ESE GATE PSUs

State Engg. Exams

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Workbook 2025

Detailed Explanations of Try Yourself *Questions*

Civil Engineering

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1 Water Demand

Detailed Explanation of Try Yourself Questions

T1 : Solution

(i) Arithmetic increase method

Year	Population (thousands)	Increase
2010	26	
		3
2011	29	
		6
2012	35	
		8
2013	43	
		4
2014	47	

$\bar{x} = 5.25$

$\therefore$ Population in 2020

$$P_{2020} = P_{2014} + 6\bar{x}$$
$$= 47000 + 6\,(5250)$$
$$= 78500$$

(ii) Geometric increase method

Year	Population (thousands)	Increase	Growth rate
2010	26		
		3	$3/26 \times 100 = 11.5$
2011	29		
		6	$6/29 \times 100 = 20.68$
2012	35		
		8	$8/35 \times 100 = 22.8$
2013	43		
		4	$4/43 \times 100 = 9.3$
2014	47		

$r = 16.07\%$

Population in 2020,

$$P_{2020} = P_{2014} + (1 + r)^6$$
$$= 47 + (1 + 0.1607)^6 = 114.5 \text{ thousands}$$

(iii) Incremental increase method

Year	Population (thousands)	Increase	Increase in increase
2010	26		
		3	
2011	29		3
		6	
2012	35		2
		8	
2013	43		–4
		4	
2014	47		

y = 0.33

Population in 2010, $P_{2020} = P_{2014} + 6\bar{x} + \frac{6(6+1)}{2}y$

$= 47 + 6x\,(5.25) + 21\,(0.33)$

$= 85.43$ thousands

T2 : Solution

Given,

$P_0 = 40000,\ t = 0$

$P_1 = 160000,\ t_1 = 20$

$P_2 = 280000,\ t_2 = 40$

(*i*) **By logistic curve method,**

Saturation population, $P_s = \dfrac{2P_0P_1P_2 - P_1^2(P_0+P_2)}{P_0P_2 - P_1^2}$

Substituting the value of P_0 , P_1 and P_2 in above equation, we get

$$P_s = \frac{2\times 40000\times 160000\times 280000-(160000)^2(40000+280000)}{40000\times 280000-(160000)^2}$$

$= 320000$

(*ii*) **Using logistic curve method,**

$$P = \frac{P_s}{1+m\log_e^{-1}(nt)}$$

where, $m = \dfrac{P_s - P_0}{P_0} = \dfrac{320000-40000}{40000} = 7$

and $n = \dfrac{2.3}{t_1}\log_{10}\left[\dfrac{P_0(P_s-P_1)}{P_1(P_s-P_0)}\right]$

$$= \frac{2.3}{20}\log_{10}\left[\frac{40000(320000-160000)}{160000(320000-40000)}\right] = -0.097$$

$$P = \frac{320000}{1+7\log_e^{-1}(-0.097\,t)}$$

(*iii*) When t = 55 years, then

$$P = \frac{320000}{1+7\log_e^{-1}(-0.097\times 55)} = \frac{320000}{1+7\log_e^{-1}(-5.335)}$$

Let, $x = \log_e^{-1}(-5.335)$

$\therefore$ $$P = \frac{320000}{1+7x}$$

Now, we find out the value of x

$\because$ $x = \log_e^{-1}(-5.335)$

$$\log_e x = -5.335$$

or $2.3\log_{10} x = -5.335$

$$\log_{10} x = \frac{-5.335}{2.3} = -2.319$$

$$x = 0.004797$$

Substituting the value of x in equation (iii), we get

$$P = \frac{320000}{1+7\times 0.004797} = 309604$$

Hence, the population after 15 more years will be

$$= 309604$$

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2 Sources of Water Supply & Well Hydraulics

T1 : Solution

$$\text{Dia. of well} = 60 \text{ mm} \qquad \therefore r_w = 30 \text{ cm}$$

$$Q = 1360 \text{ lit/min}$$

$$r_1 = 6 \text{ m} \qquad s_1 = 6 \text{ m}$$

$$r_2 = 15 \text{ m} \qquad s_2 = 1.5 \text{ m}$$

Discharge as per theims theory in unconfined aquifer

$$Q = \frac{\pi k\left(h_2^2 - h_1^2\right)}{2.306 \log_{10}\left(\frac{r_2}{r_1}\right)}$$

$$h_1 = H - s_1 = 90 - 6 = 84 \text{ m}$$

$$h_2 = H - s_2 = 90 - 1.5 = 88.5 \text{ m}$$

$$\frac{1360 \times 10^{-3}}{60} = \frac{\pi \times k \times \left(88.5^2 - 84^2\right)}{2.303 \times \log_{10}\left(\frac{15}{6}\right)}$$

$$k = 8.5 \times 10^{-6} \text{ m/sec}$$

To find drawdown (h_0)

$$Q = \frac{\pi k\left(h_1^2 - h_0^2\right)}{2.303 \log_{10}\left(\frac{r_1}{r_w}\right)}$$

$$\frac{1360 \times 10^{-3}}{60} = \frac{\pi \times 8.5 \times 10^{-6}\left(84^2 - h_0^2\right)}{2.303 \times \log_{10}\left(\frac{6}{0.3}\right)}$$

$$h_0 = 67.2 \text{ m}$$

$$s = 22.8 \text{ m}$$

To find R use Dupit theory

$$Q = \frac{\pi k\left(H^2 - {h_0}^2\right)}{2.306\log_{10}\left(\frac{R}{r_w}\right)}$$

$$\frac{1360 \times 10^{-3}}{60} = \frac{\pi \times 8.5 \times 10^{-6} \times \left(90^2 - 67.2^2\right)}{2.306\log_{10}\left(\frac{R}{30 \times 10^{-2}}\right)}$$

$$\log_{10}\left(\frac{R}{30 \times 10^{-2}}\right) = 1.8104$$

$$R = 20.5 \text{ m}$$

For specific capacity using Dupit theory,

$$SC = \frac{\pi k\left(H^2 - h_0^2\right)}{2.306\log\left(\frac{R}{r_w}\right)}$$

$$s = 1; \qquad H = 90$$

$$h_0 = (90 - 1) = 89 \text{ m}$$

$$SC = \frac{\pi \times 8.5 \times 10^{-6} \times \left(90^2 - 89^2\right)}{2.306 \times \log\left(\frac{20.5}{30 \times 10^{-2}}\right)}$$

$$= 1.13 \times 10^{-3} \text{ m}^3/\text{sec}$$

$$= 67.8 \text{ L/min}$$

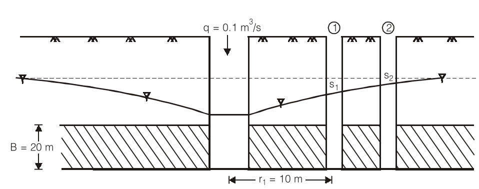
To find max. discharge, $h_0 = 0$

$$Q_{max} = \frac{\pi k(H^2)}{2.306\log_{10}\left(\frac{R}{r_w}\right)}$$

$$= \frac{\pi \times 8.5 \times 10^{-6} \times 90^2}{2.306\log\left(\frac{20.5}{0.3}\right)}$$

$$= 0.051 \text{ m}^3/\text{sec} = 30.78 \text{ L/min.}$$

T2 : Solution



Given that, $s_1 = 4$ m; $s_2 = 3$ m

We know that, $$q = \frac{2\pi kB(s_1 - s_2)}{2.303\log_{10}\left(\frac{r_2}{r_1}\right)} \qquad [\because kB = T]$$

$$\Rightarrow \quad q = \frac{2\pi T(s_1 - s_2)}{2.303\log_{10}\left(\frac{r_2}{r_1}\right)} \Rightarrow 0.1 = \frac{2\pi T(4-3)}{2.303\log_{10}\left(\frac{60}{10}\right)}$$

$$\Rightarrow \quad T = \frac{2.303 \times 0.1 \times \log_{10} 6}{2\pi} \Rightarrow T = 0.0285 \text{ m}^2/\text{sec}$$

$$\therefore \quad k = \frac{T}{B} = \frac{0.0285}{20} = 1.425 \times 10^{-3} \text{ m/sec}$$

Again, $$q = \frac{2\pi T(s_w - s_1)}{2.303\log_{10}\left(\frac{r_1}{r_w}\right)}$$

where, s_w is drawdown in well

r_w is radius of well $= \frac{0.5}{2} = 0.25$ m

$$\therefore \quad 0.1 = \frac{2\pi \times 0.0285 \times (s_w - 4)}{2.303\log_{10}\left(\frac{10}{0.25}\right)}$$

$$\Rightarrow \quad s_w = \frac{0.1 \times 2.303 \times \log_{10} 40}{2\pi \times 0.0285} + 4 = 6.06 \text{ m}$$

Thus, coefficient of permeability of test well, $k = 1.425 \times 10^{-3}$ m/s and drawdown in the test well, $s_w = 6.06$ m.

Darcy's law is valid only for saturated soils in which laminar flow condition prevails. The Reynolds number should be less than 1 for laminar flow conditions in soils. According to William Hazen for laminar flow diameter of soil grains should be less than or equal to 3 mm.

