landfill gas movement is essential to prevent odour, fire, and explosion hazards. This is achieved by providing gas vents or gas collection wells, using low-permeability cover materials, and installing gas barriers to restrict lateral migration of methane and carbon dioxide.

Control of leachate movement is done by providing bottom liners of compacted clay or geomembranes, along with leachate collection pipes and drainage layers. Proper surface drainage and daily or final cover systems reduce rainwater infiltration and minimise leachate generation, thereby protecting groundwater quality.

6. (c) Solution:

Given data, $Q = 3.5 \text{ MLD}$

$$R = 1.5$$

$$\text{BOD of raw sewage} = 220 \text{ mg/l}$$

$$\text{BOD removal efficiency of PST} = 25\%$$

\therefore BOD of sewage entering into trickling fether

$$S_o = 220 \left(1 - \frac{25}{100} \right)$$

$$S_o = 165 \text{ mg/l}$$

Effluent BOD required $S = 30 \text{ mg/l}$

Efficiency of TF required,

$$\eta = \frac{S_o - S}{S_o} \times 100 = \frac{165 - 30}{165} \times 100 = 81.82\%$$

Efficiency of high rate TF is also given as

$$\eta = \frac{100}{1 + 0.0044 \sqrt{\frac{Q_o S_o}{VF}}} \quad \dots(i)$$

Where,

$$\begin{aligned} Q_o S_o &= \text{BOD applied in kg/day} \\ &= 3.5 \times 165 = 577.5 \text{ kg/day} \end{aligned}$$

V = Filter volume in hac-m

$$F = \frac{1 + R}{(1 + 0.1R)^2} = \frac{1 + 1.5}{(1 + 0.1 \times 1.5)^2} = 1.89$$

Putting in equation above,

$$81.82 = \frac{100}{1 + 0.0044 \sqrt{\frac{577.5}{V \times 1.89}}}$$

$$V = 0.119819 \text{ hac m}$$

$$V = 1198.19 \text{ m}^3$$

For diameter

$$V = 1198.19$$

$$\frac{\pi D^2}{4} \times H = 1198.19$$

$$\frac{\pi D^2}{4} \times 1.5 = 1198.19$$

$$D = 31.89 \text{ m}$$

If standard rate TF is to be used then

$$\eta = \frac{100}{1 + 0.0044 \sqrt{\frac{Q_o S_o}{V}}}$$

$$81.82 = \frac{100}{1 + 0.004 \sqrt{\frac{577.5}{V}}}$$

$$V = 2264.58 \text{ m}^3$$

$$\frac{\pi D^2}{4} \times 1.5 = 2264.58$$

$$D = 43.84 \text{ m}$$

$$\% \text{ change in diameter} = \frac{43.84 - 31.89}{31.89} \times 100 = 37.47\%$$

7. (a) (i) **Solution:**

Given:

$$Q = 45 \text{ MLD}$$

$$\text{Alum required} = 16 \text{ mg/l}$$

$$\text{Initially Alkalinity} = 5 \text{ mg/l as CaCO}_3$$

⇒ Total Alkalinity required for coagulation by Alum

$$= 16 \times 0.45 \text{ mg/l alkalinity as CaCO}_3$$

(1 gm alum require 0.45 of alkalinity as CaCO₃)

$$\therefore \text{Total alkalinity} = 7.2 \text{ mg/l as CaCO}_3$$

$$\therefore \text{External alkalinity required} = 7.2 - 5 = 2.2 \text{ mg/l as CaCO}_3$$

$$100 \text{ mg of CaCO}_3 \text{ require} = 56 \text{ mg of CaO}$$

$$\therefore \text{CaO required for } 2.2 \text{ mg/l}$$

$$\Rightarrow \frac{2.2}{100} \times 56 = 1.232 \text{ mg/l as CaO.}$$

$$\therefore \text{Quantity of pure CaO required} = 1.232 \times 45 \times 10^6 \times 10^{-6} \text{ kg/day}$$

$$= 55.44 \text{ kg/day}$$

$$\therefore \text{CaO required} = \frac{55.44}{0.85} = 65.22 \text{ kg/day}$$

$$\text{Annually CaO required} = \frac{65022 \times 365}{1000} \text{ tonne/year}$$

$$= 23.8 \text{ tonne/year}$$

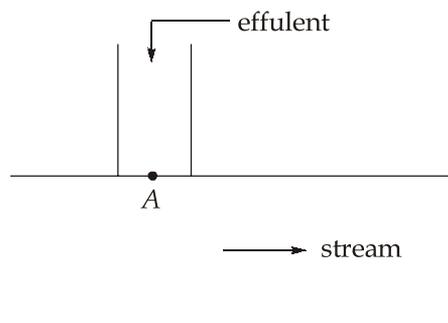
7. (a) (ii) Solution:

