ESE Main Examination

Civil Engineering : Paper-II

(Previous Years Solved Paper 1999)

Contents

MADE EASY PUBLICATIONS

1 Fluid Mechanics, Hydraulic Machines and OCF

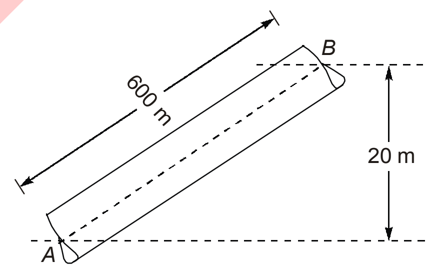
5. Fluid Dynamics, Flow Measurements and Vortex Flow

5.1 A steel pipe of 15 cm diameter carries water at a rate of 30 liters per second from point *A* to *B* along the pipe, the point *B* being 20 m higher than point A and 600 m apart along the pipe. If the pressure at *B* is to be 2.8 kg/cm^2, what pressure must be maintained at *A*, if friction factor for the pipe is 0.024? What will be the capacity of the pipe after 15 years of service, when the friction factor is tripled? Assume that the same pressures are to be maintained at *A* and *B*, (kg = unit of force in metric system).

[15 marks : 1999]

Solution:

The brief schematic given below shows the arrangement clearly.



Given data:

$$Q = 30 \text{ litres per second} = 30 \times 10^{-3} = 0.03 \text{ m}^3/\text{s}$$

$$d = 15 \text{ cm} = 0.15 \text{ m}$$

$$p_B = 2.8 \text{ kg/cm}^2 = \frac{2.8 \times 9.81}{(10^{-2})^2} = 2.75 \times 10^5 \text{ N/m}^2$$

$$L = 600 \text{ m};\ f = 0.024$$

Assuming point *A* as datum, we have $Z_A = 0$ and $Z_B = 20$ m

Applying Bernoulli's equation between *A* and *B*, we get,

$$\frac{p_A}{w} + \frac{V_A^2}{2g} + Z_A = \frac{p_B}{w} + \frac{V_B^2}{2g} + Z_B + h_f$$

$$\Rightarrow \quad \frac{p_A}{w} + \frac{V^2}{2g} + 0 = \frac{2.75 \times 10^5}{9810} + \frac{V^2}{2g} + 20 + \frac{fLQ^2}{12.1d^5}$$

$$\Rightarrow \quad \frac{p_A}{w} = 28 + 20 + \frac{0.024 \times 600 \times (0.03)^2}{12.1 \times (0.15)^5}$$

$$\Rightarrow \quad p_A = 62.10 \times 9810 = 6.1 \times 10^5 \text{ N/m}^2$$

When friction factor is tripled

$$f = 3 \times 0.024 = 0.072$$

$$p_A = p_B$$

Again applying Bernoulli's equation between *A* and *B*, we get

$$\frac{p_A}{w} + \frac{V_A^2}{2g} + Z_A = \frac{p_B}{w} + \frac{V_B^2}{2g} + Z_B + h_f \qquad [V_A = V_B]$$

$$\frac{6.096 \times 10^5}{9.81 \times 1000} + 0 + \frac{V_A^2}{2g} = \frac{2.75 \times 10^5}{9.81 \times 1000} + 20 + \frac{V_B^2}{2g} + \frac{8fLQ^2}{\pi^2 gD^5}$$

$$62.141 = 28.033 + 20 + \frac{0.072 \times 8 \times 600Q^2}{\pi^2 \times 9.81 \times 0.15^5}$$

$$Q = 0.0173 \text{ m}^3/\text{s}$$

10. Boundary Layer Theory

10.1 Explain: (a) Boundary layer thickness (b) Displacement thickness (c) Form Drag

The velocity distribution in a laminar boundary layer is given by $\frac{u_x}{U_\infty} = 3\left(\frac{y}{\delta}\right) - 2\left(\frac{y}{\delta}\right)^2$

where

u_x = velocity in x-direction at a distance y from the boundary

δ = boundary layer thickness

U_∞ = free stream velocity.

Find the ratio of the displacement thickness to the boundary layer thickness.

[15 marks : 1999]

Solution:

(i) **Boundary layer Thickness:** It is defined as the distance from the boundary surface to the point where the velocity reaches 99% of the free stream velocity. It is also known as nominal boundary thickness and is denoted by (δ). In other words, the boundary layer thickness may be considered equal to the distance y from the boundary surface at which $u = 0.99\ U_\infty$

(ii) **Displacement Thickness:** It is defined as the distance by which the boundary would have to be displaced in order to compensate for the reduction in mass flow rate on account of boundary layer growth. It is denoted by (δ^*). Mathematically,

$$\delta^* = \int_0^\delta \left(1 - \frac{u}{U_\infty}\right) dy$$

(iii) **Form drag:** If the surface of a immersed object along which the boundary layer forms is such that it curves away from the flow, there exists a tendency for the flowing fluid to leave the boundary. This phenomenon is known as separation of flow, which usually occurs at higher value of Reynolds number. On the downstream side of the body on account of the separation a region of low pressure is developed which is known as 'wake'. Since on the upstream side of the body the pressure being considerably high, there exists a pressure difference between the upstream and downstream sides. The pressure difference so created results in producing a drag on the body, which is known as form drag or pressure drag.

Now we have the velocity distribution as

$$\frac{u_x}{U_\infty} = 3\left(\frac{y}{\delta}\right) - 2\left(\frac{y}{\delta}\right)^2$$

Displacement thickness is given by

$$\delta^* = \int_0^\delta \left(1 - \frac{u_x}{U_\infty}\right) dy$$

$$\therefore \quad \delta^* = \int_0^\delta \left(1 - \frac{u_x}{U_\infty}\right) dy = \int_0^\delta \left(1 - 3\left(\frac{y}{\delta}\right) + 2\left(\frac{y}{\delta}\right)^2\right) dy$$

$$= \int_0^\delta \left(1 - \frac{3y}{\delta} + \frac{2y^2}{\delta^2}\right) dy = \left(y - \frac{3y^2}{2\delta} + \frac{2y^3}{3\delta^2}\right)_0^\delta = \delta - \frac{3\delta}{2} + \frac{2\delta}{3} = \frac{6\delta - 9\delta + 4\delta}{6}$$

$$\delta^* = \frac{\delta}{6}$$

The ratio of displacement thickness (δ^*) to boundary layer thickness (δ) will be

$$\frac{\delta^*}{\delta} = \frac{\delta/6}{\delta} = \frac{1}{6}$$

13. Impact of Jets and Turbines

13.1 An inward flow reaction turbine works under a head of 30 m and discharge of 10 m^3/s. The speed of the runner is 300 rpm. At inlet tip of runner vane, the peripheral velocity of wheel is $0.9\sqrt{2gH}$ and the radial velocity of flow is $0.3\sqrt{2gH}$, where H is the head on the turbine. If the overall efficiency and the hydraulic efficiency of the turbine are 80% and 90% respectively, determine

(i) the power developed in kW.
(ii) diameter and width of runner at inlet.
(iii) guide blade angle at inlet.
(iv) inlet angle of runner vane.
(v) diameter of runner at outlet.

Assume that the discharge at outlet is radial.

[15 marks : 1999]

Solution:

Given data:

Discharge, $Q = 10\ m^3/s$

Head, $H = 30\ m$

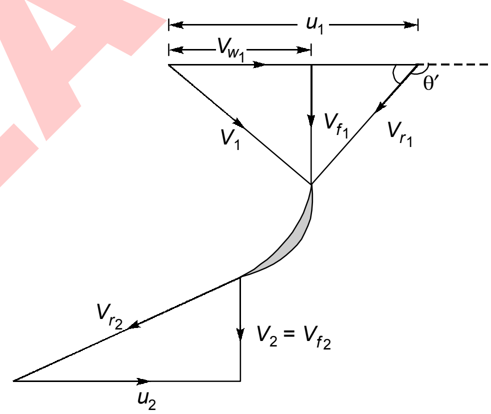
Speed of runner, $N = 300$ rpm

Peripheral velocity at inlet, $u_1 = 0.9\sqrt{2gH}$

Radial velocity of flow at inlet, $V_{f_1} = 0.3\sqrt{2gH}$

Overall efficiency, $\eta_0 = 80\%$

Hydraulic efficiency, $\eta_h = 90\%$



(i) The power developed by the turbine is given by

$$P = \gamma Q H \times \eta_0$$

$$= 9.81 \times 1000 \times 10 \times 30 \times \frac{80}{100} = 2354.40 \text{ kW } \textbf{(Ans.)}$$

