ACE Engineering Academy
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ESE – 2019 MAINS
OFFLINE TEST SERIES

CIVIL ENGINEERING

TEST – 12
SOLUTIONS

All Queries related to **ESE – 2019 MAINS Test Series** Solutions are to be sent to the following email address
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01. (a)

Sol: $U = U_{max}\left(1-\frac{r}{R}\right)^3$

$$\Rightarrow dQ = U_{max}\left(1-\frac{r}{R}\right)^3 2\pi r dr$$

$$\Rightarrow Q = 2\pi U_{max}\int_o^R r\left(1-\frac{r}{R}\right)^3 dr$$

Consider $\left(1-\frac{r}{R}\right) = z \Rightarrow r = R - Rz$

$$\Rightarrow dr = -R.dz$$

$$\Rightarrow Q = 2\pi U_{max}\int_1^0 (R-Rz)z^3(-Rdz)$$

$$\Rightarrow Q = 2\pi R^2 U_{max}\int_0^1 (z^3 - z^4)dz$$

$$\Rightarrow Q = 2\pi R^2 U_{max}\left(\frac{z^4}{4}-\frac{z^5}{5}\right)\Bigg|_0^1$$

$$\Rightarrow Q = 2\pi R^2 U_{max}\left(\frac{1}{20}\right) = \frac{\pi R^2 U_{max}}{10}$$

$$\overline{U} = \frac{Q}{A} = \frac{U_{max}}{10}$$

$$\alpha = \frac{1}{A\overline{U}^3} = \int U^3 dA = \frac{1}{\frac{\pi R^2 U_{max}^2}{10^3}}\int_o^R U_{max}^2\left(1-\frac{r}{R}\right)^9 2zrdr$$

$$\Rightarrow \alpha = \frac{2\times 10^3}{R^2}\int_0^R r\left(1-\frac{r}{R}\right)^9 dr = \frac{2\times 10^3}{R^2}\int_1^0 R(1-z)z^9(-Rdz)$$

$$\alpha = 2\times 10^3\int_0^1 (z^9 - z^{10})dz$$

$$= 2\times10^{3}\left(\frac{z^{10}}{10}-\frac{z^{11}}{11}\right)\Bigg|_0^1$$

$$\Rightarrow 2\times10^{3}\left(\frac{1}{110}\right)$$

$$\Rightarrow \alpha = \frac{200}{11}$$

01. (b)

Sol: Given scale ratio, $L_r = 1/20$

Speed of prototype, $V_p = 8$ m/s

Model fluid = air

Prototype fluid = sea water

(i) As submarine is to overcome the viscous resistance, Reynolds law of similarity is essential for dynamic similarity between model and prototype

$(Re)_m = (Re)_p$

$$\left(\frac{VL}{\nu}\right)_m = \left(\frac{VL}{\nu}\right)_p$$

$$V_r = \frac{\nu_r}{L_r}$$

$$\frac{V_m}{V_p} = \frac{\nu_m}{\nu_p}\times\frac{L_p}{L_m}$$

$$V_m = \frac{1.64\times10^{-1}}{1.21\times10^{-2}}\times20\times8$$

$= 2168.6$ m/s

(ii) Also drag force, F = ma

$F = \rho L^2V^2$

Ratio,

$$\frac{F_m}{F_p} = \frac{\rho_m}{\rho_p}\times\left(\frac{L_m}{L_p}\right)^2\times\left(\frac{V_m}{V_p}\right)^2$$

$$= \frac{1.34}{1027} \times \frac{1}{20^2} \times \left(\frac{2168.6}{8}\right)^2$$

$$\frac{F_m}{F_p} = \frac{1}{4.17}$$

01. (c)

Sol: $f = 1.76\sqrt{d_{mm}} = 1.76\sqrt{2} = 2.49$

Canal bed slope $S = \dfrac{f^{5/3}}{3340 Q^{1/6}} = 7.135 \times 10^{-4}$

Existing ground slope is 1.5×10^{-4} , which is much lesser

Therefore, the median size of the sediment which the channel would be able to carry can be determined by finding f ′ for $S = 1.5 \times 10^{-4}$

$$S = \frac{f^{5/3}}{3340 Q^{1/6}} \Rightarrow 1.5\times10^{-4} = \frac{(f')^{5/3}}{3340\ \ 50^{1/6}}$$

$\Rightarrow f = 0.976$

$f = 1.76\sqrt{d_{mm}}$

$$d = \left(\frac{0.976}{1.76}\right)^2 = 0.31 \text{ mm}$$

Therefore the material coarser than 0.3 mm will have to be removed for the efficient.

Functioning of the canal

$$V = \left(\frac{Qf^2}{140}\right)^{1/6} = \left(\frac{50\times 0.976^2}{140}\right)^{1/6} = 0.8355 \text{ m/s}$$

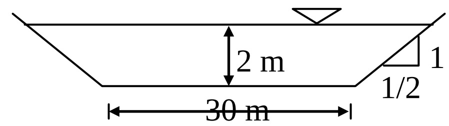
Hydraulic radius $R = 2.5\ \dfrac{V^2}{f} = \dfrac{2.5(0.8355)^2}{0.976}$

$= 1.8$ m

Wetted perimeter $P = 4.75\sqrt{Q} = 4.75\sqrt{50}$

$= 33.6$ m

Side slope is fixed $\left(\dfrac{1}{2} H : 1V\right)$



$P = B + \sqrt{5}D = 33.6$

$B = 33.6 - 2.236\ D$

$$A = BD + \frac{D^2}{2} = PR = 33.6 \times 1.8 = 59.9$$

$33.6\ D - 2.24\ D^2 + 0.5\ D^2 = 59.9$

$\Rightarrow D = 2$ m

Other value is irrational (17.5 m)

$B = 33.6 - 2.236\ (2)$

$= 33.6 - 4.472$

$= 29.1$ m say 30 m

01. (d) (i)

Sol: Slope of a streamline in a 2D flow is given by $M_1 = \left.\frac{dy}{dx}\right|_{\text{Streamline}} = \frac{V}{u} \rightarrow (1)$

For equipotential lines $d\phi = 0$

$$\Rightarrow \frac{\partial\phi}{\partial x}dx + \frac{\partial\phi}{\partial y}dy = 0$$

$$\Rightarrow -Udx - Vdy = 0$$

$$\Rightarrow \left.\frac{dy}{dx}\right|_{EPL} = -\frac{U}{V} = m_2 \rightarrow (2)$$

$$m_1 m_2 = \left(\frac{V}{U}\right)\left(\frac{-U}{V}\right)$$

$\Rightarrow m_1\ m_2 = -1$

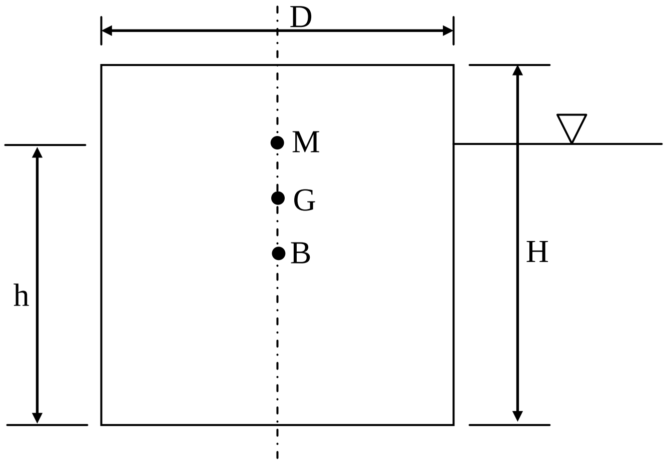
$\therefore$ stream line & equipotential lines are

Orthogonal

Hence Proved

01. (d) (ii)

Sol:



Given:

$\rho_s = 850\ kg/m^3$

$\rho = 1000\ kg/m^3$

D = 1m

H = 0.8m

for vertical equilibrium

$F_B = W$

$$\rho \times \frac{\pi}{4} D^2 \times h \times g = \rho_s \times \frac{\pi}{4} D^2 \times H \times g$$

$$\therefore \quad h = \frac{\rho_s}{\rho} \times H = \frac{850}{1000} \times 0.8 = 0.68 \text{ m}$$

$$GM = \frac{I_{min}}{\forall} - BG$$

$$= \frac{\frac{\pi}{64} \times 1^4}{\frac{\pi}{4} \times (1)^2 \times 0.68} - \left[\frac{0.8 - 0.68}{2}\right]$$

$= 0.0319$ m

$$T = 2\pi \sqrt{\frac{K^2}{gGM}}$$

$$= 2\pi \times \sqrt{\frac{(I/A)}{gGM}}$$

$$= 2\pi \times \sqrt{\frac{\left(\frac{\pi}{64}\times 1^4\right)/\left(\frac{\pi}{4}\times 1^2\right)}{9.81 \times 0.0319}}$$

$$= 2\pi \times \sqrt{\frac{(1/16)}{9.81 \times 0.0319}}$$

= 2.81 seconds

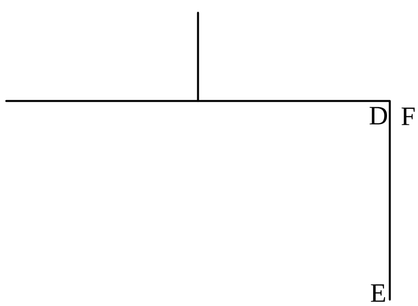
01. (e)

Sol: At Downstream Pile:

$$\alpha = \frac{b}{d} = \frac{48}{10} = 4.8 \text{ m}$$

$$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2}$$

$$= \frac{1+\sqrt{24.04}}{2} = 2.95$$



$$\phi_D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda-2}{\lambda}\right) = 39\%$$

$$\phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda-1}{\lambda}\right)$$

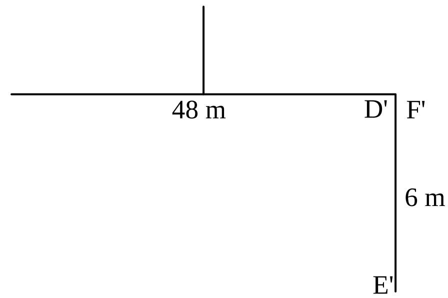
$$= \frac{1}{\pi}\cos^{-1}\left(\frac{1.95}{2.95}\right) = 27\%$$

$$\phi_F = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda}{\lambda}\right) = 0\%$$

To calculate ϕ values at upstream pile, principle of mirror images is to be used

For this, downstream pile depth must be takes as upstream pile depth

Virtual Downstream Pile:



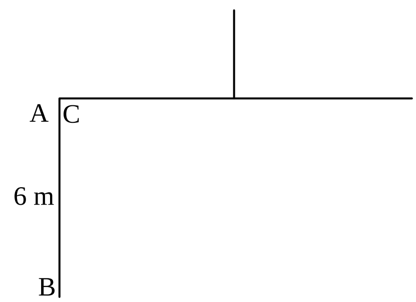
$$\alpha = \frac{48}{6} = 8$$

$$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2} = 4.53$$

$$\phi_{D'} = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda-2}{\lambda}\right) = 31\%$$

$$\phi_{E'} = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda-1}{\lambda}\right) = 21.56\%$$

$$\phi_{F'} = 0\%$$



Now, at upstream pile,

$$\phi_A = 100 - \phi_F{}' = 100\%$$

$$\phi_B = 100 - \phi_{E'}$$

$$= 100 - 21.56 = 78.44\%$$

$$\phi_c = 100 - \phi_{D'}$$

$$= 100 - 31\% = 69\%$$

02. (a)

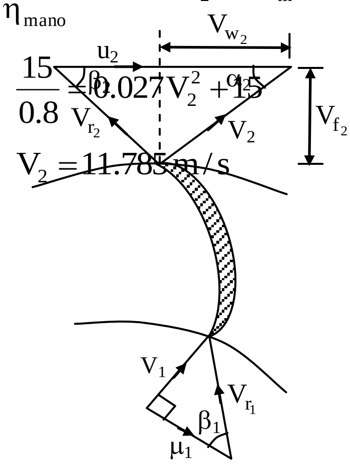
Sol: The energy balance for the impeller gives

$$H_e = h_L + H_m$$

$$\frac{H_m}{\eta_{mano}} = 0.027V_2^2 + H_m$$

$$\frac{15}{0.8} = 0.027V_2^2 + 15$$

$$V_2 = 11.785 \text{ m/s}$$



$$\sin\alpha_2 = \frac{V_{f_2}}{V_2} = \frac{3.2}{11.785}$$

$\Rightarrow \alpha_2 = 15.76°$

$$V_{w_2} = V_2\cos\alpha_2 = 11.342\text{ m/s}$$

$$\eta_{mano} = \frac{gH_m}{U_2V_{W_2}}$$

$$0.8 = \frac{9.81\times15}{U_2\times11.342}$$

$\Rightarrow \quad U_2 = 16.217$ m/s

$$\tan\beta_2 = \frac{V_{fl2}}{U_2 - V_{W_2}} = \frac{3.2}{16.217-11.342}$$

$\beta_2 = 33.28°$

$$U_2 = \frac{\pi D_2 N}{60}$$

$$\therefore \quad D_2 = \frac{60U_2}{\pi N} = \frac{60\times16.217}{\pi\times750} = 0.413\text{ m}$$

$$Q = A_{f_2}V_{f_2} = \pi D_2 B_2 V_{f_2}$$

$$B_2 = \frac{Q}{\pi D_2 V_{f_2}} = \frac{0.6}{\pi\times0.413\times3.2} = 0.144\text{ m}$$

$$\frac{U_1}{U_2} = \frac{D_1}{D_2} = 0.5$$

$U_1 = 0.5 \times 16.5217 = 8.109$ m/s

$$\tan\beta_2 = \frac{V_{f_1}}{U_1}$$

$$\beta_2 = \tan^{-1}\left(\frac{3.2}{8.109}\right) = 21.54°$$

02. (b)

Sol:

Sl. No	Year	Peak Flood (m^3/s)	Deviation from mean ($x - \bar{x}$)	Square of deviation $(x-\bar{x})^2$
1	1971	3200	–1770	3132900
2	1972	4250	–720	518400
3	1973	6250	1280	1638400
4	1974	3100	–1870	3496900
5	1975	2800	–2170	4708900
6	1976	3500	–1470	2160900
7	1977	8500	3530	12460900
8	1978	8900	3930	15444900
9	1979	4200	–770	592900
10	1980	5000	30	900
		Σx = 49,700		$\Sigma(x-\bar{x})^2 = 44156000$

$$\text{Mean}\,\bar{x} = \frac{\Sigma x_i}{n} = \frac{49700}{10} = 4970 \ \ m^3/s$$

Standard deviation

$$\sigma = \sqrt{\frac{\Sigma(x_i-\bar{x})^2}{n-1}} = \sqrt{\frac{44156000}{9}}$$

= 2214.99 = 2215

Let T be the time of years of recurrence interval

Given $Q_T = 10000\ m^3/s$

As per Gumbel's method

$$Q_T = \bar{x} + k.\overline{\sigma_{n-1}}$$

$$k = \frac{Q_T - \bar{x}}{\overline{\sigma_{n-1}}} = \frac{10000-4970}{2215}$$

= 2.27088

Also,

$$k = \frac{y_T - y_n}{s_n}$$

As data is available for 10 years, $y_n = 0.4952$

$S_n = 0.9496$

$$2.27088 = \frac{y_T - 0.4952}{0.9496}$$

$y_T = 2.651628$

Also, $y_T = -\ell n\left(\ell n\left(\frac{T}{T-1}\right)\right) = 2.651628$

$$\ell n\left(\frac{T}{T-1}\right) = 0.070536$$

$= 1.0730832$

$$\frac{T}{T-1} = 0.93189 \Rightarrow \frac{1}{T} = 0.06811$$

$$T = \frac{1}{0.06811} = 14.68 \text{ years}$$

Probability of getting a flood discharge of 10000 m^3/s is

$$P = \frac{100}{T} = 6.81\%$$

02. (c)

Sol: Q = 10 MLD

$C_{in} = 200$ mg/l

$\eta_{ST} = 70\%$

Concentration of VS = $200 \times \frac{70}{100} = 140 \text{ mg}/\ell$

$S_V = 2$, $S_{NV} = 2.7$

Concentration of NVS = $200 \times \frac{30}{100} = 60 \text{ mg}/\ell$

Concentration of solids in sludge = $\eta_{ST} \times C_{in}$

$$= \frac{70}{100} \times 200$$

$= 140$ mg/l

$P_1 = 96\%$, $P_2 = 90\%$

% VS destroyed = 70%

Digestion period, t = 50 days

Before Digestion:

$$\frac{100}{S_{sol}} = \frac{\%VS}{S_{VS}} + \frac{\%NVS}{S_{NVS}}$$

$$\frac{100}{S_{sol}} = \frac{70}{2} + \frac{30}{2.7}$$

$\Rightarrow \quad S_{sol} = 2.168$

$$\frac{100}{S_{sol}} = \frac{4}{2.168} + \frac{96}{1}$$

$S_{sol} = 1.022$

$\Rightarrow \quad \rho_{slu} = 1022\ kg/m^3$

$M_1 = Q \times \eta_{ST} \times C_{in}$

$$= 10 \times \frac{70}{100} \times 200$$

$= 1400$ kg/day

$$V_1 = \frac{100}{100 - \rho_1} \times \frac{M_1}{\rho_{slu}}$$

$$= \frac{100}{100 - 96} \times \frac{1400}{1022}$$

$= 34.246\ m^3$/day

After Digestion:

Concentration of solids in sludge = VS + NVS

$$= \left[140 - 140 \times \frac{70}{100}\right] + 60$$

$= 102$ mg/l

$M_2 = Q_2 \times$ Cone of solids in sludge

$= 10 \times 102$

$= 1020$ kg/day

$$\% \text{ of VS} = \frac{42}{102} \times 100 = 41.176\%$$

$$\% \text{ of NVS} = \frac{60}{102} \times 100 = 58.824\%$$

$$\frac{100}{S_{sol}} = \frac{41.176}{2} + \frac{58.824}{2.7}$$

$\Rightarrow S_{sol} = 2.36$

$$\frac{100}{S_{slu}} = \frac{10}{2.36} + \frac{90}{1}$$

$\Rightarrow S_{slu} = 1.061$

$\rho_{slu} = 1061$ kg/m^3

$$V_2 = \frac{100}{100 - P_2} \times \frac{M_2}{\rho_{slu}}$$

$$= \frac{100}{100 - 90} \times \frac{1020}{1061}$$

$= 9.613$ m^3/day

Capacity of digester, $V = \frac{V_1 + V_2}{2} \times t$

$$V = \frac{34.246 + 9.613}{2} \times 50$$

$= 1096.489$ m^3

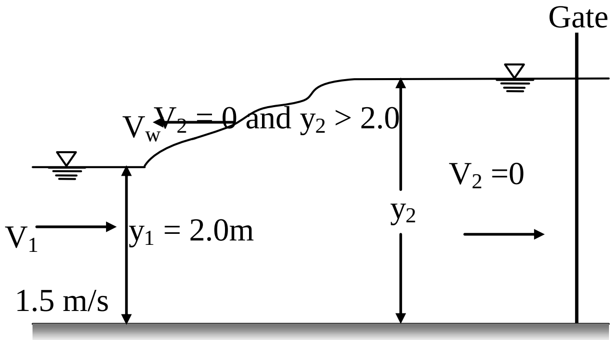
03. (a) (i)

Sol: Given b = 4 m, y_1 = 2m, Q = 12 m^3/s, $V_2 = 0$

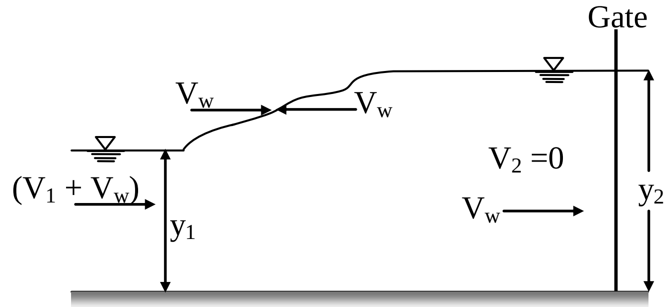
$$\Rightarrow V_1 = \frac{Q}{by} = \frac{12}{4 \times 2} = 1.5 \text{ m/s}$$

Since the flow is completely stopped at down stream, $V_2 = 0$

A positive surge of velocity ($-V_w$) i.e, travelling upstream, will be generated as a result of the sudden stopping of the flow. By superimposing a velocity (V_w) on the system, a steady flow is simulated as shown in figure



