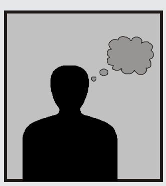
ESE GATE PSUs

State Engg. Exams

MADE EASY Workbook 2026



Detailed Explanations of Try Yourself *Questions*

Civil Engineering

Environmental Engineering

MADE EASY PUBLICATIONS

1 Water Demand

Detailed Explanation of Try Yourself Questions

T1 : Solution

(i) Arithmetic increase method

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

$\bar{x} = 5.25$

$\therefore$ Population in 2020 $\quad P_{2020} = P_{2014} + 6\bar{x}$

$= 47000 + 6\,(5250)$

$= 78500$

(ii) Geometric increase method

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

r = 16.07%

Population in 2020, $\quad P_{2020} = P_{2014} \times (1 + r)^6$

$= 47 \times (1 + 0.1607)^6 = 114.5$ 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, $P_{2020} = P_{2014} + 6\overline{x} + \frac{6(6+1)}{2}y$

$= 47 + 6x\,(5.25) + 21\,(0.33) = 85.43$ thousands

T2 : Solution

Statement-3 does not directly relate to the design period. It refers to the design criteria used for hydraulic structures.

Specically, the design flood or return period used to design structures like dams, spillways etc.

Design period refers to the duration for which a structure is planned to serve effectively, considering factors like economy, durability and future demand. Hence correct option is (c).

T3 : Solution

Average daily requirement of water $= 70{,}000 \times 150 = 10500\ m^3/d$

This water shall be withdrawn in 10 hours. So

$$\text{Intake load} = \frac{10500}{60 \times 10 \times 60} = \frac{10.5}{36} = \frac{3.5}{12} = 0.29\ m^3/s$$

So, option (b) is correct.

T4 : Solution

Distribution system is to be designed for greater of maximum hourly demand or maximum daily demand and fire demand.

Units	Design parameters
Water treatment units	Maximum daily demand
Main supply pipes (water mains)	Maximum daily demand
Wells and Tubewells	Maximum daily demand
Demand Reservoir	Average annual demand
Distribution System	Maximum hourly demand coincident + draft } Max.

Note: Coincident draft = Max. daily demand+ Fire demand

So, option (d) is correct.

T5 : Solution

Birth Rate: The number of live births per thousand of population per year.

Death Rate: The ratio of deaths to the population of a particular area or during a particular period of time, usually calculated as the number of deaths per one thousand people per year.

Migration: Migration also affects the no. of deaths and births in an area. Migrations take into account emigration from and immigration to the city. So, option (b) is correct.

T6 : Solution

Estimation of population by geometrical increase method

$$P_n = P_0\left(1+\frac{r}{100}\right)^n$$

$$P_{2020} = P_{2000}\left(1+\frac{35}{100}\right)^2$$

Where, $P_0 = 82300$, $n = 2$, $r = 35\%$

$$= 82300\left(1+\frac{35}{100}\right)^2 = 149991.75 \simeq 1{,}50{,}000$$

So, option (c) is correct.

T7 : Solution

Given:

Year	Population in thousand	Increase in Population in thousand	%age Increase in Population (Growth rate)
1981	82		
		25	30.49
1991	107		
		19	17.76
2001	126		
		16	12.70
2011	142		

Now, Geometric mean of past growth rates,

$$r = (r_1 r_2 r_3)^{1/3} = (30.49 \times 17.76 \times 12.70)^{1/3}$$

$$= 19.02\% \text{ per decade}$$

By geometric increase method,

$$P_{2051} = P_{2011} \times \left[1+\frac{2}{100}\right]^4 = 142000 \times \left[1+\frac{19.02}{100}\right]^4$$

$$P_{2051} = 284950$$

T8 : Solution

Given: $P_o = 60{,}000$ $\quad t = 0$

$P_1 = 1{,}30{,}000$ $\quad t_1 = 10$ years

$P_2 = 2{,}00{,}000$ $\quad t_2 = 20$ years

$P_s = 3{,}20{,}000$

Population at any time, t

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

Where,

$$m = \frac{P_s - P_o}{P_o} = \frac{320000-60000}{60000} = 4.33$$

$$n = \frac{1}{t_1}\log_e\left[\frac{P_o(P_s-P_1)}{P_1(P_s-P_o)}\right] = \frac{1}{10}\log_e\left[\frac{60000(320000-130000)}{130000(320000-60000)}\right] = -0.1087$$