(ii) Peripheral velocity at inlet is given by

$$u_1 = 0.9\sqrt{2gH}$$

But, $$u_1 = \frac{\pi D_1 N}{60}$$

where D_1 is the diameter of runner at inlet

$$\Rightarrow \quad 0.9\sqrt{2gH} = \frac{\pi D_1 N}{60}$$

$$\Rightarrow \quad 0.9\sqrt{2 \times 9.81 \times 30} = \frac{\pi \times D_1 \times 300}{60}$$

$$\Rightarrow \quad D_1 = 1.39 \text{ m } \textbf{(Ans.)}$$

Also, $Q = \pi D_1 B_1 V_{f_1}$ where B_1 is the width of runner at inlet

$$\Rightarrow \quad 10 = \pi \times 1.39 \times B_1 \times 0.3\sqrt{2gH}$$

$$\Rightarrow \quad 10 = \pi \times 1.39 \times B_1 \times 0.3\sqrt{2 \times 9.81 \times 30}$$

$$\Rightarrow \quad B_1 = 0.315 \text{ m } \textbf{(Ans.)}$$

(iii) Hydraulic efficiency is given by

$$\eta_h = \frac{u_1 V_{w_1} - u_2 V_{w_2}}{gH}$$

where V_{w_1} is whirl velocity at inlet and V_{w_2} is whirl velocity at outlet

But it is given that discharge at outlet is radial.

$$\therefore \quad V_{w_2} = 0$$

$$\Rightarrow \quad \eta_h = \frac{u_1 V_{w_1} - u_2 \times 0}{gH}$$

$$\Rightarrow \quad \eta_h = \frac{u_1 V_{w_1}}{gH}$$

$$\Rightarrow \quad \frac{90}{100} = \frac{0.9\sqrt{2 \times 9.81 \times 30} \times V_{w_1}}{9.81 \times 30}$$

$$\Rightarrow \quad V_{w_1} = 12.13 \text{ m/s}$$

If α is the guide vane angle at inlet, then

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

$$\Rightarrow \quad \tan\alpha = \frac{0.3\sqrt{2 \times 9.81 \times 30}}{12.13}$$

$$\Rightarrow \quad \alpha = \tan^{-1}(0.6)$$

$$\Rightarrow \quad \alpha = 30.96° \approx 31° \text{ (Ans.)}$$

(iv) If θ is the inlet angle at runner vane, then

$$\tan\theta = \frac{V_{f_1}}{u_1 - V_{w_1}}$$

$$\Rightarrow \quad \tan\theta = \frac{0.3\sqrt{2 \times 9.81 \times 30}}{0.9\sqrt{2 \times 9.81 \times 30} - 12.13}$$

$$\Rightarrow \quad \theta = 36.87° \text{ (Ans.)}$$

(v) Diameter of runner at outlet is given by

$$D_2 = \frac{D_1}{2} = \frac{1.39}{2} = 0.695 \text{ m}$$

15. Open Channel Flow

15.1 A rectangular channel 2.4 m wide carries uniform flow of water at a rate of 7 cum/sec at a depth of 1.5 m. If there is a local rise of 15 cm in the bed, calculate the change in water level. What is the maximum rise in bed that will be permissible so that there is no change in the upstream depth of flow?

[15 marks : 1999]

Solution:

Discharge per unit width, $q = \frac{Q}{B} = \frac{7}{2.4} = 2.92 \text{ m}^3\text{/s/m}$

Velocity, $V_1 = \dfrac{q}{y_1} = \dfrac{2.92}{1.5} = 1.95$ m/s

Froude number, $F_1 = \dfrac{V_1}{\sqrt{gy_1}} = \dfrac{1.95}{\sqrt{9.81 \times 1.5}} = 0.508 < 1$

Hence the upstream flow is subcritical and the local rise will cause a drop in the water surface elevation.

Specific energy, $E_1 = y_1 + \dfrac{V_1^2}{2g} = 1.5 + \dfrac{(1.95)^2}{2 \times 9.81} = 1.694$ m

Specific energy in the downstream may be given as

$$E_2 = E_1 - \Delta Z \text{ rise}$$

$$\Rightarrow \quad E_2 = 1.694 - \frac{15}{100} = 1.544 \text{ m}$$

Critical depth, $y_c = \left(\dfrac{q^2}{g}\right)^{1/3} = \left[\dfrac{(2.92)^2}{9.81}\right]^{1/3} = 0.954$ m

Specific energy at the critical depth will be

$$\therefore \quad E_c = \frac{3}{2}y_c = \frac{3}{2} \times 0.954 = 1.431 \text{ m}$$

The minimum specific energy E_c is less than E_2, the available specific energy at the downstream. Hence $y_2 > y_c$ and the upstream depth y_1 will remain unchanged.

Now $E_2 = y_2 + \dfrac{V_2^2}{2g}$ $\quad \left[V_2 = \dfrac{q}{y_2}\right]$

$$\Rightarrow \quad E_2 = y_2 + \frac{q^2}{2gy_2^2}$$

$$\Rightarrow \quad 1.544 = y_2 + \frac{(2.92)^2}{2 \times 9.81 \times y_2^2} = y_2 + \frac{0.4346}{y_2^2}$$

$$\Rightarrow \quad 1.544\, y_2^2 = y_2^3 + 0.4346$$

$$\Rightarrow \quad y_2^3 - 1.544\, y_2^2 + 0.4346 = 0$$

$$\Rightarrow \quad y_2 = 1.278 \text{ m}$$

and $y_2 = 0.7312$ m $\quad (Fr_2 > 1)$

Thus, the change in water level is 1.5 – 1.278 = 0.222 m fall.

So $y_2 = 1.278$ m

The rise in the bed may be increased till the specific energy at the downstream becomes minimum i.e. $E_2 = E_c$

$$\therefore \quad E_2 = E_1 - \Delta Z_{max}$$

$$\Rightarrow \quad E_c = E_1 - \Delta Z_{max}$$

$$\Rightarrow \quad \Delta Z_{max} = E_1 - E_c = 1.694 - 1.431$$

$$\Rightarrow \quad \Delta Z_{max} = 0.263 \text{ m} = 26.3 \text{ cm}$$

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2 Engineering Hydrology

3. Infiltration, Runoff and Hydrograph

3.1 What are the assumptions and limitations of Sherman's Unit Hydrograph Theory?

[10 marks : 1999]

Solution:

Assumptions of Unit Hydrograph Theory are:

(i) Time invariance: The runoff produced from a given drainage basin due to a given effective rainfall shall always be the same irrespective of the time of its occurrence. In other words, the runoff response of a basin to a given effective rainfall is assumed to be time invariant.

(ii) Linear response: The runoff response of a drainage basin to the excess rainfall is assumed to be linear. It means that if the excess rainfall of *D* cm occurs in a duration of T hour, then the resulting runoff hydrograph will have its ordinates equal to *D* times the ordinates of a unit hydrograph.

Limitations of Unit Hydrograph Theory:

The basic assumptions made in defining a UH were that:

(a) the excess rain should occur uniformly over the entire basin and

(b) that its intensity should be constant during the entire duration.

In actual practice, however these two conditions are never strictly satisfied, since storms do have non-uniform areal distribution, and their intensities also vary during the specified duration.

Even under non-uniform areal distribution of rainfall, unit hydrographs can be used, if the variations in areal extent are consistent within the different storms, which is reasonably so for intense rains falling in smaller catchments. This however imposes a limit on the size of the catchment, since for very large basins, the centre of the storm can vary from storm to storm, and each can give different surface runoff hydrograph under otherwise identical conditions. Unit hydrographs therefore cannot give reliable results for basins exceeding about 5000 sq. km or so. Unit hydrographs are also generally not developed and used for very small basins, having areas lesser than 2 km^2.

Variations in rainfall intensity during the specified duration can also affect the accuracy of unit hydrographs, which can be minimized by choosing a smaller duration which according to Sherman was not to exceed the time of concentration.

4. Floods, Flood Routing & Flood Channel

4.1 The amounts of water flowing from a certain catchment area at the proposed dam site are as tabulated below:

Determine:

(i) The minimum capacity of reservoir if water is to be used to feed the turbines of hydropower plant at an uniform rate and no water is to be spilled over.

(ii) The initial storage required to maintain the uniform demand as above.

