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UPSC

Engineering Services Exam

ESE (Main) Examination 2020

Civil Engineering
Paper-II

Detailed Solutions

ESE Mains 2020 Civil Engineering Detailed Weightage Analysis

Paper-2					
S.no.	Subjects	Difficulty Level	Marks in 2020	% Marks in 2020	% Marks in 2019
1	Flow of Fluids & Hydraulics	Moderate	67	12.9	12.9
2	Hydrology & Water Resources Engg.	Easy	67	22.7	12.9
3	Environmental Engineering	Moderate	116	14.4	24.2
4	Geotechnical Engineering	Moderate	99	20.6	20.6
5	Surveying and Geology	Moderate	42	10.8	10.8
6	Transportation Engineering	Easy	89	18.5	18.5
Total			480	100.00	100.00

CIVIL ENGINEERING

Paper-II

SECTION-'A'

1.(a) (i) Find out the pH of a mixture formed by mixing the following two water solutions :

Solution A : Volume 450 mL; pH = 7.5

Solution B : Volume 550 mL; pH = 6.5

Sol. For solution A, VA = 450 ml

$pH_A = 7.5$

hydrogen ion concentration in moles/L = $[H^+]_A = 10^{-7.5}$

pH = – log $[H^+]$

for solution B, V_B = 550 ml

$pH_B = 6.5$

$[H^+]_B = 10^{-6.5}$

$$\left[H^+\right]_{mix} = \frac{450 \times 10^{-7.5} + 550 \times 10^{-6.5}}{450 + 550}$$

$= 1.88 \times 10^{-7}$ moles/lt

$$pH_{mix} = -\log\left[1.88 \times 10^{-7}\right] = 6.725$$

(ii) Compute the theoretical oxygen demand of 108.75 mg// of glucose.

Sol.

$$\underset{180}{C_6H_{12}O_6} + \underset{192}{6O_2} \rightarrow 6\,CO_2 + 6\,H_2O$$

180 mg/L Glucose → 192 mg/L O_2

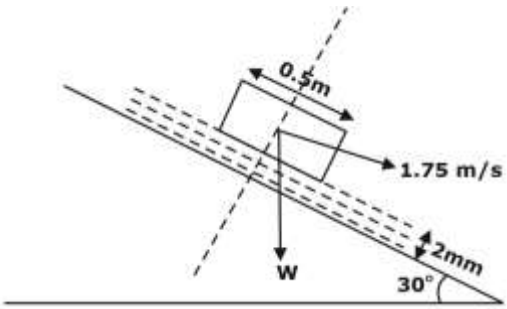
$\therefore$ 1 mg/L Glucose → $\frac{192}{180}$ mg/L O_2

$\therefore$ 108.75 mg/L Glucose → $108.75 \times \frac{192}{180}$ mg/L O_2

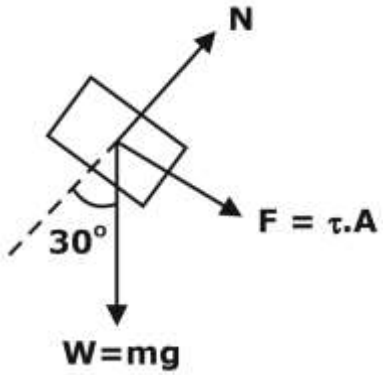
= 116 mg/l of O_2 ThOD

1.(b) (i) A rectangular plate of 0.5 m x 0.5 m dimensions, weighing 500 N slides down an inclined plane making 30° angle with the horizontal at a velocity of 1.75 m/s. If the 2 mm gap between the plate and inclined surface is filled with a lubricating oil, find its viscosity in poise.

Sol.



From the free body diagram we have



u = 1.75 m/s

= 175 cm/s

$$\frac{du}{dy} = \frac{175\,cm/s}{2\,mm}$$

$$= \frac{175}{0.2}\,S^{-1}$$

F = W sin 30°

τ.A = mg sin 30°

$$\mu\frac{du}{dy}A = mg\sin 30^{o}$$

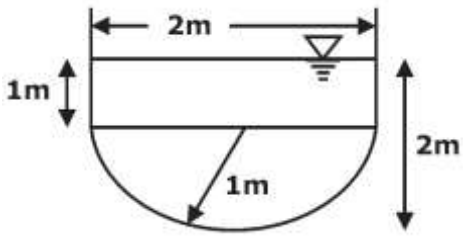
= W sin 30°

$$\mu\left(\frac{175}{0.2}\right)(50\,cm \times 50\,cm) = 500 \times 10^{5}(dyne) \times \frac{1}{2}$$

μ = 11.4286 poise

(ii) A channel has two sides vertical and semi-circular bottom of 2 m diameter. Calculate the discharge of water through the channel, when depth of flow is 2 m. Take C = 70 and slope of bed as 1 in 1000.

Sol.



Area of flow (A) = $1 \times 2 + \frac{\pi \times 1^2}{2}$

$$= \frac{\pi + 4}{2}\,m^2$$

Wetted perimeter (P) = π . 1 + 1 + 1

= (π + 2) m

Hydraulic mean depth (R) = $\frac{A}{P} = \frac{\pi + 4}{2(\pi + 2)}$

Discharge, $Q = CA\sqrt{RS}$ $\{\text{chezy's equation}\}$

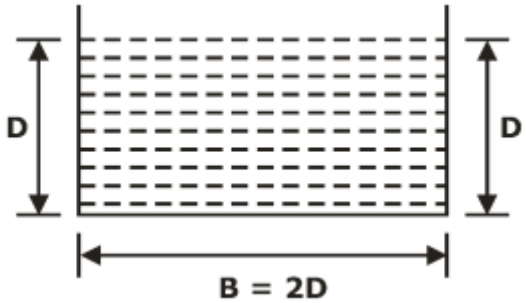
$= 70 \times \frac{(\pi + 4)}{2} \sqrt{\frac{(\pi + 4)}{2(\pi + 2)} \times \frac{1}{1000}}$

= 6.587 m^3/s

1.(c) A rectangular sewer with width twice its depth is hydraulically equivalent to a circular sewer. Find the relation between the width of the rectangular sewer and the diameter of the circular sewer assuming that sewer is running completely full.

Sol. Two sections are said to be hydraulically equivalent, if the discharge carried by them are same.

Rectangular Section

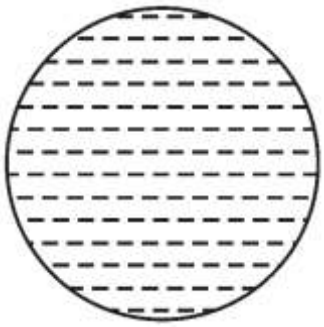


A = B × D = 2D × D

P = D + B + D = D + 2D + D = 4D

∴ for rectangular section, $\frac{A}{P} = R = \frac{2D \times D}{4D} = \frac{D}{2}$

Circular Section



$A = \frac{\pi}{4} d^2$

$P = \pi d$

So,

$R = \frac{A}{P} = \frac{\frac{\pi}{4} d^2}{\pi d} = \frac{d}{4}$

For hydraulically equivalent channel

$\therefore Q_R = Q_C$

$$\Rightarrow \left(\frac{1}{n} \times R_R^{2/3} \times S^{1/2}\right) \times 2D \times D = \frac{1}{n} \times R_C^{2/3} \times S^{1/2} \times \left[\frac{\pi}{4} d^2\right]$$

$$\Rightarrow \left(\frac{D}{2}\right)^{2/3} \times 2D^2 = \left(\frac{d}{4}\right)^{2/3} \times \frac{\pi}{4} d^2$$

$$\Rightarrow 2^{1/3} \times D^{2+2/3} = \frac{\pi}{4} \times \left(\frac{1}{4}\right)^{2/3} \times d^{2+2/3}$$

$$D^{8/3} = \frac{\pi}{4} \times \left(\frac{1}{4}\right)^{2/3} \times 2^{-1/3} \times d^{8/3}$$

$\therefore D^{8/3} = 0.247 \times d^{8/3}$

$$\therefore \left(\frac{D}{d}\right)^{8/3} = 0.247 \qquad \therefore \frac{D}{d} = 0.59$$

$\therefore D = 0.59\ d$

1.(d) After how many days will you supply water to soil (clay loam) in order to ensure efficient irrigation of the given crop, if :

Field capacity of soil	= 27%
Permanent wilting point	= 14%
Density of soil	= 1.5 g/cm^3
Effective depth of root zone	= 75 cm

Daily consumptive use of water for the given crop = 11 mm

Sol. Given,

FC = 0.27; D = 75 cm = 750 mm

PWP = 0.14 C_u = 11 mm/day

ρ_s = 1.5 g/cc

$$d_{aw} = D \times \frac{\rho_s}{\rho_w}(FC - PWP)$$

$$= 750 \times \frac{1.5}{1} \times (0.27 - 0.14)$$

d_{aw} = 146.25 mm

Assuming readily available water depth to be 75% of the available water depth.

d_{raw} = 146.25 × 75/100 = 109.69 mm

Now,

If 11 mm water is consumed per day

$\therefore$ 1 mm water will be consumed in $\frac{1}{11}$ days

$\therefore$ 109.69 mm water will be consumed in $\frac{1}{11} \times 109.69$ = 9.97 day $\simeq$ 10 day

$\therefore$ After every 10 day, irrigation needs to be done.

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1.(e) (i) A town with a population of 3 lakh produces solid waste at a rate of 2.5 kg/capita/day. If the waste is compacted to a density of 1500 kg/m^3, how much volume of landfill site is needed in a year ? Assuming that the ratio of solid waste to cover is 4 : 1, what volume of cover soil is needed in a year? What type of soil would you recommend as cover?

Sol. Town population = 3 lakh

Solid Waste generation rate = 2.5 kg/capita/day

$\therefore$ Total SW generation per day = $3 \times 10^5 \times 2.5$ kg/day

$= 7.5 \times 10^5$ kg/day

Compacted density of SW = 1500 kg/m^3

$\therefore$ volume of SW filled in landfill per day $= \dfrac{7.5 \times 10^5 \text{ kg/d}}{1.5 \times 10^3 \text{ kg/m}^3}$

= 500 m^3/d

$\therefore$ In year, a total of (500 × 365) m^3= 182500 m^3 of landfill site is Required.

Again, $\dfrac{\text{volume of cover}}{\text{volume of SW}} = \dfrac{1}{4}$

$\therefore$ volume of cover = volume of SW × $\dfrac{1}{4}$

$= 182500 \times \dfrac{1}{4}$

= 45625 m^3/year

Normally, impermeable clayey soils would be preferred (for example → Bentonite, or geotextiles) as a cover, in order to prevent leachate.