$\Rightarrow$ For $t = 70$ years,

$$P = \frac{320000}{1+4.33\log_e^{-1}(-0.1087\times 70)} = 319315$$

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2 Sources of Water Supply & Well Hydraulics

Detailed Explanation *of* Try Yourself Questions

T1 : Solution

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

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

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

$H = 100$ m

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

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

Using Thiem's equation for unconfined aquifers, we have

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

$\Rightarrow$ $\pi k = 3.193 \times 10^{-5} \Rightarrow k = 1.016 \times 10^{-5}$ 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

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

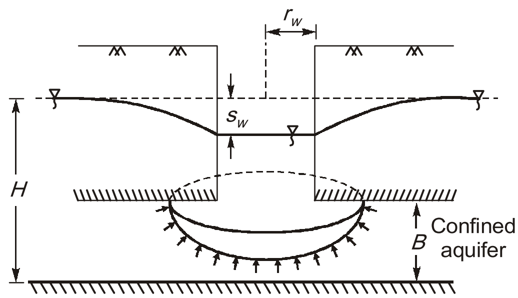
$\Rightarrow$ $2488.3 = 94^2 - h_w^2$

$\Rightarrow$ $h_w = 79.67$ m

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

T2 : 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, $$Q_s = vA$$

$\Rightarrow$ $$Q_s = kiA$$

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

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

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

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

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

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

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

Using this equation and putting numerical values

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

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

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

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

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

T3: Solution

The radius of influence (R) is commonly known as radius of drawdown, is the radius measured from the centre of the wall to the point at which drawdown is measured.

It is given by $R = 3000\ S\sqrt{k}$

where, R = radius of drawdown

S = drawdown

k = permeability of aquifer soil

So, option (b) is correct.

T5: Solution

Storage Coefficient:

- In general, storage coefficient (S) is defined as the volume of water that aquifer releases or stores per unit surface area per unit decline or rise of water table.
- For an unconfined aquifer, storage coefficient is generally taken as equal to the specific yield.
- For a confined aquifer, it is defined as equal to the volume of water released from the aquifer of full height and unit cross-sectional area when the piezometric surface declines by unity.

  So, option (b) is correct.

T6: Solution

Pumping Test: A pump is first of all, installed so as to draw sufficient supplies of water from the open well, and to cause heavy drawdown in its water level. The rate of pumping is then changed and so adjusted that the water level in the well become constant.

Pressure Test: The purpose of pressure test is to investigate the various limits of the pipeline such as reliability, maximum capacity, leaks, fittings and pressure. It is a test in which a test pipeline is filled with water and is allowed to stand for some time and then pressure is applied.

Recuperation test: Although the pumping test gives accurate value of safe yield, it becomes very difficult to adjust the rate of pumping. So as to keep the well water level constant. In such circumstances recuperation test is adopted.

In this method, the water is first of all drained from the well at a fast rate so as to cause sufficient drawdown.

The pumping is then stopped the water level in the well will start rising the time taken by the other measured level is taken noted.

Jar test: The common test which is performed to determine the optimum dosage of coagulent is known as jar test. So, option (b) is correct.

T7: Solution

- Horizontal tunnels constructed through water bearing stratum at the shallows depth of 3m to 5m, drilled into ground with the help of drilling equipment are known as infiltration galleries.
- Infiltration wells are brick masonary structures constructed along the banks of river for tapping their water seeping from their bottom.
- Springs are the locations where water from the aquifers flows to surface.

  So, option (c) is correct.

T8: Solution

Discharge from well (without interference)

$$Q = \frac{2\pi kB(H-h_w)}{\log_e\left(\frac{R}{r_w}\right)} = \frac{2\pi\times1.5\times10^{-3}\times12\times4.5}{\log_e\left(\frac{300}{0.15}\right)}$$

$$Q = 0.0669 \text{ m}^3/\text{s}$$

Discharge from well (with interference)

$$Q' = \frac{2\pi kB(H-h_w)}{\log_e\left(\frac{R}{r_w}\times\frac{R}{D}\right)} = \frac{2\pi\times1.5\times10^{-3}\times12\times4.5}{\log_e\left(\frac{300}{0.15}\times\frac{300}{150}\right)}$$

$$Q' = 0.06136 \text{ m}^3/\text{s}$$

$$\text{\%age reduction in discharge} = \left(\frac{Q-Q'}{Q}\right)\times100 = \frac{0.0669-0.06136}{0.0669}\times100 = 8.28\%$$

So, option (a) is correct.