(a) Positive surge moving upstream



(b) Simulated steady flow

In simulated steady flow,

By applying continuity equation

$$y_1 (V_w + V_1) = y_2 (V_w + V_2)$$

$$2 (V_w + 1.5) = y_2 V_w$$

$$2 V_w + 3 = V_w y_2$$

$$\Rightarrow V_w(y_2 - 2) = 3$$

$$\Rightarrow V_3 = \frac{3}{y_2 - 2}$$

By applying momentum equation

$$F_{h_1} - F_{h_2} = \rho q(V_{d/s} - V_{u/s})$$

$$\frac{\gamma y_1^2}{2} - \frac{\gamma y_2^2}{2} = \rho y_2 (V_2 + V_w)[V_w - (V_1 + V_w)]$$

$$(\because q = y_2(V_2 + V_w))$$

$$\frac{\gamma y_1^2}{2} - \frac{\gamma y_2^2}{2} = \rho y_2 V_w(-V_1)$$

$$y_2^2 - y_1^2 = 2\frac{\rho}{\gamma} y_2\left(\frac{3}{y_2 - 2}\right)V_1$$

$$\Rightarrow y_2^2 - 4 = \frac{0.9174 y_2}{y_2 - 2}$$

Solving for y_2, we get $y_2 = 2.728$

$\therefore$ Height of surge, $\Delta y = y_2 - y_1$

$$= 2.728 - 2 = 0.728 \text{ m}$$

Velocity of surge, $V_w = \frac{3}{y_2 - 2}$

$$= \frac{3}{2.728 - 2} = 4.121 \text{ m/s}$$

03 (a) (ii)

Ans: Capillarity is a phenomenon by which a liquid rises or falls in a thin annular space due to the combined effect of cohesion & adhesion of liquid particles

Derivation:

γ = sp weight of fluid (N/m^3)

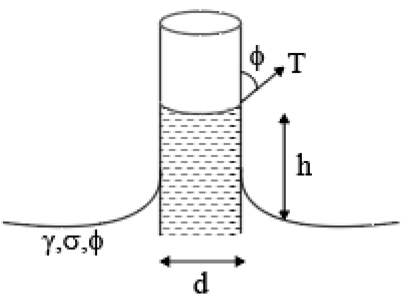
σ = surface tension (N/m)

ϕ = Meniscus angle

d = diameter (m)

h = Capillary rise (m)

T cosϕ = W

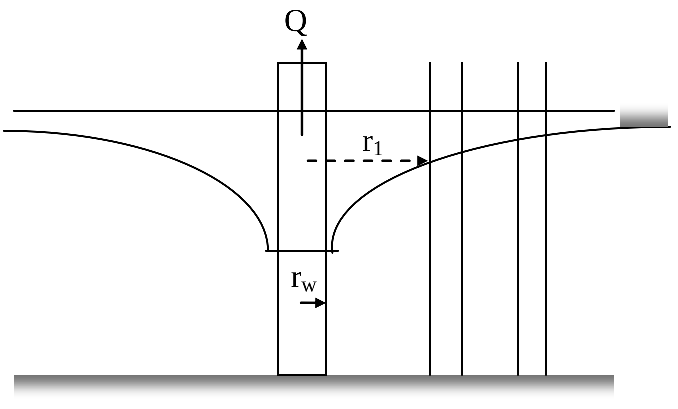


$$\Rightarrow \sigma \pi d \cos\phi = \gamma \frac{\pi}{4} d^2 h$$

$$\Rightarrow h = \frac{4\sigma \cos\phi}{\gamma d}$$

03. (b)

Sol:



Form darcy's law V = ki

$$V = k\frac{dh}{dr}$$

Q = AV

$$= 2\pi rh \quad k\frac{dh}{dr}$$

$$\frac{Q}{2\pi k}\frac{dr}{r} = hdh$$

At $r = r_1$ $\quad h = h_1$ $\quad s = s_1$

$r = r_2$ $\quad h = h_2$ $\quad s = s_2$

$r = r_w$ $\quad h = h_w$ $\quad s = s_w$

$$\frac{Q}{2\pi k}\int_{r_1}^{r_2}\frac{dr}{r} = \int_{h_1}^{h_2} hdh$$

$$\frac{Q}{2\pi K}\log_e\left(\frac{r_2}{r_1}\right) = h_2^2 - h_1^2$$

$$Q = \frac{k\pi\left(h_2^2 - h_1^2\right)}{\log_e\left(\frac{r_2}{r_1}\right)}$$

From well:

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

(i) Q = 1800 *l*pm = 30×10^{-3} m^3/s

r_w = 0.15 m

r_1 = 15 m $\quad$ r_2 = 45 m

s_1 = 1.7 m $\quad$ S_2 = 0.8 m

H_1 = 50 – 1.7 $\quad$ H_2 = 50 – 0.8

H_1 = 48.3 m $\quad$ H_2 = 49.2 m

H = 50

$$30\times10^{-3}=\frac{\pi k\left(49.2^2-48.3^2\right)}{\log_e\left(\frac{45}{15}\right)}$$

$\Rightarrow K = 1.1951\times10^{-4}$ m/s

$= 1.1951\times10^{-4}\times 86400$

$= 10.32$ m/day

$T = KH = 10.32\times 50$

$= 516.2\ m^2/day$

(ii) $$Q=\frac{\pi k\left(h_1^2-h_w^2\right)}{\log_e\left(\frac{r_1}{r_w}\right)}$$

$$30\times10^{-3}\times 86400=\frac{\pi(10.32)\left(48.3^2-h_w{}^2\right)}{\log_e\left(\frac{15}{0.15}\right)}$$

$\Rightarrow h_w = 44.327$ m

S_w = draw down at well

$= H - h_w = 50 - 44.327 = 5.67$ m

03. (c)

Sol:

(i) Size and number of filter beds:

The maximum water demand per day

= population × Maximum daily rate of water supply

= Population × 1.8 × Average daily rate of water supply

$= 75000\times1.8\times150 = 20.25\times10^6$ litre per day = 20.25 MLD

Assuming that 4% of filtered water is required for washing of the filter every day

Total filtered water required per day $=\frac{20.25}{0.96}$ = 21.09 ML

Also, assuming that 0.5 hour is lost every day is washing the filter

Filtered water required per hour $=\frac{21.09}{23.5}\times24$ = 21.538 MLD

$$\frac{21.538}{24} = 0.897 \text{ MLH}$$

Given rate of filtration = 100 L/min/m^2

$$\text{Rate of filtration} = \frac{\text{Filtered water required per hour}}{\text{Area of filter}}$$

$$\Rightarrow 100 \times 60 = \frac{0.897 \times 10^6}{\text{Area of filter}}$$

$\Rightarrow$ Area of filter = 149.5 m^2

Number of units (filter beds) may be roughly estimated by the equation developed by Morell and Wallace. It states that

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

Where N is number of units Q is plant capacity in MLD

$\therefore\ N = 1.22\sqrt{21.09} = 5.6$

Thus, providing 7 filter units = 6 operational + 1 stand by

$\therefore$ Area of each filter unit $= \frac{149.5}{6} = 24.92 \text{ m}^2 = 25 \text{ m}^2$

Assuming the length of filter bed (L) as 1.5 times the width of the filter bed (B)

Now, L × B = 25

$\Rightarrow$ 1.5B × B = 25

$\Rightarrow$ B = 4.1 m

and L = 1.5 × B = 1.5 × 4.1 = 6.15 m

Take length of filter (L) say 6.3 m

$$\therefore B = \frac{6.3}{1.5} = 4.2 \text{ m}$$

Hence, adopting 6 filter units each of dimensions 6.3 × 4.2

ii) Design of Manifold Lateral under Drainage System:

Let a manifold and lateral system be provided below the filter bed, for receiving the filter water and to allow back washing for cleaning the filter. The consists of a central manifold pipe with laterals having perforations at their bottom. To design this system, let us assume area of perforations to be 0.2% of filter area.

$\therefore$ Total area of perforations $= 6.3 \times 4.2 \times \frac{0.2}{100}$ = 0.05292

Now assuming the area of each lateral

= Four times the area of perforations

$\therefore$ Total area of laterals = 4 × 0.05292 = 0.21168 m^2

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

Area of manifold = 2 × 0.21168 = 0.4234 m^2

$\therefore$ Diameter of manifold (d) is given by

$$\frac{\pi}{4} d^2 = 0.4234$$

$$\Rightarrow d = 0.734 \text{ m} = 73.4 \text{ cm}$$

Hence, using a 75 cm diameter manifold pipe laid lengthwise along the centre of the filter bottom. Laterals running perpendicular to the manifold (i.e. width wise) emanating from the manifold may be laid at a spacing of say 15 cm (max 30 cm). The number of laterals is then given as

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

$\therefore$ Number of laterals on each side $= \frac{6.3 \times 100}{15}$ = 42

Hence, use 84 laterals in all, in each filter unit. The diameter of the laterals is adopted as 6 mm.

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

$$= \frac{4.2}{2} - \frac{0.75}{2} = 1.725 \text{ m}$$

Now, total number of perforations in all 84 laterals × Area of each lateral = Total area of perforations

$\therefore$ Total number of perforations in all 84 laterals $\times \frac{\pi}{4} \times \left(\frac{6}{1000}\right)^2$ = 0.05292

$\Rightarrow$ Total number of perforations in all 84 laterals = 1871.66 $\simeq$ 1872

$\therefore$ Number of perforations in each laterals $= \frac{1872}{84}$ = 22.82 say 23

$\therefore$ Area of perforations per lateral $= 23 \times \frac{\pi}{4} \times (0.6)^2 = 6.5 \text{ cm}^2$

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

$$= 4 \times 6.5 = 26 \text{ cm}^2$$

$\therefore$ Diameter of each lateral $= \sqrt{\frac{26 \times 4}{\pi}} = 5.75$ cm

Hence, use 84 laterals each of 5.75 cm diameter at 15 c/c, each having 23 perforations of 6 mm size with 75 cm diameter manifold.

Check: $\frac{\text{Length of each lateral}}{\text{Diameter of lateral}} = \frac{1.725 \times 100}{5.75} = 30 < 60$ (Hence OK)

iii) Wash Water Discharge:

Given rate of washing of filter = 45 rise/min

$\therefore$ Wash water discharge $= \frac{0.45 \times 6.3 \times 4.2}{60} = 0.198 \text{ m}^3/\text{s}$

$\therefore$ Velocity of flow in the lateral of wash water

$$= \frac{0.198}{84 \times \frac{\pi}{4} \times \left(\frac{5.75}{100}\right)^2} = 0.91 \text{ m}^3/\text{s}$$

Similarly, velocity of flow in the manifold

$$= \frac{\text{Discharge}}{\text{Area}} = \frac{0.198}{\frac{\pi}{4} \times (0.75)^2} = 0.448 \text{ m/sec}$$

Thus velocity of flow in less than 1.8 to 2.4 m/s (Hence OK)

04. (a)

Sol: For Pipe P_1 & P_3

$$\frac{h_1}{h_3} = \frac{f\frac{L_1 Q_1^2}{D_1^5}}{f\frac{L_3 Q_3^2}{D_3^5}} = 1 \qquad (\because h_1 = h_3)$$

$\Rightarrow Q_1 = Q_3 \quad (\because L_1 = L_3; D_1 = D_3)$

Let the initial discharge be Q & the final discharge be 1.25 Q.