In a slow sand filter, purification of water takes place mainly due to biological and physical processes rather than simple straining. The most important mechanism is the formation of a biological layer called the schmutzdecke on the top surface of the sand bed. This layer consists of algae, bacteria, protozoa, and organic matter, which trap, adsorb, and biologically degrade pathogenic organisms and organic impurities. Mechanical straining and sedimentation also contribute to filtration, where suspended particles are trapped in the fine pores of the sand and settle within the voids between sand grains. As water flows slowly through the sand bed, heavier particles are retained near the surface while finer particles penetrate deeper layers.

Adsorption plays a significant role, as colloidal and dissolved impurities adhere to the surface of sand grains and the biological film. Additionally, biological oxidation occurs within the filter bed, where microorganisms oxidize organic matter and convert harmful substances into more stable forms, improving water quality.

Overall, the combined action of biological activity, straining, adsorption, and sedimentation makes the slow sand filter highly effective in removing bacteria, turbidity, and organic impurities from water.

7. (b) Solution:



Given,

$$k_{D/20^{\circ}\text{C}} = 0.087 \text{ per day}$$

$$k_{R/20^{\circ}\text{C}} = 0.174 \text{ per day}$$

First, we will determine DO, BOD and temperature of mixture at just dis of point A

$$\text{DO}_{\text{mix}} = \frac{\text{DO}_s \times Q_s + \text{DO}_R Q_R}{Q_s + Q_R}$$

$$\Rightarrow \text{DO}_{\text{mix}} = \frac{1.5 \times 0.3 + 8 \times 0.65}{0.30 + 0.65} = 5.947 \text{ mg/l}$$

$$\text{BOD}_{5, 20^\circ} \text{ of mix} = \frac{50 \times 0.3 + 2 \times 0.65}{0.3 + 0.65} = 17.158 \text{ mg/l}$$

Similarly, temperature of the mix

$$T_{\text{mix}} = \frac{27 \times 0.3 + 23 \times 0.65}{0.3 + 0.65} = 24.263^\circ\text{C}$$

Ultimate BOD of the mixture (L)

$$L_{(o)\text{mix}} = \frac{\text{BOD}_{5/20^\circ\text{C}} \text{ of mixture}}{1 - 10^{-k_{D,20^\circ\text{C}} \times 5}} = \frac{17.158}{1 - 10^{-0.087 \times 5}}$$

$$L_{(o)\text{mix}} = 27.118 \text{ mg/l}$$

Saturation DO at mixture temperature of 24.26°C (by linear interpolation from table)

$$\text{DO}_{\text{sat}} = 8.53 - \frac{8.53 - 8.38}{25 - 24} \times (24.26 - 24)$$

$$\text{DO}_{\text{sat}} = 8.491 \text{ mg/l}$$

\therefore Initial DO deficit of mixture

$$\begin{aligned} D_o &= \text{DO}_{\text{sat}} - \text{DO}_{\text{mix}} \\ &= 8.491 - 5.947 = 2.544 \text{ mg/l} \end{aligned}$$

Now temperature adjusted k_D and k_R

$$\begin{aligned} k_{D,24.26^\circ} &= k_{D/20^\circ\text{C}} (\theta)^{T-20} = 0.087 \times (1.04)^{(24.26 - 20)} \\ &= 0.103 \text{ per day} \end{aligned}$$

$$\begin{aligned} k_{R,24.26^\circ} &= k_{R/20^\circ\text{C}} (\theta)^{T-20} = 0.174 \times (1.02)^{(24.26 - 20)} \\ &= 0.189 \text{ per day} \end{aligned}$$

Now, the time after which critical DO deficit (DC) occurs is given by

$$t_c = \frac{1}{k_D(f-1)} \log_{10} \left[\left\{ 1 - (f-1) \frac{D_o}{L} \right\} f \right] \quad \dots(i)$$