[15 marks : 1999]

Month	Inflow ($\times 10^5$ m^3)
Jan	2.83
Feb	4.25
Mar	5.66
Apr	18.40
May	22.64
June	22.64
July	19.81
Aug	8.49
Sep	7.10
Oct	7.10
Nov	5.66
Dec	5.66

Solution:

Total inflow in the year = Summation of the given monthly inflows

$$= (2.83 + 4.25 + 5.66 + 18.40 + 22.64 + 22.64 + 19.81 + 8.49 + 7.10 + 7.10 + 5.66 + 5.66) \times 10^5$$

$$= 130.24 \times 10^5 \text{ m}^3$$

Since the hydropower plant is providing water at a uniform rate, therefore average monthly rate at which water can be withdrawn to avoid any wastage i.e. max. average monthly rate

$$= \frac{130.24 \times 10^5}{12} = 10.8533 \times 10^5 \text{ m}^3$$

Now, to calculate, the storage capacity, the computations are tabulated as follows:

Month	Monthly inflow (x 10^5 m^3)	Monthly outflow (x 10^5 m^3)	Monthly deficit (x 10^5 m^3) col.3 – col.2	Monthly surplus (x 10^5 m^3) col.2 – col.3	Cumulative deficit (x 10^5 m^3)	Cumulative surplus (x 10^5 m^3)	Net water available in reservoir (x 10^5 m^3)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Jan	2.83	10.8533	8.0233	—	—	—	—
Feb	4.25	10.8533	6.6033	—	—	—	—
Mar	5.66	10.8533	5.1933	—	19.82	—	(–) 19.82
Apr	18.40	10.8533	—	7.5467	—	—	—
May	22.64	10.8533	—	11.7867	—	—	—
June	22.64	10.8533	—	11.7867	—	—	—
July	19.81	10.8533	—	8.9567	—	40.08	(+) 20.26
Aug	8.49	10.8533	2.3633	—	—	—	—
Sep	7.10	10.8533	3.7533	—	—	—	—
Oct	7.10	10.8533	3.7533	—	—	—	—
Nov	5.66	10.8533	5.1933	—	—	—	—
Dec	5.66	10.8533	5.1933	—	20.26	—	0

(i) The highest value in col.6 and col.7 is 40.08×10^5 m^3 which represents the minimum storage capacity required to meet the demand without any spilling.

(ii) To compute the minimum initial storage, the net storage left in the reservoir with the above inflows and outflows is calculated. The maximum negative storage here works out to be 19.82×10^5 m^3. In order the reservoir fully meet the demand, there should be no negative storage in it, and in the limiting case, the maximum initial storage should be equal to zero. Hence the minimum initial storage in the reservoir should be 19.82×10^5 m^3.

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3 Water Resources Engineering

1. Water Requirement of Crops

1.1 A sandy loam soil holds water at 140 mm/m depth between field capacity and permanent wilting point. The root depth of the crop is 30 cm and the allowable depletion of water is 35%. The daily water use by the crop is 5 mm/day. The area to be irrigated is 60 ha and water can be delivered at 28 litres per second. The surface irrigation application efficiency is 40%. There are no rainfall and ground water contribution.

Determine,

(i) allowable depletion depth between irrigations
(ii) frequency of irrigation
(iii) net application depth of water
(iv) volume of water required
(v) time to irrigate 4 ha plot [15 marks : 1999]

Solution:

Given,

Moisture storing capacity of soil = 140 mm per m root zone depth
Root depth of crop = 30 cm
Allowable depletion of water = 35%
Consumptive use of crop = 5 mm per day
Area to be irrigated = 60 ha = 60×10^4 m^2
Rate at which water can be delivered = 28 litres per second
Irrigation application efficiency = 40%

(i) Moisture storing capacity of soil = Depth of water stored in root zone

$\therefore$ Depth of water stored in root zone = 140 mm per m root zone depth

$$= 140 \times \frac{30}{100} = 42 \text{ mm}$$

$\therefore$ Allowable depletion depth between irrigation $= \frac{35}{100} \times 42 = 14.7$ mm

(ii)
$$\text{Frequency of irrigation} = \frac{\text{Allowable depletion depth between irrigations}}{\text{Consumptive use of crop}}$$

$$= \frac{14.7}{5} = 2.94 \text{ days}$$

(iii)
$$\text{Irrigation application efficiency} = \frac{\text{Water stored in root zone during irrigation}}{\text{Water delivered to the field}}$$

$$\Rightarrow \quad \frac{40}{100} = \frac{14.7}{\text{Water delivered to the field}}$$

$$\Rightarrow \quad \text{Water delivered to the field} = 36.75 \text{ mm}$$

This water delivered to the field is in terms of depth only i.e. it is the net application depth of water.

(iv) **Volume of water required** = Area to be irrigated × net application depth of water

$$= 60 \times 10^4 \times \frac{36.75}{1000} \times 10^3 \times 10^{-6} = 22.05 \text{ M litres}$$

(v) **Volume of water required to irrigate 4 ha plot**

$$= 4 \times 10^4 \times \frac{36.75}{1000} \times 10^3 \times 10^{-6} = 1.47 \text{ M litres}$$

$$\therefore \quad \text{Time to irrigate 4 ha plot} = \frac{\text{Volume of water required}}{\text{Rate at which water is delivered}}$$

$$= \frac{1.47 \times 10^6}{28} = 52{,}500 \text{ seconds} = 14.583 \text{ hrs.}$$

2. Design of Stable Channels and Canals

2.1 **A river discharges 1000 m³/sec of water at high flood level of *RL*= 103. A weir is constructed for flow diversion with a crest length of 255 m and total length of concrete floors as 40 m. The weir has to sustain the under seepage at a maximum static head of 2.4 m. The silt factor and the safe exit gradient for the river bed material are 1.1 and 1/6 respectively. Determine the depth of cut-off required at the downstream end of the concrete floor. Take the level of downstream concrete floor as *RL* = 100. Check for exit gradient.**

[15 marks : 1999]

Solution:

Given,

High flood discharge, $Q = 1000 \text{ m}^3\text{/sec}$

Length of weir, $L = 255 \text{ m}$

∴ Discharge per unit length of weir is given by

$$q = \frac{Q}{L} = \frac{1000}{255} = 3.92 \text{ m}^3\text{/s}$$

Now for a discharge intensity, q, the normal depth of scour is given by Lacey's equation i.e.

$$R = 1.35\left[\frac{q^2}{f}\right]^{1/3} = 1.35\left[\frac{(3.92)^2}{1.1}\right]^{1/3} = 3.25 \text{ m}$$

Adopting downstream cut off upto 1.5 *R* below the downstream water level

$$\therefore \quad 1.5\,R = 1.5 \times 3.25 = 4.875 \text{ m}$$

Now, upstream water level = Upstream HFL = 103 m (given)

H = maximum static head causing seepage = 2.4 m (given)

∴ Downstream water level = Upstream HFL – H = 103 – 2.4 = 100.6 m

Hence, *RL* of bottom of downstream cut off = 100.6 – 4.875 = 95.725 m

Now *RL* of downstream floor = 100 m (given)

∴ Depth of downstream cut off = 100 – 95.725 = 4.275 m

Hence $$d = 4.275 \text{ m}$$

Check for exit gradient

$$G_E = \frac{H}{d} \times \frac{1}{\pi\sqrt{\lambda}} \qquad \text{where } \lambda = \frac{1+\sqrt{1+\alpha^2}}{2}$$

where $$\alpha = \frac{b}{d} = \frac{\text{Length of weir floor}}{\text{Depth of downstream cutoff}}$$

$\therefore$ $$\alpha = \frac{b}{d} = \frac{40}{4.275} = 9.357$$

$\therefore$ $$\lambda = \frac{1+\sqrt{1+(9.357)^2}}{2} = 5.2$$

$\therefore$ $$G_E = \frac{2.4}{4.275} \times \frac{1}{\pi\sqrt{5.2}}$$

$$= \frac{2.4}{30.64} = \frac{1}{12.77} < \frac{1}{6} \text{ (Safe exit gradient)}$$

Hence, the weir is safe from exit gradient considerations with bottom of downstream cut off at *RL* of 95.725 m.

4 Environmental Engineering

1. Water Demand

1.1 The census record of a particular town shows the population figures as follows:

Years	1960	1970	1980	1990
Population	55,500	63,700	71,300	79,500

Estimate the population for the year 2020 by Decreasing Rate of Growth.