T3 : Solution

Given data: $$Q = 1500 \text{ litres per minute} = \frac{1500 \times 10^{-3}}{60} \text{ m}^3/\text{s} = 0.025 \text{ m}^3/\text{s}$$

$r_1 = 6$ m, $s_1 = 6$ m

$r_2 = 16$ m, $s_2 = 2$ m

$H = 100$ m

$\therefore \quad h_1 = H - s_1 = 100 - 6 = 94$ m

$h_2 = H - s_2 = 100 - 2 = 98$ m

Using Thiem's equation for unconfined aquifers, we have

$$Q = \frac{\pi k(h_2^2 - h_1^2)}{2.303\log_{10}\left(\frac{r_2}{r_1}\right)} \Rightarrow 0.025 = \frac{\pi k(98^2 - 94^2)}{2.303\log_{10}\left(\frac{16}{6}\right)}$$

$$\Rightarrow \quad \pi k = 3.193 \times 10^{-5} \Rightarrow k = 1.016 \times 10^{-5} \text{ m/s}$$

Thus the coefficient of permeability, $k = 1.016 \times 10^{-5}$ m/s

Radius of gravity well, $r_w = \dfrac{0.5}{2} = 0.25$ m

Using Theim's equation again, we get

$$Q = \frac{\pi k\left(h_1^2 - h_w^2\right)}{2.303 \log_{10}\left(\dfrac{r_1}{r_w}\right)} \Rightarrow 0.025 = \frac{3.193 \times 10^{-5} \times (94^2 - h_w^2)}{2.303 \log_{10}\left(\dfrac{6}{0.25}\right)}$$

$$\Rightarrow \quad 2488.3 = 94^2 - h_w^2$$

$$\Rightarrow \quad h_w = 79.67 \text{ m}$$

$\therefore$ Drawdown in the pumped well $= H - h_w = 100 - 79.67 = 20.33$ m

T4 : Solution

Diameter of tube well = 200 mm
Spacing between tube wells = 150 m
Radius of influence, R = 200 m

$\therefore$ Radius of tube well, $r_w = \dfrac{200}{2} = 100$ mm

The discharge from a fully penetrating tubewell in a confined aquifer is given by

$$Q_1 = \frac{2\pi kHs}{2.3 \log_{10}\left(\dfrac{R}{r_w}\right)}$$

where, H = thickness of confined aquifer
s = drawdown
k = coefficient of permeability

The discharge from two tubewells at a distance B apart is given by

$$Q_2 = Q_3 = \frac{2\pi kHs}{2.3 \log_{10}\left(\dfrac{R^2}{r_w B}\right)}$$

$\therefore$ Total discharge from both wells,

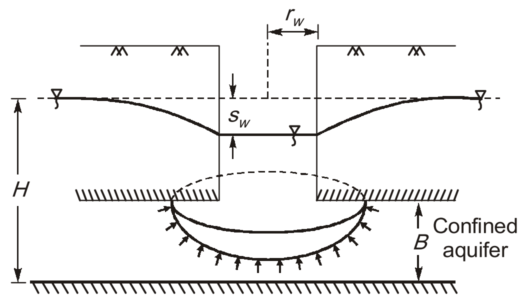
$$Q_4 = Q_2 + Q_3 = \frac{2 \times 2\pi\, kHs}{2.3 \log_{10}\left(\dfrac{R^2}{Br_w}\right)}$$

$$\frac{Q_1}{Q_4} = \frac{\log_{10}\left(\dfrac{R^2}{Br_w}\right)}{2\log_{10}\left(\dfrac{R}{r_w}\right)}$$

$$\Rightarrow \quad \frac{Q_1}{Q_4} = \frac{\log_{10}\left(\dfrac{200^2}{150 \times 100 \times 10^{-3}}\right)}{2\log_{10}\left(\dfrac{200}{100 \times 10^{-3}}\right)} \Rightarrow \frac{Q_1}{Q_4} = 0.52$$

T5 : Solution

The flow will be spherical for the partially penetrated well.



Since, flow is spherical, therefore the flow area,

$$A = 2\pi r_w^2 \qquad \text{(Surface area of a hemi sphere)}$$

Discharge, $$Q_s = \text{vA}$$

$\Rightarrow$ $$Q_s = \text{kiA}$$

$\Rightarrow$ $$Q_s = k \times \frac{s_w}{r_w} \times 2\pi r_w^2$$

$\Rightarrow$ $$Q_s = 2\pi k s_w r_w$$

$\Rightarrow$ $$s_w = \frac{Q_s}{2\pi k r_w}$$

$\therefore$ $$Q_s = 2\pi k r_w s_w = 2\pi r_w k (H - h_w) \qquad \text{...(i)}$$

But, discharge through a fully penetrating well in confined aquifer under steady state condition is given by

$$Q_r = \frac{2\pi kB(H - h_w)}{2.303 \log_{10}\left(\frac{R}{r_w}\right)} \qquad \text{...(ii)}$$

where, B isthickness of confined aquifer
H isinitial height of water table from bottom of well
h_w isartesian pressure in the well
R is radius of influence
r_w is radius of well
s_w =Drawdown in the well = $H - h_w$

Dividing (i) by (ii), we get

$$\frac{Q_s}{Q_r} = \frac{2\pi r_w k (H - h_w)}{2\pi kB(H - h_w) / 2.303 \log_{10}\left(\frac{R}{r_w}\right)}$$

$\Rightarrow$ $$\frac{Q_s}{Q_r} = \frac{\left[2.303 \log_{10}\left(\frac{R}{r_w}\right)\right] \times r_w}{B}$$

$\Rightarrow$ $$\frac{Q_s}{Q_r} = 2.303\left(\frac{r_w}{B}\right)\log_{10}\left(\frac{R}{r_w}\right)$$

(i) $$Q = \frac{2\pi kH \cdot S}{2.303 \log_{10}(r/r_w)}$$

where, H = thickness of the confined aquifer; S = drawdown; r = radius of circle of influence; r_w = radius of the well

$$\therefore \quad 2000 = \frac{2\pi \times 30 \times 15 \cdot S}{2.303 \log_{10}(135/0.15)} = \frac{2827.43}{6.80} \cdot S$$

$$S = \frac{2000 \times 6.80}{2827.43} = \mathbf{4.81\ m}$$

(ii) $$Q = \frac{2\pi kHS}{2.303 \log_{10}(r/r_w)}$$

Putting numerical values, $$Q = \frac{2\pi \times 21 \times 15 \times 2}{2.303 \log_{10}(400/0.05)} = \frac{1718.8}{\log_{10}(8000)} = \frac{1718.8}{3.903}$$

$$= \mathbf{440.40\ m^3/day}$$

$$Q_1 = Q_2 = Q_3 = \frac{2\pi kD(H-h)}{2.3 \log_{10}\left(R^3/rL^2\right)}$$

Using this equation and putting numerical values

$$Q_1 = Q_2 = Q_3 = \frac{2\pi \times 21 \times 15 \times 2}{2.303 \log_{10}\left(\frac{400^3}{0.05 \times 15^2}\right)} = \frac{3958.4}{15.56} = 254.5\ \text{m}^3/\text{day}$$

Hence, percentage reduction in discharge = $\frac{(440.4 - 254.4)}{440.4} \times 100 = \mathbf{42.23\%}$

(iii) $$Q = \frac{2\pi kHS}{2.303 \log_{10}(R/r_w)}$$

or $$0.10 \times 60 \times 60 \times 24 = \frac{2\pi \times 60 \times 30 \times 5}{2.303 \log_{10}(280/r_w)} = \frac{24554}{\log_{10}(280/r_w)}$$

or $$\log_{10}\left(\frac{280}{r_w}\right) = \frac{24554}{8640} = 2.842$$

$$\frac{280}{r_w} = 10^{2.842}$$

$$r_w = 0.40\ \text{m} = \mathbf{40\ cm}$$

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3 Quality Characteristics of Water

T1 : Solution

Hardness is due to multixalent cations.

Total hardness in mg/*l* as $CaCO_3$

$$= \left[Ca^{++} \text{ in mg}/l \times \frac{\text{Combining weight of } CaCO_3}{\text{Combining weight of } Ca^{++}}\right]$$

$$+ \left[Mg^{++} \text{ in mg}/l \times \frac{\text{Combining weight of } CaCO_3}{\text{Combining weight of } Mg^{+}}\right]$$

$$= \left[50 \times \frac{50}{20} + 72 \times \frac{50}{12}\right] = 125 + 300 = 425 \text{ mg}/l$$

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4 Treatment of Water

T1 : Solution

$FeSO_4 .7H_2O$ and $Ca(OH)_2$ are added in 1 mole each, having their molecular weight as 278 and 112 respectively

Thus, 1 gm of Ferrous sulphate

$$= \frac{112}{278} = 0.403 \text{ gm of lime}$$

So, 1 mg/L of ferrous sulphate = 0.403 mg/L of lime

$\Rightarrow$ 12 mg/L = 12 × 0.403 = 4.836 mg/L of lime

$\Rightarrow$ Total lime required = 4.836 × 16 = 77.38 kg/day

T2 : Solution

Let us assume $d < 0.1$ mm

Setling velocity,
$$V_s = 418(G-1)d^2\left(\frac{3T+70}{100}\right)$$

$$V_S = \frac{10\times10^6\times10^{-3}\times10^3}{300\times24\times3600}\text{mm/s} = 418(2.65-1)d^2\left(\frac{3\times26+70}{100}\right)$$

$$\Rightarrow \quad d^2 = \frac{0.3858}{1020.75}$$

$$\Rightarrow \quad d = 0.01944 \text{ mm}$$

Hence our assumption is correct.