T9: Solution

Discharge is given by

$$q = kiA = 0.001\times2\times20$$

(Assuming L = 1 m)

$$= 0.04 \text{ m}^3/\text{sec} = 0.04\times1000\times60 = 2400 \text{ lpm}$$

So, option (a) is correct.

T10: Solution

1. Overpumping of groundwater can lower the water table, which in turn reduces the baseflow to nearby lakes and streams. Even if rainfall is normal, surface water bodies may dry up due to lack of groundwater support.
2. Over extraction can cause intrusion of saline water (especially in coastal areas) or draw in contaminated water from other layers, leading to water quality degradation.
3. Excessive pumping can remove the water pressure that supports soil structure, causing the land above to compact and subside.
4. There is no strong scientific correlation that ground water pumping leads to increased seismic activity. Earthquakes are usually caused by tectonic plate movements, not ground water withdrawl.
5. As water table drops, more energy and deeper drilling is needed to extract water, increasing the cost. Hence, the correct option is (b).

T11 : Solution

The dependable discharge of a lone circular open well is most easily improved by deepening the well, which increases contact with the aquifer and allows more water to enter the well. Hence, the correct option is (c).

T12 : Solution

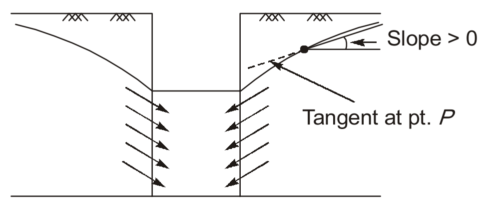
When a confined aquifer is tapped by non-pumping wells (i.e. wells that are not drawing water), the water level in these wells rises above the top of the aquifer due to hydrostatic pressure. This level is known as the piezometric level.

A piezometric surface is an imaginary surface that represents the level to which water will rise in tightly caused wells that penetrate a confined aquifer. Hence, the correct option is (c).

T13 : Solution

Any point on drawdown curve will have pressure equal to atmospheric pressure but slope of the tangent at this point being > 0, the curve exponentially gets lowered towards the well from where water is being drawn and thus it is called as DRAWDOWN CURVE.

So, option (c) is correct.



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3 Quality Characteristics of Water

Detailed Explanation *of* Try Yourself Questions

T1 : Solution

Hardness is due to multixalent cations.

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

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

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

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

T2: Solution

Alkalinity of water sample is due to presence of $[HCO_3^-]$ and $[CO_3^{2-}]$

Total alkalinity = 1 mole of $[HCO_3^-]$

+ 2 mole of $[CO_3^{2-}]$

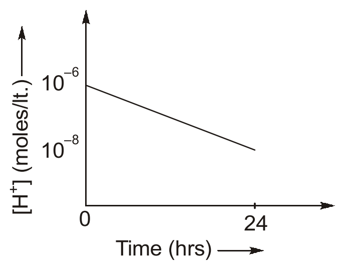
in terms of $CaCO_3$

$$= (2 \times 10^{-3} \times 50 + 2 \times 3.04 \times 10^{-4} \times 50) \times 10^3 \text{ mg}/l$$

$$= 130.4 \text{ mg}/l \text{ as } CaCO_3$$

T3: Solution

Given, initial pH value = 6.0
after 24 hour, pH = 8.0



$$[H^+]_{at\ t=0} = 10^{-6}$$
$$[H^+]_{t=24\ hr.} = 10^{-8}$$
$$\therefore \text{Mean of } [H^+] = \frac{10^{-6}+10^{-8}}{2}$$
$$= \frac{10^{-6}(1+10^{-2})}{2}$$
$$= \frac{1.01\times10^{-6}}{2} = 0.505 \times 10^{-6}$$

So, $\text{Mean of pH} = -\log_{10}(0.505\times10^{-6})$
$$= -\log_{10}(0.505) + 6$$
$$= 0.2967 + 6 = 6.296$$

T4: Solution

$$[pH]_A = 4.2 = -\log\,[H^+]_A$$
$\Rightarrow$ $[H^+]_A = 10^{-4.2}$
As $[H^+]\times[OH^-] = 10^{-14}$
$\therefore$ $[OH^-]_A = 10^{-9.8}$
$\Rightarrow$ $[OH^-]_B = 2\times10^{-9.8}$
Now, $p[OH]_B = -\log\,[OH^-]_B$
$\Rightarrow$ $p[OH]_B = 9.8 - \log 2$
As, $pH + pOH = 14$
$\Rightarrow$ $p[H]_B = 4.5$

T5: Solution

Virusus are the smallest biological structure, containing all information necessary for its reproduction. They are obligate parasites, and as such, require a host in which to live.
So, option (d) is correct.