(ii) The sound power from a voice shouting is 0.002 W. What is the Sound Power Level ? What are the Sound Intensity, Sound Intensity Level, the Sound Pressure and the Sound Pressure Level at a distance of 10 metres from the source ? Assume that sound radiates from the source in all directions.

Sol. Sound power level,

(Given, W = 0.002 w)

W= 2×10^{-3} w

$w_0 = 10^{-12}$ w

$$L_p = 10 \log_{10}\left(\frac{w}{w_0}\right)$$

$$= 10 \log_{10}\left(\frac{2 \times 10^{-3}}{10^{-12}}\right)$$

= 10 × 9.301

= 93.01 dB

$$\text{sound intensity} = \frac{\text{Power}}{\text{Area}} = \frac{2 \times 10^{-3} \text{w}}{\pi \times 10^2 \text{ m}^2} \quad (\because r = 10 \text{ m})$$

= 6.366×10^{-6} w/m^2

sound intensity level, $L_I = 10 \log_{10}\left(\frac{I}{I_0}\right)$

$= 10\log_{10}\left(\frac{6.366 \times 10^{-6}}{10^{-12}}\right)$

= 68.04 dB

We know,

Sound intensity is, $I = \frac{P_{rms}^2}{\rho c}$

We know ρ = 1.2 kg/m³

c = 330 m/s

$\therefore I = 6.366 \times 10^{-6} = \frac{P_{rms}^2}{1.2 \times 300}$

$\therefore P_{rms}^2 = 2.29 \times 10^{-3}$

$\therefore P_{rms}$ = 0.048 Pa

Sound Pressure level

$L_P = 20\log_{10}\left(\frac{P}{P_0}\right)$

$\because P_0$ = 20 μPa,

P = 0.048 Pa = 0.048 × 10⁶ μPa

$L_P = 20\log_{10}\left(\frac{0.048 \times 10^6}{20}\right)$

= 67.58 dB

2.(a) (i) What is ϕ index ? How is it estimated ? What are the factors that affect ϕ index ?

Sol. **Φ -index**

• The ϕ index is the average rainfall above which the rainfall volume is equal to the runoff volume.

• ϕ index is average infiltration rate during the period of rainfall excess.

Rainfall excess is the rainfall contributing to runoff and the period during which such a rainfall takes place is called period of rainfall excess.

• ϕ index is derived from the rainfall hyetograph with the knowledge of the resulting runoff volume. The initial loss is also considered as part of infiltration.

• ϕ index can be assumed as 0.10 cm/hr far estimating maximum floods for design purposes.

It is estimated as,

$\phi = \left(\frac{P - R}{t}\right)$

Where, P — Precipitation in effective duration

R — Runoff

t— Effective duration of rainfall.

Φ index depends on the type of surface like infiltration will be more in sandy sail and less and clayey sail.

(ii) A storm with 10 cm precipitation produced a direct runoff of 5.8 cm. The time distribution of the storm is given below. Estimate the ϕ index of the storm.

Time from start (h)	Incremental rainfall in each hour (cm)
1	0.4
2	0.9
3	1.5
4	2.3
5	1.8
6	1.6
7	1.0
8	0.5

Sol. Let ϕ index < 0.4 cm/hr

Then, effective rainfall = 10 - 8ϕ

10 - 8ϕ = 5.8

8ϕ = 4.2

φ = 0.525 cm/hr > 0.4 cm/hr

Hence, our assumption is wrong.

Let 0.4 < ϕ index < 0.5

(0.9 - ϕ) + (1.5 – ϕ) + (2.3 – ϕ) + (1.8 – ϕ) + (1.6 – ϕ) + (1.0 – ϕ)+ (0.5 – ϕ) = 5.8

9.6 – 7ϕ = 5.8

7ϕ = 3.8

ϕ = 0.543 cm/hr > 0.5 cm/hr

Again our assumption is wrong.

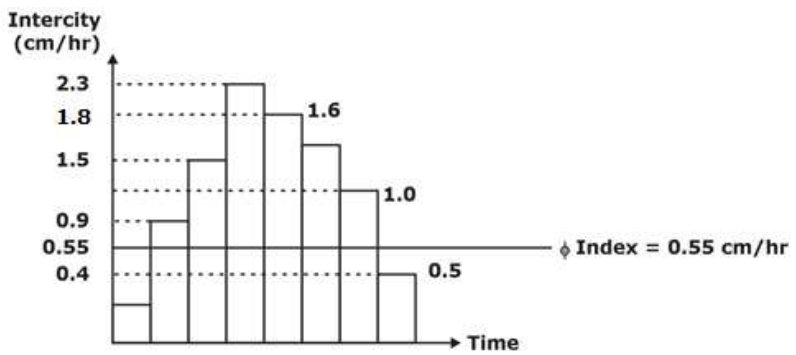
Let, 0.5 <ϕ index < 0.9

(0.9 – ϕ) + (1.5 – ϕ) + (2.3 – ϕ) + (1.8 – ϕ) + (1.6 – ϕ) + (1.0 – ϕ) = 5.8

9.1 – 6ϕ = 5.8

ϕ = 0.55 cm/hr > 0.5

Here, our assumption is correct.



2.(b) (i) A bed of uniform sand, having particle size 0.65 mm diameter and specific gravity 2.66, porosity 0.48 and depth 75 cm is to be washed hydraulically. Compute

(a) Backwash rate so that expansion will be 50 percent.

(b) Head loss at this rate.

Take kinematic viscosity of water as 1.3 x 10^{-2} cm^2/sec and assume C_D = 24/R.

Sol. d = 0.65 mm = 0.65 × 10^{-3} m

G = 2.66; n = 0.48; D = 75 cm

= 0.75 m

D_e = expanded bed depth

= 1.5 × 0.75 = 1.125 m

Now $n_e = \left(\frac{V_B}{V_t}\right)^{0.22}$ so, to get V_B, we need n_e & V_t

We know

D (1 – n) = D_e × (1 – n_e)

→ 0.75 (1 – 0.48) = 0.75 × 1.5 × (1 – n_e)

∴ 0.52 = 1.5 × (1 – n_e)

∴ n_e = 0.65

V_t → will be calculates by Stoke's law, as

$C_D = \frac{24}{R}$ → laminar flow (given)

$$\therefore V_t = \frac{g(G-1)d^2}{18\nu}$$

$$= \frac{9.81 \times (2.66-1) \times (0.65 \times 10^{-3})^2}{18 \times 1.3 \times 10^{-6}}$$

$$= \frac{9.81 \times 1.66 \times (0.65)^2 \times 10^{-6}}{18 \times 1.3 \times 10^{-6}} \text{ m/s}$$

= 0.294 m/s

$$\therefore 0.65 = \left(\frac{V_B}{0.294}\right)^{0.22}$$

$$\therefore \frac{V_B}{0.294} = 0.14$$

Rate of backwashing

V_B = 0.041 m/s

(ii) Briefly explain various factors affecting bactericidal efficiency of chlorine on water treatment process.

Sol. Important Factors Affecting Bactericidal Efficiency of Chlorine

1. Turbidity ↑ — Bactericidal Efficiency ↓

2. Metallic compound (Iron, Manganese): Chlorine is utilized in oxidizing these metallic compounds. Hence, bactericidal efficiency decreases.

3. Ammonia compounds form chloramines which reduces efficiency.

4. pH of water: pH range of 5-7 is most effective. Beyond this range, bactericidal efficiency decreases.

5. Temperature: Requirement of chlorine increases with decreases in temperature and increase in pH beyond 7.

6. Time of contact ↑ → Efficiency ↑ .

2.(c) (i) A flat plate of 2 m width and 4 m length is kept parallel to air plate flowing at 5 m/s velocity at 15°C. Determine the length of the plate over which boundary layer is laminar, shear at the location where boundary layer ceases to be laminar and total force on both sides on that portion of plate the boundary layer is laminar.

Take $\rho = 1.208$ kg/m^3 and $v = 1.47 \times 10^{-5}$ m^2/s.

Sol. B = 2 m, L = 4 m, V = 5 m/s temp = 15°C

$$R_{eL} = \frac{U_\infty L}{v}$$

$$5.0 \times 10^5 = \frac{5 \times L}{1.47 \times 10^{-5}}$$

L = 1.47 m

$$\tau = \frac{\rho U_\infty^2}{2}\left(\frac{0.664}{\sqrt{Re_{1.47}}}\right) = \frac{1.208 \times 5^2}{2} \times \frac{0.664}{\sqrt{5\times10^5}}$$

$\tau = 0.141$ N/m^2

$$F_D = C_D \frac{1}{2}\rho A U^2$$

$$C_D = \frac{1.328}{\sqrt{5 \times 10^5}} = 1.87 \times 10^{-3}$$

$$F_D = \frac{1.87 \times 10^{-3}}{2} \times 1.2 \times 1.47 \times 2 \times 5^2 = 0.082 \text{ N}$$

Total drag force on both sides,

$F_{D,total} = 0.164$ N.

(ii) What are the functions of a surge tank?

Sol. Surge tank is a stand pipe at the end of a cloud aqueduct, feeder, dam, barrage pipe to absorb sudden rise of pressure.

So, basically they are pressure neutralizers in hydropower water conveyance system. When there is a closure of the gate it experiences a water hammer which is then compensated by providing the surge tank. It also works as reservoir in few cases. So, it improves the regulation characteristics of a hydraulic turbine.

3.(a) (i) In a factory, coal is burnt at a rate of 1 kg/second. Analysis of the coal reveals a sulphur content of 3 percent. The sulphur in the ash is 5 percent of the input sulphur. What is the annual rate of emission of sulphur dioxide ?

Sol. Rate of burning of coal = 1 kg/second.

= 1 × 86400 × 365 kg/gram= 31536000 kg/year

Now, Sulphur content = 3%

$\therefore$ Rate of Burning of Sulphur= $31536 \times 10^3 \times \frac{3}{100}$ = 946080 kg/year

Again, sulphur in the ash is 5% of the input sulphur.

Rest of the 95% goes along with the flue gas.