T3 : Solution

$$\text{Total surface area} = \frac{\text{Design flow rate}}{\text{Design loading rate}} = \frac{0.5}{200/(24\times60\times60)}$$

$$= \frac{24\times60\times60\times0.5}{200} = 216 \text{ m}^2$$

T4 : Solution

$$\text{Number of filters} = \frac{\text{Total surface area}}{\text{Surface area of each filter box}}$$

$$= \frac{216}{50} = 4.32 \simeq 5$$

Provide one additional filter as a stand-by filter to be used during cleaning, maintenance, etc.
Hence, number of filters = 5 + 1 = 6.

T5 : Solution

The disinfection of industrial water supplies is necessary in food processing, distillery (alcohol), etc.

T6 : Solution

$$\text{Chlorine usage} = 9\ \text{kg/d}$$

$$Q_0 = 25000\ \text{m}^3/\text{d}$$

$$\text{Chlorine usage} = \frac{9\times10^6}{25000\times10^3} = 0.36\ \text{mg}/l$$

$$\text{Chlorine usages} = \text{chlorine demand} + \text{residual chlorine}$$

$$0.36 = \text{chlorine demand} + 0.2$$

$$\text{Chlorine demand} = 0.36 - 0.2 = 0.16\ \text{mg}/l$$

T7 : Solution

Total water to be filtered

$$= 99 \times 1.05\ \text{MLD} = 103.95\ \text{MLD}$$

(Addition of 5% to be used for backwashing)

$$\frac{L}{B} = 1.35 \text{ where } B = 5.2\ \text{m}$$

$\therefore$ $L = 7.02$ m

$\therefore$ Surface area of each filter $= 36.504\ \text{m}^2$

Total surface area required

$$= \frac{\text{Discharge through filter}}{\text{Rate of filtration}}$$

$$= \frac{103.95\times10^3}{6\times24} = 721.875\ \text{m}^2$$

Total no. of working units required

$$= \frac{721.875}{36.504} = 19.77 \text{ filters} = 20 \text{ filters}$$

1 unit is to added as standby, thus total no. of units required = 21

T8 : Solution

$$\text{Flow rate, } Q_0 = 0.2\ \text{m}^3/\text{sec}$$

Plan area, $(PA) = LB = 32 \times 8 = 256\ \text{m}^2$

(OFR) over flow rate

$$= \frac{Q_0}{PA} = \frac{0.2}{256} = 7.8125 \times 10^{-4}\ \text{m/s}$$

Now, settling velocity of particle of size 25 μm be u_s

$$u_s = \frac{(G-1)\gamma_w d^2}{18\mu}$$

$$= \frac{(2.5-1)9.81\times10^{-3}\left(25\times10^{-6}\right)^2}{18\times0.01\times10^{-3}\times10^2}$$

$$= 5.10 \times 10^{-4}\ \text{m/sec}$$

$$\eta_{\text{removal}} = \frac{u_s}{OFR}\times 100$$

$$= \frac{5.10\times10^{-4}}{7.8125\times10^{-4}}\times100 = 65.28\% \simeq 65\%$$

T9 : Solution

During disinfection variations of micro-organism is given by

$$N_t = N_o e^{-kt}$$

N_t = No. of micro-organism at time t

N_o = No. of micro-organism at time 0

So, disinfection efficiency at any time 't',

$$\eta_t = \frac{N_o - N_t}{N_o}\times 100$$

For $t = 3$ min; $\eta_3 = 50\%$

$$\eta_3 = \frac{N_o - N_o e^{-k\times3}}{N_o}\times100 = 50$$

$$k = 0.231\ \text{min}^{-1}$$

Now for $\eta_t = 99\%$

$$\eta_t = \frac{N_o - N_t}{N_o}\times100 = 99$$

$$\frac{N_o - N_o e^{-0.231\times t}}{N_o}\times100 = 99;\quad t = 19.93\ \text{min}$$

T10 : Solution

Using principle of gram equivalent, 1 gm - equivalent of calcium as calcium carbonate will react with 1 gm-equivalent of lime.

Now, equivalent weight of calcium carbonate = 50 gm

Equivalent weight of lime = 28 gm

So, 50 g of calcium carbonate require = 28 gm of lime

Hence, 72 mg/L of calcium carbonate require = $\frac{28\times72}{50}$ = 40.32 mg/L of lime

But, as the lime is 82% pure, therefore requirement of lime is $\left(\frac{40.32}{0.82}\right)$ i.e. 49.17 mg/L.

T11 : Solution

Type-I Settling: Particles whose shape, size, specific gravity do not change with time are called as "DISCRETE PARTICLES" and particles whose surface properties are such that they coalesce/combine with other particles upon contact thereby changing shape, size and specific gravity of particles are called "FLOCCULATING PARTICLES" settling of discrete particles in dilute suspension is called as Type-I settling.

When a particle is suspended in water, initially it has only two forces acting upon it viz.

1. Force of gravity = $F_g = \rho_p V_p g$
   Where ρ_p and V_p are density and volume of particles respectively.
2. Buoyant force = $F_B = \rho_w V_p g$
   Where ρ_w is the density of water.
   Now, $F_g = F_B$ is $\rho_p = \rho_w$ and no acceleration of the particles will take place.
   If $\rho_p \neq \rho_w$ which usually always happens, a net force acts on the particles and particle accelerates in the direction of net force (F_{net}).
   Thus, $F_{net} = (\rho_p - \rho_w) g V_p$ = Driving force for acceleration.
   Once motion of particles has started, a third force come into play due to viscous friction. This force is called as "DRAG FORCE" (F_D), given by

$$F_D = \frac{1}{2} C_D A_P \rho_w V_P^2$$

Where C_D = Drag coeff.

$$\therefore \quad (\rho_p - \rho_w) g V_p = \frac{1}{2} C_D A_P \rho_w V_P^2$$

For spherical particles,
$$\frac{V_P}{A_P} = \frac{\frac{\pi}{6} d^3}{\frac{\pi}{4} d^2} = \frac{2}{3} d$$

$$\therefore \quad V_P^2 = \frac{4}{3} g \frac{(\rho_P - \rho_w) d}{C_D f_w} \qquad \text{...(i)}$$

Drag coeff.
$$(C_D) = \begin{cases} \dfrac{24}{\text{Re}} & \text{for laminar flow.} \\ 0.4 & \text{for turbulent flow.} \end{cases}$$

Here Re = Reynold's no. = $\dfrac{v \rho_w d}{\mu}$

Substuting C_D in (i),
$$v = \frac{g d^2 \rho_w (G-1)}{18\mu}$$
G = Sp. gravity of particles.

$d = 4 \times 10^{-3}$ cm/s $= 4 \times 10^{-5}$ m/s, $G = 2.65$

Assuming laminar flow,
$$v = \frac{9.81(16 \times 10^{-10})(2.65 - 1)}{18 \times 1.02 \times 10^{-6}} = 14.106 \times 10^{-4} \text{ m/s}$$

$$\therefore \quad Re = \frac{14.106 \times 10^{-4} \times 4 \times 10^{-5}}{1.02 \times 10^{-6}} = 55.318 \times 10^{-3} < 1$$

$\Rightarrow$ Assumptions of laminar flow is true.

T12 : Solution

$$n' = \text{Porosity of expanded bed}$$

$$n' = \left(\frac{V_B}{V_s}\right)^{0.22}$$

$$0.65 = \left(\frac{V_B}{4.5\text{ cm/s}}\right)^{0.22}$$

$$V_B = 6.35 \times 10^{-3}\text{ m/s}$$

5 Distribution System

T1 : Solution

Time (Hour)	Cumulative Demand (ML)	Cumulative Supply	(i) Cumulative Supply – demand	Cumulative Supply	(ii) Cumulative Supply – demand
0 - 2	0.450	3	2.55	0	–0.45
2 - 4	0.975	6	5.025	0	–0.975
4 - 6	1.95	9	7.05 (A)	6	4.05
6 - 8	4.95	12	7.05	12	7.05 (A)
8 - 10	10.95	15	4.05	18	7.05
10 - 12	16.50	18	1.50	18	1.50
12 - 14	19.20	21	1.80	18	–1.20 (B)
14 - 16	21.75	24	2.25	24	2.25
16 - 18	26.20	27	0.3	30	3.30
18 - 20	31.70	30	–1.80	36	4.20
20 - 22	35.80	33	–2.1 (B)	36	0.90
22 - 44	36.00	36	0	36	0

(i) If pumping is constant

$$\text{Rate of supply} = \frac{36}{24} = 1.5 \text{ ML/hr}$$

$$\begin{aligned}\text{Balancing storage} &= A + B \\ &= 7.05 + 2.1 \\ &= 9.15 \text{ ML}\end{aligned}$$