$\therefore$ for pipe P_1

$$Q_{final} = \frac{1.25Q}{2}$$

$$= 0.625\ Q$$

$$\frac{(h_1)_f}{(h_1)_i} = \frac{f\dfrac{L_1(Q_1)_f^2}{D_1^5}}{f\dfrac{L_1(Q_1)_i^2}{D_1^5}}$$

$(h_1)_f = (h_1)_i\ (0.625)^2$

$(h_1)_f = 0.390625(h_1)_i$

Similarly for P_2

$$\frac{(h_2)_f}{(h_2)_i} = \frac{f\dfrac{L_2(Q_2)_f^2}{D_2^5}}{f\dfrac{L_2(Q_2)_i^2}{D_2^5}}$$

$(h_2)_f = (h_2)_i\ (1.25)^2$

$= 1.5625(h_2)_i$

We know $(h_1)_i + (h_2)_i = 25$ -------- (1)

$(h_1)_f + (h_2)_f = 25$

$\Rightarrow 0.390625\ (h_1)_i + 1.5625\ (h_2)_i = 25$ -------- (2)

Solving (1) & (2)

$(h_1)_i = 12$ m

$(h_2)_i = 13$ m

$$\therefore L_1 = L_3 = \frac{12}{25} \times L$$

$$\therefore\ L_1 = L_3 = \frac{12}{25} \times 1500$$

$$= 720 \text{ m}$$

$\therefore$ The junction has to be at 720 m from upstream end (or) 780 m from down stream end.

$(h_1)_f = 0.390625\ (h_1)_i$

$= 0.390625 \times 12$

$= 4.6875$ m

∴ The head at the junction with respect to down stream water level is (25 – 4.6875) m

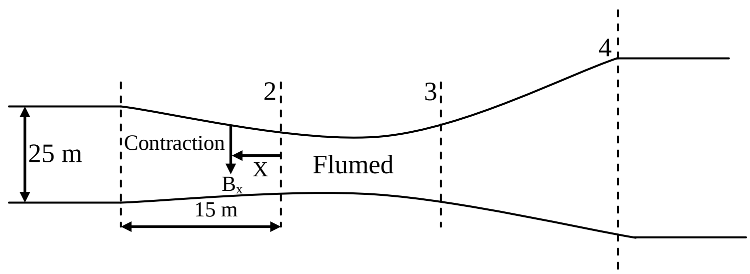
= 20.3125 m

04. (b)

Sol: The Bed width at any section at a distance x from flumed section as per Mitra's method is given by

$$B_x = \frac{B_o B_f L_f}{L_f B_o - (B_o - B_f)x}$$

B_o = Normal width of canal section; B_f = flumed width; B_x = width at 'x' from flumed section



$$B_x = \frac{25 \times 10 \times 15}{15 \times 25 - (25 - 10)x}$$

$$= \frac{3750}{375 - 15x} = \frac{250}{25 - x}$$

For various values of x (from 0 to 15 m), values of B_x are worked out and mentioned as below:

S. No	X from flumed section (m)	B_x (m)
1.	0	10
2.	3	11.36
3.	6	13.16
4.	9	15.63
5.	12	19.23
6.	15	25

As per Chaturvedi's Semi Cubical equation, various distances from the flumed section can be found out by choosing various convenient values of B_x.

$$x = \frac{L_f B_o^{3/2}}{B_0^{3/2} - B_f^{3/2}}\left[1-\left(\frac{B_f}{B_x}\right)^{3/2}\right]$$

$$x = \frac{15\times(25)^{3/2}}{(25)^{3/2} - (10)^{3/2}}\left[1-\left(\frac{10}{B_x}\right)^{3/2}\right]$$

For B_x =10, corresponding value of x is determined as follows:

$$x = \frac{15\times(25)^{3/2}}{(25)^{3/2} - (10)^{3/2}}\left[1-\left(\frac{10}{B_x}\right)^{3/2}\right]$$

For x=0, B_x = 10 m

Similarly for other values of x, B_x is calculated and is mentioned as below:

S. No	X from flumed section (m)	B_x (m)
1.	0	10
2.	3	11.14
3.	6	12.67
4.	9	14.865
5.	12	18.35
6.	15	25

Comparison:

S. No	X from flumed section (m)	B_x (m) as per Mitra's method	B_x (m) as per Chaturvedi's Method)
1.	0	10	10
2.	3	11.36	11.14
3.	6	13.16	12.67
4.	9	15.63	14.865
5.	12	19.23	18.35
6.	15	25	25

04. (c) (i)

Sol: Assume the following data

DT = 24 hrs = 1 day

Rate of sludge production RSP = 30 lit/person/year

Desludging period = 2 years

Q_{DWF} = number of users × percapita DWF

$= 200 \times 150 = 30\ m^3/day$

Volume of settling zone

$V_{sett} = Q_{DWF} \times DT = 30 \times 1 = 30\ m^3$

Assume liquid depth, H = 1.5 m

Surface area of septic tank, $A_S = \frac{V_{sett}}{H} = \frac{30}{1.5} = 20\ m^2$

Assume L : B = 2 : 1 ⇒ L = 2 B

$\therefore\ 2\ B \times B = 10$

$B = \sqrt{\frac{20}{2}} = 3.162\ m$

L = 2 B = 6.324 m

Volume of sludge zone, V_{slu} = RSP × Number of users × Desludging period

$V_{slud} = 30 \times 200 \times \frac{2}{10^3} = 12\ m^3$

Total volume = $V_{sett} + V_{slud}$

Depth of sludge zone, $H_{slud} = \frac{V_{slud}}{L \times B} = \frac{12}{20}$

$= 0.6\ m$

Provide a FB of 0.3 m

Overall depth = 1.5 + 0.6 +0.3

= 2.4 m

∴ Provide a septic tank of size

L = 6.324 m

B = 3.162 m

Overall depth = 2.4 m

04. (c) (ii)

Sol:

AEROSOLS: The tiny liquid or solid particles floating in the air as suspend matter about a millionth of centimetre in diameter, consisting of sulpahtes, soot, organic carbon, and mineral dust are called as Aerosols.

Composition of a typical aerosol	
Component	**Percent value**
Potassium	2
Sea-salt + nitrate	1
Minor inorganics	2
Mineral dust	10
Sulphate	32
Ammonium	8
Fly ash	5
Black carbon	14
Organics	26

Sources of Aerosols

A natural aerosols found over almost the entire globe is a sea slat, including both NaCI and sulphates as magnesium sulphate. The formation begins with the burst of one of the myriad of bubbles that are found on the surface of the sea. The small droplets are released into the air. About 90 per cent of airborne aerosols are naturally occurring substances like dust and particulate matter from volcanic eruptions, and sea spray. Overall, humans are responsible for roughly 10 per cent of the aerosols,, mainly as exhausts from automobile, industrial and biomass burning. In the Indian ocean study, however, scientists inferred that as much as 85 per cent of aerosols were of anthropogenic origin.

Aerosol sources are of two types-primary and secondary.

Primary aerosols are emitted directly as tiny particles, such as smoke from forest or bush fires, soot from burning fossil fuels in industries, vehicles, trains, aeroplanes, airborne dust and sea-salt particles produced when sea spray dries out.

Secondary aerosols are produced from gaseous precursors. Chemicals reactions in the air, converts the primary gaseous pollutants (SO_2, N_2O) into gases with reduced volatility, some of which condense into particulates. Emissions arising from vegetation (as pollen grains, plant debris) and microbial particles (algae, bacteria, viruses etc) also form secondary aerosols. These are biogenic aerosols.

The tiny particles can range in size from 0.01 □m to several tens of micrometre.

Under normal conditions, the majority of aerosols form they are washed out by rain within a week or so. A severe volcanic eruption can push large amounts of aerosol into the upper atmosphere.

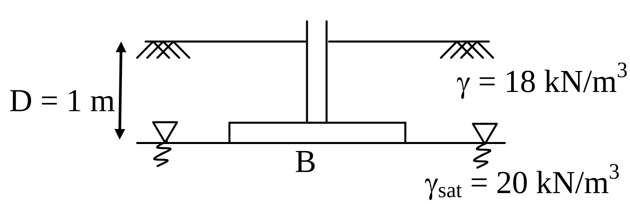
Threats Caused due to Aerosols

- Harmful to human health, being easily passed to lungs.
- Heavy cloud formation, affecting world's major freshwater bodies, as lakes, groundwater supplies, glaciers etc.
- Alteration in temperature and climate, affecting rainfall and monsoon patterns.
- Reduced agricultural yield.

Fine particles less than 2.5 μm in diameter are believed to damage human health, because they move past to the lungs introducing infections microbes to lung tissues. Dust particles as large as 10 μm (dia) can deposit in the lung airways and cause bronchial airway constriction. Particles upto 4 μm can be inhated and interfere with the lung function by penetrating into their gaseous-exchange regions.

05. (a)

Sol:



Given: strip footing, $q_u = CN_c + \gamma DN_q + 0.5\gamma' BN_\gamma$

Design condition

$q = q_s$

$$q = \frac{q_{nu}}{F} + \gamma D$$

$$\frac{Q}{(B\times 1)} = \frac{\gamma D(N_q - 1) + 0.5\gamma' B N_\gamma}{F} + \gamma D$$

$$\frac{Q_g}{(B\times 1)} = \frac{\gamma D(N_q + 2) + 0.5\gamma' B N_\gamma}{3}$$

$$\frac{1000}{(B\times 1)} = \frac{18\times 1(20.4) + 0.5\times(20-9.81)\times B\times 22.4}{3}$$

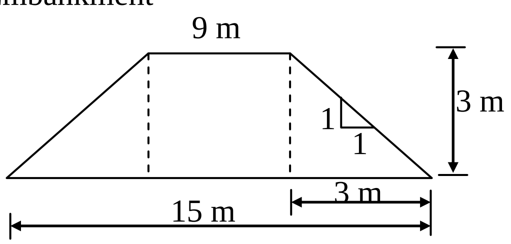
$$\frac{3000}{B} = 367.2 + 114.24\ B$$

$114.24\ B^2 + 367.2\ B - 3000 = 0$

By solving $B = 3.76$ m

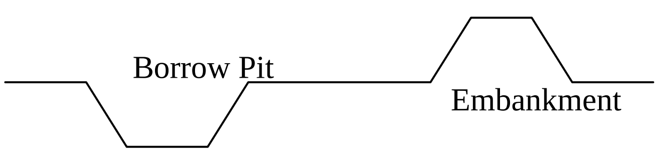
05. (b)