$\therefore$ Sulphur is the flue gas = $946080 \times \frac{95}{100}$ = 898776 kg/year

Now, $\underset{32}{S} + O_2 \rightarrow \underset{32+2\times16=64}{SO_2}$

So, from 32 kg/yr Sulphur = 64 kg/yr SO_2 is emitted

$\therefore$ from, 1 kg/yr Sulphur — $\frac{64}{32}$ = 2 kg/yr SO_2 is emitted

$\therefore$ 898776 kg/gm Sulphur — 2 × 898776 = 1797552 kg/yr= 1797.552 kg/yr

So, annual rate of emission of SO_2 is 1797.552 kg/year

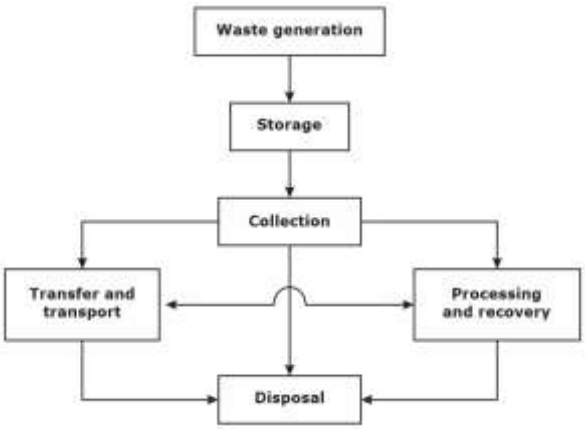
(ii) Describe various functional elements of a solid waste management system.

Sol. **Functional Elements of Solid Waste Management**

The activities involved with the management of solid wastes from the point of generation to final disposal have been grouped into six functional elements.

1. Waste generation
2. On-site handling, storage and processing
3. Collection
4. Transfer and transport
5. Processing and recovery
6. Disposal

Description of the functional elements of a solid waste management system	
Component	**Description**
Waste generation	Those activities in which materials are identified as no longer being of value and are either thrown away or gathered together for disposal
On site handing, storage, and processing	Those activities associated with the handling, storage and processing of solid waste at or near the point of generation.
Collection	Those activities associated with the gathering of solid wastes and the hauling of wastes after collection to the location where the collection vehicle is emptied
Transfer and transport	Those activities associated with (1) the transfer of wastes from the smaller collection vehicles to the larger transport equipment and (2) the subsequent transport of the wastes, usually over long distance, to the disposal site.
Processing and recovery	Those techniques equipment and facilities used to improve the efficiency of the other functional elements and to recover usable materials, conversion products or energy from solid wastes
Disposal	Those activities associated with ultimate disposal of solid wastes, including those wastes collected and transported directly to a landfill site, semisolid wastes (sludge) from wastewater treatment plants, incinerator residue, compost or other substances from the various solid-waste processing plants that are of no further use.



Interrelationship of functional elements comprising a solid-waste management system.

3.(b) (i) What are the effects of water logging ?

Sol. **Effect of Water Logging**

(i) Inhibiting activity of soil bacteria

(ii) Decrease in available capillary water

(iii) Fall in soil temperature

(iv) Rise in level of salts in the surface soil

(v) Delay in cultivation operations

(vi) Growth of wild flora (leading to decrease in crop yield)

(vii) Adverse effect on community health.

(ii) A centrifugal pump runs at 1000 rpm against a head of 16 m and carries 145 liters/s of water discharge. The impeller diameter at the outlet is 300 mm and the width there is 60 mm. If the vane angle ϕ at the outlet is 40°, determine the manometric efficiency.

Sol. N = 1000 rpm.

D_2 = 300 mm = 0.3 m

H_m = 16 m

B_2 = 60 mm = 0.06 m

Q = 145 lps

φ = 40°

$Q = \pi B_2 D_2 Vf_2$

$145 \times 10^{-3} = \pi \times 0.06 \times 0.3 \times V_{f_2}$

V_{f_2} = 2.5641 m/sec

$$\tan = \phi = \frac{V_{f_2}}{U_2 - V_{w_2}}$$

$$U_2 = \frac{\pi D_2 N}{60} = \frac{\pi \times 0.3 \times 1000}{60}$$

U_2 = 15.70 m/sec

$$\tan 40 = = \frac{2.56}{15.70 - V_{w_2}}$$

V_{w_2} = 12.64 m/sec

$$\eta_{man} = \frac{gH_m}{V_{w2} \times U_2} = \frac{9.81 \times 16}{12.64 \times 15.70} \times 100 = 79.09\%$$

3.(c) A municipality has directed to upgrade its primary wastewater treatment unit to a secondary unit that can meet an effluent standard of 20 mg// BOD_5 and 20 mg/l total suspended solids. They have selected a completely mixed activated sludge system. BOD5 of total suspended solids is 63% of TSS concentration. Estimate the required volume of aeration tank. The following data is available from existing primary plant :

Flow = 0.150 m^3/s, BOD_5 = 80 mg//.

Assume the following values for half velocity constant = 95 mg/l of BOD_5; Maximum growth rate constant = 2.5/day; Decay rate of micro-organism = 0.050/day; Yield coefficient = 0.50 mg VSS per mg BOD_5 removed; MLVSS = 2000 mg/l.

Sol. Given, BOD_5 of effluent = 20 mg/L =S

And, X_e = 20 mg/L

K_E = 0.05/day

Y= 0.5 mg VSS per mg BOD_5 removed= 0.5

MLVSS, X = 2000 mg/l

Flow, Q = 0.15 m^3/s

It is also given that BOD_5 of the total suspended solid is 63% of total suspended solid concentration.

S = Maximum BOD allowed – BOD_5 in SS

= 20 – 20 ×0.63 = 7.4 mg/l

Half velocity constant $\frac{u}{u_{max}} = 0.5 = \frac{s}{K_s + s}$

Given, s = 95 mg/l

$$0.5 = \frac{95}{K_s + 95}$$

K_s = 95 mg/l

Now,

$$u = u_{max}\frac{s}{K_s + s} = \frac{1}{\theta_c} + K_E$$

Given, K_E = decay rate of microorganism = endogenous rate constant = 0.05 d^{-1}

$$\therefore 2.5 \times \frac{7.4}{95 + 7.4} = \frac{1}{\theta_c} + 0.05$$

θ_C = 7.653 days

Ander steady state condition,

$$\frac{1}{\theta_c} = YU - k_e$$

Here, U = Specific substrate utilization rate

$$U = \frac{Q(S_o - S_e)}{VX} = \frac{0.15 \times (80 - 7.4)}{V \times 2000} = \frac{1}{183.655V}/s$$

$$U = \frac{1}{183.655V}/s = \frac{86400}{183.655V}/d = \frac{470.477}{V}/d$$

Now, $\frac{1}{\theta_C} = YU - k_e = \frac{1}{7.653} = 0.5 \times \frac{470.447}{V} - 0.05$

$\therefore \frac{1}{V} \times 235.23 = 0.18$

$\therefore V = \frac{235.23}{0.18} = 1302 m^3$

4.(a) An outward flow turbine running at 200 rpm, works on a discharge of 5 m^3/s under a head of 40 m. Internal and external diameters of the wheel are 2 m and 2.5 in respectively while the width at the inlet and outlet is 200 mm. Assuming the discharge to be radial at the outlet, determine angles of the turbine at the inlet and outlet. Also draw the velocity triangles for outward flow turbine.

Sol. $u_1 = \frac{\pi D_1 N}{60}$

$= \frac{\pi \times 2 \times 200}{60} = 20.944 m/s$

$u_2 = \frac{\pi D_2 N}{60}$

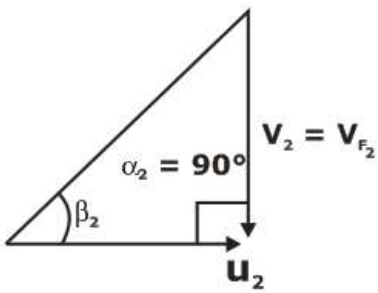
$= \frac{\pi \times 2.5 \times 200}{60} = 26.18 m/s$

$V_{f_1} = \frac{Q}{\pi D_1 B_1} = \frac{5}{\pi \times 2 \times \frac{200}{1000}} = 3.98 m/s$

$V_{f_2} = \frac{Q}{\pi D_2 B_2} = \frac{5}{\pi \times 2.5 \times \frac{200}{1000}} = 3.18 m/s$

Since discharge is radial at outlet

$\alpha_2 = 90°$, $V_{w_2} = 0$ & $V_2 = V_{f_2}$

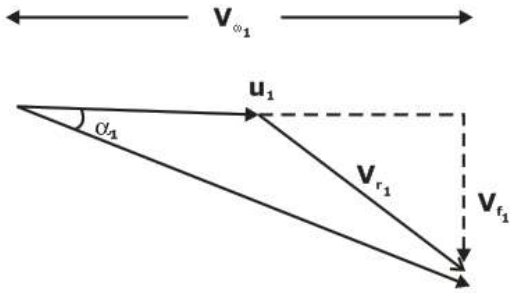


$\tan\beta_2 = \frac{V_{f_2}}{u_2} = \frac{3.18}{26.18}$

$\beta_2 = 6.92°$

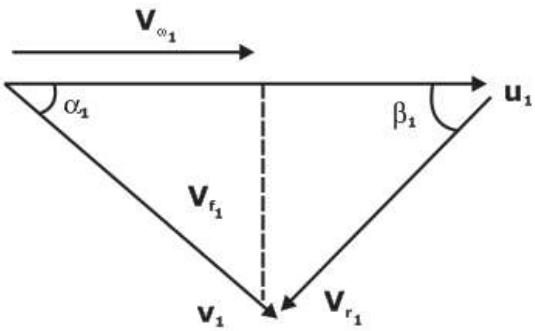
$H = \frac{V_{w_1} u_1}{g}$

$40 = \frac{V_{w_1} \times 20.944}{9.81} = V_{w_1} = 18.74 m/s$



$u_1 > V_{w_1}$

Hence the velocity triangle will be



$$\tan\alpha_1 = \frac{V_{f_1}}{V_{w_1}}$$

$$\tan\alpha_1 = \frac{3.98}{18.74}$$

$\alpha_1 = 12°$

$$\tan\beta_1 = \frac{V_{f_1}}{u_1 - V_{w_1}} = \frac{3.98}{20.944 - 18.74}$$

$\beta_1 = 61°$

4.(b) (i) Explain the factors that cause sludge bulking in activated sludge process for wastewater treatment.

Sol. A sludge that exhibits poor settling characteristics is called a bulking sludge. Foam formation and poorly settling sludge are the two most common problems of the activated sludge process. Filamentous micro-organism are responsible for a bulked sludge. Large surface area to volume ratios of these micro-organisms retard their settling velocities. Fungi are the most familiar filamentous micro-organisms. The vegetative structure of most fungi is composed of filaments, which contain several nuclei. Fungi, however, are not commonly significant in wastewater treatment. Some other bacteria, particularly of type 021 N bacteria are among the most frequently reported filamentous micro-organisms found in bulking sludge. These bacteria are usually found to develop in activated sludge systems which are characterized by the low or variable nutrient concentrations. These bacteria may live on a variety of carbon and nitrogen sources present in the activated sludge system.