T6: Solution

Specific conductance and concentration of TDS are not related on one-to-one basis, only ionized substances contribute to specific conductance. Organic molecules and compounds that dissolve without ionizing are not measured. Additionally, the magnitude of specific conductance is influenced by the valancy of the ions in solution, there mobility and relative numbers.

So, option (a) is correct.

T8: Solution

$$(\text{Total hardness, Alkalinity})_{min} = \text{Carbonate hardness}$$

So, option (b) is correct.

T9: Solution

Threshold odour number is given as

$$\text{TON} = \frac{A+B}{A}$$

where A is the volume of odorous water (mL) and B is the volume of odor-free water required to produce a 200 mL mixture.

$$\therefore \quad \text{TON} = \frac{12.5+187.5}{12.5} = 16$$

So, option (d) is correct.

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4 Treatment of Water

Detailed Explanation of Try Yourself Questions

T1 : Solution

The operational troubles which are generally encountered during the operation of rapid gravity filters are.

Air binding: Development of negative pressure in the bottom sand layer of rapid sand filter leads to air binding. The negative pressure so developed, tends to release the air dissolved in water. It causes the formation of bubbles, which stick to the sand grains, and affects the working of filter.

Mud balls: The mud from the atmosphere usually accumulates on the sand surface, so as to form a dense mat. During inadequate washing of the filter, this mud may sink down into the sand bed. This mud then sticks to the sand grains and other arrested impurities, thereby forming mud balls.
The fine sand contained in the top layers of the filter bed, shrinks and causes the development of shrinkage cracks in the sand bed.
So, option (a) is correct.

T2: Solution

Filters are also able to remove even particles of size smaller than the size of voids present in filter however these smaller particles are retained in mat layer.
So, option (d) is correct.

T3: Solution

During an epidemic of infective hepatitis, the supplied water is super chlorinated because of avoidance of future contamination but the spore forming bacteria, do not cause hepatitis, it is caused by virus.
So, option (c) is correct.

T4 : Solution

Settling velocity (Stoke's Law), $v_s = \dfrac{gd^2}{18\nu}(G_s - 1) = \dfrac{9.81 \times \left(5 \times 10^{-3}\right)^2}{18 \times 1.01 \times 10^{-2}} \times 1.65$

$= 0.0022 \text{ m/sec} = 0.22 \text{ cm/s}$

So, option (a) is correct.

T5 : 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 = $12 \times 0.403 = 4.836$ mg/L of lime

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

T6 : Solution

Let us assume $d < 0.1$ mm

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

$$V_S = \frac{10 \times 10^6 \times 10^{-3} \times 10^3}{300 \times 24 \times 3600} \text{mm/s} = 418(2.65-1)d^2\left(\frac{3 \times 26 + 70}{100}\right)$$

$\Rightarrow$ $d^2 = \dfrac{0.3858}{1020.75}$

$\Rightarrow$ $d = 0.01944$ mm

Hence our assumption is correct.

T7 : Solution

$$\text{Total surface area} = \frac{\text{Design flow rate}}{\text{Design loading rate}} = \frac{0.5}{200/(24 \times 60 \times 60)}$$

$$= \frac{24 \times 60 \times 60 \times 0.5}{200} = 216 \text{ m}^2$$

So, option (c) is correct.

T8 : 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.
So, option (c) is correct.

T9 : Solution

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

T10 : Solution

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

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

$$\text{Chlorine usage} = \frac{9 \times 10^6}{25000 \times 10^3} = 0.36 \text{ mg}/l$$

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

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

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

So, option (c) is correct.

T11 : 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 m^2

Total surface area required

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

$$= \frac{103.95 \times 10^3}{6 \times 24} = 721.875 \text{ m}^2$$

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
So, option (c) is correct.