Sol: Embankment



Area of embankment $= \frac{3}{2}\times(9+15) = 36\ m^2$

For 1 km length of embankment, Volume = $36 \times 10^3\ m^3$



$w_1 = 15\%$ $\gamma_{d_2} = 18\ kN/m^3$

$e_1 = 0.69$ $V_2 = 36\times 10^3\ m^3$

$G_1 = 2.7$

$V_1 = ?$

For borrow pit: $\gamma_{d_1} = \dfrac{G\gamma_w}{1+e_1}$

$$\gamma_{d_1} = \frac{2.7 \times 9.81}{1+0.69} = 15.67 \text{ kN/m}^3$$

(i) $V \propto \dfrac{1}{\gamma_d}$

$$\frac{V_1}{V_2} = \frac{\gamma_{d_2}}{\gamma_{d_1}}$$

$$V_1 = \frac{18}{15.67} \times 36 \times 10^3 = 41.35 \times 10^3 \text{m}^3$$

Volume of sandy soil to be excavated = 41.35×10^3 m^3

(ii) Each truck carry 10 m^3

No of truck load = $\dfrac{41.352 \times 10^3}{10} = 4135.2 \approx 4136$ trucks

(iii) $W = \dfrac{W_W}{W_s}$

Weight of water in sandy soil

$W_W = w_1 \times W_s$

$= w_1 \times \gamma_{d_1} \times V_1$

$= 0.15 \times 15.67 \times 41.35 \times 10^3$

$= 97.19 \times 10^3$ kN

$$W_w / \text{truck load} = \frac{97.19 \times 10^3}{4136} = 23.499 = 23.5 \text{ kN}$$

(iv) For sandy soil (Insitu)

$e_1 S_{r_1} = w_1 G$

$$S_{r_1} = \frac{0.15 \times 2.7}{0.69}$$

$= 0.5869$

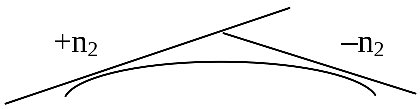
$= 58.69\%$

05. (c)

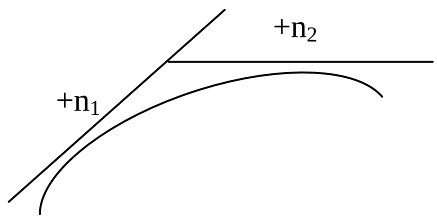
Sol:

(a) Summit curves are provided when

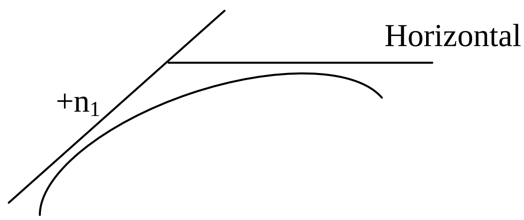
Ascending gradient meets descending gradient



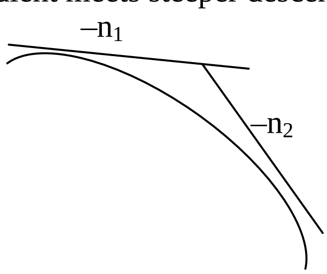
Ascending gradient meets milder ascending gradient



Ascending gradient meets level stretch



Descending gradient meets steeper descending gradient



Given

$$n_1 = \frac{1}{100}\,;\ n_2 = \frac{-1}{100}$$

V = 80 kmph

Deviation angle $N = n_1 - n_2$

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

$$= \frac{2}{100} = \frac{1}{50} \text{ (Positive } \therefore \text{ Summit curve is provided)}$$

Stopping sight distance:

$$SSD = 0.27vt + \frac{V^2}{254f} \text{ (V in kmph)}$$

t = 2.5 sec; f = 0.35

$$\therefore SSD = 0.278 \times 80 \times 2.5 + \frac{80^2}{254 \times 0.35}$$

$$= 127.59 \text{ m}$$

Assuming length of summit curve (L) ≥ SSD

$$L = \frac{NS^2}{\left(\sqrt{2H} + \sqrt{2h}\right)^2}$$

For SSD; H = 1.2 m ; h = 0.15 m

$$\therefore L = \frac{NS^2}{4.4}$$

$$= \frac{1}{50} \times \frac{127.59^2}{4.4}$$

$$= 73.99 < SSD$$

$\therefore$ Assumption is wrong

Assuming L < SSD

$$L = 2S - \frac{\left(\sqrt{2H} + \sqrt{2h}\right)^2}{N} = 2S - \frac{4.4}{N}$$

$$= 2 \times 127.59 - \frac{4}{\left(\frac{1}{50}\right)}$$

$= 55.18 \text{ m} < SSD$

$\therefore$ Length of summit curve required to provide stopping sight distance = 55.18 m

Minimum radius of parabolic summit curve is $R = \frac{L}{N}$

$$= \frac{55.18}{\frac{1}{50}} = 2759 \text{ m}$$

05. (d)

Sol:

Factors effecting selection of site for airport

Regional plan: The site selected should fit well into the regional plan there by forming it an integral part of the national network of airport.

Purpose: The selection of site depends upon the use of an airport. Whether for civilian or for military operations. However during the emergency civilian airports are taken over by the defence. Therefore the airport site selected should be such that it provides natural protection to the area from air roads. This consideration is of prime importance for the airfields to be located in combat zones.

Proximity to other airport: The site should be selected at a considerable distance from the existing airports so that the aircraft landing in one airport does not interfere with the movement of aircraft at other airport. The required separation between the airports mainly depends upon the volume of air traffic.

Ground accessibility: The site should be so selected that it is readily accessible to the users. The airline passenger is more concerned with his door to door time rather than the actual time in air travel. The time to reach the airport is therefore an important consideration especially for short haul operations.

Topography: This includes natural features like ground contours, trees, streams etc. A raised ground a hill top is usually considered to be an ideal site for an airport.

Obstructions: When aircraft is landing or taking off it loses or gains altitude very slowly as compared to the forward speed. For this reason long clearance areas are provided on either side of runway known as approach areas over which the aircraft can safely gain or loose altitude.

Visibility: Poor visibility lowers the traffic capacity of the airport. The site selected should therefore be free from visibility reducing conditions such as fog smoke and haze. Fog generally settles in the area where wind blows minimum in a valley.

Wind: Runway is so oriented that landing and take off is done by heading into the wind should be collected over a minimum period of about five years.

Noise nuisance: The extent of noise nuisance depends upon the climb out path of aircraft type of engine propulsion and the gross weight of aircraft. The problem becomes more acute with jet engine aircrafts. Therefore the site should be so selected that the landing and take off paths of the aircrafts pass over the land which is free from residential or industrial developments.

Grading, drainage and soil characteristics: Grading and drainage play an important role in the construction and maintenance of airport which in turn influences the site selection. The original ground profile of a site together with any grading operations determines the shape of an airport area and the general pattern of the drainage system. The possibility of floods at the valley sites should be investigated. Sites with high water tables which may require costly subsoil drainage should be avoided.

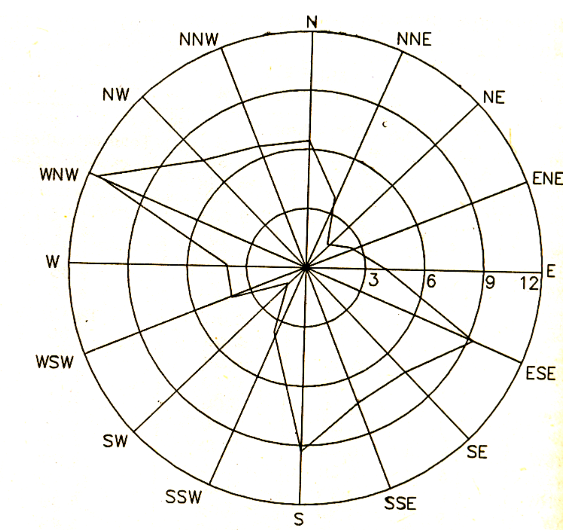
Future development: Considering that the air traffic volume will continue to increase in future more member of runways may have to be provided for an increased traffic.

Head Wind:

The runway is usually oriented in the direction of the prevailing winds. The head wind indicates the wind from the opposite direction of the head or nose of the aircraft while it is landing or taking off. The orientation of runway along the head wind grants the following two advantages:

1) During landing, it provides a breaking effect, and the aircraft comes to a stop in a short
length of the runway.

2) During take off, it provides greater lift on the wings of the aircraft.



Cross Wind :

It is not possible to get the direction of opposite wind parallel to the centre – line of the runway length everyday or throughout the year. For some period of the year at least, the wind may blow making some angle θ with the direction of the centre – line of the runway length as shown in figure:

If V kmph is the velocity of the inclined opposing wind, its component, Vsinθ which is normal to the centre – line of the runway length is called the cross wind component. If this component is in excess, it will interrupt the safe landing and take off operations. The orientation of the runway should therefore be such that this component is kept to a minimum. For light and medium weight aircrafts, the cross wind component should not exceed 25 kmph.

05. (e) (i)

Overbreak:

Rock excavated in excess of the neat lines of a tunnel or cutting. Quick, simple, reliable, and inexpensive measurements of overbreak and underbreak are needed for proper evaluation of tunnelling by the drill and blast method. Problems causing rock damage can be identified and remedied while the work is still in progress. The measurements are also useful in identifying causes of overbreak and overbreak, and in helping to settle contractual disputes relating to payment fo replacement concrete and secondary blasting of 'tights' (zones of underbreak). A newly developed method to measure underbreak and overbreak is presented here. The light sectioning method (LSM) uses a radial sheet of light to define the tunnel profile. An image of the final tunnel profile is acquired and digitized, using digital image analysis. This profile is superimposed over the design profile, and from this zones of overbreak and underbreak are identified, quantified, and presented graphically.