Organic acids form an important class of carbon sources for the growth of filamentous sulfur bacteria. These bacteria, however, do not develop well at low pH values. The other types of bacteria which are most commonly found in foaming and bulking sludges of activated plants are Nocardia amorae, Microthrix parvicella, N. amarae like organisms, N.pinesis like organisms, and type 0092, etc. Nocardia growth is supported by high sludge ages, low F: M ratios, and higher waste temperatures. The most successful methods to control these organisms, as per the latest research, are :

(i) reduction of the sludge age to less than 6 days; and

(ii) chlorination of return activated sludge.

Foam removal is also a logical and beneficial control measure since Nocardia filaments are usually concentrated in foam compared to the mixed liquor.

Classical control measures like adjustment of the F: M (Food: Micro-organisms) ratio**, raising or lowering D.O., or applying a disinfectant (chlorine) are found to help in controlling filamentous bacteria. Carbohydrate-rich waters are more prone to give rise to filamentous populations. Exclusion of such wastewaters may, therefore, sometimes help in controlling sludge bulking.

Nutrient deficiencies as said above, cause the growth of filamentous micro-organisms. One of the first checks made on a bulking activated plant, therefore, is to measure concentrations to determine whether they are sufficient for the amount of sludge being produced. If not, the problem is readily solved by the addition of nutrients to the wastewater.

(ii) Differentiate and compare anaerobic digestion process and composting process used for solid waste treatment.

Sol. First of all the two processes are very much the same. The crucial difference is that composting is the decomposition of organic matter in the presence of air (oxygen) and anaerobic digestion (AD) is the decomposition of organic matter, without air (and most importantly oxygen) present. Now we make the case for each process by listing the advantages and disadvantages of below:

Advantages of Anaerobic Digestion

1. It is a net energy-producing process that produces renewable energy in the form of biogas.
2. It produces a liquid and a fibrous fertilizer.
3. It sanitizes the feedstock/waste which is put through it, as long as the temperature is held above a required temperature for a pre-defined period.
4. It reduces odour below unprocessed waste odour levels.
5. It is much less likely to cause environmental pollution than spreading untreated organic waste on land.
6. The effect of the fertilizer is longer lasting than for untreated organic waste.

Disadvantages of Anaerobic Digestion

1. When carried out at a commercial scale on farms and at wastewater treatment works (WwTWs) it requires a high level of investment in large tanks and other process vessels.
2. If run inefficiently AD can cause an odor nuisance.
3. Does not convert as large a proportion of the carbon in the biomass to biogas as can be achieved using gasification.

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Advantages of Composting

1. Lower initial capital investment needed to start a composting facility than is needed for an AD Plant.

2. A slightly lower level of training is needed to run a composting plant the is required for an AD Plant.

3. It produces a solid output/fertilizer only. For some, this will be an advantage, but in general, this digest product is a neutral factor, neither positive nor negative for composting.

4. It has the same advantages as for AD (items 3 to 6 inclusive) overspreading untreated organic waste material on land.

Disadvantages of Composting

1. If the composted materials contain waste types such as animals or food waste there is a risk that diseases (for example "foot and mouth") may be spread in the compost. In those circumstances, in many countries the Animal By-products regulations must be complied with for all commercial sales/spreading on land. Like anaerobic digestion, it is necessary to pasteurize the composted material to ensure that all infectious agents have been effectively removed. Commercial composting companies ensure that they meet the relevant regulations by applying the PAS 100 code for Quality Compost.

2. This can be done by ensuring that it composts rapidly and raises its temperature, through the heat produced during composting. The required hot temperature must be held constant for a stated minimum period. Ensuring that every batch self-sanitizes itself this way requires very good operating practice and detailed monitoring to demonstrate successful pasteurization to the local environmental regulating body.

3. This disadvantage is the biggest disadvantage of composting! Composting requires the input of quite large energy inputs to fuel and operate the equipment needed to aerate and turn the compost piles.

By comparison, anaerobic digestion wins hands-down for "greenness" by providing its power to do this.

Composting does not contribute to reducing the carbon footprints of the business that use the composting process.

4.(c) (i) What do you understand by galleries and shafts and why are they provided in gravity dams?

Sol. Galleries are openings or passageways left in the dam body. They may be provided parallel or normal to the dam axis at various elevations. The galleries are interconnected by steeply sloping passages or by vertical shafts fitted with lifts. The shape and size of the gallery depending on the size of the damned and the function served.

The functions for which the galleries are provided are :

1. Drainage: To cater to the drainage of the dam section by intercepting seepage from the water face and carry it away from the downstream face.

2.Inspection: To provide access to the interior of the mass comprising the dam to inspect the structure and study the structural behavior of the dam in the post-construction period.

3. Drilling: To provide access for carrying out drilling and grouting of foundations, etc.

4. Operation of gates and control equipment: To provide access to mechanical equipment for the operation of rates and control equipment.

5. Post-construction grouting: To provide space for header and return pipes for post-construction grouting of longitudinal joints of the dam. Also, to provide access for grouting the construction joints which cannot be done from the face of the dam.

Classification of Galleries

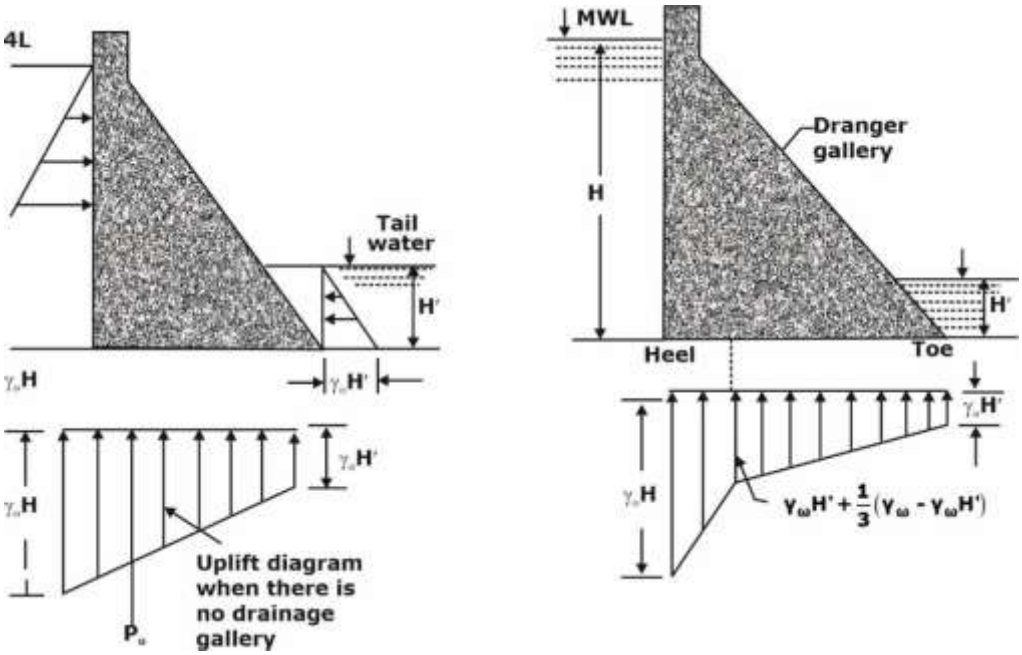
Foundation gallery: It caters for (i) The drainage of water percolating from upstream face or seepage through the foundations in the dam, (ii) Provides space for drilling and grouting of holes from the floor of the gallery for the main grout curtain, and (iii) For drilling foundation drain holes after foundation grouting has been completed.

Drainage gallery: It serves for intercepting seepage from the water face and conducting it away from the downstream face and drilling and draining the downstream portion of the foundation.

Inspection gallery: It is a gallery to provide access to the interior of the mass comprising the dam to inspect the structure and study the structural behavior of the dam in the post-construction period.

Gate gallery: It is a gallery made in a dam to provide access to and to house the mechanical equipment required for the operation of gates in outlet conduits, power penstocks, or spillway crest.

Grouting gallery: It is a gallery to locate the supply, return and vent headers of the grout pipe system, as also the piping system for artificial cooling of the blocks terminates in this gallery.



- Dams are also provided with shafts. The shaft is a vertical opening in the dam.
- Shafts connect galleries at various levels. Shafts are generally provided with lifts to conduct an effective inspection of the dam and also to facilitate quick approach anywhere in the dam.
- They are sometimes used to measure the deflection of the dam also.

Shafts Are :

required for locating headers of the post cooling system and for locating measuring devices. @Shafts are also required for the movement of elevators and the hoisting equipment. Sometimes shafts are constructed inclined to connect two galleries or the same gallery at two different elevations by a staircase or a lift arrangement.

(ii) During a recuperation test, the water in an open well was depressed by 2.5 m by pumping and it recuperated 1.8 m in 80 minutes. Find yield from a well of 4 m diameter under a depression head of 3 m.

Sol. $\frac{C}{A} = \frac{\ell n(s_1/s_2)}{T}$

C : Specific capacity

A : Area of the well

S_1 : Initial drawdown = 2.5 m

S_3 = Final drawdown = 2.5 – 1.8

= 0.7 m

Yield per unit volume $\frac{C}{A} = \frac{\ell n(2.5/0.7)}{80 \times 60}$ s^{-1}

Yield $= \frac{C}{A} \times$ vol.

