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PART - A

2 Marks Questions

1.
Sol. Factors affecting the growth of population :
The growth of population at a particular area depends on many factors, some of them are as follows:
 - Scope of employment
 - Connectivity of the area with other cities/locations
 - Availability of water
 - Presence of infrastructure like electricity, telephones, sanitation etc.
 - Status of law and order
2.
Sol.
 - Typhoid fever (caused by salmonella typhi bacteria)
 - Cholera (caused by vibrio-cholera bacteria)
 - Poliomyelitis (caused by polio virus)
 - Giardiasis (Caused by giardia lambalia)
3.
Sol.

Parameters	Accetable limit
TDS	500 ppm
Chlorides	250 ppm
Fluorides	1 ppm
Nitrates	45 ppm
4.
Sol. Most probable number (MPN) is an index to suggest the quantity of coliforms present in water. This index is calculated by multiple tube permentation technique.
5.
Sol.

	Acceptable limit	Rejection limit
(a) pH	7 - 8.5	< 6.5 & > 9.2
(b) Turbidity	1 NTU	5 NTU
(c) Hardness	200 ppm	600 ppm
(d) Flouridrs	1 ppm	1.5 ppm
6.
Sol. Carbonate hardness : If bicarbonates and carbonates of calcium and magnesium are present in water, the water is rendered hard temporarily, as this hardness can be removed to some extent by simple boiling or to full extent by adding lime to the water. Such a hardness is known as temporary hardness or Carbonate Hardness.
7.
Sol. Non-carbonate hardness : If sulphates, chlorides and nitrates of calcium or magnesium are present in water, they cannot be removed at all by simple boiling and therefore, such water requires special treatment for their hardness removal. Such a hardness is known as permanent hardness or non-carbonate hardness.
It is the total hardness in excess of the alkalinity. If alkalinity is equal to or greater than the total hardness, there is no-carbonate hardness.
8.
Sol. It is the sum of total ammonia nitrogen and the organic nitrogen.
9.
Sol.
 - Lime - soda process
 - Zeolite method
 - Ion exchange method
10. **Match List-I with List-II :**

List-I	List-II
--------	---------



- | | |
|-----------------------------|--------------------------|
| A. Grit chamber | 2. Zone setting |
| B. Secondary setting tank | 1. Stoke's law |
| C. Activated sludge process | 3. Aerobic |
| D. Trickling filter | 4. Contact stabilisation |

11.

Sol. It is very necessary to remove disease causing micro-organisms from water and disinfection is the process which used to remove this type of micro-organisms. Hence it is most important step in water treatment.

12.

- Sol.** (i) Minor Methods
- Disinfection using ozone (O_3)
 - Disinfection using $KMnO_4$
- (ii) Major Methods
- Chlorination

13.

- | | | |
|-------------|------------------------------------|------------------------------------|
| Sol. | List-I
(Treatment units) | List-II
(Detention time) |
| | A. Grit chamber | 2. Two minutes |
| | B. Primary sedimentation | 3. Two hours |
| | C. Activated sludge | 1. Six hours |
| | D. Sludge digestion | 4. Twenty days |

14.

Sol.

Given, Dose of chlorine = 0.5 mg/l; Daily flow = 3,00,000 l/d
 Quantity of chlorine daily required = $(3,00,000) \times (0.5 \times 10^{-6}) \text{ kg} = 0.15 \text{ kg/day}$

Amount of bleaching powder required = $0.15 \times \frac{100}{30} = 0.5 \text{ kg/day}$

15.

Sol.

Given,
 Population of town = 80,000
 Water supply to town = 150 litres/capita/day
 Water supplied to town = $80000 \times 150 = 12 \times 10^6 \text{ litre/day}$

$\Rightarrow$ Chlorine usage/dose = $\frac{5 \times 10^6 \text{ mg/day}}{12 \times 10^6 \text{ litres/day}} = 0.417 \text{ mg/l}$

$\therefore$ Residual chlorine after 10 minutes contact = 0.20 mg/l

We know, Chlorine dose, mg/L = Chlorine demand, mg/L + Chlorine residual, mg/L

$\Rightarrow$ Chlorine demand, mg/L = $0.417 \text{ mg/L} - 0.20 \text{ mg/L} = 0.217 \text{ mg/L}$

16.