(ii) Intermittant supply

$$\text{Rate of supply} = \frac{36}{12} = 3 \text{ ML/hr}$$

$$\begin{aligned}\text{Balancing storage} &= A + B \\ &= 7.05 + 1.2 \\ &= 8.25 \text{ ML}\end{aligned}$$

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6 Quality, Characteristics and Biochemical Reactions of Waste Water

T1 : Solution

Given $$[BOD_4]_{20^\circ C} = \frac{75}{100} \times BOD_u$$

We know, $$[BOD_t]_{T^\circ C} = BOD_u\left[1 - 10^{-K_D \times t}\right]$$

$$[BOD_4]_{20^\circ C} = BOD_u\left[1 - 10^{-K_D \times 4}\right]$$

$$\Rightarrow \quad \frac{75}{100} \times BOD_u = BOD_u\left[1 - 10^{-4K_D}\right]$$

$$\Rightarrow \quad 0.75 = 1 - 10^{-4K_D} \quad \Rightarrow \quad 10^{-4K_D} = 0.25$$

Taking $\log_{10}$ both sides

$$\Rightarrow \quad -4\,K_D = -0.6020$$

$$\therefore \quad K_D = 0.1505 \text{ day}^{-1}$$

T2 : Solution

"Bio-chemical oxygen demand (BOD) is used as a measure of the quantity of oxygen required for oxidation of bio-degradable organic matter present in wastewater sample by aerobic bio-chemical action".

Determination of BOD_5:

The standard 5 day BOD (BOD_5) is determined in the laboratory by mixing a known volume of a sample of wastewater with known volume of pure water and calculating the dissolved oxygen (D.O.) of this diluted sample. The diluted sample is then incubated for 5 days at 20°C. The dissolved oxygen (D.O.) of the diluted sample, after this period of incubation is again calculated. Then BOD_5 in mg/l is calculated as

$$BOD_5 = (D.O_i - D.O_f) \times \frac{\text{Vol. of the diluted sample}}{\text{Vol. of the undiluted sewage sample}}$$

where, $D.O_i$ = initial D.O. of diluted sample

$D.O_f$ = Final D.O. of diluted sample after 5 days incubation at 20°C

Given, Vol. of Waste water = 5 ml

D.O. of waste water = 0 mg/l

Vol. of pure water = 300 – 5 = 295 mg/l

D.O. of pure water = 9.2 mg/l

Initial D.O. of diluted sample, $$D.O_i = \frac{V_{waste} \times BOD_5 + V_{pure} \times BOD_5}{V_{waste} + V_{pure}}$$

$$= \frac{5 \times 0 + 295 \times 9.2}{300} = 9.05 \text{ mg}/l$$

After incubating the bottle for 5 day, D.O. of mixture was found 5.0 mg/l

$$\therefore \quad D.O_f = 5.0 \text{ mg}/l$$

$$\therefore \quad BOD_5 = (D.O_i - D.O_f) \times \frac{\text{Vol. of diluted sample}}{\text{Vol. of undiluted sewage sample}}$$

$$= (9.05 - 5.0) \times \frac{300}{5} = 242.8 \text{ mg / L}$$

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7 Disposal of Sewage Effluents

T1 : Solution

Per capita BOD of the domestic sewage = 72 gm/day = 72×10^3 mg/day

Per capita sewage produced = 240 lit/day

$\therefore$ BOD per litre of the domestic sewage $= \dfrac{72 \times 10^3}{240} = 300$ mg/L

Amount of domestic waste water produced per day $= 30000 \times 240 = 7.2 \times 10^6$ litres

$\therefore$ Net BOD of all waste waters (domestic + industrial) $= \dfrac{7.2 \times 300 + 4 \times 1500}{7.2 + 4} = 728.57$ mg/L

Total waste water discharge $= \dfrac{(7.2+4) \times 10^6}{24 \times 60 \times 60} = 129.63$ lit/sec

Total waster water discharge with 10% expansion $= 129.63 + \dfrac{10}{100} \times 129.63 = 142.593$ lit/sec

Now, Initial *DO* of saturated stream water = 7 mg/L

Assuming that the *DO* of waste water is nil, at the starting point.

$$DO \text{ of the mixture} = \frac{DO \text{ of river} \times Q_r + DO \text{ of sewage} \times Q_s}{Q_r + Q_s}$$

where $Q_r = 4500$ lit/sec; $Q_s = 142.593$ lit/sec

$$\therefore \quad DO \text{ of mixture} = \frac{7 \times 4500 + 0 \times 142.593}{4500 + 142.593} = 6.785 \text{ mg/L}$$

$$\therefore \quad \text{Initial deficit in } DO = D_o = 7 - 6.785 = 0.215 \text{ mg/L}$$

Given that
$$f = \frac{k_R}{k_D} = \frac{0.3}{0.1} = 3$$

$$D_c = 7 - 4 = 3 \text{ mg/L}$$

$$D_o = 0.215 \text{ mg/L}$$

We know that

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

$$\Rightarrow \quad \left(\frac{L_o}{3 \times 3}\right)^{3-1} = 3\left[1 - (3-1) \times \frac{0.215}{L_o}\right]$$

$$\Rightarrow \quad \frac{L_o^2}{81} = 3\left[1-\frac{0.43}{L_o}\right]$$

$$\Rightarrow \quad \frac{L_o^2}{3\times 81} = \frac{L_o - 0.43}{L_o}$$

$$\Rightarrow \quad L_o^3 = 243L_o - 104.49$$

$$\Rightarrow \quad L_o^3 - 243L_o + 104.49 = 0$$

$$\Rightarrow \quad L_o = 15.37 \text{ mg/L}$$

Maximum permissible 5 day BOD of the mix at mix temperature

$$= L_o\left[1-10^{-k_D t}\right] = 15.37\,[1 - 10^{-0.1\times 5}] = 10.51 \text{ mg/L}$$

$$\text{Again BOD}_{mix} = \frac{C_s \times Q_s + C_r\, Q_r}{Q_s + Q_r}$$

$$\Rightarrow \quad 10.51 = \frac{C_s \times 142.593 + 0 \times 4500}{142.593 + 4500}$$

where C_s = Maximum permissible BOD_5 of waste water

$$\Rightarrow \quad C_s = \frac{10.51\times(142.593 + 4500)}{142.593}$$

$$\Rightarrow \quad C_s = 342.19 \text{ mg/L}$$

$\therefore$ Degree of treatment required

$$= \left(\frac{\text{Initial BOD of city waste water} - \text{Max. permissible BOD of waste water}}{\text{Initial BOD of city waste water}}\right)\times 100$$

$$= \frac{728.57\times 128.57 - 342.19\times(128.57\times 1.1)}{728.57\times 128.57}$$

$$= 48.4\%$$

T2 : Solution

$$\text{BOD of mixture} = \frac{Q_w Y_w + Q_R \times Y_R}{Q_w + Q_R} = \frac{8\times 100 + 20\times 6}{8+20}$$

$$= 32.857 \text{mg}/l = 32.857 \text{ gm/m}^3$$

Deoxygenation rate constant with base 10,

$$K_D = 0.434\,K = 0.434 \times 0.252 = 0.1094$$

$$\text{Area of river} = 80 \text{ m}^2$$

$$\text{Flow of river} = 20 + 8 = 28 \text{ m}^3/\text{sec}$$

$$\text{Stream velocity} = \frac{28}{80} = 0.35 \text{ m/sec}$$

$$Y_t = Y_0[1-10^{-K_D \cdot t}]$$

$$\Rightarrow \quad 5 = 32.857\,[1 - 10^{-0.1094\times t}]$$

$$\Rightarrow \quad t = 0.6553 \text{ days} = 56620.87 \text{ sec}$$

Distance from downstream mixing point = Velocity × Time

$$= 0.35 \times 56620 = 19817 \text{ m} \simeq 19.82 \text{ km}$$

T3 : Solution

$$v = 0.85 \text{ m/s}$$

$$\text{Distance} = 48.3 \text{ km}$$

$$\text{Time} = \frac{d}{v} = \frac{48.3 \times 10^3}{0.85 \times 86400} = 0.657 \text{ days}$$

$$L_0 = 20 \text{ mg}/l$$

$$\text{Initially D.O.} = 10 \text{ mg}/l$$

$$\Rightarrow \quad D = 10 - 10 = 0$$

$$D_t = \frac{k_D L_0}{k_R - k_D}\left[10^{-k_D t} - 10^{-k_R t}\right] + 10^{-k_R t} D$$

$$= \frac{0.2 \times 20}{0.4 - 0.2}\left[10^{-0.2 \times 0.657} - 10^{-0.4 \times 0.657}\right] + 10^{-0.4 \times 0.657} \times 0$$

$$= 3.059 \text{ mg/l}$$

$$\therefore \quad \text{Dissolved oxygen} = DO_{sat} - D_t$$

$$= 10 - 3.059$$

$$= 6.94 \text{ mg}/l$$

$\Rightarrow$ DO at 48.3 km downstream = 6.94 mg/l

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8 Treatment of Waste Water

T1 : Solution

Efficiency of treatment,
$$\eta = \frac{Q_0S_0 - Q_0S_e}{Q_0S_0} \times 100 = \frac{120-20}{120} \times 100 = 83.3\%$$

$$\eta = \frac{100}{1+0.0044\sqrt{\dfrac{W_1}{V_1F_1}}}$$

Amount of BOD entering,
$$W_1 = Q_0S_0 \text{ kg/day}$$
$$= 2200 \times 10^3 \times 120 \times 10^{-6} = 264 \text{ kg/day}$$

$$F_1 = \frac{1+R}{(1+0.1R)^2}$$

$$R = \frac{Q_R}{Q_0} = \frac{4000}{2200} = 1.81$$

$$F_1 = \frac{1+1.81}{(1+0.1\times 1.81)^2} = 2.01$$

$$83.3 = \frac{100}{1+0.44\sqrt{\dfrac{264}{V_1 \times 2.01}}}$$

$$V_1 = 637 \text{ m}^3$$

$\because$ Depth = 1.5 m

$$\text{Plan area} = \frac{637}{1.5}$$

$$\frac{\pi D^2}{4} = 425$$

$$D = 23.3 \text{ m}$$

Note: The diameter of trickling filter is limited upto 60 m. as it is the maximum available size of rotatory distribution (if more than 60 m steel truss will bend at its ends due to self weight).