$= \frac{\ell n(25/7)}{80 \times 60} \times \frac{\pi}{4} \times 4^2 \times 3$ (depression head)

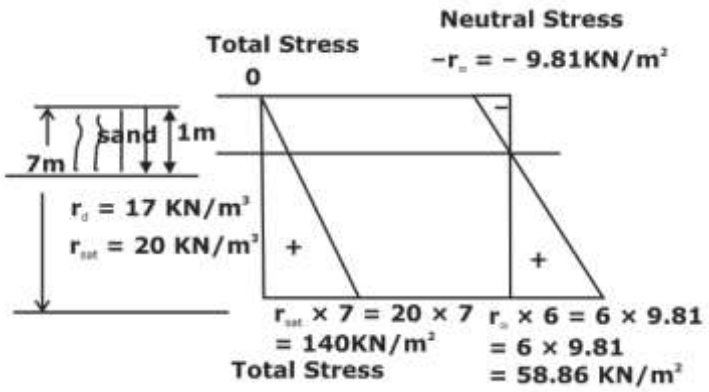
= 0.0114 m^3/s

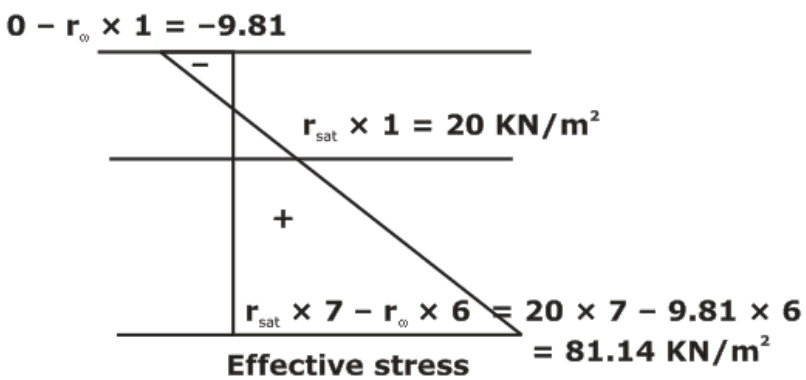
= 11.4 lit/s

SECTION-'B'

5.(a) The soil profile in a particular site consists of 7 m thick sandy layer overlain by a layer of clay. The water table is at 1 m below the ground surface. Above the water table, the sand is saturated with capillary moisture. The dry unit weight of sand is 17 kN/m^3 and its saturated unit weight is 20 kN/m^3. Plot the total stress, neutral stress and effective stress with depth up to a depth of 7 m.

Sol.





(b) What is meant by N value? Why should we apply corrections for the N value obtained from the field? Briefly explain the corrections.

Sol. N value

(i) The standard penetration test is conducted in 3 stage. Each stage has penetration value of 150 mm.

(ii) The number of blows for first 150 mm Penetration are ignored and no of blows in last 300 mm penetration are noted are called observed SPT number of N number

If the test site has water table and have overburden pressure then these N value must be corrected. These correct must be applied in following sequence

1. Overburden pressure correction (C_1)

→SPT value at low level of depth is less but at greater depth the penetration value increase. In order to normalize SPT number following correction is done

$$N_1 = N_0 \times \frac{350^\circ}{\bar{\sigma} + 70}$$

N_0 = Observed SPT

N_1 = Correction SPT

$\bar{\sigma}$ = Effective stresses

if $\bar{\sigma}$ > 280 KN/m → No need to apply this correction

2. Water table correction (C_2)

→In fine sand or silt below W.T, the observed value of N is higher than actual value hence following correction is needed

$$N = 15 + \frac{1}{2}(N_1 - 15)$$

N_1 = Correct SPT value after overburden pressure

N = final corrected value

If N_1< 15 then no need to apply this correction

Let test is conducted at different level say at P, Q, R, S then SPT is taken as

$$N = \frac{N_P + N_Q + N_P + N_S}{4}$$

SPT value	**Relative density**
4	very loose
10	loose
20	Moderate dense
30	Medium dense
50	Dense
>50	very dense

(c) Define optimum signal cycle time. Design two phase traffic signal with pedestrian crossing by Webster's method for an average normal flow of traffic on cross roads A and B during design hour as 480 PCU and 250 PCU per hour, the saturation flows on roads A and B are given as 1200 PCU and 1000 PCU per hour respectively. All red time required for pedestrian crossing is 12 seconds and amber times of 2 seconds for clearance in each phase is to be provided.

Sol. Cycle time is the time taken by a signal to complete one full cycle of iterations i.e. one complete rotation through all signal indications. Optimum cycle length is one in which average delay is minimum. If the vehicle is supposed to be moving but it does not utilize that time, then it is considered as lost time.

Normal flow for road A = q_A = 480 PCU

Normal flow for road B = q_B = 250 PCU

Saturation flow for road A = S_A = 1200 PCU

Saturation flow for road B = S_B = 1000 PCU

All red time = R = 12 seconds

Amber time = t = 2 sec in each phase

As per the Webster method, optimum cycle length C_0,

$$C_0 = \frac{1.5L + 5}{1 - y}$$

$$y = y_A + y_B$$

here, $y_A = \frac{q_A}{S_A} = \frac{480}{1200} = 0.4$

$y_B = \frac{q_B}{S_B} = \frac{250}{1000} = 0.25$

y = 0.4 + 0.25 = 0.65

L = 2n + R = 2 × 2 + 12 = 16 secs

{n = no. of phase)

$C_0 = \frac{1.5 \times 16 + 5}{1 - 0.65} = 82.85\,\text{sec} \simeq 83\,\text{sec}$

$G_A = \frac{y_A}{y}(C_0 - L)$

$= \frac{0.4}{0.65}(82.85 - 16) = 41.14\text{s} \simeq 41\text{s}$

$G_B = \frac{y_B}{y}(C_0 - L)$

$\frac{0.25}{0.65}(82.85 - 16) = 25.71\text{s} \simeq 26\text{s}$

Total cycle time = 41 + 26 +12 +2 +2 = 83 sec

(d) Calculate lead and radius of a turnout on a Broad Gauge railway track with the following data :

Heel divergence = 130 mm

Straight length between theoretical nose of crossing and tangent point of crossing = 1.3 m

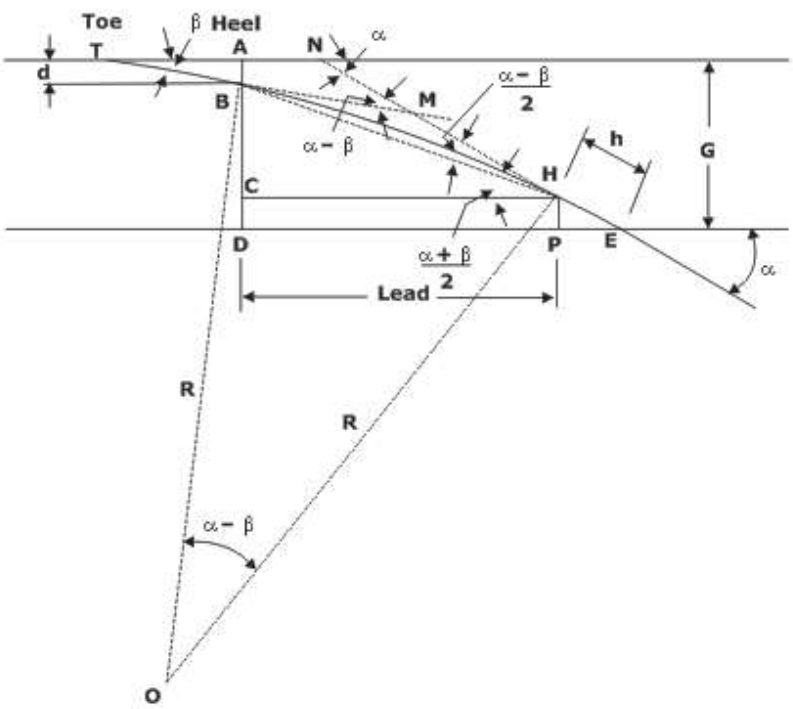
Angle of crossing = 4° 45' 49"

Angle of switch = 1° 08' 00"

Broad Gauge Width = 1.676 m

Show the values on a neat sketch of turnout.

Sol. **Lead of crossing (L)**



Level of Crossing In Δ BMH.

BM = MH (as both are tangents)

$$\angle MHB = \angle MBH = \frac{\alpha - \beta}{2}$$

BC = AD - (AB + CD) = G - (d + h sin α)

Therefore, crossing lead

$$L = (G - d - h\sin\alpha)\cot\frac{\alpha - \beta}{2} + h\cos\alpha$$

Radius of curve (R) Δ OBH,

$$\angle BOH = \alpha - \beta$$

$$BH = 2R\sin\frac{\alpha - \beta}{2} \text{.....(i)}$$

In Δ BHC,

$$BH = \frac{BC}{\sin\frac{\alpha + \beta}{2}} = \frac{G - d - h\sin\alpha}{\sin\frac{\alpha + B}{2}} \text{.....(ii)}$$

Equating Equations (i) and (ii)

$$2R\sin\frac{\alpha - \beta}{2} = \frac{G - d - h\sin\alpha}{\sin\frac{\alpha + \beta}{2}}$$

or

$$R = \frac{G - d - h\sin\alpha}{2\sin\frac{\alpha + \beta}{2} \times \frac{\alpha - \beta}{2}}$$

$$= \frac{G - d - h\sin\alpha}{\cos\beta - \cos\alpha}$$

Given,

$\alpha = 4°4.5'49''$

$\beta = 1°8'$

G = 1.676

d = 0.13m

$\therefore \cot\alpha = 12$

Hence No. of crossing ⇒ N = 1.3 m

$$\text{Radius, } R = \frac{G - d - h\sin\alpha}{\cos\beta - \cos\alpha}$$

$$= \frac{1.676 - 0.13 - 1.3\sin(4°45'49')}{\cos(1°8') - \cos(4°45'49'')}$$

= 441.31 m

(ii) Lead = h cos α + (G - d - h sinα) $\cot\left(\frac{\alpha + \beta}{2}\right)$

$$= 1.3\cos(4°45'49'')$$

$$+(1.676 - 0.13 - 1.3\sin 4°45'49'') \times \cot 2°56'54''$$

= 1.2955 + 1.438 × 19.416

= 29.21 m

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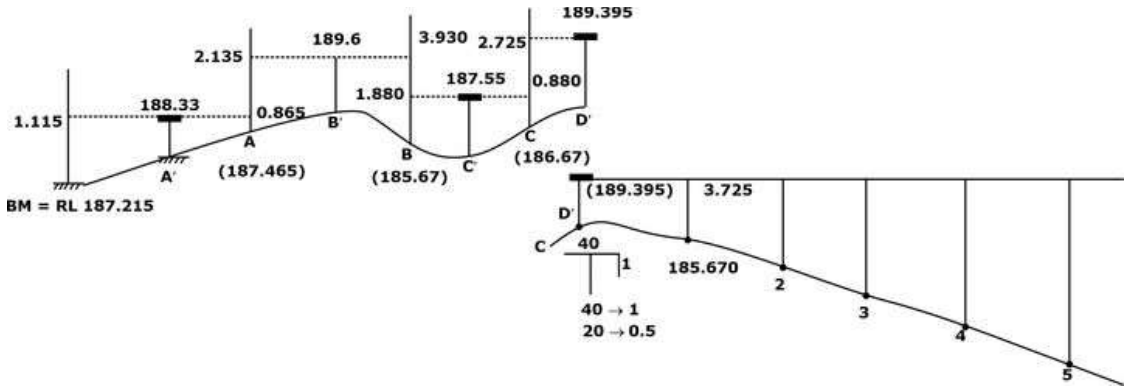
(e) In a running fly level from a benchmark of RL 187.215, the following readings were obtained.