05. (e) (ii)

Sol:

(i) According to Trapezoidal Rule Area =

$$\frac{\text{common distance}\left[\left(\text{1st ordinate}+\text{last ordinate}\right)+2\left(\text{sum of other ordinates}\right)\right]}{2}$$

$$= \frac{5[(2.72+1.6)+2(3.46+5.23+6.8+4.86+3.35+3.00+2.5)]}{2}$$

$= 156.8\ m^2$

(ii) Simpson rule: is applicable because number of ordinates are odd.

$$A = \frac{d}{3}[(\text{first}+\text{last})+4(\text{Even})+2(\text{Odd})]$$

$$= \frac{5}{3}[(2.72+1.6)+4(3.46+6.82+3.35+2.50)+2(5.23+4.86+3.00)]$$

$= 158.23\ m^2$

06. (a)

Sol: Although water table is much below we can consider S = 1 (Saturated soil)

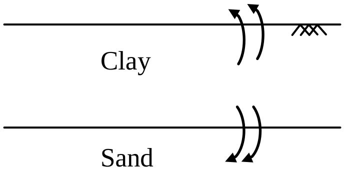
$$\gamma_{bulk} = \frac{(G_s+Se)}{1+e}\gamma_w$$

$$\Rightarrow 19 = \frac{(2.7+e)}{1+e}10$$

e = 0.88

(i) Given clay layer settles by 60 mm in 2.5 years

t = 2.5 years



Δh = 60 mm = Intermediate settlement

$$d = \frac{H}{2} = 5 \text{ (double drainage)}$$

Time factor, $T_v = \frac{C_v t}{d^2} = \frac{1\times 2.5}{25} = 0.1$

$$T_v = \frac{\pi}{4}\left(\frac{U}{100}\right)^2 \qquad U \le 60\%$$

U is degree of consolidation

U = 35.7% $\qquad U = \frac{\Delta h}{\Delta H}\times 100$

$\Rightarrow \Delta H$ = 168.15 mm

$\simeq$ 0.168 m

(ii) Ultimate settlement $\Delta H = \frac{C_c H}{1+e_o}\log_{10}\frac{\overline{\Delta\sigma}+\overline{\sigma}}{\overline{\sigma}}$

e_o = 0.88 (Initial void ratio)

H = 10 m (Thickness of clay layer)

$\overline{\Delta\sigma}$ = Increase in effective stress = 60 kN/m^2

$\overline{\sigma}$ = Effective stress at mid depth of clay layer = $\gamma_{bulk}\times\frac{H}{2} = 19\times 5$ = 95 kN/m^2

$$\Delta H = \frac{C_c H}{1+e_o}\log_{10}\frac{\overline{\Delta\sigma}+\overline{\sigma}}{\overline{\sigma}}$$

$$\Rightarrow 0.168 = \frac{C_c \times 10}{1.88}\log_{10}\frac{95+60}{95}$$

$\Rightarrow$ C_c = 0.15

$$\frac{\Delta e}{1+e_o} = \frac{\Delta H}{H}$$

$$\Rightarrow \Delta e = \frac{0.168}{10}\times(1.88) = 0.0316$$

Final void ratio = 0.88 – 0.0316 = 0.8484

06 (b) (i)

Ans:

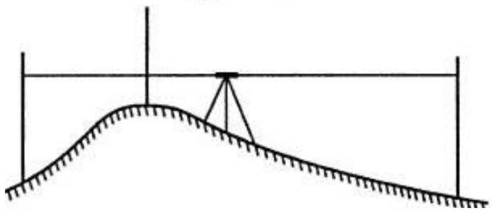
1. Levelling on Steep Slope.
2. Levelling on Summits and Hollows/Undulating terrain
3. Taking Level of an Overhead Point.
4. Levelling Ponds and Lakes too Wide t o be Sighted across.
5. Levelling across River.
6. Levelling on Past High Wall.

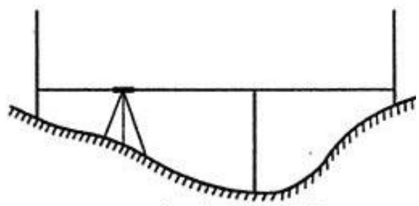
1. Levelling on Steep Slope:

- Du et o short lengths ,it is difficult to obtain the FS and BS accurately.
- In this case, the distance between the instrument station and the FS station is kept approximately equal to the distance between the instrument station and BS station.

2. Levelling on Summits and Hollows/Undulating terrain:

- It is a difficult process, because it requires large number of stations.
- In levelling over summit, the level should be set up sufficiently high, so that the summit can be sighted without extra setting.
- In levelling over hollow, the level should be set up sufficiently low.





Levelling over summit **Levelling over Hollow**

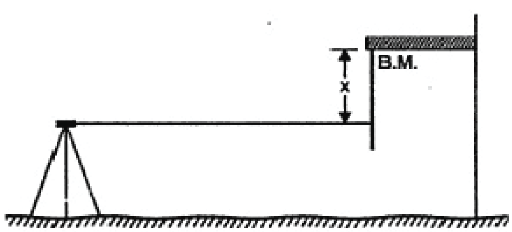
3. Taking Level of an Overhead Point:-

- It occurs when the point under observation / benchmark is higher than the line of sight.
- This can be overcome by,

  (i) Holding the staff inverted on the overhead point keeping the foot of the staff touching the point

  (ii) The RL o ft he line of sight can be calculated as,

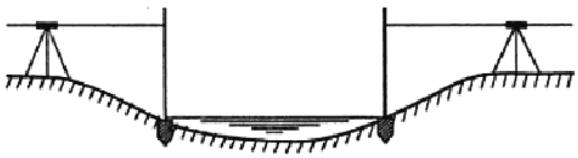
  RL of line of sight = RL of B.M –Staff Reading



4. Levelling Ponds and Lakes too Wide to be Sighted across:-

- While levelling a long distance, due to the obstructions such as lakes the following operations may be carried out.

1. Take FS reading on A from t he station ‘ P’
2. Shift the instrument to Q and take the BS reading on B. In this case also the RL of stations A and B are assumed to be the same.



06. (b) (ii)

Sol: The ground coordinates can be found as follows:

For P, X-coordinate $= \dfrac{(H-h_a)}{f} = (2500-600)\times\dfrac{35}{210} = 316.7\text{m}$

Y-coordinate $= (2500-600)\times\dfrac{25}{210} = 226.2\text{m}$

Ground coordinates of P are (316.7, 226.2).

For Q, X-coordinate $= (2500-600)\times\dfrac{20}{210} = 209.5\text{m}$

Y-coordinate $= (2500-300)\times\dfrac{50}{210} = 523.8\text{m}$

Coordinates of Q are (209.5, 523.8)

The length PQ $= \sqrt{\left[(316.7-209.5)^2+(226.2-523.8)^2\right]} = 316.3\text{m}$

06. (b) (iii)

Ans: Analytic lens is an additional lens placed between the diaphragm and the objective at a fixed distance from the objective. This lens will be fitted in ordinary transit theodolite. After fitting this additional lens the telescope is called as external focusing analytic telescope. The purpose of fitting the analytic lens is to reduce the additive constant to zero.

06. (c)

Sol: K = 100 and C = 0.3. When the staff is held vertical,

Horizontal distance D = Ks $\cos^2\theta$ + C $\cos\theta$

V = (1/2) Ks sin 2 θ + C sin θ

Observation to Q: Vertical angle θ = 5^0 30', S = 2.835 - 2.105 = 0.73 m:

D = 100 × 0.73 × $\cos^2$ (5^0 30',) + 0.3 cos (5^0 30') = 72.638 m

V = (1/2) × 100 × 0.73 × sin(2 × 5^0 30') + 0.3 × sin (5^0 30',) = 6.992 m

Observation to R: Vertical angle = 1^0 08', S = 2.905 - 2.215 = 0.69 m:

$D = 100 \times 0.69 \times \cos^2 (1^0\ 08',) + 0.3 \times \cos (1^0\ 08',) = 69.273$ m

$V = (1/2\ ) \times 100 \times 0.69 \sin(2^0\ 16') + 0.3 \times \sin (1^0\ 08',) = 1.367$ m

In triangle PQR, PQ = 72.638 m, PR = 69.273 m. Angle QPR = 58^0 30'. The distance QR can be calculated using the cosine rule;

$QR^2 = PQ^2 + PR^2 - 2PQPR \cos (58^0\ 30')$

$= 72.638^2 + 69.273^2 - 2\times 72.638 \times 69.273\times \cos (58^0\ 30')$

$= 4816.75$

QR = 69.4 m

RL of BM = 285.35 m

RL of line of sight = 285.35 + 2.255 = 287.605 m

RL of Q = 287.605 + 6.992 - 2.47 = 292.127 m

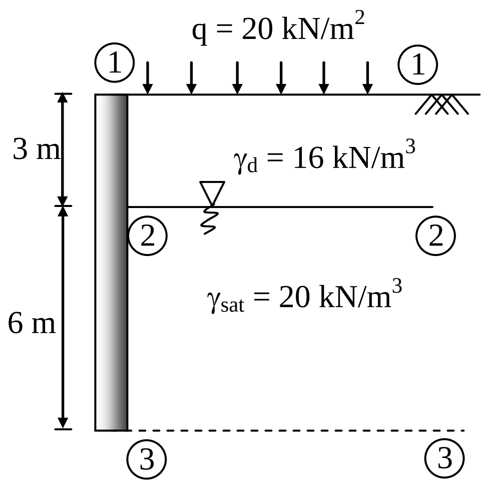
RL of R = 287.605 +1.367 - 2.56 = 286.412 m

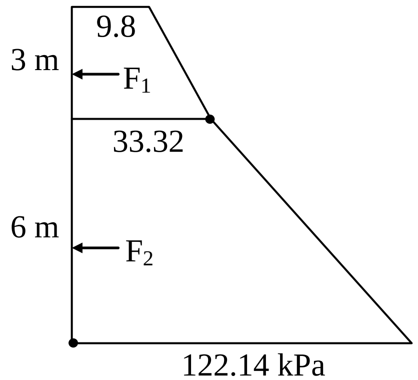
Difference in elevation of Q and R = 292.127 - 286.412 = 5.715 m

Gradient from Q to R = 5.715/69.4 = 0.0823 (about 1 in 12)

07. (a) (i)

Sol:





Active earth pressure distribution

Given: $\phi = 20^o$

$$\Rightarrow \quad k_a = \frac{1-\sin 20^o}{1+\sin 20^o} = 0.49$$

At (1) – (1):- $\sigma_V = q = 20$ kN/m^2

$p_a = k_a\, \sigma_V$

$= 0.49 \times 20$

$p_a = 9.8$ kPa

At (2) – (2):- $\sigma_V = q + \gamma_d \times 3$

$= 20 + 16 \times 3 = 68$ kPa

$p_a = k_a (q + \gamma_d \times 3)$

$= 0.49 \times 68 = 33.32$ kPa

At (3) – (3):- $p_a = k_a (q + \gamma_d \times 3 + \gamma' \times 6) + \gamma_w \times 6$

$= 0.49 (20 + 16 \times 3 + (20 - 9.81) \times 6) + 9.81 \times 6$

$p_a = 122.14$ kPa

Total active thrust, $F_a = F_1 + F_2$

$$F_1 = \frac{3}{2}(9.8 + 33.32) = 64.68 \text{ kN/m}$$

$$F_2 = \frac{6}{2}(33.32 + 122.14) = 466.38 \text{ kN/m}$$

$\therefore F_a = 64.68 + 466.38$

$F_a = 531.06$ kN/m

Line of action:

For F_1:-

$$\text{Centroid } \overline{H}_1 = 6 + \frac{3}{3}\left(\frac{33.32 + 2 \times 9.8}{33.32 + 9.8}\right)$$

$\overline{H}_1 = 7.23$ m from base

For F_2 :-

$$\overline{H}_2 = \frac{6}{3}\left(\frac{122.14 + 2 \times 33.32}{122.14 + 33.32}\right)$$

$= 2.43$ m from base

$$\text{Total line of action} = \frac{F_1 \times \overline{H}_1 + F_2 \times \overline{H}_2}{F_a}$$

$$= \frac{64.68 \times 7.23 + 466.38 \times 2.43}{531.06}$$

$\overline{H} = 3.02$ m from base

07. (a) (ii)

Sol: Mean deflection = $\frac{\Sigma D}{n}$

$$= \frac{1.29 + 1.26 + 1.31 + 1.34 + 1.3 + 1.35 + 1.4 + 1.38 + 1.6}{9}$$

= 1.337 mm

Standard deviation = $\sqrt{\frac{\Sigma\left(D - \overline{D}\right)^2}{n - 1}}$

= 0.05

Characteristic deflection D_c = $\overline{D} + \sigma$

= 1.337 + 0.05 = 1.387 mm

Deflection after temperature correction

$D_t = D_c - (t_c - 35^oC) \times 0.0065$

= 1.387 – (30 – 35) × 0.0065

= 1.4195 mm

Deflection after subgrade moisture correction:

= 1.4195 × 1.4 = 1.9875 mm

Overlay thickness required for WBM / granular layer

$$h_o = 550 \log_{10} \frac{D_c}{D_a}$$

Given allowable deflection D_a = 1 mm

$\therefore$ $h_o = 550 \log_{10} \frac{1.9875}{1}$

= 164.07 mm $\simeq$ 16.4 cm

When bituminous concrete with is provided as overlay as per IRC equivalent factor = 2

$\therefore$ Design thickness of overlay = $\frac{16.4}{2}$

= 8.2 cm

07. (b)

Sol:

From stress criteria, Equivalent single wheel load is that single wheel load producing the same value of stress of that of dual wheel assembly at the desired depth.