T12 : Solution

$$\text{Flow rate, } Q_0 = 0.2 \text{ m}^3/\text{sec}$$
$$\text{Plan area, } A = \text{LB} = 32 \times 8 = 256 \text{ m}^2$$
$$\text{(OFR) over flow rate} = \frac{Q_0}{A} = \frac{0.2}{256} = 7.8125 \times 10^{-4} \text{ m/s}$$

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

$$v_s = \frac{(G-1)\gamma_w d^2}{18\mu}$$
$$= \frac{(2.5-1)9.81\times10^{-3}(25\times10^{-6})^2}{18\times0.01\times10^{-3}\times10^2}$$
$$= 5.10 \times 10^{-4} \text{ m/sec}$$
$$\eta_{\text{removal}} = \frac{v_s}{OFR}\times100$$
$$= \frac{5.10\times10^{-4}}{7.8125\times10^{-4}}\times100 = 65.28\% \simeq 65\%$$

So, option (d) is correct.

T13 : Solution

During disinfection variations of micro-organism is given by

$$N_t = N_o e^{-kt}$$
$$N_t = \text{No. of micro-organism at time } t$$
$$N_o = \text{No. of micro-organism at time } 0$$

So, disinfection efficiency at any time 't',

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

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 $\eta_t = 99\%$

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

So, option (a) is correct.

T14 : 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 = $\frac{28 \times 72}{50}$ = 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.

T15 : 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.

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

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

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

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

Assuming laminar flow, $v = \dfrac{9.81(16\times10^{-10})(2.65-1)}{18\times1.02\times10^{-6}} = 14.106 \times 10^{-4}$ m/s $= 1.41$ mm/s

$\therefore$ $Re = \dfrac{14.106\times10^{-4}\times4\times10^{-5}}{1.02\times10^{-6}} = 55.318 \times 10^{-3} < 1$

$\Rightarrow$ Assumptions of laminar flow is true.

T16 : Solution

n' = Porosity of expanded bed

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

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

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

So, option (a) is correct.

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5 Distribution System

Detailed Explanation of Try Yourself Questions

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

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

$$\text{Balancing storage} = A + B$$
$$= 7.05 + 2.1 = 9.15 \text{ ML}$$

(ii) Intermittant supply

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

$$\text{Balancing storage} = A + B$$
$$= 7.05 + 1.2 = 8.25 \text{ ML}$$

7 Wastewater Quality Characteristics

Detailed Explanation of Try Yourself Questions

T1: Solution

$BOD_5/COD \geq 0.63$ means waste water is biodegradable. So, option (c) is correct.

T2: Solution

$$\underset{\text{(in water)}}{Mn^{2+} + 2OH^-} \longrightarrow \underset{\text{White ppt.}}{Mn(OH)_2 \downarrow}$$

...in absence of Dissolved Oxygen

$$Mn^{2+} + 2OH^- + \frac{1}{2}O_2 \longrightarrow MnO_2^- \downarrow$$

Brown ppt.
... in presence of Dissolved Oxygen

So, option (c) is correct.

T3: Solution

The Winkler Method is a technique used to measure dissolved oxygen in fresh water systems. It uses titration to determine dissolved oxygen in the water sample.

Reagent List:

- 2 ml Manganese sulphate
- 2 ml alkali-iodide-azide
- 2 ml concentrated sulfuric acid
- 2 ml starch solution
- Sodium thiosulfate

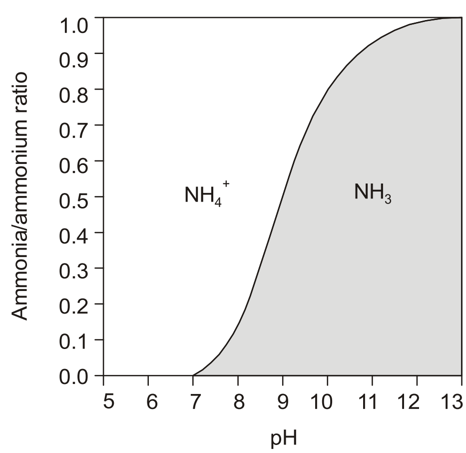
Procedure:

1. A 300-mL glass Biological Oxygen Demand (BOD) stoppered bottle is filled till brim with sample water.

2. Manganese sulphate is added to the collection bottle by inserting the calibrated pipette just below the surface of the liquid.
3. Alkali-iodide-azide reagent is also added in the same manner.
4. The sample is mixed by inverting it several times. If oxygen is present, a brownish-orange cloud of precipitate or floc will appear.
5. Then concentrated sulfuric acid is added. At this point, the sample is "fixed" and can be stored for up to 8 hours if kept in a cool, dark place.
6. This sample is then titrated with sodium thiosulfate to a pale straw color.
7. Then starch solution is added so a blue color forms.
8. Titration is done until the sample turns clear.
9. The concentration of dissolved oxygen in the sample is equivalent to the number of milliliters of titrant used. Each mL of sodium thiosulphate added in steps 6 and 8 equals 1 mg/L dissolved oxygen.