BS	1.115	2.135	1.880	2.725
FS	0.865	3.930	0.880	-

From the last position of the instrument, five pegs at 20 m intervals are to be set out on a uniformly falling gradient of 1 in 40. The first peg is to have an RL of 185.670. Work out the staff readings required for setting the tops of the pegs on the given gradient.

Sol.



*Finding RL at D (Height of Instrument method)

Station	BS	IS	FS	HI	RL	Remark
BM	1.115			188.33	187.215	BM
A	2.135		0.865	189.6	187.465	
B	1.880		3.930	187.55	185.67	
C	2.725		0.880	189.395	186.67	

1		3.725			185.670	
2		4.225			185.17	
3		4.725			184.67	
4		5.225			184.17	
5			5.725		183.67	

$\Sigma BS - \Sigma FS = \text{Last RL} - \text{First RL}$

7.855 - 11.4 = 183.67 - 187.215

= - 3.545

6.(a) Consolidated undrained type Triaxial tests were carried out to failure on two identical specimens of silty clay with pore water pressure measurements, as given below :

S.No.	Confining pressure (kPa)	Deviator stress (kPa)	Pore pressure (kPa)
1.	100	150	40
2.	200	220	75

Determine the shear strength parameters, if

(i) construction is done at a faster rate,

(ii) construction is done slowly.

Sol. (i) construction done, at a faster rate use total stresses

$$\sigma_1 = \sigma_3 \tan^2(45+\phi/2) + 2C\tan(45+\phi/2)$$

$$150+100 = 150\tan^2(45+\phi/2) + 2C \times \tan(45+\phi/2)$$

Use $\tan(45+\phi/2) = T_\phi$

→ For soil specimen no. 1

$$250 = 150\,T_\phi^2 + 2C\,T_\phi \text{ ---- (1)}$$

→ For soil specimen no. 2

$$220+200 = 220\,T_\phi^2 + 2C\,T_\phi \text{ ---- (2)}$$

Solve (1) and (2) for T_φ and C

C = −36.65 kPa

φ = 24.62°

(ii) Construction is done slowly use effective stress

$$\bar{\sigma}_1 = \bar{\sigma}_3 \tan^2(45+\phi/2) + 2C\tan(45+\phi/2)$$

$$(\sigma_1 - u) = (\sigma_3 - u)\tan^2(45+\phi/2) + 2C\tan(45+\phi/2)$$

For soil specimen (1)

$$(150+100-40) = (150-40)\,T_\phi^2 + 2CT\phi \text{ .... (3)}$$

For soil (2)

$$(220+200-75) = (220-75)\,T_\phi^2 + 2CT\phi \text{ .... (4)}$$

Solve (3) and (4)

C = − 51.6 KPa

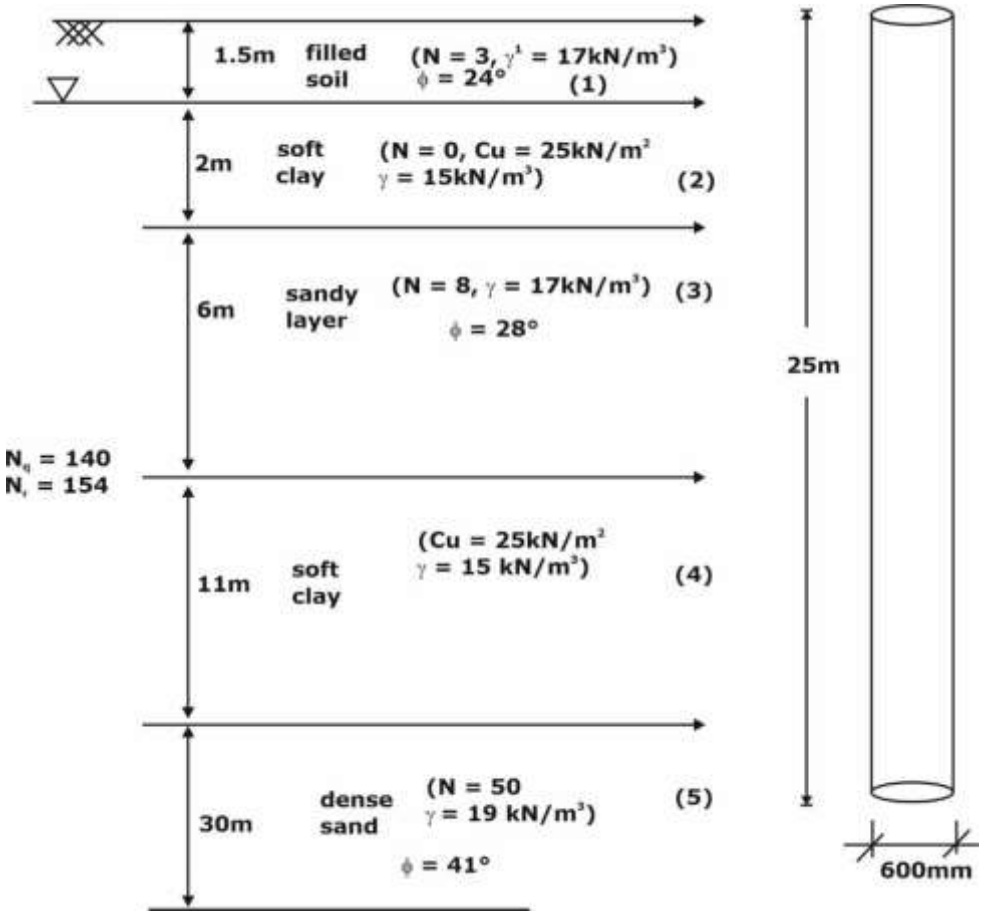
T_φ = 36.03°

(b) The soil profile in a particular site consists of a 1.5 m thick filled up soil (N = 3, γ = 17 kN/m^3) followed by 2 m thick very soft clay layer (N = 0, C_u = 5 kN/m^2, γ = 15 kN/m^3). This is followed by 6 m thick sandy layer (av. N value = 8 and γ = 17 kN/m^3), which is followed by 11 m thick stiff clay layer (av. cohesion = 25 kN/m^2, γ = 15 kN/m^3). This is followed by dense sand upto 30 m (av. N value = 50, γ = 19 kN/m^3). The water table is at 1.5 m below GL. Calculate the safe load that a 25 m long 600 mm dia bored cast in situ pile can carry.

Take for N = 3, φ =24°; N = 8, φ = 28°

for N = 50, φ = 41°, N_q = 140 and N_γ = 152.

Sol.



Ultimate pile load carrying capacity (Q_{up})

$$\boxed{Q_{up} = Q_b + Q_s}$$

(i) Find bearing resistance (Q_b)

$$Q_b = \left(\bar{\sigma} N_q\right).A_b$$

$\bar{\sigma} =$ 17 × 1.5 + (15 – 9.81) × 2 + (17 – 9.81) × 6 + (15 – 9.81) × 11 + (19 – 9.81) × 4.5

= 177.465 kN/m^2

Q_b = 177.46 × 140 × $\frac{\pi}{4}$ × 0.6^2.

= 7024.78 kN

But for bored pile

$Q_b = \frac{1}{2} Q_b$ (for driven pile)

$\frac{1}{2} \times 7024.73$

3512.39kN

(ii) Skin friction resistance (Q_s)

$Q_s = \Sigma Q_s$ for each layer

$= Q_{S_1} + Q_{S_2} + Q_{S_3} + Q_{S_4} + Q_{S_5}$

For sand

$$Q_s = \left(\frac{1}{2}K\gamma L\right)\tan\delta \times A_s$$

As for clay $Q_s = \left(\propto \bar{C}_u\right)$ As

$$Q_s = Q_{S_1} + Q_{S_2} + Q_{S_3} + Q_{S_4} + Q_{S_5}$$

use K = 1 → all sands except dense sand

= 2 → for dense sand

δ = 0.75φ

α = 1

$Q_s = \frac{1}{2}$ × 1 × 17 × 1.5 × tan (0.75 × 24) × (π × 0.6 × 1.5) + 1 × 5 × (π × 0.6 × 2) + $\frac{1}{2}$ × 1 × 17 × 6 × tan (0.75 × 28) × (π × 0.6 × 6) + 1.25 × (π × 0.6 × 11) + $\frac{1}{2}$ × 2 × 19 × tan (0.75 × 41) × 4.5 (π × 0.6 × 4.5)

Q_S = 1201.80 kN

$Q_{up} = Q_b + Q_S$

= 3512.39 + 1201.80

= 4714.2kN

Safe load on pile = $\frac{Q_{up}}{FOS}$

Use FOS = 2.5

Safe load $\frac{4714.2}{2.5}$

= 1885.67kN

(c) Mention standard conditions assumed for basic runway length. Design the runway length for a proposed airport site at an altitude of 420 m above mean sea level. Use the following data :

Basic runway lengths for take-off and landing are 2000 m and 2400 m respectively.

Airport reference temperature is 23°C.

Effective gradient along the proposed runway is 0.4%.

Sol. Standard condition assumed for basic runway length

i) Airport altitude is at sea level

ii) Temperature at airport is standard (15° c)

iii) Runway is levelled in longitudinal direction.

iv) No wind blowing on runway

v) Aircraft is loaded at its full capacity

Numerical

Given,

Elevation of airport = 420m

BRL for landing = 2400m

BRL for take off = 2000m

Airport reference temp = 23°c

i) For takeoff length

Correction for elevation

As per ICAO, correction is 7% per 300m

$\therefore$ correction = $\frac{7}{100} \times \frac{420}{300} \times 2000$

= 196m

$\therefore$ correction length (L_e) = 2000 + 196

= 2196m

For landing, only correction to elevation is needed

$\therefore$ Correction $= \frac{7}{100} \times \frac{420}{300} \times 2000$

= 196 m

$\therefore$ Correction length $= 2196\,\text{m}$

Correction for temperature

As per ICAO, correction due to temp.

is 1% for every 1° C rise

$\therefore$ Rise in temp = 23 – T_S

and standard temp (T_S) at 420 m elevation

= 15 – 0.0065 × 420

= 12.27° C

$\therefore$ Rise in temp = 23 – 12.27

= 10.73° C

$\therefore$ Correction = $\frac{1}{100} \times 10.73 \times L_e$

$\frac{1}{100} \times 10.73 \times 2196$

= 235.63m

$\therefore$ Correction length (L_T) = 2431.63m

$\therefore$ Total correction = 235.2 + 196

= 431.63m

$\therefore$ Total percentage correction

= $\frac{431.63}{2000} \times 100$

= 21.58 < 35% (OK)

Correction for effective gradient

FAA guidelines adopted by DGCA i.e. 20% for every 1% of effective gradient

$\therefore$ Correction length

= $2431.63 + \left[\frac{20}{100} \times 0.4 \times 2431.63\right]$

= 2626.2 m

ii) For Landing

Only correction for elevation is needed.