Where

$$f = \text{Self purification factor} = \frac{k_R}{k_D}$$

$$f = \frac{0.189}{0.103} = 1.835$$

Putting values from above in equation (i)

$$t_c = \frac{1}{0.103(1.835-1)} \log_{10} \left[\left\{ 1 - (1.835-1) \times \frac{2.544}{27.118} \right\} \times 1.835 \right]$$

$$t_c = 2.653 \text{ days}$$

∴ Velocity of river stream = 0.3 m/sec

$$\begin{aligned} \text{Distance} &= 0.3 \text{ m/sec} \times 2.653 \times 24 \times 60 \times 60 \text{ sec} \\ &= 68.765 \text{ kms} \end{aligned}$$

and critical deficit D_c is given by

$$\left[\frac{L}{D_c f} \right]^{f-1} = f \left[1 - (f-1) \times \frac{D_o}{L} \right]$$

$$\Rightarrow \left[\frac{27.118}{D_c \times 1.835} \right]^{0.835} = 1.835 \left[1 - 0.835 \times \frac{2.544}{27.118} \right]$$

$$D_c = 7.876 \text{ mg/l}$$

OR

$$\text{Critical deficit, } D_c = \frac{L_o}{f} (10)^{-K_D t_c}$$

$$\Rightarrow D_c = \frac{27.118}{1.835} (10)^{-0.103 \times 2.653}$$

$$D_c = 7.876 \text{ mg/l}$$

7. (c) (i) Solution:

The settling of suspended particles in water and sewage treatment plants occurs in different ways depending on the concentration and nature of solids present. These settling behaviours are classified into four types.

Discrete settling (Type I) occurs when particles settle independently without any interaction with other particles. The size, shape, and density of the particles remain unchanged during settling. This type of settling is commonly observed in grit chambers and plain sedimentation tanks handling relatively clean water.

Flocculent settling (Type II) takes place when fine particles collide and form larger flocs during the settling process. As floc size increases, the settling velocity also increases with time. This type of settling is typical in sedimentation tanks following coagulation in water treatment plants and in primary settling tanks of sewage treatment plants.

Zone or hindered settling (Type III) occurs at high suspended solids concentrations where particles are so close that they settle together as a mass. A distinct interface develops between the clear liquid above and the settling sludge blanket. This type of settling is observed in secondary clarifiers of activated sludge processes.

Compression settling (Type IV) occurs at very high solids concentrations near the bottom of the tank, where particles are in direct contact and support each other. Settling takes place due to compression and expulsion of water from the sludge. This type of settling is commonly seen in sludge thickeners and at the bottom of settling tanks.

7. (c) (ii) **Solution:**

Given: Depth of tank, $H = 4$ m

Length of tank, $L = 55$ m

Flow velocity, $v_f = 1.20$ cm/sec

Kinematic viscosity, $\nu = 0.01$ cm²/sec = 0.01×10^{-4} m²/sec

Assume the minimize size of particle to be removed effectively is d

We know

$$\therefore \frac{H}{v_s} = \frac{L}{v_f} = td$$

\therefore Surface overflow rate or setting velocity

$$v_s = v_f \times \frac{H}{L} = 1.20 \times \frac{4}{55}$$

$$\Rightarrow v_s = 0.0873 \text{ cm/sec}$$

Assuming stokes law valid (for $R_e < 1$),

$$\text{Setting velocity } v_s = \frac{(G-1)\rho g d^2}{18\mu} \quad \text{since } \nu = \frac{\mu}{\rho}$$

$$\Rightarrow v_s = \frac{(G-1)gd^2}{18\nu}$$

$$\Rightarrow 0.0873 \times 10^{-2} = \frac{(2.65-1) \times 9.81 \times d^2}{18 \times 0.01 \times 10^{-4}}$$

$$\Rightarrow d = 3.116 \times 10^{-5} \text{ m}$$

$$\Rightarrow d = 3.116 \times 10^{-5} \times 10^3 \text{ mm}$$

$$\Rightarrow d = 0.0312 \text{ mm}$$

Now check the applicability of Stoke's law

$$R_e = \frac{v_s d}{\nu} = \frac{0.0873 \times 10^{-2} \times 3.116 \times 10^{-5}}{0.01 \times 10^{-4}}$$