[10 marks : 1999]

Solution:

The decreasing rate of growth method is as follows:

Year	Population	Increase in Population	% Increase in Population (r)	Decrease in% Increase (r')
1960	55,500			
		8,200	14.77	
1970	63,700			2.84
		7,600	11.93	
1980	71,300			0.43
		8,200	11.50	
1990	79,500			

Avg. value of $r' = \dfrac{2.84 + 0.43}{2} = 1.635$

$$r = 11.50$$

$$P_{2000} = P_{1990} + \left(\frac{r - r'}{100}\right) \times P_{1990} = 79500 + \left(\frac{11.50 - 1.635}{100}\right) \times 79500 = 87,343$$

$$P_{2010} = P_{2000} + \left(\frac{r - 2r'}{100}\right) \times P_{2000} = 87343 + \left(\frac{11.50 - 2 \times 1.635}{100}\right) \times 87343 = 94,531$$

$$P_{2020} = P_{2010} + \left(\frac{r - 3r'}{100}\right) \times P_{2010} = 94531 + \left(\frac{11.50 - 3 \times 1.635}{100}\right) \times 94531 = 1,00,765$$

The population for the year 2020 is 1,00,765.

2. Conduits for Transporting Water and Distribution Systems

2.1 A town of 200,000 population is to be supplied water from a source 2500 m away. The lowest water level in the source is 15 m below the water works of the town. The demand of water is estimated as 150 lit/capita/day. A pump of 300 HP is operated for 15 hours. If the maximum demand is 1.5 times the average demand, the velocity of flow through the rising main is 1.3 m/sec and the pump efficiency is 70%, determine

(i) hydraulic gradient and (ii) friction factor for the pipe.

[15 marks : 1999]

Solution:

$$\text{Maximum demand} = \text{Population} \times 1.5 \times \text{average demand}$$
$$= 200{,}000 \times 1.5 \times 150 = 45 \text{ MLD}$$

The pumping is done for 15 hours a day, hence maximum discharge required for pumping,

$$Q = \frac{45 \times 10^6}{10^3 \times 15 \times 60 \times 60} = 0.8333 \text{ m}^3\text{/sec}$$

$$\text{Power of pump} = 300$$

But, we know that $$P\,(HP) = \frac{\gamma_w Q H}{0.7457 \times \eta}$$

$\Rightarrow$ $$300 = \frac{9.81 \times 0.8333 \times H}{0.7457 \times 0.70}$$

$\Rightarrow$ $$H = 19.16 \text{ m}$$

But total head (H) = head difference between source and water works + head loss due to friction in rising main

$\Rightarrow$ $$19.16 = 15 + h_f$$

$\Rightarrow$ $$h_f = 19.16 - 15$$

$\Rightarrow$ $$h_f = 4.16 \text{ m}$$

Now length of pipe, $$L = 2500 \text{ m}$$

(i) Hydraulic gradient $= \dfrac{h_f}{L} = \dfrac{4.16}{2500} = \dfrac{1}{601}$

(ii) We know that $$h_f = \frac{fLV^2}{2gd}$$

Now $$\frac{\pi d^2}{4} = \frac{Q}{V} = \frac{0.8333}{1.3}$$

$\therefore$ $$d = \sqrt{\frac{0.8333}{1.3} \times \frac{4}{\pi}}$$

$\Rightarrow$ $$d = 0.9 \text{ m}$$

$\because$ $$h_f = \frac{fLV^2}{2gd}$$

$\Rightarrow$ $$f = \frac{4.16 \times 2 \times 9.81 \times 0.9}{2500 \times (1.3)^2} = 0.0174$$

4. Quality Control of Water Suppliers

4.1 Define MPN and CI. Explain membrane filtration technique.

[10 marks : 1999]

Solution:

Most Probable Number (MPN):

- It is an index to suggest quantity of coliforms present in a water sample

- In this test, 3 sample sizes, with each size having 5 samples and volume 10 mL, 1 mL and 0.1 mL respectively are tested for positive presumptive test and afterwards confirmatory test.
- This index is calculated by multiple tube fermentation technique.

Coliform Index (CI)

- It is an index to rate purity of water based on coliform bacteria present in water sample.
- It is defined as the reciprocal of the smallest quantity of a sample which would give a positive coliform test.

Membrane Filter Technique

- This technique gives a direct count of coliform bacteria.
- The water sample is filtered through a sterile membrane of pore size 5 to 10 μm.
- Bacteria retained an membrane is then put is contact with nutrients (M-Endo broth) and incubated at 35°C for 20-22 hours.
- Coliform bacteria will produce characteristic colonies which are pink to dark red.
- The final result is then reported in number of organism per 100 mL of water.

5. Water Treatment

5.1 A rapid sand filter is proposed for water supply treatment plant for a town with a population of 75000. The average rate of water supply is 150 litre/capita/day and rate of filtration is to be 100 litre/min/sq.m. Design:
(i) Size and number of filter beds
(ii) Manifold lateral under drainage system
(iii) Wash water discharge required, if the rate of washing is 45 cm rise/min.

[18 marks : 1999]

Solution:

(i) Size and number of filter beds

The maximum water demand per day

$$= \text{Population} \times \text{Maximum daily rate of water supply}$$
$$= \text{Population} \times 1.8 \times \text{Average daily rate of water supply}$$
$$= 75000 \times 1.8 \times 150 = 20.25 \times 10^6 \text{ litres per day} = 20.25 \text{ MLD}$$

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

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

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

$$\text{Filtered water required per hour} = \frac{21.09}{23.5} \times 24 = 21.538 \text{ MLD}$$

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

$$\text{Given rate of filtration} = 100 \text{ L/min/m}^2$$

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

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

$$\Rightarrow \quad \text{Area of filter} = 149.5 \text{ m}^2$$

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

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

Thus, providing 7 filter units = 6 operational + 1 standby

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

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

Now, $L \times B = 25$

$\Rightarrow$ $1.5B \times B = 25$

$\Rightarrow$ $B = 4.1$ m

and $L = 1.5 \times B = 1.5 \times 4.1 = 6.15$ m

Take length of filter (L) say 6.3 m.

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

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

(ii) Design of manifold lateral under drainage system

Let a manifold and lateral system be provided below the filter bed, for receiving the filtered water and to allow backwashing for cleaning the filter. This 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.

$$\therefore \quad \text{Total area of perforations} = 6.3 \times 4.2 \times \frac{0.2}{100} = 0.05292 \text{ m}^2$$

Now assuming the area of each lateral = four times the area of perforations in it

$$\therefore \quad \text{Total area of laterals} = 4 \times 0.05292 = 0.21168 \text{ m}^2$$

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

$$\text{Area of manifold} = 2 \times 0.21168 = 0.4234 \text{ m}^2$$

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

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

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

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. widthwise) 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

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

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

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

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

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

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

$$= \text{Total area of perforations}$$

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

$$\Rightarrow \text{Total number of perforations in all 84 laterals} = 1871.66 \approx 1872$$

$$\therefore \text{ Number of perforations in each lateral} = \frac{1872}{84} = 22.28 \text{ say } 23$$

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

$$\text{Now area of each lateral} = 4 \times \text{area of perforations in it}$$

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

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

Hence, use 84 laterals each of 5.75 cm dia @ 15 cm 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

$$\text{Given rate of washing of filter} = 45 \text{ cm rise/min}$$

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

$$\therefore \text{ Velocity of flow in the lateral for wash water} = \frac{0.198}{84 \times \frac{\pi}{4} \times \left(\frac{5.75}{100}\right)^2} = 0.91 \text{ m/sec}$$

$$\text{Similarly, Velocity of flow in the manifold} = \frac{\text{Discharge}}{\text{Area}} = \frac{0.198}{\frac{\pi}{4} \times (0.75)^2} = 0.448 \text{ m/sec}$$

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

9. Treatment of Sewage

9.1 **Compare 'Trickling Filter' and 'Recirculation with Bio-filters'.**
Describe with a neat sketch the working of a 'Sludge digestion tank with floating cover'.