Correction = $\frac{7}{100} \times \frac{420}{300} \times 2400 = 235.2$m

So, corrected length = 2400 + 235.2 = 2635.2 m

Hence Runway length of 2635.2 m will be constructed. ($\because$ 2635.2 >2626.2)

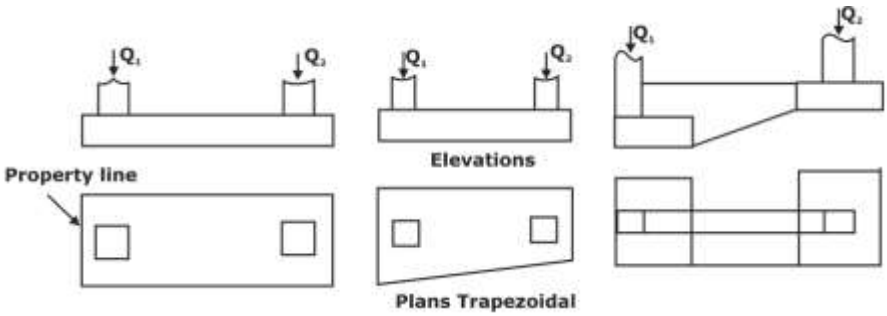
7.(a) (i) What is the basis for classifying foundations into shallow and deep? Briefly explain the situations in which different types of shallow foundations are adopted.

Sol. **Strap footings:**

- A strap footing consists to two or more spread footings connected by a beam called 'strap'. These footings are also known as 'cantilever footing' or 'pump-handle foundation.
- It may be used where the distance between the column is so great that the combined trapezoidal footing become quite narrow with high bending moment.

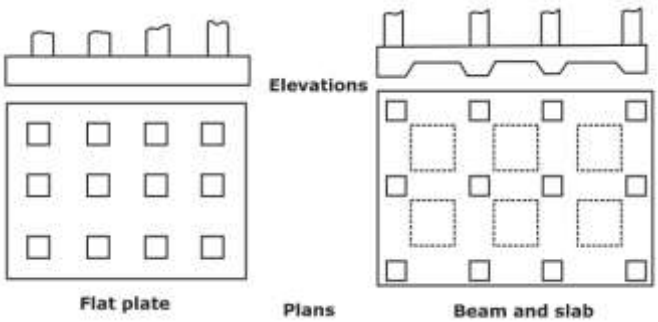
Combined footing

- Combined footing is provided when two columns are close together and separate footing would overlap.
- A combined footing may be rectangular or trapezoidal in shape.



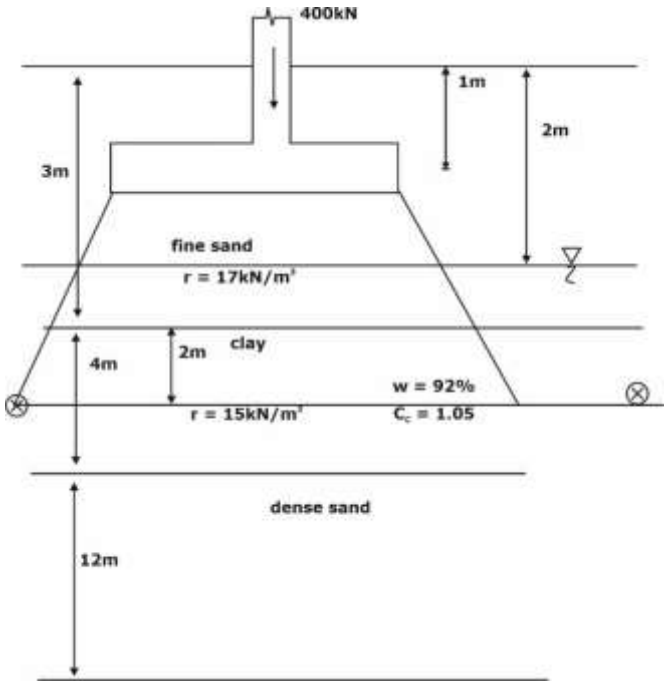
Raft foundation (Mat):

- Raft foundations are used in situation when the loads transmitted by the columns in a structure are so heavy or the allowable soil pressure so small that individual footings would cover more than about one-half of the area.
- This is considered suitable when differential settlement arising out of footing on weak soils are to be minimized.



(ii) A square footing (2 m x 2 m) founded at a depth 1 m below GL has to support a column load of 400 kN. The soil profile consists of fine sand (γ = 17 kN/m^3) up to a depth of 3 m, followed by a 4 m thick layer of silty clay (γ = 15 kN/m^3, NMC = 92%, C_c = 1.05). This is followed by dense sandy layer up to 12 m. The WT is at 2 m below the GL. Compute the possible consolidation settlement and state whether it is within permissible limits.

Sol.



$$H_c = \frac{H_o C_c}{1+e_o}\log_{10}\left(\frac{\bar{\sigma}+\Delta\bar{\sigma}}{\bar{\sigma}}\right)$$

$\bar{\sigma}$ at (x) – (x)

$\bar{\sigma}$ = 17 × 3 + 15 × 2 – 9.81 × 3

= 51.57 kN/m^2

$\Delta\bar{\sigma}$ at (x) – (x)

$$\Delta\bar{\sigma} = \frac{400}{(2+4)(2+4)}$$

= 11.11 kN/m^2

Se = WG {Assume G = 2.7)

e = 0.92 × 2.7

e = 2.484

$$H_c = \frac{4\times 1.05}{1+2.484}\left(\log_{10}\frac{51.57+11.11}{51.57}\right)$$

H_c = 10.21 mm

Since Total settlement is less then 40mm,

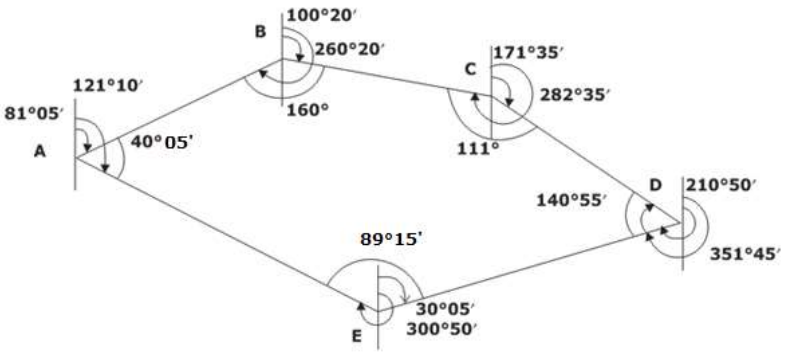
Hence it is within permissible limits

(b) Determine the correct magnetic bearings of the lines of closed traverse having the following bearings as observed:

Line	AB	BC	CD	DE	EA
FB	81°05′	100°20′	171°35′	210°50′	300°50′
BB	260°20′	282°35′	351°45′	30°05′	121°10′

Sol.

LINE	AB	BC	CD	DE	EA
FB	81°05′	100°20′	171° 35′	210° 50′	300° 50′
BB	260° 20′	282° 35′	351° 45′	30. 05′	121° 10′
FB ± BB	179° 15′	182° 15′	180° 10′	180° 45′	175° 40′
Error	45′	2° 15′	10′	45′	20′



$\sum$ interior angles = 541° 15′

$\sum$ interior angles = (2n – 4) 90

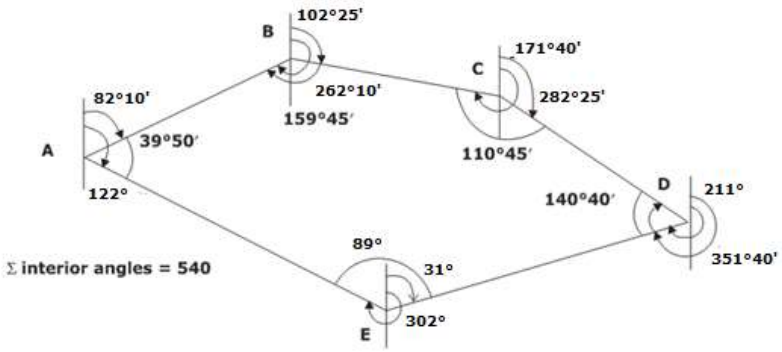
= (2 × 5 – 4)90 = 540

Error in angles = 1° 15′

Distributing error equally = $\frac{75}{5} = 15'$

Line CD has least error.

So, adjusted FB of CD = 171°35′ + 5′ = 171°40′



CORRECTED BEARING

LINE	AB	BC	CD	DE	EA
FB	82° 10′	102° 25′	171° 40′	211°	302°
BB	262° 10′	282 25′	351° 40′	31°	122°

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(c) Describe tunnel lining and various materials used for it.

Sol. A tunnel is an underground or underwater passage that is primarily horizontal. Relatively small-diameter ones carry utility lines or function as pipelines. Tunnels that transport people by rail or by automobile often comprise two or three large, parallel passages for opposite-direction traffic, service vehicles, and emergency exit routes.

Tunnels in loose rock and soft soils are liable to disintegrate and, therefore, a lining is provided to strengthen their sides and roofs so as to prevent them from collapsing. The objectives of a lining are as follows.

(a) Strengthening the sides and roofs to withstand pressure and prevent the tunnel from collapsing.

(b) Providing the correct shape and cross section to the tunnel.

(c) Checking the leakage of water from the sides and the top.

(d) Binding loose rock and providing stability to the tunnel.

(e) Reducing the maintenance cost of the tunnel.

Materials used in tunnels vary with the design and construction methods chosen for each project. Grout used to stabilize soil or fill voids behind the tunnel lining may contain various materials, including sodium silicate, lime, silica fume, cement, and bentonite (a highly absorbent volcanic clay). Bentonite-and-water slurry is also used as a suspension and transportation medium for muck (debris excavated from the tunnel) and as a lubricant for objects being pushed through the tunnel (e.g., TBMs, shields). Water is used to control dust during drilling and after blasting, which is often done with a low-freezing gelatine explosive. Water-and-salt brine or liquid nitrogen are common refrigerants for stabilizing soft ground by freezing. The most common modern lining material, concrete reinforced by either steel or fiber, may be sprayed on, cast in place, or prefabricated in panels.

(d) Classify wet docks and write advantages and disadvantages of each of them.