   So, option (c) is correct.

T4: Solution



From the above curve, it is evident that at pH 7.0, NH_4^+ will be predominant.
So, option (c) is correct.

T5: Solution

(i) Total BOD $= 1000 \times 75 \text{ gm} = 75000 \text{ gm}$

$\therefore$ Population equivalent $= \dfrac{75000}{55} = 1364$

(ii) Total suspended solids $= 40000 \times 1800 \times 10^{-3} \text{ gm}$
$= 72000 \text{ gm}$

$\therefore$ Population equivalent $= \dfrac{72000}{90} = 800$

T6: Solution

Given: $(BOD)_4 = 0.75\,L_o$

$\Rightarrow$ $L_o(1 - 10^{-k\times 4}) = 0.75\ L_o$

$0.25 = 10^{-4k}$

$\Rightarrow$ $-4k = \log_{10}{}^{0.25}$

$k = 0.15\ \text{day}^{-1}$

T7: 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**

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

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

Give: Vol. of Waste water = 5 mL

DO of waste water = 0 mg/L

Vol. of pure water = 300 – 5 = 295 mL

DO of pure water = 9.2 mg/L

$$\text{Initial DO of diluted sample,}DO_i = \frac{V_{waste}\times BOD_5 + V_{pure}\times BOD_5}{V_{waste}+V_{pure}} = \frac{5\times 0 + 295\times 9.2}{300} = 9.0467\ \text{mg/L}$$

After incubating the bottle for 5 day, DO of mixture was found 5.0 mg/L

$\therefore$ $DO_f = 5.0$ mg/L

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

$$= (9.0467 - 5.0)\times\frac{300}{5} = 242.8\ \text{mg / L}$$

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8 Disposal of Sewage Effluents

Detailed Explanation of Try Yourself Questions

T1 : 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,

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

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

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

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

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

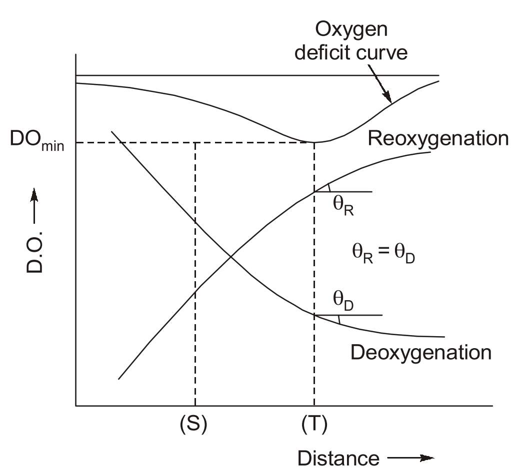
$\Rightarrow$ $$5 = 32.857\ [1 - 10^{-0.1094 \times t}]$$

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

Distance from downstream mixing point = Velocity × Time

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

T2: Solution

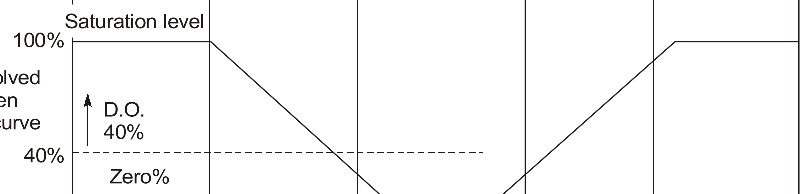


- Before point T, rate of aeration < rate of degradation, so DO continuously decrease from S till T.
- At point T, rate of aeration = Rate of degradation. Hence DO is minimum at T.
- After T, rate of aeration > Rate of degradation, so DO starts increasing.

Hence, DO is minimum at some point downstream of S. So, option (d) is correct.

T3: Solution

Table: Showing zone of pollution along a river stream

	← Zones of pollution →				
	Clear water	Zone of degradation	Zone of active decomposition	Zone of recovery	Zone of clearer water
Dissolved oxygen sag curve					
Physical Indices	Clear, water, no bottom sludge, no colour	Floating solids; bottom sludge present, colour getting turbid	Darker and greyish colour, evolution of gases like CH_4, CO_2, H_2S etc. lot of sludge coming to the surface forming an ugly scum layer at top	Turbid with bottom sludge	Clear water with no bottom sludge
Fish presence	Ordinary fish like game, pan, food & forage etc. present.	Tolerant fishes like carp, buffalo, gary, etc. present	No fish present	Tolerant fish like carp, buffalo, etc. are present	Ordinary fish like game, pan, food, and forage, etc. present
Bottom Animals					
Algae & Protozoa etc. called plankton					

So, option (d) is correct.