[7 + 8 = 15 marks : 1999]

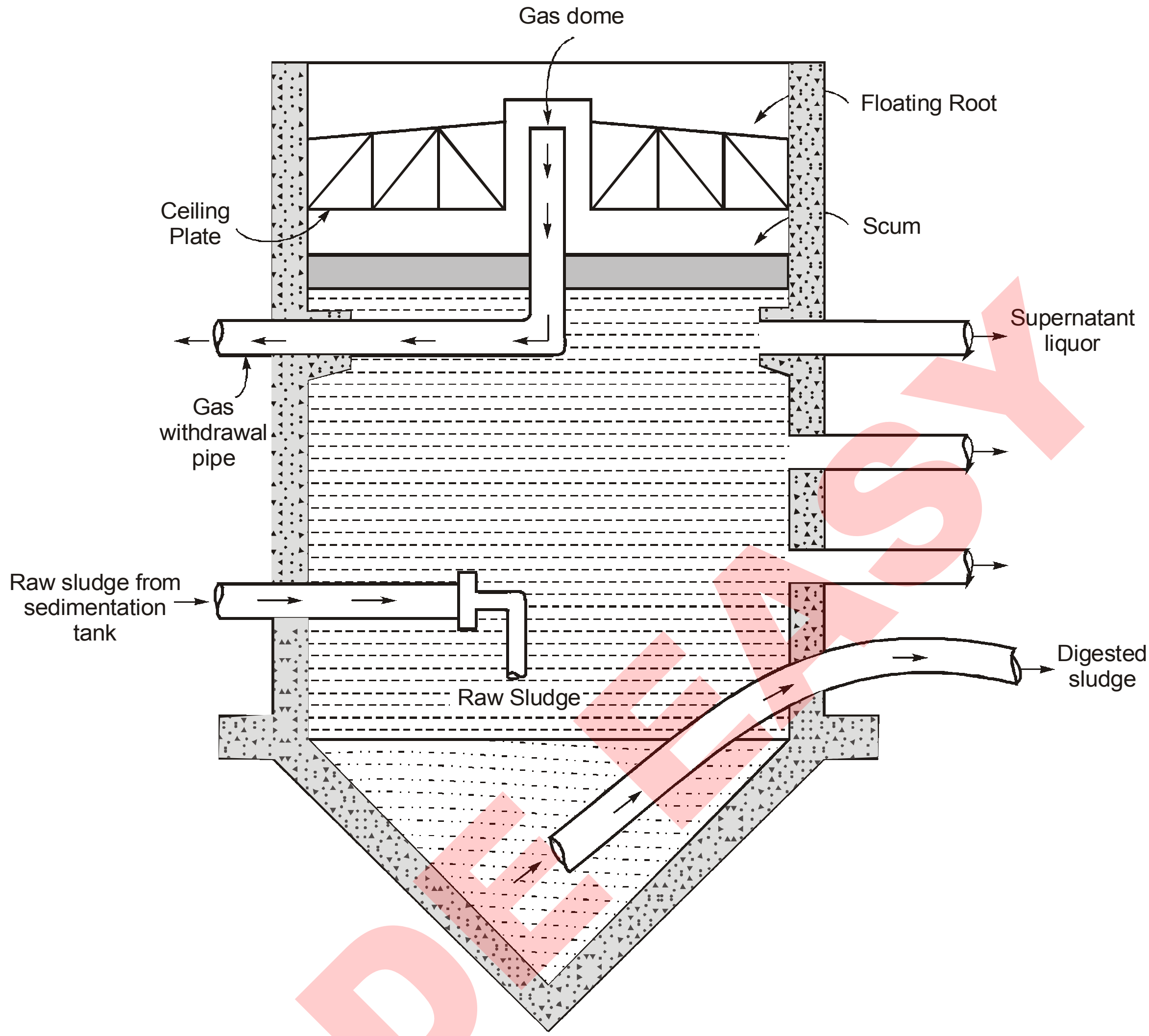
Solution:

Trickling filters are classified in two types:

(i) Standard rate trickling filter

(ii) High rate trickling filter

The construction details of these filters are same, except that provision is made in high rate trickling filters for recirculation of sewage through the filter, by pumping a part of the filtered effluent to the primary sedimentation tank, and repassing through it and the filter.



The high rate trickling filters make it possible to pass sewage at greater loadings, thus require lesser space and lesser filter media. The value of hydraulic loading for conventional (standard rate) trickling filters may vary between 22 to 44 M litre/ha/day. The hydraulic loading can still be increased to about 110 to 330 M litre/ha/day in high rate trickling filters. The value of organic loading for conventional filters may vary between 900 to 2200 kg of BOD_5 per ha-m. This organic loading value can be further increased to about 6000 to 18000 kg of BOD_5 per ha-m in high rate trickling filters. The depth of high rate trickling filters varies between 1.2 to 1.8 m.

Bio filters are essentially high rate filters, but comparatively shallower than trickling filters. The depth varies between 1.2 m to 1.5 m (the depth is kept less on the consideration that the main action of treatment is involved in the upper surface layer of the filter). The filter utilizes recirculation of a portion of the filter effluent to the PST for a second passage through the filter. If additional treatment is necessary to lower the BOD content in the effluent, such as in the case of strong sewages, a second stage filter may be provided. Also the quality of the final effluent can also be changed by altering the loading rate and the recirculation ratio. These filters are capable of giving any degree of treatment by appropriate selection of flow diagram and recirculation ratio. Organic loading normally ranges between 9000 to 11000 kg of BOD_5 per ha-m per day. The total hydraulic loading may range between 110 to 330 million litres per day per hectare.

A sludge digestion tank consists of a circular RCC tank with hoppered bottom and having a floating roof over its top. The raw sludge is pumped into the tank, and when the tank is first put into operation, it is seeded with the digested sludge from another tank. A screw pump with an arrangement for circulating the sludge from bottom to top of the tank or vice versa is commonly used for stirring the sludge. Sometimes, power driven mechanical devices may be used for stirring the sludge. A typical sludge digestion tank is shown above.

In cold countries, the tank may have to be provided with heating coils through which hot water is circulated in order that the temperature inside the tank is maintained at optimum digestion temperature level.

The gases of decomposition (chiefly methane and carbon dioxide) are collected in a gas dome. The digested sludge which settles down to the hoppered bottom of the tank is removed under hydrostatic pressure, periodically once a weak or so. The supernatant liquor lying between the sludge and the scum is removed at suitable elevations, through a number of withdrawal pipes. The supernatant liquor, being higher in BOD and suspended solids content is sent back for treatment along with the raw sewage in the treatment plant. The scum formed at the top surface of the supernatant liquor is broken by the recirculating flow or through the mechanical rockers called scum breakers.

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5 Soil Mechanics and Foundation Engineering

1. Properties of Soils

1.1 The void ratio and specific gravity of a sample of clay are 0.73 and 2.7 respectively. If the voids are 92% saturated, find the bulk density, the dry density and the water content.
What would be the water content for complete saturation, the void ratio remaining the same?
[10 marks : 1999]

Solution:

Given data: $e = 0.73$, $G = 2.7$, $S = 92\%$

We know that $Se = Gw$

$$\Rightarrow \quad w = \frac{Se}{G}$$

$$\Rightarrow \quad w = \frac{0.92 \times 0.73}{2.7}$$

$$\Rightarrow \quad w = 0.2487 \text{ or } 24.87\%$$

Now,
$$\gamma = \frac{(G + Se)\gamma_w}{1 + e}$$

$$\Rightarrow \quad \gamma = \left[\frac{2.7 + (0.92 \times 0.73)}{1 + 0.73}\right] \times 9.81$$

$$\Rightarrow \quad \gamma = 19.12 \text{ kN/m}^3$$

But Bulk density,
$$\rho = \frac{\gamma}{g} = \frac{19.12 \times 10^3}{9.81} = 1949.0316 \text{ kg/m}^3$$

Dry density,
$$\rho_d = \frac{\rho}{1 + w} = \frac{1949.0316}{1 + 0.2487} = 1560.85 \text{ kg/m}^3$$

When $S = 100\%$

then $Se = Gw$

$$\Rightarrow \quad w = \frac{Se}{G} = \frac{1 \times 0.73}{2.7}$$

$$\Rightarrow \quad w = 0.2703 \text{ or } 27.03\%$$

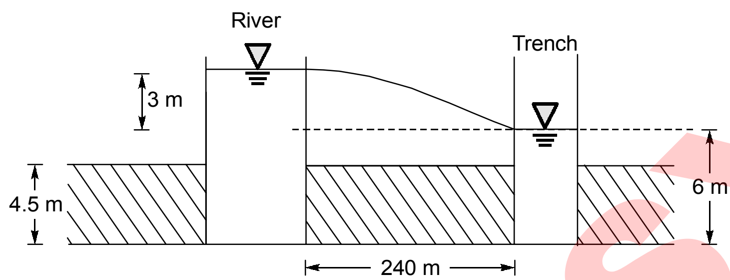
2. Effective Stresses, Permeability and Seepage Analysis

2.1 A trench is to be excavated 240 m away from a river and in a confined aquifer 4.5 m thick. The trench has to run parallel to the river and is to be 300 m long. Water level in the trench is to be maintained 6 m above the lower confining layer and 3 m below the water level in the river. Determine the rate at which water should be pumped from the trench, if the hydraulic conductivity of the aquifer is 4.5 m/day. Assume there is no contribution to flow from the land side of the trench. State the condition when the assumption can be valid.