Sol. Given:

$P = 20,000$; BOD_5 of persons = 70 gm/c; BOD_5 of industry = 450 mg/l;

Discharge of industries = 50000 l/d

Total BOD_5 of combined sewage = $(20,000 \times 70) + (450 \times 50,000 \times 10^{-3}) = 1422500 \text{ gm/day}$

Population equivalent = $\frac{\text{Total } BOD_5}{80 \text{ gm/day/person}} = \frac{1422500}{80} = 17781.25$

17.

Sol. The reservoir trap efficiency is defined as the ratio of deposited sediment to the total sediment in flow for a given period within the reservoirs economic life time.

18.



- Sol.** Layouts of Distribution System
- Dead end or tree system
 - Grid system or reticular system
 - Ring or circular system
 - Radial system
19. **Sol.** The corrosion of metal pipes leads to formation of ferrous hydroxide, which on reacting with oxygen forms ferric hydroxide. This ferric hydroxide gets deposited on the pipe surface. This deposit of ferric hydroxide is known as tuberculation and it increases the roughness of the pipe and thereby reducing its carrying capacity.
20. **Sol.** In septic tank suspended growth anaerobic reaction takes place. Detention time = 24 hrs.
21. **Sol.** In Aerobic treatment process oxygen is required, whereas in anaerobic treatment process oxygen is not required.
22. **Sol.** The amount of oxygen required to oxidise bio-degradable organic material known as B.O.D.
23. **Sol.**
- | List-I (Process) | List-II (Biological agent) |
|---|---|
| A. Oxidation ditch | 3. Aerobic bacteria (suspended culture) |
| B. Waste stabilisation pond | 1. Facultative bacteria |
| C. Imhoff tank | 2. Anaerobic bacteria |
| D. Rotating Biological Contractor (RBC) | 4. Aerobic bacteria (attached culture) |
24. **Sol.** It is the burning of refuse in the presence of very sparse air or oxygen, so that the wastes do not completely burn, but only smoulder to produce charcoal like product.
25. **Sol.** The most common reason is that distilled water itself does not contain any source of nutrients and minerals, so that it is not potable, Which are beneficial for human body.

PART - B

5 Marks Questions

26. **Sol.** **Design Period:** The number of years for which the system is to be adequate is called design period.
- Population forecasting:** Design of water supply and sanitation scheme is based on the projected population of a particular city, estimated for the design period. Any underestimated value will make system inadequate for the purpose intended, similarly overestimated value will make it costly.
- The present and past population record for the city can be obtained for the census population record. After collecting these population figures, the population at the end of design period is predicted using various methods as suitable for that city considering the growth pattern followed by the city.
- Incremental Increase method:** This method is modification of arithmetical increase method and it is suitable for an average size town under normal condition where the growth rate is found to be in increasing order. While adopting this method, the increase in increment is considered for calculating future population. The incremental increase is determined for each decade from the past population and the average value is added to the present population along with the average rate of increase.

Hence, population after n^{th} decade is $P_n = P + nX + \left\{ \frac{(n+1)n}{2} \right\} Y$

Where,
 P_n = Population after n^{th} decade
 X = Average increase of population of known decades
 Y = Average Incremental increase of known decades

- Advantages of incremental increase method:
 1. This method gives/predict more correct value of population.
 2. This method embodies the advantage of arithmetic average method and geometrical average method.

27.

Sol.

Given: $[Na^+] = 20 \text{ mg/l}$; $[Mg^{2+}] = 60 \text{ mg/l}$; $[Ca^{2+}] = 45 \text{ mg/l}$

$$\text{Hardness as CaCO}_3 \text{ (mg/l)} = \left[\frac{[Ca^{2+}]}{\text{Equivalent Wt. of Ca}^{2+}} \times \text{Equivalent Wt. of CaCO}_3 + \frac{[Mg^{2+}]}{\text{Equivalent Wt. of Mg}^{2+}} \times \text{Equivalent Wt. of CaCO}_3 \right]$$

$$\Rightarrow \text{Hardness} = \left[\frac{45}{20} \times 50 + \frac{60}{12} \times 50 \right] = 362.5 \text{ mg/l}$$

28.

Sol.