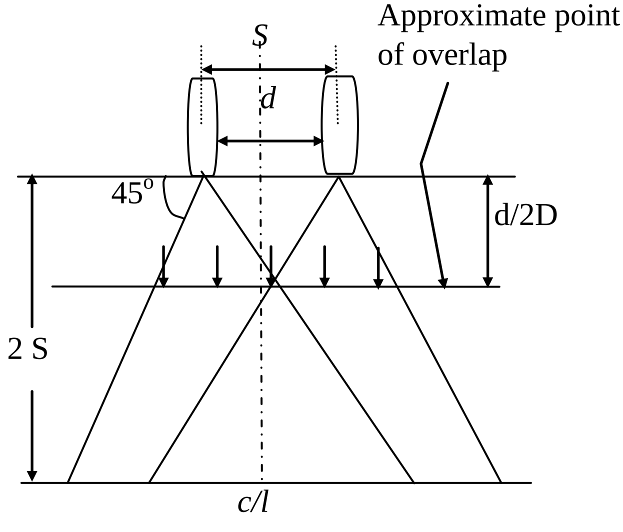
From strain criteria, Equivalent single wheel load is that single wheel load having same contact pressure producing the same value of maximum deflection of that of dual wheel assembly at the desired depth.

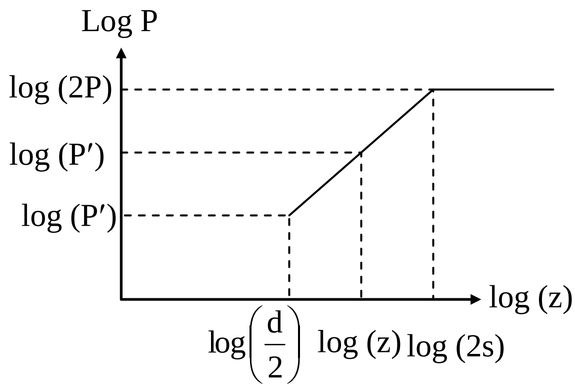
Significance:

Dual wheel assemblies are provided to maintain the maximum wheel load within specified limit and to carry greater load. In the case of dual wheel assembly, the stress at any depth is more than that of single wheel and cannot be obtained directly by adding the pressure caused by each wheel load as it is in between that of single load and twice of each wheel. Hence, to use the effective load in pavement design, multiple wheel loads are converted to ESWL using the load dispersion technique.

Derivation:

If 'd' is the clear spacing between the wheels and 'S' is the centre to centre spacing of the wheels, from the figure assuming the load dispersion as 45^o , upto depth d/2, each load acts independently as there is no overlap and after depth of 2S, stress is due to effect of both wheels as the overlap is considerable. In between d/2 and 2S, the value of stress with respect to depth is assumed to be liner on log-log scale.





For any depth 'z', such that $\frac{d}{2} < z < 2s$; Let ESWL be 'P'. Since variation is linear, slope is constant.

$$\text{i.e}\ \frac{\log(P') - \log(P)}{\log z - \log\frac{d}{2}} = \frac{\log 2P - \log P}{\log 2s - \log\frac{d}{2}}$$

$$\Rightarrow \frac{\log P' - \log P}{\log\left(\frac{z}{d/2}\right)} = \frac{\log 2}{\log\left(\frac{2s}{d/2}\right)}$$

$$\Rightarrow \log P' - \log P = 0.3010\ \frac{\log\left(\frac{z}{d/2}\right)}{\log\left(\frac{2s}{d/2}\right)}$$

$$\log P' = \log P + 0.3010\ \frac{\log\left(\frac{z}{d/2}\right)}{\log\left(\frac{2s}{d/2}\right)}$$

Conclusion:

For $Z \le \frac{d}{2}$; ESWL = P

$\frac{d}{2} < Z < 2s$; ESWL is given by

$$\text{Log (ESWL)} = \log P + 0.3010 \frac{\log\left(\frac{z}{d/2}\right)}{\log\left(\frac{2s}{d/2}\right)}$$

$d \geq Zs$; ESWL = 2P

(b) Given P = 2268 kg

S = 27 cm

a = 8 cm

$\therefore$ Clear spacing between tyres 'd' = S-2a

= 27 – 16

= 11 cm

Thickness of pavement 'z' = 25 cm

$$\frac{d}{2} = \frac{11}{2} = 5.5 \text{ cm}$$

2S = 54 cm

$$\therefore \frac{d}{2} < z < 2s$$

ESWL is given by

$$\log P' = \log(2268) + 0.3010 \frac{\log\left(\frac{25}{5.5}\right)}{\log\left(\frac{54}{5.5}\right)}$$

= 3.555

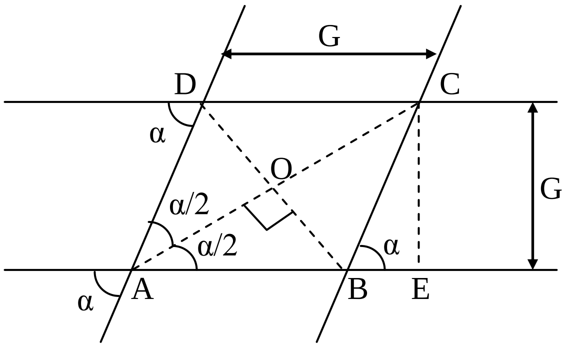
$\therefore$ P′ = 3590.57 kg

$\therefore$ ESWL = 3590.57 kg

07.(c) (i)

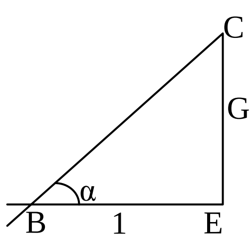
Sol: Diamond crossing:

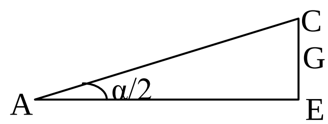
- When straight tracks or curved tracks of same (or) different gauges cross each other at an angle less than 90^{o}, a diamond crossing is formed.
- It consists of two acute angel crossing (α) and two obtuse angle crossings (β) and four check rails.
- Indian standards specify the limit of flattest diamond to be 1 m 10 BG tracks and 1 in 8/2 for other gauges.
- Diamond crossing should be avoided to the maximum extent on curves as they require speed restriction.



From triangle CBE :

Side of diamond CB = G cosec α





In triangle ACE :

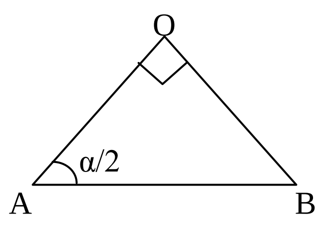
Diagonal AC = CE cosec $\alpha/2$

$= G \operatorname{cosec} \alpha/2$

In triangle BOA:

OB = OA tan $\alpha/2$

$$= \frac{AC}{2} \tan\alpha / 2$$



$$= \frac{G\,\text{cosec}\,\alpha/2}{2}\tan\frac{\alpha}{2}$$

$$= \frac{G}{2}\sec\frac{\alpha}{2}$$

∴ Diagonal BD = 2OB = 6 sec $\frac{\alpha}{2}$

In 1 in 10 crossing

N = 10 cot α

A = 5° 42′ 38.14″ = 5.71°

∴ Side of diamond = AB = BC = CD = AD = G cosec α

= G cosec 5.71

= 1.676 × cosec 5.7

= 16.84 m

Diagonal AC = G cosec α /2

$$= 1.676\ \text{cosec}\ \frac{5.71}{2} = 33.65\ \text{m}$$

Diagonal BD = G sec $\frac{\alpha}{2}$

$$= 1.676\ \sec\ \frac{5.71}{2} = 1.678\ \text{m}$$

07. (c) (ii)

Sol: Docks are enclosed areas for berthing ships, to keep them float at a uniform level, to facilitate loading and unloading cargo.

Functions of docks:

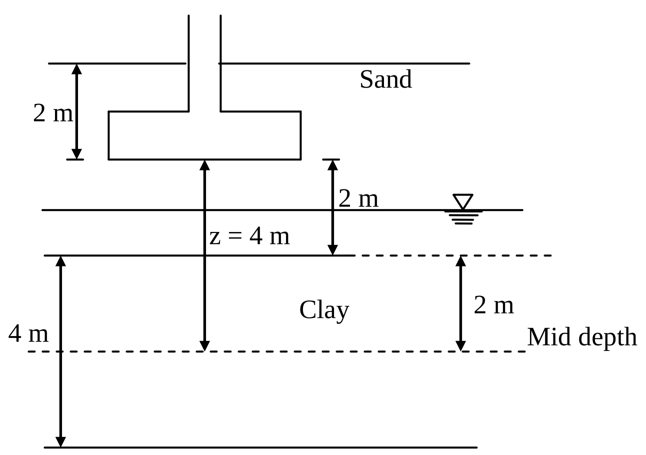
- To maintain uniform level of water for handling cargo
- Passenger exchange
- Loading , unloading , building and repair of ships

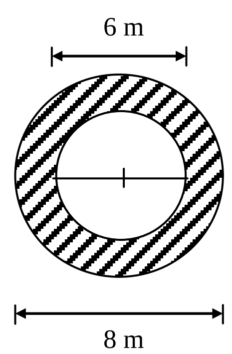
The following are the differences between floating dock and dry dock

	Floating Dock	Dry dock
	It is a floating vessel which can lift a ship out of water and retain it above water by means of its own buoyancy	It is a long, excavated chamber, having side walls, a semicircular end wall and a floor.
	It is a generally a steel structure	The dock is constructed of concrete or masonry
	Time required for construction is less	Time required for construction is more.
	It can be transferred from point to point	This is a fixed structure
	It has no elaborate entrance or gate arrangements	The open end of the chamber is provided with a gate and acts as the entrance to the dock.