T4: Solution

Self purification of streams include physical, chemical and biological process.

A. **Physical processes:**
   Dilution; sedimentation and resuspension; filtration; gas transfer; and heat transfer.

B. **Chemical Processes:**
   Chemical conversion (oxidation and reduction)

C. **Biological process:**

Metabolic processes in micro-organisms. So, option (b) is correct.

T5: Solution

When sewage is applied continuously on a piece of land, the soil pores or voids may get filled up and clogged with sewage matter retained in them. Thus free circulation of air will be prevented and anaerobic conditions will develop within the pores.

Sewage sickness is the condition when soil pores get filled up and clogged with sewage matter due to continuous application of waste water effluents. This develops anaerobic conditions and foul gases like methane, carbon-dioxide and hydrogen sulphide are evolved.

In order to prevent sewage sickness:

(i) Sewage should be given primary treatment
(ii) The soil chosen for effluent irrigation/sewage farming should be sandy or loamy.
(iii) A proper under drainage system (open jointed drains) should be designed.
(iv) Land should be given rest for some time and ploughed thoroughly.
(v) Rotation of crops to be followed.
(vi) Shallow depths of water should be applied.

So, option (d) is correct.

T6: Solution

Critical dissolved oxygen (DO) deficit occurs in zone of active decomposition.
(**Refer table of T3 Solution).** So, option (b) is correct.

Here,
Zone-I : Zone of degradation
Zone-II : Zone of active decomposition
Zone-III : Zone of recovery
Zone-IV : Zone of clear water

T7: Solution

$$\text{Temperature of the mix, } T = \frac{Q_w T_w + Q_s T_s}{Q_w + Q_s}$$

$$= \frac{8640 \times 25 + 1.2 \times 24 \times 60 \times 60 \times 15}{8640 + 1.2 \times 24 \times 60 \times 60} = 15.77°\text{C}$$

So, option (b) is correct.

T8: Solution

$$DO_{mix} = \frac{Q_W DO_W + Q_R DO_R}{Q_W + Q_R}$$

$$= \frac{(1.10 \times 2.00) + (8.70 \times 8.3)}{1.10 + 8.70} = 7.6 \text{ mg/L}$$

So, option (c) is correct.

9 Treatment of Waste Water

Detailed Explanation of Try Yourself Questions

T1 : Solution

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

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

Amount of BOD entering, $W_1 = Q_0S_0$ kg/day

$= 2200 \times 10^3 \times 120 \times 10^{-6} = 264$ kg/day

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

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

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

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

$V_1 = 637\ m^3$

$\because$ Depth $= 1.5$ m

Plan area $= \dfrac{637}{1.5}$

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

$D = 23.3$ 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

$$v_h = 4.5\sqrt{gd(S_s - 1)} = 4.5\sqrt{9.81 \times 0.2 \times 10^{-3} \times (2.65 - 1)}$$
$$= 0.26 \text{ m/s}$$

(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, $Q = v_h \times A$

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

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

$$D \times B = A$$

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

$\Rightarrow$ $B = 1.78$ 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

$$3B^2 = 12.6$$

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

$\therefore$ Provide width = 2.1 m; and
Provide length of the tank = 6 m
$\therefore$ 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

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

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

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

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.
Sewage flow = 19800 l/d
Percolation rate = 1250 l/m^3/d
$\therefore$ Volume (of filtering media) required for the soak pit

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

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

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

$\therefore$ Diameter of soak pit required $= \sqrt{\dfrac{7.92 \times 4}{\pi}} = 3.17$; say 3.20 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 = $10^4\ m^2 \cdot m = 10^4\ m^3$

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

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

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

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

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

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

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

Volume of tank = $20 \times 15 \times 5$

= $1500 \text{ m}^3 = 1500 \times 10^3$ litre

$Q = 0.08 \text{ m}^3/\text{sec}$ = $0.08 \times 24 \times 60 \times 60 \times 10^3$ litre/day

= 6.912×10^6 litre/day

Hydraulic retention time, $HRT = \frac{V}{Q}$

$$= \frac{1500\times 10^3}{6.912\times 10^6} = 0.217 \text{ day} = 5.21 \text{ hrs}$$

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

T8 : Solution

Daily sewage flow = $Q = 180 \times 35000$ l/day = 6300 m^3/day

BOD of sewage coming to aeration = $Y_0 = 70\% \times 220$ mg/l = 154 mg/l

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

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

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

$\therefore$ BOD removed in activated plant = 154 – 33 = 121 mg/l

$\therefore$ 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 conventional activated plant. Since efficiency required is on lower side, we can use moderate figures for F/M ratio and MLSS.