- (i) **Hardness in water:** Hardness in water is that characteristic of water which prevents the formation of leather of foam when such hard water is mixed with soap. Hardness is basically caused due to the presence of multivalent cations (like Ca^{+2} , Mg^{+2} etc.) in water.

Carbonate hardness: This type of hardness is caused due to the presence of carbonates and bicarbonates of calcium and magnesium. Carbonate hardness is also referred to as temporary hardness as this hardness can be removed by simple boiling (to some extent) or by adding lime to water (to full extent). When this water is boiled, carbon dioxide gas escapes out and the insoluble calcium carbonate gets precipitated. Temporary hardness cause deposition of calcium scales in the boilers.

Non-carbonate hardness: This type of hardness is caused due to the presence of sulphates, chlorides and nitrates of calcium or magnesium. This type of hardness cannot be removed by simple boiling and such waters require special treatment for softening Therefore it is also called as permanent hardness.

Total hardness: Total hardness of water is the sum of total carbonate hardness (or alkalinity, whichever is less) and the non-carbonate hardness.

Thus, Total hardness = Min (Carbonate hardness and alkalinity) + Non-carbonate hardness

- (ii) Given, Alkalinity = 200 mg/l as $CaCO_3$
 Concentration of $Ca^{+2} = 160 \text{ mg/l}$
 Concentration of $Mg^{+2} = 40 \text{ mg/l}$
 $pH = 8.1$

$$\text{Total hardness} = 160 \times \frac{50}{20} + 40 \times \frac{50}{12} = 400 + 166.67 = 566.67 \text{ mg/l as CaCO}_3$$

Now carbonate hardness = Minimum of (Total hardness or Alkalinity) = 200 mg/l as $CaCO_3$

$$\therefore \text{Non-carbonate hardness} = \text{Total hardness} - \text{Carbonate hardness} \\ = 566.67 - 200 = 366.67 \text{ mg/l as CaCO}_3$$

29.

Sol.

$$210 \text{ gm CO}_3^{2-} \text{ will have } = \frac{210}{60/2} = \frac{210}{30} = 7 \text{ gm equivalent}$$

Here, $7 \text{ gm equivalent of CO}_3^{2-} = 7 \text{ gm equivalent of CaCO}_3$



$$122 \text{ gm of } \text{HCO}_3^- \text{ will have } = \frac{122}{61/1} = 2 \text{ gm equivalent of } \text{HCO}_3^-$$

$$2 \text{ gm equivalent of } \text{HCO}_3^- = 2 \text{ gm equivalent of } \text{CaCO}_3$$

$$68 \text{ gm of } \text{OH}^- \text{ will have } = \frac{68}{17/1} = 4 \text{ gm equivalent of } \text{CaCO}_3$$

$$\therefore 4 \text{ gm equivalent of } \text{OH}^- = 4 \text{ gm equivalent of } \text{CaCO}_3$$

$$\text{Now, Total alkalinity of water} = 2 + 2 + 4 = 8 \text{ gm equivalent of } \text{CaCO}_3$$

$$\therefore \text{Weight in gm of } \text{CaCO}_3 = \frac{100}{2} \times 8 = 400 \text{ gm of } \text{CaCO}_3$$

$$\therefore \text{Total alkalinity of water} = 400 \text{ gm expressed as } \text{CaCO}_3$$

30.

Sol. The presence of nitrogen in water may be found to occur in one or more of the following forms.

Type	Indication
1. Free ammonia	Indicates recent pollution.
2. Organic ammonia (Alubiminoid)	Indicates quantity of nitrogen before decomposition has started.
3. Nitrite (NO_2^-)	Indicate partly decomposed condition.
4. Nitrate (NO_3^-)	Indicate old pollution (fully oxidised)

- Presence of nitrogen in water indicates presence of organic matter.
- Presence of free ammonia in water indicates its recent pollution. Free ammonia should not be more than 0.15 mg/l and it can be measured by simply boiling the water and measuring the liberated ammonia by distillation process.
- The presence of organic ammonia indicates decomposition has just started and limiting value in potable water is 0.3 mg/l and it is measured by boiling a sample of a already boiled water plus strong alkaline like KMnO_4 and measuring the ammonia so liberated.
- Among all form of nitrogen, nitrite is highly dangerous, hence its permissible limit is zero and it is measured by colour matching technique. The colour for nitrite is developed by sulphuric acid Naphthamine.
- Nitrate is not harmful because it is fully oxidised. But too much of nitrate affects infants. The reason behind this is it causes blue baby disease or methemoglobinemia. Nitrate concentration should not be more than 45 mg/l.
- Nitrate concentration is measured by colour matching technique. Colour is formed by phenol-di-sulphonic + Potassium hydroxide.
- Free Ammonia + Organic Ammonia = Kjeldahl Ammonia.