$$= 0.0272 < 1 \quad (\text{Ok}) \quad \text{Stoke's Law is valid.}$$

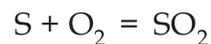
8. (a) Solution:

$$\begin{aligned}\text{Power} &= 915 \text{ MW} \\ &= 915 \times 3600 \text{ MJ/hr} \quad [\because 1 \text{ W} = 1 \text{ J/s} = 3600 \text{ J/hr}] \\ &= 3294 \times 10^3 \text{ MJ/hr}\end{aligned}$$

$$\begin{aligned}\text{Oil required} &= \frac{3294 \times 10^3 \times 0.725}{0.4 \times 40.5} = 147417 \text{ kg/hr} \\ &= 147.417 \text{ tonne/hr}\end{aligned}$$

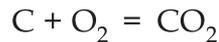
Given 80% of ash is particulates,

$$\text{So,} \quad \text{Particulates} = 0.8 \times \frac{0.04}{100} \times 147.417 = 0.047 \text{ tonne/hr}$$



\therefore 32 gm sulphur generates 64 gm of SO_2

$$\text{SO}_2 \text{ generated by 2.8\% of sulphur} = \frac{64}{32} \times \frac{2.8}{100} \times 147.417 = 8.255 \text{ t/hr}$$



\therefore 12 gm carbon generates 44 gm of CO_2

$$\therefore \text{CO}_2 \text{ generated by 85.2\% of carbon} = \frac{44}{12} \times \frac{85.2}{100} \times 147.417 = 460.53 \text{ tonne/hr}$$

\therefore Total annual production is as follows:

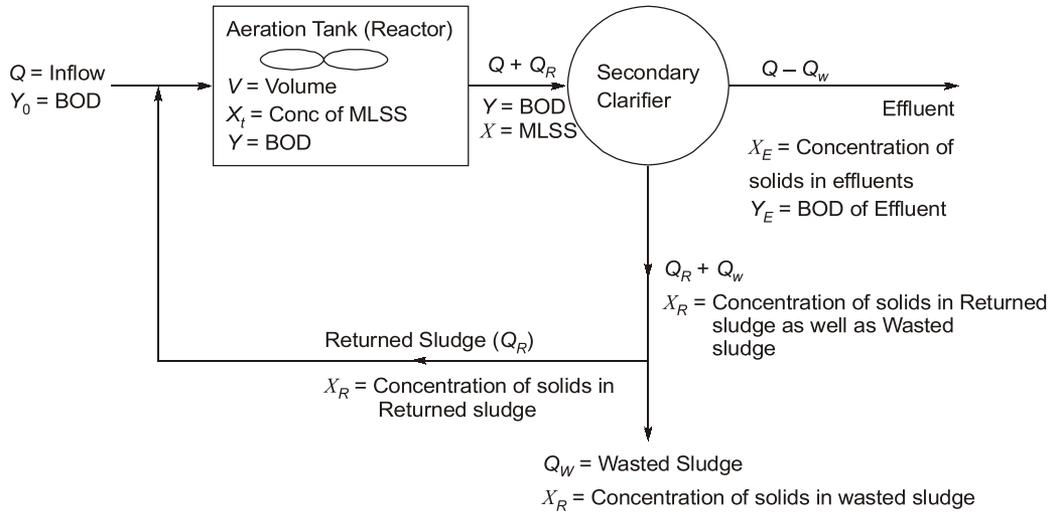
$$\text{Particulars} = 0.047 \times 24 \times 365 = 411.72 \text{ tonnes}$$

$$\text{SO}_2 = 8.255 \times 24 \times 365 = 72313.8 \text{ tonnes}$$

$$\text{CO}_2 = 460.53 \times 24 \times 365 = 4034242.8 \text{ tonnes}$$

8. (b) (i) Solution:

The flow chart of the ASP scheme is as follows-



Data given

$$Q = 40,000 \text{ m}^3/\text{day}$$

$$V = 9500 \text{ m}^3$$

$$y_0 = 240 \text{ mg/l}$$

$$y_E = 18 \text{ mg/l}$$

$$X_T = 2480 \text{ mg/l}$$

$$X_E = 30 \text{ mg/l}$$

$$X_W = 9700 \text{ mg/l}$$

$$Q_W = 220 \text{ m}^3/\text{day}$$

(a) Aeration period

$$t = \frac{\text{Volume of aerator}}{\text{inflow}}$$

$$\Rightarrow t = \frac{9500 \text{ m}^3}{40000 \text{ m}^3/\text{d}} \times \frac{24 \text{ hrs}}{\text{d}} = 5.7 \text{ hrs}$$