[12 marks : 1999]

Solution:

As there is no contribution of flow from land side of the trench, the hydraulic gradient will vary linearly between the river and trench, otherwise the flow could have been radial.



Given data:

Hydraulic conductivity (k) = 4.5 m/day, Length of trench (L) = 300 m, Thickness of aquifer (B) = 4.5 m

Hydraulic gradient between river and trench, $i = \dfrac{(6+3)-6}{240} = \dfrac{3}{240}$

Let the rate of discharge be Q, then

$$Q = kiA = kiLB = 4.5 \times \frac{3}{240} \times 300 \times 4.5 = 75.94 \text{ m}^3\text{/day}$$

where A is area through which water flows in the trench.

The above given assumption can only be valid when the flow is laminar i.e. **Darcy's law** is valid. It also means that the soil to be dewatered is homogeneous and isotropic.

7. Retaining Wall/Earth Pressure Theories

7.1 A retaining wall 6 m high supports earth with its face vertical. The earth is cohesionless with particle specific gravity 2.69, angle of internal friction 35° and porosity 40.5%. The earth surface is horizontal and level with the top of the wall. Determine the earth thrust and its line of action on the wall if the earth is water logged to level 2.5 m below the top surface. Neglect wall friction. Draw the pressure diagrams.

[15 marks : 1999]

Solution:

Given data: Height of retaining wall (H) = 6 m, Specific gravity of soil particles (G) = 2.69

Angle of internal friction (ϕ) = 35°, Porosity (n) = 40.5% = 0.405

Depth of water table below top surface = 2.5 m

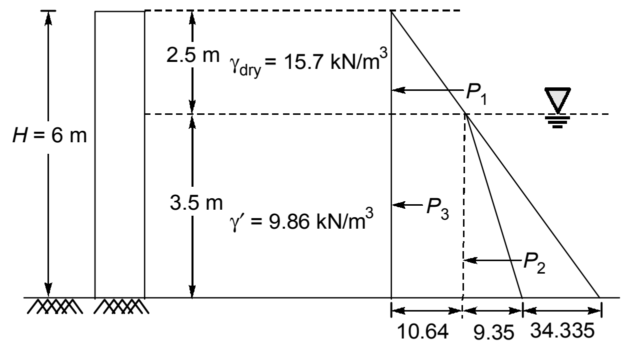
We know that: $$n = \frac{e}{1+e}$$

$\therefore$ $$e = \frac{n}{1-n} = \frac{0.405}{1-0.405} = 0.681$$

$$\gamma_{sat} = \left(\frac{G+e}{1+e}\right)\gamma_w = \left(\frac{2.69+0.681}{1+0.681}\right) \times 9.81 = 19.67 \text{ kN/m}^3$$

$$\gamma_{dry} = \frac{G\gamma_w}{1+e} = \frac{2.69 \times 9.81}{1+0.681} = 15.7 \text{ kN/m}^3$$

$$\gamma' = \gamma_{sat} - \gamma_w = 19.67 - 9.81 = 9.86 \text{ kN/m}^3.$$



$$K_a = \frac{1-\sin\phi}{1+\sin\phi} = \frac{1-\sin 35°}{1+\sin 35°} = 0.271$$

For $z = 2.5$ m

$$P_v = K_a\gamma z - 2c\sqrt{K_a} \qquad [\because \text{for cohesionless soil } c = 0]$$

$$P_v = K_a\gamma z = 0.271 \times 15.7 \times 2.5 = 10.64 \text{ kN/m}^2$$

$\therefore$ Total active thrust for z = 2.5 m is given by

$$P_1 = \frac{1}{2} \times P_v \times 2.5$$

$$= \frac{1}{2} \times 10.64 \times 2.5 = 13.3 \text{ kN/m}$$

For $z = 3.5$ m

$$P_{v_1} = K_a\gamma' z = 0.271 \times 9.86 \times 3.5 = 9.35 \text{ kN/m}^2$$

$$P_{v_2} = \gamma_w z = 9.81 \times 3.5 = 34.335 \text{ kN/m}^2$$

$$P_{v_1} + P_{v_2} = 9.35 + 34.335 = 43.685 \text{ kN/m}^2$$

$\therefore$ Total active thrust for z =3.5 m is given by

$$P_2 + P_3 = \frac{1}{2} \times (43.685) \times 3.5 + 10.64 \times 3.5 = 76.45 + 37.24 = 113.69 \text{ kN/m}$$

$\therefore$ Total active thrust on the retaining wall = $P_1 + P_2 + P_3 = 13.3 + 76.45 + 37.24 = 126.99$ kN/m

$$\text{Location of total thrust from base} = \frac{13.3 \times \left(3.5 + \frac{2.5}{3}\right) + 37.24 \times \left(\frac{3.5}{2}\right) + 76.45 \times \left(\frac{3.5}{3}\right)}{126.99} = 1.67 \text{ m}$$

9. Shallow Foundation and Bearing Capacity

9.1 A concrete strip footing rectangular in cross-section is located at ground level and extends 1.2 m below the ground level. It carries UDL of 15000 kg/m. The soil profile consists of homogeneous clay 6 m thick overlying rock. The clay properties are as under:

Saturated unit bulk weight = 1750 kg/m^3

Shear strength (undrained) = 8500 kg/m^2

Compressibility = 1×10^{-4} m^2/100 kg

Determine,

(i) width of footing for factor of safety, $F = 2$

(ii) ultimate consolidation settlement

Assume bulk unit weight of concrete = 2500 kg/m^3

Neglect the spread of load beneath the footing and any side cohesion on the foundation.

[15 marks : 1999]

Solution:

(i) As per **Terzaghi's bearing capacity equation** for strip footing on clayey soil,

$$q_u = 5.7\, c_u + \gamma D_f$$

Now, the net ultimate bearing capacity of the soil is

$$q_{nu} = 5.7\, c_u$$

Net safe bearing capacity,

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

and thus safe bearing capacity,

$$q_s = q_{ns} + \gamma D_f = \frac{q_{nu}}{F} + \gamma D_f$$

Given $\gamma = 1750$ kg/m^3; $c_u = 8500$ kg/m^2; $m_v = 1 \times 10^{-4}$ m^2/100 kg

Factor of safety, $F = 2$

Unit weight of concrete = 2500 kg/m^3

Depth of footing, $D_f = 1.2$ m

Now UDL = 15000 kg/m

Self weight of concrete = $1.2 \times B \times 2500 = 3000\,B$ kg/m

Total load = $15000 + 3000\,B$

Thus, total load carried by the footing should be equal to the safe bearing capacity of soil

$$\therefore \quad 15000 + 3000\,B = \left(\frac{q_{nu}}{F} + \gamma_{sat} D_f\right) \times B$$

$$\Rightarrow \quad 15000 + 3000\,B = \left(\frac{5.7 c_u}{F} + \gamma_{sat} D_f\right) B$$

$$\Rightarrow \quad 15000 + 3000\,B = \left[\frac{5.7 \times 8500}{2} + (1750 \times 1.2)\right] \times B$$

$$\Rightarrow \quad B = 0.643 \text{ m}$$

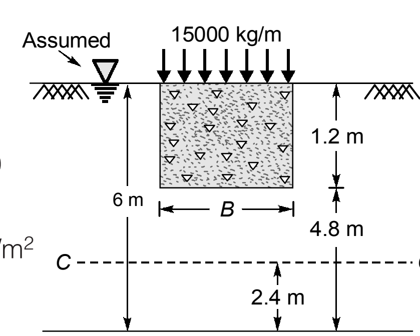
(ii) Initial effective overburden pressure at the level of C-C,

$$\bar{\sigma}_1 = (1750 - 1000) \quad 3.6 = 2700 \text{ kg/m}^2$$

Final effective overburden pressure at the level C-C,

$$\bar{\sigma}_2 = 1.2 \times 2500 + 2.4 \times 1750 + \frac{15000}{B} - 3.6 \times 1000$$

$$= 3000 + 4200 + \frac{15000}{0.643} - 3600 = 26928.14 \text{ kg/m}^2$$



$\therefore$ Change in effective overburden pressure,

$$\Delta\bar{\sigma} = \bar{\sigma}_2 - \bar{\sigma}_1 = 26928.14 - 2700 = 24228.15 \text{ kg/m}^2$$

Ultimate consolidation settlement,

$$\Delta H = m_v H_0 \Delta\bar{\sigma}$$

$$= \frac{1\times 10^{-4}}{100} \times 4.8 \times 24228.15 = 0.1163 \text{ m} = 9116.3 \text{ mm}$$