T2 : Solution

(i) 1st iteration

Let the flow in the grit chamber be laminar. Thus, the settling velocity may be calculated by Stoke's equation i.e.

$$v_t = \frac{g(S_s-1)d^2}{18\nu} = \frac{9.81\times(2.65-1)\times(0.2\times10^{-3})^2}{18\times10^{-2}\times(10^{-2})^2} = 0.036 \text{ m/s}$$

$$Re = \frac{v_t d}{\nu} = \frac{0.036\times0.2\times10^{-3}}{10^{-2}\times(10^{-2})^2} = 7.2$$

The value of Reynolds number is greater than 1 but less than 10^4. Hence, the flow is transitional.

$$\therefore \quad C_D = \frac{24}{Re}+\frac{3}{(Re)^{1/2}}+0.34 = \frac{24}{7.2}+\frac{3}{(7.2)^{1/2}}+0.34 = 4.8$$

The general formula for the calculation of settling velocity is given by

$$v_t^2 = \frac{4}{3}\times g\times\frac{(S_s-1)d}{C_D}$$

$$\Rightarrow \quad v_t^2 = \frac{4}{3}\times9.81\times\frac{(2.65-1)\times0.2\times10^{-3}}{4.8}$$

$$\Rightarrow \quad v_t = 0.03 \text{ m/s}$$

2nd iteration

Again, $$Re = \frac{v_t d}{\nu} = \frac{0.03\times0.2\times10^{-3}}{10^{-2}\times(10^{-2})^2} = 6$$

$$\therefore \quad C_D = \frac{24}{Re}+\frac{3}{(Re)^{1/2}}+0.34 = \frac{24}{6}+\frac{3}{(6)^{1/2}}+0.34 = 5.565$$

$$\therefore \quad v_t^2 = \frac{4}{3}\times9.81\times\frac{(2.65-1)\times0.2\times10^{-3}}{5.565}$$

$$\Rightarrow \quad v_t = 0.028 \text{ m/s}$$

3rd iteration

$$\Rightarrow \quad Re = \frac{v_t d}{\nu} = \frac{0.028\times0.2\times10^{-3}}{10^{-2}\times(10^{-2})^2}$$

$$\therefore \quad C_D = \frac{24}{Re}+\frac{3}{(Re)^{1/2}}+0.34 = \frac{2.4}{5.6}+\frac{3}{(5.6)^{1/2}}+0.34 = 5.893$$

$$\therefore \quad v_t^2 = \frac{4}{3}\times9.81\times\frac{(2.65-1)\times0.2\times10^{-3}}{5.893}$$

$$\Rightarrow \quad v_t = 0.027 \text{ m/s} \approx 0.028 \text{ m/s (Hence OK)}$$

Thus, the settling velocity of the 0.2 mm particles is 0.027 m/s.

(ii) Critical horizontal flow velocity can be calculated by modified Shield's formula as

$$v_h = 4.5\sqrt{gd(S_s-1)} = 4.5\sqrt{9.81\times0.2\times10^{-3}\times(2.65-1)}$$

$= 0.26$ m/s

(iii) Let the length, width and depth of grit chamber be L, B and D respectively.

Quantity of flow, $Q = 40 \text{ MLD} = \dfrac{40 \times 10^6 \times 10^{-3}}{24 \times 60 \times 60} \text{ m}^3/\text{s} = 0.463 \text{ m}^3/\text{s}$

Now, we know that, $Q = v_h \times A$

$\Rightarrow \quad A = \dfrac{0.463}{0.26} \Rightarrow A = 1.78 \text{ m}^2$

Assuming depth of tank (D) as 1 m, then

$$D \times B = A$$

$\Rightarrow \quad 1 \times B = 1.78$

$\Rightarrow \quad B = 1.78$ m say 1.8 m

$$\text{Detention time} = \frac{\text{Depth of basin}}{\text{Settling velocity}} = \frac{D}{v_t} = \frac{1}{0.027} = 37 \text{ seconds}$$

$\therefore$ Lenth of tank, L = Critical horizontal flow velocity $\times$ Detention time

$= 0.26 \times 37 = 9.6$ m

Thus, the dimensions of the tank will be 9.6 m $\times$ 1.8 m $\times$ 1 m

T3 : Solution

The quantity of water supplied = Per capita rate $\times$ Population

$= 120 \times 150$ litres/day $= 18000$ l/day

Assuming that 80% of water supplied becomes sewage, we have

The quantity of sewage produced $= 18000 \times 0.8 = 14{,}400$ l/day.

The quantity of sewage produced during the detention period (i.e. the capacity of the tank)

(Assume detention period as 24 hr)

$$= 14400 \times \frac{24}{24} = 14400 \text{ litres}$$

Now, assuming the rate of deposited sludge as 30 litres/capita/year; and also assuming the period of cleaning as 1 year, we have

The volume of sludge deposited $= 30 \times 150 \times 1 = 4500$ litres

$\therefore$ Total required capacity of the tank = Capacity for sewage + Capacity for sludge

$= 14400 + 4500 = 18900$ litres $= 18.9$ cu-m

Assuming 1.5 m as the depth of the tank, we have

$$\text{The surface area of the tank} = \frac{18.9}{1.5} \text{m}^2 = 12.6 \text{m}^2$$

If the ratio of the length to width is kept as 3 : 1, we have

$$3B^2 = 12.6$$

$\Rightarrow \quad B = \sqrt{\dfrac{12.6}{3}} = \sqrt{4.2} = 2.05 \text{m}; \text{ say } 2.1 \text{ m}$

$\therefore$ Provide width = 2.1 m; and

Provide length of the tank = 6 m

∴ Area of cross-section provided $= 6 \times 2.1 = 12.6\ m^2$ (same as required)

Thus, the dimensions of the septic tank will be

6 m × 2.1 m × (1.5 + 0.3) m overall depth [0.3 m used as free-board]

Hence, use a tank of size 6 m × 2.1 m × 1.8 m.

T4 : Solution

(i) **Design of Septic Tank:**

Quantity of sewage produced per day

$$= 110 \times 180 = 19800\ l/\text{day}$$

Assuming the detention period to be 24 hours, we have

The quantity of sewage produced during the detention period, i.e., the capacity of tank

$$= 19800 \times \frac{24}{24} = 19800 \text{ litres}$$

Now assuming the rate of sludge deposit as 30 litres/capita/year and with the given 1 year period of cleaning, we have

The quantity of sludge deposited = 30 × 180 × 1 = 5400 litres

Total required capacity of the tank

$$= 19800 + 5400 = 25200 \text{ litres} = 25.2\ m^3$$

Assuming the depth of the tank as 1.5 m, the cross-sectional area of the tank

$$= \frac{25.2}{1.5} = 16.8\ m^2$$

Using $L : B$ as 4 : 1 (given) we have

$$4B^2 = 16.8$$

$$B = \sqrt{\frac{16.8}{4}} = 2.04 \simeq 2\text{m}$$

$$L = 4 \times 2 = 8 \text{ m}$$

The dimensions of the tank will be 8 m × 2 m × (1.5 + 0.3 m) as overall depth with 0.3 m freeboard.

Hence, use a tank of size 8 m × 2 m × 1.8 m.