(b) F/M ratio-

$$\frac{F}{M} = \frac{\text{Mass of BOD applied}}{\text{Mall of mass}}$$

$$\Rightarrow \frac{F}{M} = \frac{QV_0}{VX_T} = \frac{40000 \text{ m}^3/\text{d} \times 240 \text{ mg/l}}{9500 \text{ m}^3 \times 2480 \text{ mg/l}}$$

$$\Rightarrow \frac{F}{M} = 0.407 \text{ per day}$$

(c) Percentage efficiency of BOD removal

$$\eta \% = \left(\frac{BOD_{in} - BOD_{out}}{BOD_{in}} \right) \times 100$$

$$\Rightarrow \eta \% = \frac{y_O - y_E}{y_O} \times 100 = \frac{240 - 18}{240} \times 100 = 92.5\%$$

(d) Sludge age (θ_c)-

$$\theta_c = \frac{V \cdot X_T}{Q_w X_w + (Q - Q_w) X_E}$$

$$\Rightarrow \theta_c = \frac{9500 \text{ m}^3 \times 2480 \text{ mg/l}}{220 \text{ m}^3 / \text{d} \times 9700 \text{ mg/l} + (40000 - 220) \times 30}$$

$$\Rightarrow \theta_c = 7.08 \text{ days}$$

8. (b) (ii) Solution:

The Activated Sludge Process (ASP) is a widely used biological secondary treatment process in wastewater treatment. In this process, microorganisms oxidize organic matter in the presence of oxygen, producing treated effluent and biological sludge.

Advantages:

- **Less Land Requirement:** The activated sludge process requires comparatively smaller land area than other secondary treatment methods such as trickling filters.
- **High-Quality Effluent:** It produces well-stabilized and high-quality effluent, which is suitable for discharge into receiving water bodies or for further treatment.
- **Low Head Loss:** The process involves relatively low hydraulic head loss through the treatment units.
- **No Fly or Odour Nuisance:** Unlike trickling filter systems, fly breeding and odour problems are minimal.
- **Lower Capital Cost:** In many cases, the initial capital cost is lower compared with trickling filter plants of similar capacity.
- **Improved Operational Efficiency:** Due to modern modifications and improved process control, operation of the system has become easier and more efficient.

Disadvantages

- **Continuous Supervision Required:** The process requires continuous monitoring and skilled operation to maintain proper microbial activity and treatment efficiency.

- **High Operating Cost:** Operating costs are relatively high, mainly due to energy consumption for aeration.
- **High Power Requirement:** Significant electrical power is needed for air compressors/ blowers and sludge recirculation pumps.
- **High Energy Consumption:** The typical power requirement ranges from about 55-110 HP per million litres of sewage treated.
- **Sensitive to Load Variations:** The process is sensitive to sudden changes in hydraulic load or organic concentration, which may disturb biological activity and reduce efficiency.

8. (c) (i) **Solution:**

Total demand during the 24 hrs period

$$\begin{aligned}
 &= \text{Summation of all the demands} \\
 &= 0.36 + 0.86 + 1.70 + 1.36 + 0.74 + 0.42 \\
 &= 5.44 \text{ Million litres}
 \end{aligned}$$

∴ Total supply during the 24 hrs

$$\begin{aligned}
 &= \text{Total demand} \\
 &= 5.44 \text{ million litres}
 \end{aligned}$$

Since inflow to reservoir is maintained at a uniform rate throughout 24 hrs

S_0 , uniform supply for 4 hrs period

$$\begin{aligned}
 &= \frac{5.44}{24} \times 4 \\
 &= 0.907 \text{ million litres}
 \end{aligned}$$