11. Soil Stablization and Soil Exploration

11.1 Explain with a neat sketch the Resistivity Method of Soil Exploration.

[10 marks : 1999]

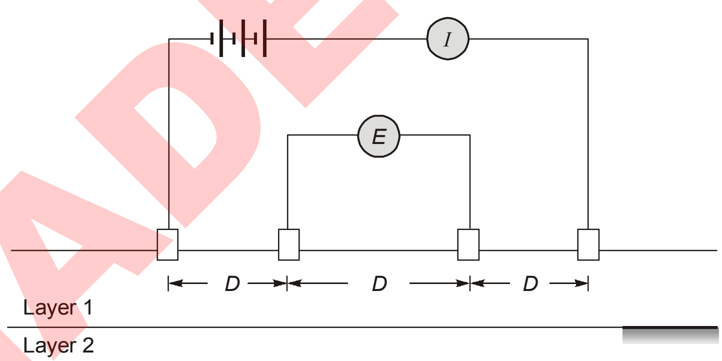
Solution:

The electrical resistivity method is based on the measurement and recording of changes in the mean resistivity or apparent specific resistance of various soils. The resistivity, ρ is usually defined as the resistance between the opposite faces of a unit cube of the material. Significant variations in resistivity can be detected between different types of soil strata or above and below the water table or between unfissured rocks and soils or between voids and soil / rock.

The test is carried out by driving four metal spikes to serve as electrodes into the ground along a straight line at equal distances. Current (I) from a battery flow through the soil between the two outer electrodes, producing an electrical field within the soil. The potential difference E between the two inner electrodes is then measured. The apparent resistivity, ρ is given by the equation

$$\rho = \frac{2\pi DE}{I}$$

It is customary to express D in cm, E in volts, I in amperes and ρ in ohm-cm. The arrangement of the devices in the method is shown in the figure below.



The apparent resistivity is the weighted average of true resistivity upto a depth D in a large volume of soil, the soil close to the surface being more heavily weighted than the soil at greater depths. If a stratum of low resistivity overlies a stratum of higher resistivity, the current is forced to flow closer to the ground surface, resulting in a higher voltage drop and hence a higher value of apparent resistivity. It would be the opposite if a stratum of high resistivity lies above a stratum of low resistivity. The method of sounding is used when the variation of resistivity with depth is required. The method of profiling is used when the lateral variation of soil strata is to be investigated. The electrical resistivity method is not as reliable as the seismic method.

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7

Highway Engineering

1. Highway Geometric Design

1.1 A highway is provided with a horizontal curve of radius 300 m in certain locality. Calculate the super elevation needed to maintain the design speed of 90 kmph. Also calculate the maximum allowable speed, if the super elevation is limited to 0.07 and safe limit of transverse coefficient of friction is 0.15. Comment on the result.

Explain the terms:

(i) Ruling Minimum Radius and

(ii) Absolute Minimum Radius of horizontal curve.

[15 marks : 1999]

Solution:

Given data: $V = 90$ kmph; $R = 300$ m; $e = 0.07$; $f = 0.15$

We know that super elevation, $e = \dfrac{V^2}{127\,R}$

where V is in kmph and R is radius of curve in m. Calculation of super elevation for mixed traffic condition is a complex problem, as different vehicles ply on the road with a wide range of speeds. To super elevate the pavement upto the maximum limit so as to counteract the centrifugal force fully, neglecting the lateral friction is safer for fast moving vehicles. But for slow moving vehicles this may be quite inconvenient. On the contrary to provide lower value of super elevation, thus relying more on the lateral friction would be unsafe for fast moving vehicles. As a compromise and from practical considerations it is suggested that the super elevation should be provided to fully counteract the centrifugal force due to 75% of the design speed, (by neglecting lateral friction developed) and limiting the maximum superelevation to 0.07.

$$\therefore \quad e = \frac{(0.75\,V)^2}{127\,R} = \frac{V^2}{225\,R} = \frac{90^2}{225\times 300} = 0.12 > 0.07$$

Hence a superelevation, $e = 0.07$ is provided.

Check: $$f = \frac{V^2}{127\,R} - 0.07 = \frac{90^2}{127\times 300} - 0.07 = 0.143 < 0.15 \text{ (Hence Safe)}$$

Now, we know that $$e + f = \frac{V_a^2}{127\,R}$$

$$\therefore \quad \frac{V_a^2}{127\times 300} = 0.07 + 0.15$$

$$\Rightarrow \quad V_a^2 = 8382$$

$$\Rightarrow \quad V_a = 91.55 \text{ kmph}$$

If the allowable speed as calculated above is higher than design speed, then design is adequate. Thus, the maximum allowable speed is 91.55 kmph.

(i) **Ruling minimum radius:** For a certain speed of a vehicle the centrifugal force is dependent on the radius of the horizontal curve. To keep the centrifugal ratio within a low limit, the radius of the curve is kept correspondingly high. The centrifugal force which is counteracted by the superelevation and lateral friction is given by

$$e + f = \frac{V^2}{127\,R}$$

$$\Rightarrow \qquad R = \frac{V^2}{127\,(e+f)}$$

This radius is called ruling minimum radius of curve for the ruling design speed of the road.

(ii) **Absolute minimum radius:** When the minimum design speed is adopted instead of ruling design speed, the corresponding radius is called absolute minimum radius. This length of radius must be provided to counteract the centrifugal forces.

3. Pavement Design

3.1 PBT conducted with 30 cm diameter plate on a soil subgrade yielded a pressure of 1 kg /cm^2 at 5 mm deflection. The test carried out over 18 cm base course yielded a pressure 5 kg/cm^2 at 5 mm deflection. Design the pavement section for wheel load of 4100 kg with a tyre pressure of 6 kg/cm^2 and allowable deflection of 5 mm. Use Burmister's method.

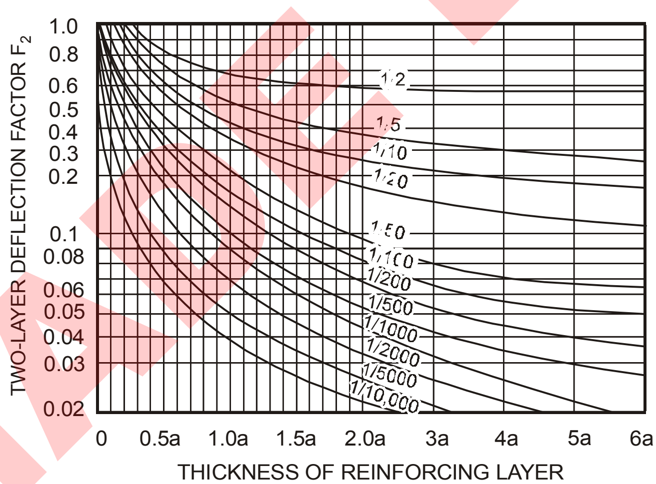


Figure : Relationship of F_2 and h in a Two-layer System (Burmister Method)

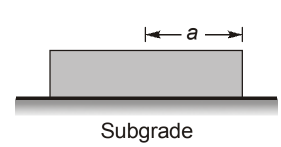
[15 marks : 1999]

Solution:

Case (i) Plate load test over soil subgrade

Diameter of plate = 30 cm

$\therefore$ Radius of contact area, $a = \frac{30}{2} = 15$ cm



Δ = 5 mm = 0.5 cm

p = 1 kg/cm^2

This is a single layer system, as per **Burmister method**

$\therefore \qquad h = 0$ and $F_2 = 1.0$

$$\therefore \quad \text{For rigid plate,} \qquad \Delta = \frac{1.18\,pa}{E_s} \times F_2$$

$$\Rightarrow \qquad 0.5 = \frac{1.18 \times 1 \times 15}{E_s} \times 1$$

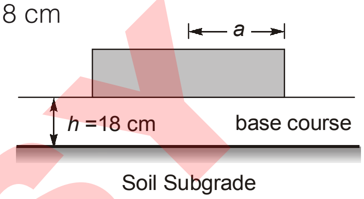
$$\Rightarrow \qquad E_s = 35.4 \text{ kg/cm}^2$$