(ii) **Design of Soak Pit:** The soak pit or soak well can be designed by assuming the percolating capacity of the filtering media say as 1250 litres per cu-m per day.

$$\text{Sewage flow} = 19800\ l/d$$

$$\text{Percolation rate} = 1250\ l/m^3/d$$

∴ Volume (of filtering media) required for the soak pit

$$= \frac{19800\ l/d}{1250\ l/m^3/d} = 15.84\ m^3$$

If the depth of the soak pit is taken as 2 m, then

$$\text{Area of soak pit required} = \frac{15.84}{2} = 7.92\ m^2$$

$$\therefore \text{Diameter of soak pit required} = \sqrt{\frac{7.92 \times 4}{\pi}} = 3.17;\ \text{say } 3.20 \text{ m}$$

T5 : Solution

1. Total 5-day BOD present in sewage = $4.5 \times 10^6 \times 160 \times 10^{-6}$ = 720 kg/day
2. Volume of the filter media required = $720 \times 10^3/160$ = 4500 m^3
3. Surface area $= \dfrac{4.5 \times 10^6}{2000} = 2250\,m^2$
4. Depth of the bed required $= \dfrac{4500}{2250} = 2\,m$
5. Efficiency of the filter is given as,

$$\eta = \frac{100}{1 + 0.0044\sqrt{u}}$$

where, u = organic loading in kg/ha-m/day

Organic loading, u = 160 gm/m^3/day (given)

Now, 1 hectare-m = 10^4 m^2·m = 10^4 m^3

$\therefore$ $$u = \frac{160}{1000} 10^4 \text{ kg/ha-m/day} = 1600 \text{ kg/ha-m/day}$$

Hence, $$\eta = \frac{100}{1 + 0.0044\sqrt{1600}} = \frac{100}{1 + 0.176} = \frac{100}{1.176} = 85.03\,\%$$

T6 : Solution

1. Total BOD present in raw sewage = 3.79 ML × 240 mg/l = 909.6 kg
2. Now, filter volume required $= \dfrac{\text{Total BOD in raw sewage in kg}}{\text{Given BOD loading rate of 11086 kg/ha-m}}$

$$= \frac{909.6}{11086} \text{ ha-m} = 0.082 \text{ ha-m}$$

3. Now, assuming that 35% of BOD is removed in primary clarifier, we have
   The amount of BOD applied to the filter = 0.65 × 909.6 kg = 591.24 kg
4. Now, using equation for efficiency of trickling filter, we have

$$\eta = \frac{100}{1 + 0.0044\sqrt{\dfrac{Y}{V \cdot F}}}$$

where, Y = Total BOD applied to the filter in kg
= 591.24 kg

$\therefore$ V = Volume of the filter in ha-m = 0.082 ha-m

$$F = \frac{1 + \dfrac{R}{I}}{\left(1 + 0.1\dfrac{R}{I}\right)^2}; \text{ where } \frac{R}{I} = 1$$

$$\therefore \qquad F = \frac{1+1}{(1+0.1)^2} = \frac{2}{1.21} = 1.65$$

$$\therefore \qquad \eta = \frac{100}{1+0.0044\sqrt{\dfrac{591.24}{0.082\times 1.65}}} = 77.47\%$$

5. The amount of BOD left in the effluent = 591.24 (1 – 0.7747) kg = 133.21 kg

$\therefore$ BOD concentration in the effluent =

$$\frac{\text{Total BOD}}{\text{Sewage volume}} = \frac{133.21\times 10^6}{3.79\times\times 10^6}\text{mg}/l = 35.15\text{ mg}/l$$

T7 : Solution

$$\text{Volume of tank} = 20\times 15\times 5$$
$$= 1500\text{ m}^3 = 1500\times 10^3\text{ litre}$$
$$Q = 0.08\text{ m}^3/\text{sec} = 0.08\times 24\times 60\times 60\times 10^3\text{ litre/day}$$
$$= 6.912\times 10^6\text{ litre/day}$$

$$\text{Hydraulic retention time, } HRT = \frac{V}{Q}$$
$$= \frac{1500\times 10^3}{6.912\times 10^6} = 0.217\text{ day} = 5.21\text{ hrs}$$

$$\text{Sludge volume index } (SVI) = \frac{V_s}{\left(\dfrac{x_f}{1000}\right)} = \frac{250}{\left(\dfrac{2000}{1000}\right)} = 125\text{ m}l/\text{gm}$$

T8 : Solution

$$\text{Daily sewage flow} = Q = 180\times 35000\ l/\text{day} = 6300\text{ m}^3/\text{day}$$
$$\text{BOD of sewage coming to aeration} = Y_0 = 70\%\times 220\text{ mg}/l = 154\text{ mg}/l$$

($\because$ 30% BOD is removed in primary settling)

$$\text{BOD left in effluent} = Y_E = 15\%\times 220\text{ mg}/l = 33\text{ mg}/l$$

($\because$ Overall 85% BOD removal is desired)

$$\therefore\ \text{BOD removed in activated plant} = 154 - 33 = 121\text{ mg}/l$$

$$\therefore\ \text{Efficiency required in activated plant} = \frac{121}{154} = 0.79$$

For efficiency of 85-92%, we use F/M ratio as 0.4 to 0.3 and MLSS between 1500 to 3000 for convenctional activated plant. Since efficiency required is on lower side, we can use moderate figures for F/M ratio and MLSS.

$$\text{So let us adopt } F/M = 0.33$$
$$\text{Similarly adopt MLSS } (X_T) = 2000\text{ mg}/l$$

Using equation,
$$\frac{F}{M} = \frac{QY_0}{VX_T}$$

where,
$$\frac{F}{M} = 0.33 \quad \text{(assumed)}$$
$$Q = 6300\ \text{m}^3/\text{day}$$
$$Y_0 = 154\ \text{mg}/l = 154\ \text{gm/m}^3$$
$$X_T = 2000\ \text{mg}/l \quad \text{(assumed)}$$
$$0.33 = \frac{6300 \times 154}{V \times 2000}$$
$$V = \text{Volume of aeration tank}$$
$$= \frac{6300 \times 154}{2000 \times 0.33} = 1470\ \text{m}^3$$

(i) Check for aeration period or H.R.T. (t)
$$t = \frac{V}{Q} \times 24h = \frac{1470}{6300} \times 24\,h$$
$$= 5.6\ h \text{ (within the limits of 4 to 6}h) \quad \text{... OK}$$

(ii) Check for S.R.T. (θ_c)
$$V\,X_T = \frac{Q(Y_0 - Y_E)\theta_c}{1 + K_e \cdot \theta_c}$$

where,
$$V = 1470\ \text{m}^3$$
$$X_T = 2000\ \text{mg}/l$$
$$Q = 6300\ \text{m}^3/\text{d}$$
$$K_e = \text{Endogeneous respiration rate constant}$$
$$= 0.06\ d^{-1}$$
$$Y_0 = \text{BOD of influent in aeration tank} = 154\ \text{mg}/l$$
$$Y_E = \text{BOD of effluent} = 33\ \text{mg}/l$$

Substituting the values, we get
$$1470 \times 2000 = \frac{6300(154 - 33)\theta_c}{1 + 0.06 \times \theta_c}$$
$$\Rightarrow \quad 1 + 0.06\theta_c = \left(\frac{6300 \times 121}{1470 \times 2000}\right)\theta_c = 0.275\theta_c$$
$$1 + 0.06\,\theta_c = 0.259\,\theta_c$$
$$\Rightarrow \quad 1 = (0.259 - 0.06)\,\theta_c$$
$$\Rightarrow \quad 1 = 0.199\,\theta_c$$
$$\theta_c = \frac{1}{0.199} = 5.02\ \text{days} = 5\ \text{days} \quad \text{... OK}$$

As it lie between 5 to 8 days.

(iii) Check for volumetric loading
$$\text{Volumetric loading} = \frac{Q \cdot Y_0}{V}\ \text{gm of BOD/m}^3 \text{ of tank volume}$$
$$= \frac{6300 \times 154}{1386}\ \text{gm/m}^3 = 700\ \text{gm/m}^3 = 0.7\ \text{kg/m}^3 \quad \text{... OK}$$

The value is within the permissible range of 0.3 - 0.7 kg/m^3.

(iv) Check for return sludge ratio $= \dfrac{Q_R}{Q} = \dfrac{X_T \text{ (i.e. MLSS)}}{\dfrac{10^6}{SVI} - X_T}$

where, SVI = 100 ml/gm (assumed since this value should be in the range of 50-150)

X_T = 2000 mg/l

$$\Rightarrow \quad \frac{Q_R}{Q} = \frac{2000}{\left(\dfrac{10^6}{100} - 2000\right)}$$

= 0.25 (i.e. within the prescribed range of 25 to 50%)

We will, for conservative purposes, however provide 33% return sludge. The adapted SVI with this return sludge ratio is then computed as:

$$0.33 = \frac{2000}{\left(\dfrac{10^6}{SVI} - 2000\right)}$$

$$\Rightarrow \quad \frac{10^6}{SVI} - 2000 = \frac{2000}{0.33} = 6060$$

$$\Rightarrow \quad SVI = \frac{10^6}{8060} = 125 \quad \text{... OK}$$

The sludge pumps for bringing recirculated sludge from the secondary sedimentation tank will thus have a capacity = 33% × Q = 33% × 6300 m^3/d = 2100 m^3/d.

Tank dimensions. Adopt aeration tank of depth 3 m and width 4.5 m. The total length of the aeration channel required.

$$= \frac{\text{Total volume required}}{B \times D} = \frac{1470}{4.5 \times 3} \text{m}$$

= 108.9 m; say 111 m

Provide a continuous channel, with 3 aeration chambers, each of 37 m length. Total width of the unit, including 2 baffles each of 0.25 m thickness = 3 × 4.5 m + 2 × 0.25 = 14 m. Total depth provided including free-board of 0.6 m will be 3 + 0.6 = 3.6 m.