08. (a)

Sol:





Plan of footing

Vertical Stress at depth z = σ_Z [below centre]

$$\sigma_Z = q\left[1-\left[\frac{1}{1+\left(\frac{r_o}{z}\right)^2}\right]^{3/2}\right] - q\left[1-\left[\frac{1}{1+\left(\frac{r_i}{z}\right)^2}\right]^{3/2}\right]$$

q = load intensity = 200 kN/m^2

z = 4 m

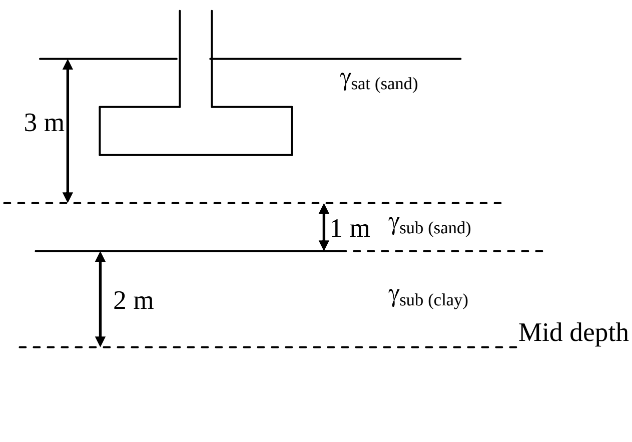
r_o = outer radius = 4m

r_i = inner radius = 3 m

$$\sigma_Z = 200\left[1-\left[\frac{1}{1+\left(\frac{4}{4}\right)^2}\right]^{3/2}\right] - \left[1-\left[\frac{1}{1+\left(\frac{3}{4}\right)^2}\right]^{3/2}\right]$$

= 200[0.646 – 0.488]

= 31.6 kN/m^2



$$\text{For sand, } e = \frac{wG_S}{S} = \frac{0.25\times 2.64}{1} = 0.66$$

$$\gamma_{sat}(\text{sand}) = \left(\frac{G_S + e}{1+e}\right)\gamma_w = 19.88 \text{ kN/m}^3$$

γ_{sub} (sand) = $\gamma_{sat} - \gamma_W$ = 9.88 kN/m^3

$$\text{For clay } e = \frac{wG_S}{S} = 0.4\times 2.72 = 1.09$$

$$\gamma_{sub}(\text{clay}) = \left(\frac{G_s - 1}{1+e}\right)\gamma_w = 8.23\ \text{kN/m}^3$$

Initial over burden pressure = $\bar{\sigma}_o$

$= 2 \times 8.23 + 1 \times 9.88 + 3 \times 19.88$

$= 16.46 + 9.88 + 59.64 = 85.98\ \text{kN/m}^2$

Liquid limit = w_L

Plastic limit = w_P = 30%

$I_P = w_L - w_P = 40\%$

$w_L = 70\%$

C_C = Compression Index = 0.009 (70 – 10) = 0.54

Settlement

$$\Delta H = \frac{C_C H}{1+e_o}\log_{10}\left(\frac{\bar{\sigma}_o + \sigma_Z}{\bar{\sigma}_o}\right)$$

$$= \frac{0.54 \times 400}{1+1.09}\log_{10}\left(\frac{85.98+31.6}{85.98}\right) = 14.05\ \text{cm}$$

08. (b) (i)

Sol: Practical capacity: It is the maximum number of vehicles that can pass a given point on a lane per hair without traffic density being considerable to cause unreasonable delay, hazard or restriction to the driver's freedom to move under prevailing road conditions.

Basic capacity: It is the maximum number of vehicles that can pass a given point on a lane or roadway during one hair under ideal roadway and traffic conditions. Two roads of same physical features will have same basic capacity/

Possible capacity: It is the maximum number of vehicles that can pass a given point on a lane during one hour under prevailing road way and traffic conditions. It varies from zero to basic capacity.

Factors affecting practical capacity:

(i) Lane width: As lane width increases, practical capacity increases.

(ii) Lateral clearance: Restricting the lateral clearance affects driving comfort, increases accident rates and reduces capacity.

(iii) Width of shoulder: Narrow shoulders reduces effective width of traffic lanes, reducing the capacity of lane.

(iv) Commerical vehicles: Large commercial vehicles travel at slow speed, occupy more space thus reducing the capacity.

(v) Alignment: Restriction to sight distance requirement reduces the capacity.

(vi) Presence of intersection at grade: Intersections reduces the free flow, affecting the traffic capacity.

Given: Design speed V = 80 kpmh

l = 7 m

Basic capacity:

$$C = \frac{1000V}{S}$$

V = 80 kmph

$S = S_g + l$

$S_g = 0.278\ Vt$

t = reaction time = 0.7 sec

$\therefore$ S = 0.278 × 80 × 0.7 +7

= 22.568 m

$$\therefore\ C = \frac{1000V}{S}$$

$$= \frac{1000 \times 80}{22.568}$$

= 3544.84 veh/hr/lane

$\simeq$ 3545 veh/hr

Practical capacity

$$C = \frac{1000V}{S}$$

S = SSD + L

$$SSD = 0.278Vt + \frac{V^2}{254f}$$

Assuming f = 0.35; l = 2.5 sec

$$SSD = 0.278 \times 80 \times 2.5 + \frac{80^2}{254 \times 0.35}$$

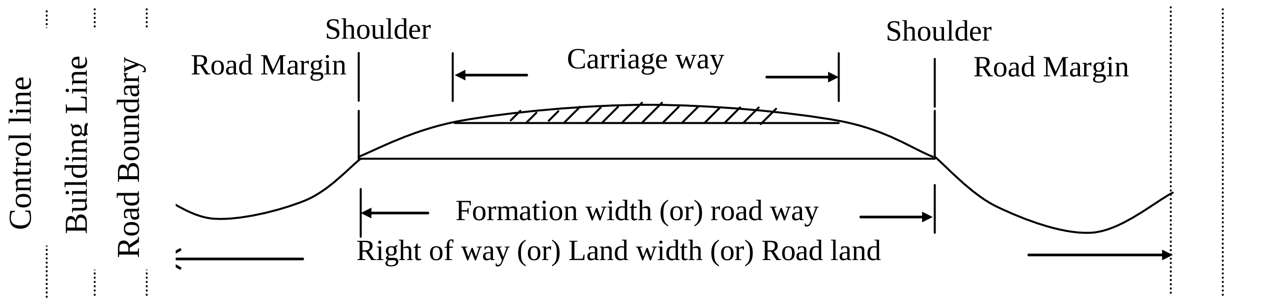
= 127.59 m

S = 127.59 + 7 = 134.59 m

$$C = \frac{1000 \times 80}{134.59} = 594.39 \text{ veh/hr/}l\text{ane}$$

08. (b) (ii)

Sol: Typical cross section elements of a road

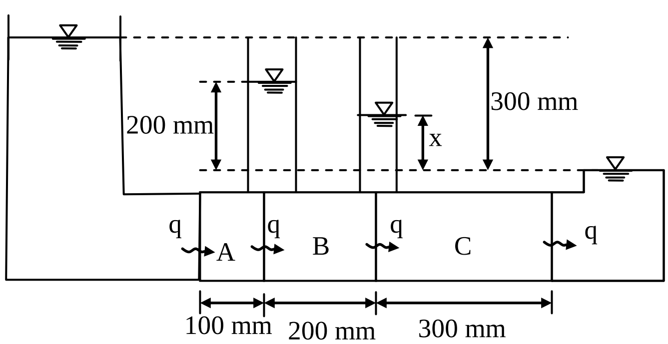


08. (c) (i)

Sol:

Flow is perpendicular to planes of stratification

So, flow will be same in each layer = $q = q_A = q_B = q_C$



(a) $q_A = q_B = q_C$

$\Rightarrow k_A\, i_A\, A_A = k_B\, i_B\, A_B = k_c\, i_c\, A_c$

$\Rightarrow k_A\, i_A = k_B\, i_B = k_c\, i_c$

$$\Rightarrow k_A\left(\frac{\Delta h_A}{L_A}\right)=k_B\left(\frac{\Delta h_B}{L_B}\right)=k_C\left[\frac{\Delta h_C}{L_C}\right]$$

$$\Rightarrow 1\times10^{-2}\times\left(\frac{300-200}{100}\right)=4\times10^{-2}\left(\frac{200-x}{200}\right)=2\times10^{-2}\left(\frac{x}{300}\right)$$

$\Rightarrow x = 150$ mm

(b) $$k_{eq}=\frac{L_A+L_B+L_C}{\frac{L_A}{k_A}+\frac{L_B}{k_B}++\frac{L_C}{k_C}}$$

$$=\frac{100+200+300}{\frac{100}{1\times10^{-2}}+\frac{200}{4\times10^{-2}}+\frac{300}{2\times10^{-2}}}$$

$$=\frac{600}{300}\times10^{-2}=2\times10^{-2}\,\text{cm/s}$$

(c) $$q=k_{eq}i\ A=2\times10^{-2}\times\left(\frac{300}{600}\right)\times10\times10$$

$= 1$ cm^3/s

(or) $q = q_A = k_A\ i_A\ A$

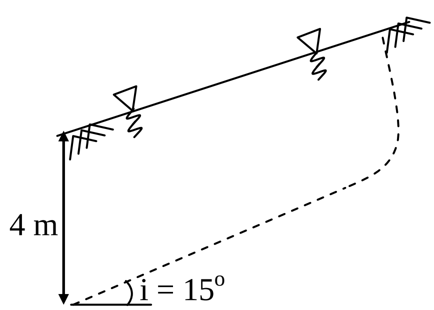
$$=10^{-2}\times\left(\frac{300-200}{100}\right)\times10\times10$$

$= 1$ cm^3/s

q can be also found by equating to q_B or q_C

08. (c) (ii)

Sol:



For infinite slope: Seepage parallel to slope surface

$$FOS = \frac{C' + \gamma' z \cos^2 i \tan\phi'}{\gamma_{sat}.z \cos i \sin i}$$

$$FOS = \frac{15 + (20 - 9.81) \times 4 \times \cos^2 15^o \times \tan 30^o}{20 \times 4 \times \cos 15^o \sin 15^o}$$

$$FOS = \frac{20.49}{20}$$

FOS = 1.0245 ≈ 1.02