Case (ii) For plate load test over 18 cm base course

$$\Delta = 0.5 \text{ cm}, \; p = 5 \text{ kg/cm}^2, \; h = 18 \text{ cm}$$

$$E_s = 35.4 \text{ kg/cm}^2$$

$$a = 15 \text{ cm}$$



$$\therefore \quad \text{For a rigid plate,} \qquad \Delta = \frac{1.18\,pa}{E_s}\, F_2$$

$$\Rightarrow \qquad 0.5 = \frac{1.18 \times 5 \times 15}{35.4} \times F_2$$

$$\Rightarrow \qquad F_2 = 0.2$$

and

$$\frac{h}{a} = \frac{18}{15} = 1.2$$

From the graph in given problem, for a value of $F_2 = 0.2$ and $\frac{h}{a} = 1.2$ or $h = 1.2\,a$, the $\frac{E_s}{E_p}$ value lies between $\frac{1}{50}$ and $\frac{1}{100}$, but it is more close to $\frac{1}{100}$. Say $\frac{E_s}{E_p} = \frac{1}{90}$

$$\therefore \qquad \frac{E_s}{E_p} = \frac{1}{90}$$

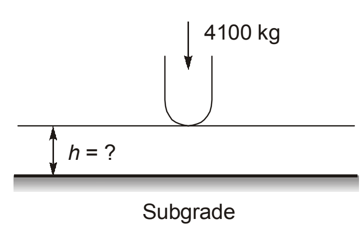
$$\Rightarrow \qquad E_p = 90\,E_s = 90 \times 35.4 = 3186 \text{ kg/cm}^2$$

Design of Pavement:

$$P = 4100 \text{ kg}; \quad p = 6 \text{ kg/cm}^2$$

$$\therefore \qquad \text{Contact area} = \frac{P}{p} = \frac{4100}{6} = 683.33 \text{ cm}^2$$

$$\therefore \quad \text{Radius of contact area,} \qquad a = \sqrt{\frac{683.33}{\pi}} = 14.75 \text{ cm}$$



$$\Delta = 0.5 \text{ cm}$$

$$\frac{E_s}{E_p} = \frac{1}{90}$$

$$\therefore \quad \text{For a flexible case,} \qquad \Delta = \frac{1.5\,pa}{E_s} \times F_2$$

$$\Rightarrow \qquad 0.5 = \frac{1.5 \times 6 \times 14.75}{35.4} \times F_2$$

$$\Rightarrow \qquad F_2 = 0.133$$

$\therefore$ For a F_2 value of 0.133 and $\frac{E_s}{E_p} = \frac{1}{90}$, the value of h in terms of a can be found easily from the graph

$\Rightarrow$ $$h = 2.3\,a = 2.3 \times 14.75 = 33.93 \text{ cm}$$

Thus the pavement thickness = 33.93 cm = 34 cm

■■■■

8 Railway Engineering

3. Geometric Design of the Track

3.1 What special measures and precautions are required in geometric features of a modern railway track?

[10 marks : 1999]

Solution:

Special measures required in geometric features of a modern railway track are:

(i) **Gauge of track:** A wider, correct and uniform gauge is desirable for attaining high speeds. Always wider is the gauge better would be the stability and hence higher the speed. The gauge over crossings is kept 1 mm tight to improve running. The gauge should be laid and maintained tight by 3 mm on straight and on curves upto 4 mm for high speed routes.

(ii) **Alignment of track:** For high speed tracks, both horizontal and vertical alignment should be perfect. For horizontal alignment, the flat curves along with transition curves and adequate super elevation should be provided. The sharp curves in tracks should either be eliminated by diverting the track or replacing them by flat curves. For vertical alignment, the steep gradients should be eliminated either by replacing them by gentle gradients or again by diverting the track or tunnelling through hills. Adequate cant deficiency should be provided.

(iii) **Other geometric elements:** Other geometric elements like cant, radius of the curve, degree of the curve, length of transition curve, etc., for high speeds should be provided.

(iv) **Track centres:** The centre to centre distance between tracks in station yards is kept more for high speeds tracks as compared to a section between stations. This offers several advantages such as safety of staff, elimination of problems of loading gauge and safety margin and possibility of allowing trains at high speeds over crossovers to greater intermediate track length.

Precautions required are as follows:

(i) Maintenance of track shall be within permanent tolerances.

(ii) Rails shall be ultrasonically tested at least once in two years for detecting cracks or flaws.

(iii) All critical points, such as points and crossings, vicinity of water columns, old rails over 10 years, portions with past history of rail breakage etc., shall be tested more frequently.

(iv) After one year of the train introduction, track irregularities shall be recorded once in two months and recording by oscillograph car atleast once in four months.

(v) There must not be less than (M + 3) effective sleepers for 13 m rail length (BG track) on the entire rate where M is length of rail in metres.

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9 Airport, Dock, Harbour and Tunnelling Engineering

2. Dock and Harbour

2.1 Explain the function and working of a Floating Dry Dock with a neat sketch. [10 marks : 1999]

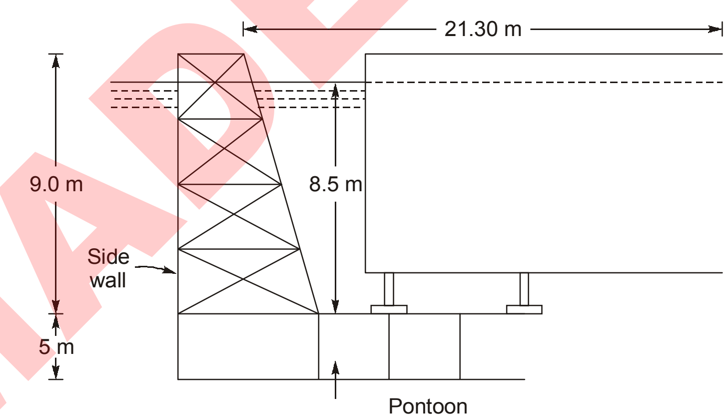
Solution:

Floating dry dock may be defined as 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 hollow structure of steel or reinforced cement concrete consisting of 2 side walls and a floor, with the ends open. To receive a ship, the structure is sunk to required depth by ballasting its interior chambers with water, the ship is then floated into position and berthed. The dock is raised bodily with the berthed ship by unballasting the chambers by pumping out the water.

There are three important types of floating docks that have been developed viz. Rigid type floating docks, self docking type floating docks and self docking offshore type floating docks.

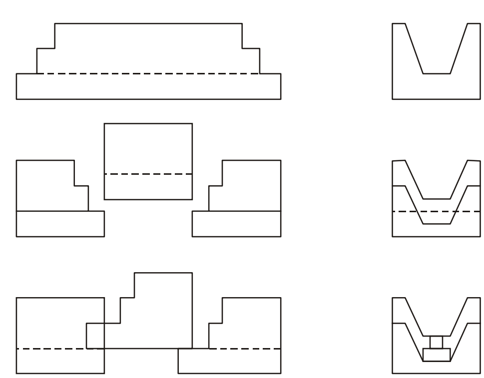
(i) Rigid type floating dry docks:

In this type, the side walls are rigidly fixed to the pontoon or bottom section. The floor portion is divided into a number of chambers, so as to assist in canting the dock if necessary to berth damaged ships, by partial unballasting of the chambers.



(ii) Self docking type floating docks:

Self docking refers to a type of floating dock, which is divided into sections longitudinally, and one of which is capable of being lifted and docked on the remainder of the dock for purposes of cleaning, painting or repairing. A typical self docking dry dock known as bolted sectional type is shown below:

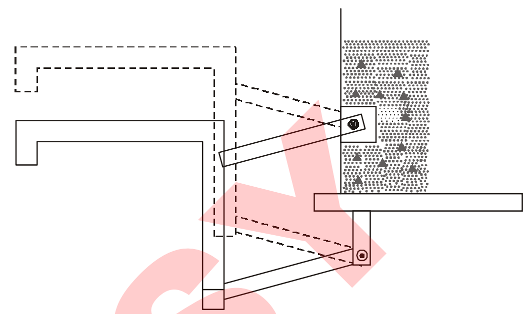


Firstly the whole dock (having minimum three sections) is shown assembled; secondly the centre section is shown detached and about to be docked on the two end sections and thirdly an end section is seen being docked on the other two.

This type is usually constructed in three equal sections, the two end sections having stepped ends to form landings during self docking. It combines the advantages of strength of the rigid type with self docking facility.

(iii) **Self docking offshore type floating docks:**

The offshore type floating dock has no side wall on the water side and has an L-shaped cross section. The side wall is connected to the shore by hinged parallel body capable of lifting or lowering the dock. The ship to be docked, could be brought on to the dock from either end or side ways.



The dock is longitudinally made into two sections, so as to dock one half on the other. The dock and the self docking operation are illustrated in the figure. This type of dock is convenient in a sheltered situation and adoptable for being attached to river quays.

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