Overalll dimensions of the Aeration tank will be 37 m × 14 m × 3.6 m.

T9 : Solution

The quantity of sewage to be treated per day

= 1000 × 200 = 200000 litres

= 0.2 ml = 200 cu-m

The BOD content per day

= 0.2 ml × 300 mg/l = 60 kg

Now, the organic loading in the pond is 600 kg/ha/day

The surface area required

$$= \frac{60 kg/day}{600 kg/ha/day} = \frac{60}{600} \times 10^4 \text{ m}^2 = 1000 \text{ m}^2$$

Using L : B as 2 : 1 (given), we have

$$2B^2 = 1000$$

$$B = \sqrt{\frac{1000}{2}} = 22.36 \approx 22.4 \text{m}$$

Use $L = 44.8$

Using a tank with operational depth as 1.2 m, we have

$$\text{The provided capacity} = 22.4 \times 44.8 \times 1.2$$
$$= 1204.22 \text{ m}^3$$

Now, Capacity = Sewage flow per day × Detention time in days

∴ Detention time in days

$$= \frac{\text{Capacity in cu-m}}{\text{Sewage flow per day in cu-m/d}}$$

$$= \frac{1204.22}{200} = 6.02 = 6 \text{ days}$$

Hence, use an oxidation pond with length = 50 m; width = 25 m and overall depth = (1.2 + 1) = 2.2 m and a detention period of 6 days.

Design of Inlet Pipe : Assume an average velocity of sewage as 0.9 m/sec and daily flow for 8 hours only.

$$\text{Discharge} = \frac{200}{8 \times 60 \times 60} \text{ cu-m}$$

∴ Area of inlet pipe required

$$= \frac{\text{Discharge}}{\text{Velocity}} = \left(\frac{200}{8 \times 60 \times 60}\right) \times \frac{1}{0.9} \text{ m}^2 = 77.16 \text{ cm}^2$$

∴ Diameter of inlet pipe

$$= \sqrt{\frac{4 \times 77.16}{\pi}} = 9.91 \text{ cm; Say 10 cm}$$

Diameter of outlet pipe may be taken as 1.5 times that of the inlet; Say 15 cm.

T10 : Solution

$$\text{Volume of grit chamber, } V = 12 \text{ m} \times 1.5 \text{ m} \times 0.8 \text{ m}$$
$$= 14.4 \text{ m}^3$$
$$\text{Discharge in chamber, } Q = 720 \text{ m}^3/\text{hr}$$

So, $$\text{detention time, } t_d = \frac{V}{Q} = \frac{14.4 \text{m}^3 \times 60}{720 \text{m}^3} = 1.2 \text{ minutes}$$

$$\text{Surface loading rate, } V_s = \frac{\text{Discharge}}{\text{Surface area}} = \frac{720 \text{ m}^3/\text{hr}}{12 \text{m} \times 1.5 \text{ m}}$$
$$= 40 \text{ m}^3/\text{hr/m}^2$$
$$= 4000 \text{ Lph/m}^2$$

T11 : Solution

$$Q = 2670\ \text{m}^3/\text{d};$$

$$N_t = \text{No. } e^{-0.145t}$$

Let x be the no. of microorganisms (M.O.) present initially.

98% kill of M.O. imlies that at time 't' 2% of M.O. are still surviving

$\therefore$ M.O. surviving at time 't' $= \dfrac{2}{100}x$

$$\therefore \quad \frac{2}{100}x = x \cdot e^{-0.145t}$$

$$\therefore \quad t = 26.979 \text{ min} = 0.018736 \text{ days}$$

$$\therefore \quad \text{Volume} = Q.t$$

$$= 2670 \times 0.018736 = 50.0244\ \text{m}^3$$

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9 Design of Sewers & Sewerage System

T1 : Solution

Form Manning's formula, we have

$$v = \frac{1}{n} . r^{2/3} \sqrt{s}$$

At full depth, using capital letters, we have

$$V = \frac{1}{N} . R^{2/3} \sqrt{S}$$

Using
$$V = 0.90 \text{ m/sec}$$
$$N = 0.013$$
$$R = \frac{D}{4} = \frac{300}{4} = 75 \text{ mm} = 0.075 \text{ m}$$

We have
$$0.90 = \frac{1}{0.013}(0.075)^{2/3}\sqrt{S}$$

or
$$\sqrt{S} = \frac{0.90 \times 0.013}{0.178} = 0.0657$$

or
$$S = 0.0043 \text{ (i.e., } 4.3\%)^*$$

and
$$Q = A.\ V$$
$$= \frac{\pi}{4}(0.3)^2\, 0.90 \text{ cumecs} = 0.064 \text{ cumecs}$$

Now, at a depth (d) equal to 0.3 times the full depth (D), we have

$$\frac{d}{D} = 0.3 \quad \text{(variations of n to be neglected, as given)}$$

$$\frac{a}{A} = 0.252; \qquad \frac{r}{R} = 0.684$$

Now for the sewer to be the same self-cleansing at 0.3 depth (d), as it will be at full depth, we have the gradient (s_s) required as

$$s_s = \left(\frac{R}{r}\right)S = \frac{1}{0.684}S = \frac{1}{0.684} \times 0.0043 = 0.0063 \text{ (i.e., } 0.63\%)$$

$$= \frac{1}{158.73} \approx \frac{1}{159}$$

T2 : Solution

Sewage discharge computations

Average quantity of water consumed per day

$$= 170 \times 8000 \text{ litres/day}$$

Average quantity of water consumed in cumecs

$$= \frac{170 \times 8000}{1000 \times 24 \times 60 \times 60} \text{ cumecs} = 0.0157 \text{ cumecs}$$

Assuming that 80% of water consumed appears as sewage, we have

Average quantity of sewage discharge

$$= 0.8 \times 0.0157 \text{ cumecs} = 0.0126 \text{ cumecs.}$$

Assuming the peak sewage discharge to be three times the average discharge, we have

Maximum rate of sewage produced

$$= 0.3 \times 0.0126 \text{ cumecs} = 0.038 \text{ cumecs}$$

Storm run-off computations

Assuming the coefficient of run-off (K) for the area as 0.55, we have, by using Rational formula

Peak storm run-off

$$Q_p = \frac{1}{36} Kp_c.A = \frac{1}{36} \times 0.55 \times 4 \times 36 \text{ cumecs} = 2.2 \text{ cumecs}$$

Combined maximum discharge

$$= 2.2 + 0.038 = 2.238 \text{ cumecs}$$

Now, assuming that the sewer while carrying this combined peak discharge possesses 10% extra capacity, we have

The design discharge which the sewer should carry while flowing full

$$= \frac{2.238}{0.9} \text{ cumecs} = 2.49 \text{ cumecs}$$

Now, using Manning's formula, we have

$$Q = \frac{1}{N}.AR^{2/3}\sqrt{S}$$

Using the same gradient as in available i.e., $\frac{1}{900}$ as the first proposition, and Manning's $N = 0.013$ for smooth concrete or vitrified clay sewer, we have

$$2.49 = \frac{1}{0.013}\left(\frac{\pi D^2}{4}\right)\left(\frac{D}{4}\right)^{2/3}\frac{1}{\sqrt{900}}$$

Where D is the dia. of the equivalent circular section.

$$\therefore \quad D^{8/3} = \frac{2.49 \times 0.013 \times 4 \times 2.52 \times 30}{\pi}$$

or $$D^{8/3} = 3.12$$

or $$D = (3.12)^{\frac{3}{8}=0.375} = 1.533 \text{ m; say } 1.54 \text{ m}$$

Now, velocity generated $$= \frac{Q}{A} = \frac{2.49}{\frac{\pi}{4}(1.54)^2} = 1.33 \text{ m/sec}$$

This is satisfactory.

Note: The velocity can be increased further by steepening the slope and changing the size of the sewer accordingly. This will no doubt increase the ground excavations but will make the sewer more efficient at low flows, as the presently designed sewer may give very low velocities at low flows during non-monsoon seasons.

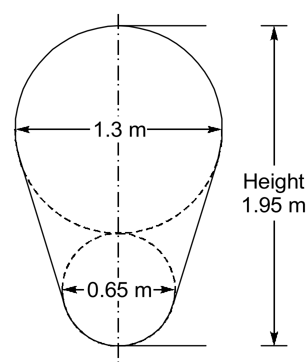
Check for lone sewage discharge

When maximum sewage is passing (once a day) in non-monsoon periods, the $\frac{q}{Q}$ will be equal to $\frac{0.038}{2.49} = 0.0152$. For this ratio of $\frac{q}{Q} = 0.0152$, from fig. we have

$$\frac{v}{V} = 0.3$$

or $$n = 0.3 \times 1.35 = 0.4 \text{ m/sec}$$ (which is just sufficient for non-silting)

Hence, in this sewer, deposition will take place during average and minimum lone sewage flow. The efficiency can be further increased by providing a steeper gradient, or by providing egg shaped section, which provide comparatively larger proportionate velocities at low depths.