Time in Hrs	Demand (ML)	Cumulative demand (ML)	Supply (ML)	Cumulative Demand (ML)	Excess of Demand col 3 - col 5 (+ve) value only	Excess of Supply col 5 - col 3 (+ve) values only
(1)	(2)	(3)	(4)	(5)	(6)	(7)
0 - 4	0.36	0.36	0.907	0.907		0.547
4 - 8	0.86	1.22	0.907	1.814		0.594
8 - 12	1.70	2.92	0.907	2.721	0.199	
12 - 16	1.36	4.28	0.907	3.628	0.652	
16 - 20	0.74	5.02	0.907	4.535	0.485	
20 - 24	0.42	5.44	0.907	5.44	-	-

From the table

Maximum excess of demand, $A = 0.652$ ml

Maximum excess of supply, $B = 0.594$ ml

Hence, the total storage required = $A + B$

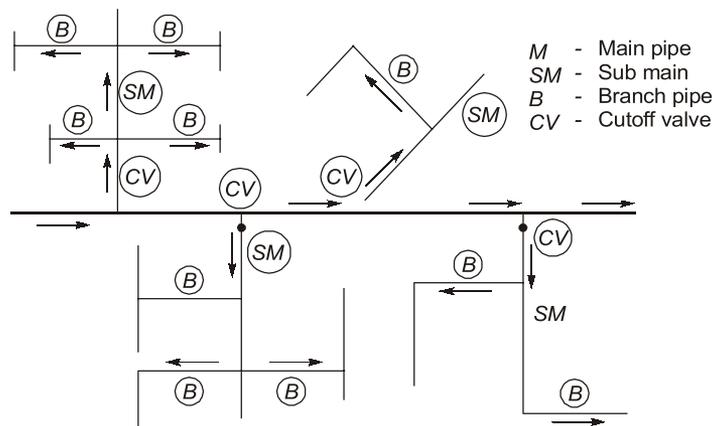
$$= 0.652 + 0.594$$

$$= 1.246 \text{ million litres}$$

8. (c) (ii) Solution:

A water distribution network is the part of a water supply system that conveys treated water from service reservoirs or treatment plants to the consumers in sufficient quantity, at adequate pressure, and with acceptable quality. It comprises pipes (mains, sub-mains and service connections), valves, pumps, storage reservoirs, hydrants, and other appurtenances. The primary objectives of a water distribution network are to ensure continuous supply, maintain required pressure at all points, minimize leakage losses, and provide water for domestic use, fire fighting, and public purposes. Based on the layout and method of distribution, water distribution networks are broadly classified into the following types:

- **Dead-end (Tree) System**, where pipes branch out like a tree; it is economical and simple but may suffer from poor circulation and water stagnation.
- **Grid Iron System**, in which pipes are interconnected in a rectangular or grid pattern; it ensures good circulation, uniform pressure, and better water quality.
- **Ring (Circular) System**, where the main distribution pipe forms a closed loop around the area; it provides reliable supply and balanced pressure.
- **Radial System**, in which the area is divided into zones and water is supplied radially from a central reservoir; it offers good pressure control and efficient distribution.



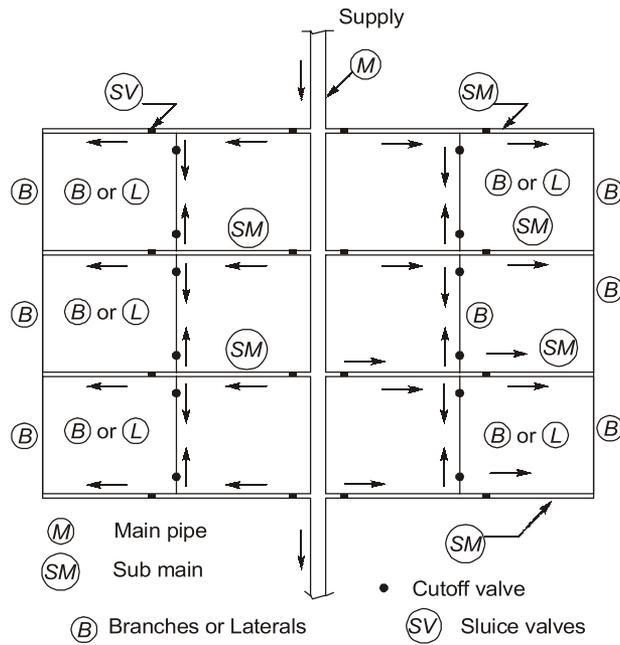


Fig. Grid System