So let us adopt $F/M = 0.33$

Similarly adopt MLSS $(X_T) = 2000$ 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 = 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)

$$t = \frac{V}{Q} \times 24h = \frac{1470}{6300} \times 24\,h$$

$= 5.6\ h$ (within the limits of 4 to 6h) ... OK

(ii) Check for S.R.T. (θ_c)

$$VX_T = \frac{Q(Y_0 - Y_E)\theta_c}{1 + K_e \cdot \theta_c}$$

where, $V = 1470 \text{ m}^3$

$X_T = 2000$ mg/l

$Q = 6300 \text{ m}^3\text{/d}$

K_e = Endogeneous respiration rate constant

$= 0.06\ d^{-1}$

Y_0 = BOD of influent in aeration tank = 154 mg/l

Y_E = BOD of effluent = 33 mg/l

Substituting the values, we get

$$1470 \times 2000 = \frac{6300(154 - 33)\theta_c}{1 + 0.06 \times \theta_c}$$

$$\Rightarrow \quad 1 + 0.06\theta_c = \left(\frac{6300 \times 121}{1470 \times 2000}\right)\theta_c = 0.275\theta_c$$

$$1 + 0.06\,\theta_c = 0.259\,\theta_c$$

$$\Rightarrow \quad 1 = (0.259 - 0.06)\,\theta_c$$

$$\Rightarrow \quad 1 = 0.199\,\theta_c$$

$$\theta_c = \frac{1}{0.199} = 5.02 \text{ days} = 5 \text{ days} \qquad \text{... OK}$$

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 $= \dfrac{Q_R}{Q} = \dfrac{X_T \text{ (i.e. MLSS)}}{\dfrac{10^6}{\text{SVI}} - X_T}$

where, SVI = 100 m*l*/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:

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

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

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

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

$= 0.2 \text{ m}l = 200 \text{ cu-m}$

The BOD content per day

$= 0.2 \text{ m}l \times 300 \text{ mg}/l = 60 \text{ kg}$

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

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

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

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

$$= 40 \text{ m}^3/\text{hr/m}^2$$

$$= 40000 \text{ Lph/m}^2$$

So, option (d) is correct.

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' $= \frac{2}{100}x$

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

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

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

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

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10 Design of Sewers & Sewerage System

Detailed Explanation of Try Yourself Questions

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 $V = 0.90$ 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 $S = 0.0043$ (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

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

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

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

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.

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

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.

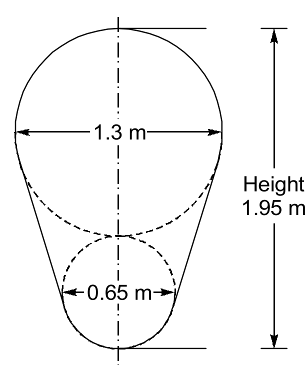
Check for lone sewage discharge

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 $n = 0.3 \times 1.35 = 0.4$ 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, $D = 1.54$ 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.

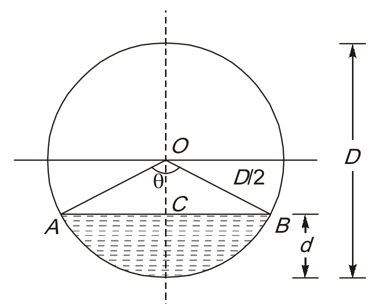
T3 : Solution

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

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

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

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

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

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

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

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

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

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

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

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

⇒ $$D = 0.835 \text{ m}$$

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.

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

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

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

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

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

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

(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,
$$Q = A \times V$$

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

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

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

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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13 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									

$$\therefore \quad L_{eq} = 10\log_{10}\sum_{1}^{10}(10)^{\frac{L_i}{10}} \times t_i = 10 \times \log_{10}(59016760.6) = 87.70 \text{ dB}$$