(b) Equivalent egg shaped sewer

Now, $$D = 1.54 \text{ m}$$

If D' is the width of the standard equivalent egg shaped sewer, we have

$$D' = 0.84\, D$$

or $$D' = 0.84 \times 1.5 = 1.295 \text{ m say } 1.3 \text{ m}$$

Thus, the top width of the egg shape section = 1.3 m

and the height or vertical diameter of the egg shape section

$$= 1.5\, D' = 1.5 \times 1.3 = 1.95 \text{ m}$$

Hence use a standard egg shaped section 1.3 m × 1.95 m, as shown in figure.

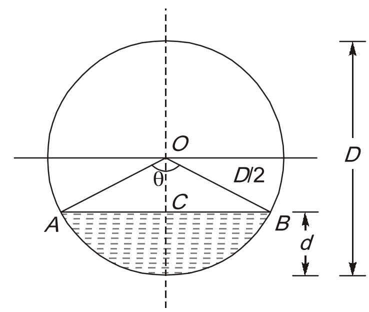
T3 : Solution

$$\text{Water supplied} = 100000 \times 200 = 20 \times 10^6 \text{ litres/day}$$

$$= \frac{20 \times 10^6}{10^3 \times 24 \times 3600} = 0.2315 \text{ cumecs}$$

Assuming that 80% of the water supplied to the town appears as sewage, we have average discharge in the sewer

$$= 0.8 \times 0.2315 = 0.185 \text{ cumecs}$$



At a peak factor of 3.

$$\text{Maximum discharge} = 3 \times 0.185 = 0.556 \text{ cumecs}$$

Since the sewer is to be designed as running 0.7 times the full depth,

$$\frac{d}{D} = 0.7 \text{ and } q = 0.556 \text{ cumecs}$$

For a sewer running partially full

$$\cos\frac{\theta}{2} = \frac{\frac{D}{2} - d}{D/2} = 1 - 2\frac{d}{D} = 1 - 2 \times 0.7 = -0.4$$

$$\therefore \quad \frac{\theta}{2} = 113.58°; \quad \theta = 227.16°; \quad \sin\theta = -0.7332$$

$$a = \frac{\pi}{4}D^2\left[\frac{\theta}{360} - \frac{\sin\theta}{2\pi}\right]$$

$$= \frac{\pi}{4}D^2\left[\frac{227.16}{360} - \frac{(-0.7332)}{2\pi}\right] = 0.587\, D^2$$

$$p = \pi D\frac{\theta}{360} = \pi D\frac{227.16}{360} = 1.982\, D$$

$$r = \frac{a}{p} = \frac{0.587\, D^2}{1.982\, D} = 0.296\, D$$

Now,
$$q = \frac{1}{n} a r^{2/3} S^{1/2}$$

$$0.556 = \frac{1}{0.013} \times 0.587\, D^2 (0.296\, D)^{2/3}\left(\frac{1}{500}\right)^{1/2}$$

$$D^{8/3} = 0.6190$$

$$\Rightarrow \quad D = 0.835 \text{ m}$$

Check for self cleansing velocity at maximum discharge

$$r = 0.296\, D = 0.296 \times 0.835 = 0.247 \text{ m}$$

$$v = \frac{1}{n} r^{2/3} S^{1/2} = \frac{1}{0.013}(0.247)^{2/3}\left(\frac{1}{500}\right)^{1/2} = 1.356 \text{ m/s}$$

This is much more than the self cleansing velocity of 60 cm/sec.

Check for self cleansing velocity at minimum discharge

Let us assume minimum flow equal to $\frac{1}{3}$ times the average flow.

$$\therefore \quad q_{min} = \frac{1}{3} \times 0.185 = 0.0617\,\text{m}^3/\text{s}$$

$$\text{Full flow discharge} = \frac{1}{n}\left(\frac{D}{4}\right)^{2/3} S^{1/2} \cdot \frac{\pi D^2}{4}$$

$$= \frac{1}{0.013}\left(\frac{0.835}{4}\right)^{2/3}\left(\frac{1}{500}\right)^{1/2} \times \frac{\pi}{4}(0.835)^2 = 0.6625\ \text{m}^3/\text{s}$$

$$\frac{q_{min}}{Q} = \frac{0.185}{3 \times 0.6625} = 0.093$$

$$V_{full} = \frac{0.6625}{\frac{\pi}{4}(0.835)^2} = 1.21\ \text{m/s}$$

For
$$\frac{q}{Q} = 0.093$$

$$\frac{v}{V} = 0.622$$

$$v = 0.753\ \text{m/s} > 0.6\ \text{m/s (Self cleansing Velocity)}$$

T4 : Solution

(i) **Rectangular section:**

Let D = Depth of rectangular section.

$\therefore$ Width, $B = 1.5\,D$

$$A = D \times 1.5\,D = 1.5\,D^2$$

$$P = D + 1.5D + D = 3.5D$$

$$R = \frac{A}{P} = \frac{1.5D^2}{3.5D} = 0.428D$$

$$Q = A \times V = A \times \frac{1}{N} R^{2/3} S^{1/2}$$

or
$$Q = (1.5D^2) \times \frac{1}{N}(0.428D)^{2/3} S^{1/2}$$

or,
$$Q = 0.852\,D^{8/3} \times \frac{S^{1/2}}{N} \quad \text{... (i)}$$

(ii) **Circular Section:**

Let, d = Diameter

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

$$P = \pi d$$

$$\therefore \quad R = \frac{A}{P} = \frac{\pi}{4} d^2 \times \frac{1}{\pi d}$$

$$R = \frac{d}{4}$$

Now,

$$Q = A \times V$$

$$= A \times \frac{1}{N} R^{2/3} S^{1/2}$$

$$= \frac{\pi}{4} d^2 \times \frac{1}{N} \left(\frac{d}{4}\right)^{2/3} S^{1/2}$$

$$Q = 0.312\, d^{8/3} \times \frac{S^{1/2}}{N} \qquad \text{... (ii)}$$

For the two sewers to be hydraulically equivalent Q, N and S are the same. Hence from equations (i) and (ii), we get

$$0.852\, D^{8/3} = 0.312\, d^{8/3}$$

or,

$$\left(\frac{D}{d}\right)^{8/3} = 0.366 \quad \text{or} \quad \frac{D}{d} = (0.366)^{3/8}$$

$$\frac{D}{d} = 0.686$$

Hence,

$$D = 0.686\, d$$

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12 Noise Pollution

T1 : Solution

L_{eq} is defined as the constant noise level, which, over a given time, expands the same amount of energy, as is expanded by the fluctuating levels over the same time.

The L_{eq} is calculated as

$$L_{eq} = 10\log_{10}\sum_{i=1}^{i=n}(10)^{\frac{Li}{10}} \times t_i$$

where, L_i = The noise level of any i^{th} sample

t_i = Time duration of i^{th} sample expressed as fraction of total sample time

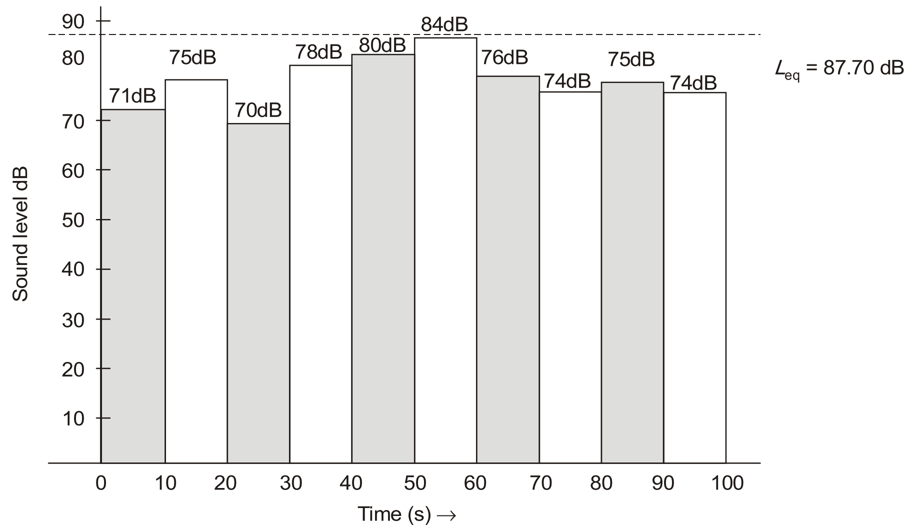
Here, Total sample time = 100 sec

Time (in s)	10	20	30	40	50	60	70	80	90	100
Noise (dBA) L(t)	71	75	70	78	80	84	76	74	75	74
$t_i = \frac{t}{100}$	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1
$\frac{L_i}{10}$	7.1	7.5	7.0	7.8	8.0	8.4	7.6	7.4	7.5	7.4
$\Sigma(10)^{\frac{L_i}{10}} \times t_i$	590167630.6									

$\therefore$
$$L_{eq} = 10\log_{10}\sum_{1}^{10}(10)^{\frac{L_i}{10}} \times t_i$$

$$= 10 \times \log_{10}(59016760.6)$$

$$= 87.70 \text{ dB}$$



90
80
70
60
50
40
30
20
10
Sound level dB
71dB
75dB
70dB
78dB
80dB
84dB
76dB
74dB
75dB
74dB
L_{eq} = 87.70 dB
0
10
20
30
40
50
60
70
80
90
100
Time (s) →

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