Sol. **Dock**

- An artificial enclosure for the reception of ships.
- May be regulated by lock gates or open to the tides.
- Docks can be broadly classified as wet docks and dry docks.

Wet Docks

- Dock used for berthing of vessels to facilitate loading and unloading of passengers and cargo.

The water level inside the wet dock is almost constant. A wet dock consists of:

1. Dock entrance
2. An enclosed basin
3. A dock-wall known as quay-wall
4. Facilities for handling and storing of cargo and passengers.

Classification of Wet Docks

1. West docks in tidal basin: Are on the open sea coast protected by an outlying breakwater.

Advantages:

- Vessel can come in and berth or leave any time.
- Costly arrangement like lock gates for entrance closing not required.

Disadvantage:

• If tidal range is more, then operations of loading and unloading are seriously effected.

• The fluctuating water levels causes rubbing of sides of ships against the berths.

2. Enclosed wet dock:

• In enclosed wet dock, water level remains constant by lock gates so that cargo handling and commercial activities increases and becomes easy.

• Disadvantage is that it is costly and because of lock gates and ships will take time for entry and exit.

Advantages:

• Helps in handling cargo.

• The prevent the rubbing of the ship sides.

• Useful when there is increase in draft of the vessel.

• Useful where considerable silting takes place.

8.(a) (i) Comment on the statement "The net bearing capacity of a shallow foundation in clayey soil is unaffected by the position of water table, whereas in sandy soil, it is very much affected".

Sol. Let the net bearing capacity of shallow foundation in q_{net}, and bearing capacity of soil q_u

For strip footing

$q_u = CN_c + \gamma D_f N_f + 0.5 B\gamma N_\gamma$

For clayey soil $N_c = 5.7$, $N_q = 1$ $N_r = 0$

$q_u = 5.7C + \gamma D_f$

$q_{net} = q_u - \gamma D_f$

$q_{net} = 5.7\ C$

Hence q_{net} does not depend on position of water table.

For Sand, C = 0

$q_u = \gamma D_f N_q + 0.5B\gamma N_\gamma$

$q_{net} = \gamma D_f N_q + 0.5\ B\gamma N_\gamma - \gamma D_f$

$q_{net} = (N_q - 1)\ \gamma D_f + 0.5\ B\gamma N_\gamma$

Since q_{net} is depend on density of soil and as water table positions changes the density changes hence bearing capacity (net) is effected by change in water table

(ii) With respect to a compaction curve, explain how one can plot the zero air voids line, 90% saturation line and 10% air voids line.

Sol. **Zero Air void line**

At constant moisture content, the dry unit weight reaches its theoretical maximum value when all the air is expelled from the void spaces, i.e. when degree of saturation is 100%. Therefore, the zero air void line is a line joining points having dry unit weight corresponding to 100% saturation at different moisture contents. Therefore, it is also called the saturation line.

Zero air void lines can be defined as "The lines showing the dry density as a function of water content for soil containing no air voids."

We can derive its equation as follows:

$$\gamma_d = \frac{G_s \gamma_w}{1+e} \qquad ...(i)$$

For any degree of saturation,

$$e = \frac{wG_s}{S}$$

Therefore we can write the expression for dry unit weight corresponding to any degree of saturation s as

$$\gamma_{d.s} = \frac{G_s \gamma_w}{1+\frac{wG_s}{S}} = \frac{\gamma_w}{\left(\frac{1}{G_s}\right)+\left(\frac{w}{S}\right)} \qquad ...(ii)$$

where, $\gamma_{d.s}$ = dry unit weight at degree of saturation

γ_w = unit weight of water, e = void ratio

w = water content, G_s = sp. gravity of solids

For zero air voids, degree of saturation becomes 100%. Therefore equation (ii) becomes

$$\gamma_{d_0} = \frac{\gamma_w}{\left(\frac{1}{G_s}\right)+w} \qquad ...(iii)$$

where, γ_{d_0} = dry unit weight at zero air void

Constant Percentage Air Void Lines

These are lines which shows the water content, dry density relation for the compacted soil containing a constant percentage air void is known as an air voids line.

By the definition of percentage air voids, we have

$$\frac{n_a}{100} = \frac{V_a}{V_v} = \frac{V_v - V_w}{V_v} = 1 - S$$

$$\text{or} \quad S = 1 - \frac{n_a}{100} \qquad ...(iv)$$

Substituting value of S into equation (ii), we have

$$\gamma_{d.a} = \frac{G_s \gamma_w \left(1-\frac{n_a}{100}\right)}{\left(1-\frac{n_a}{100}\right)+wG_s}$$

where, $\gamma_{d.a}$ = dry unit weight at constant percentage air voids

(b) Discuss the geological characteristics necessary for the design and construction of reservoirs.

Sol. It is almost impossible to select a perfect ideal reservoir site. But its selection is guided by the following factors:

(i) A suitable dam site is available. The cost of the dam is generally a controlling factor in the selection of a reservoir site.

(ii) The geological formations for the reservoir banks, walls, etc. should be such as to entail minimum leakage.

(iii) The geology of the catchment area should be such as to entail minimum water losses through absorption and percolation.

(iv) The site should be such that a deep reservoir is formed. A deep reservoir is preferred to a shallow one, because of lower land cost per unit of capacity, less evaporation loss, and less possibility or weed growth.

(v) The reservoir site must have adequate capacity.

(vi) Too much silt laiden tributaries should be avoided as far as possible.

(vii) The reservoir basin should have a deep narrow opening in the valley, so that the length of the dam is minimum.

(c) Discuss how the sensors are classified in Remote Sensing and briefly explain their salient features.

Sol. Remote sensing instruments are of two primary types—

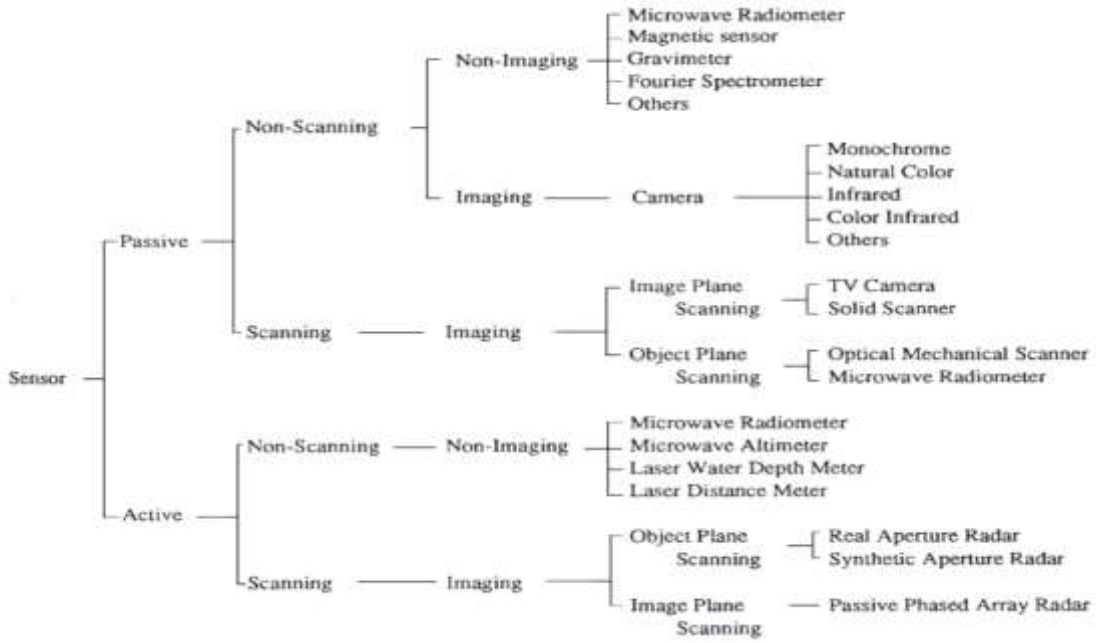
active and passive. Active sensors, provide their own source of energy to illuminate the objects they observe. An active sensor emits radiation in the direction of the target to be investigated. The sensor then detects and measures the radiation that is reflected or backscattered from the target. Passive sensors, on the other hand, detect natural energy (radiation) that is emitted or reflected by the object or scene being observed. Reflected sunlight is the most common source of radiation measured by passive sensors.

Active Sensors

The majority of active sensors operate in the microwave portion of the electromagnetic spectrum, which makes them able to penetrate the atmosphere under most conditions. An active technique views the target from either end of a baseline of known length. The change in apparent view direction (parallax) is related to the absolute distance between the instrument and target.

Passive Sensors

Passive sensors include different types of radiometers and spectrometers. Most passive systems used in remote sensing applications operate in the visible, infrared, thermal infrared, and microwave portions of the electromagnetic spectrum. Passive remote sensors include the following:



(d) Design the length of transition curve to be provided on a horizontal curve of radius 484 m on a National Highway with double lane passing through heavy rainfall area. Following design data is given :

Ruling design speed = 80 kmph

Type of terrain = Rolling terrain

Rate of introduction of superelevation = 1 in 150

Wheel base of design vehicle = 6 m

Sol. Given:

R = 484m, V = 80 kmph

Given terrain - rolling terrain

$e = \frac{1}{150}$, wheel base = 6m

Number of lane = 2

Length of transition curve

(i) According to rate of change of centrifugal acceleration

$C = \frac{80}{75 + V}$ V = 80 kmph

$C = \frac{80}{75 + 80} = 0.516 \text{ m/sec}^{3.}$

$L = \frac{0.0215V^3}{CR} = \frac{0.0215 \times 80^3}{0.516 \times 484}$

L = 44 m

(ii) According to rate of change super elevation

(a) width of pavement for double lane = 7 m

it is given that lane was passing through heavily rain fall area means pavement must be rotated about inner edge to tackle drainage problem.

$e_{allow} = \frac{1}{150} = \frac{1}{N}$

N = 150

$W_e = \frac{n\ell^2}{2R} + \frac{V}{9.5\sqrt{R}} = \frac{2 \times 6^2}{2 \times 484} + \frac{80}{9.5\sqrt{484}}$

= 0.0744 + 0.383= 0.457 m

Total width of pavement = 7 + 0.457

= 7.457 m

$e = \frac{V^2}{225R} = \frac{80 \times 80}{225 \times 484}$

= 0.059

L = e × B × N = 0.059 × 7.457 × 150

= 65.99 $\cong$ 66m

(ii) As per IRC empirical formula

$L_e = \frac{2.7V^2}{R} = \frac{2.7 \times 80 \times 80}{484}$ = 35.71m

The length of transition curve will be maximum of above three values i.e. 66 m.

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Chaitanya Reddy

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Muralikrishna B

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