31.

Sol. **Classification of Sedimentation**

Depending upon the concentration of suspended matter and the characteristics of particles, sedimentation process can be classified in following four categories.

- First type (Discrete)** : In this, particle have little or no tendency to flocculate in a dilute suspension. Such particles are known as discrete particles.
 - They settle as individual entities and there is no significant interaction with neighbouring particles.
 - The process is as in case of inorganic sand particles.
- Second type (Flocculant)** : In the second type, there is settling of flocculent particles in a dilute suspension.
 - Particles agglomerate during settling which causes increase in size, shape and density resulting in settlement at faster rate.
- Third type (Hindered or zone)** : The third type of sedimentation occurs when flocculent suspensions settle in an intermediate concentration.
 - Due to proximity of particles with each other, they tend to remain in fixed positions with respect to each other and settle as a large mass rather than as individuals.



- (iv) **Fourth type (compression settling) :** The fourth type is settlement of flocculent particles in a very high concentration.
- Due to high concentration, particles come into physical contact and form a structure and further settling occurs due to compression of the structure.
 - The process is also known as compression settling.

32.

Sol. Comparison of Alum and Iron Salts as Coagulant

The alum and the iron salts are having their own advantages and disadvantages as summarized below:

- Iron salts produce heavy floc and can, therefore, remove much more suspended matter than the alum.
- Iron salts, being good oxidising agents, can remove hydrogen sulphide and its corresponding tastes and odours from water.
- Iron salts can be used over a wider range of pH values.
- Iron salts cause staining and promote the growth of iron bacteria in the distribution system.
- Iron salts impart more corrosiveness to water than that which is imparted by alum.
- The handling and storing of iron salts require more skill and control, as they are corrosive and deliquescent. Whereas, no such skilled supervision is required for handling alum.
- Iron salts are used as coagulants more frequently for treating sewage and alum is used more frequently for treating raw water.

33.

Sol. Given that

Discharge,

$$Q = 10^6 \text{ litres per day}$$

Detention period,

$$t = 2.5 \text{ hours}$$

Velocity of flow,

$$v_h = 8 \text{ cm/min}$$

Depth of water and sediment = 4.5 m

(i) We know that

$$v_h \times 1 = \text{Length of tank}$$

$\Rightarrow$

$$L = 8 \times 10^{-2} \times 2.5 \times 60$$

$\Rightarrow$

$$L = 12 \text{ m}$$

(ii) When an allowance of 1.5 m is made for sediment, then depth of water, $H = 4.5 - 1.5 = 3 \text{ m}$

But,

$$v_h = \frac{Q}{BH}$$

$\Rightarrow$

$$B = \frac{Q}{Hv_h}$$

$\Rightarrow$

$$B = \frac{10^3 \text{ m}^3}{\frac{24 \times 60 \text{ min}}{3 \times 8 \times 10^{-2}}} = 2.9 \text{ m}$$

(iii)

$$\text{Overflow rate} = \frac{Q}{BL} = \frac{10^6 \times (10^{-3} / 24)}{2.9 \times 12} = 1.1973 \text{ m}^3 / \text{hr} / \text{m}^2$$

34.

Sol.

Water treated per day = 20,000 cum.

$$= 20,000 \times 10^3 \text{ litres}$$

$$= 20 \times 10^6 \text{ litres} = 20 \text{ M.l.}$$

Chlorine consumed per day = 8 kg = 8 M.mg.

$\therefore$ Chlorine used per litre of water

$$= \frac{8 \text{ M.mg}}{20 \text{ M.l.}} = 0.4 \text{ mg/l}$$

$\therefore$ Hence, the given chlorine dosage = 0.4 mg/l.

Now, residual chlorine left = 0.2 mg/l (given)



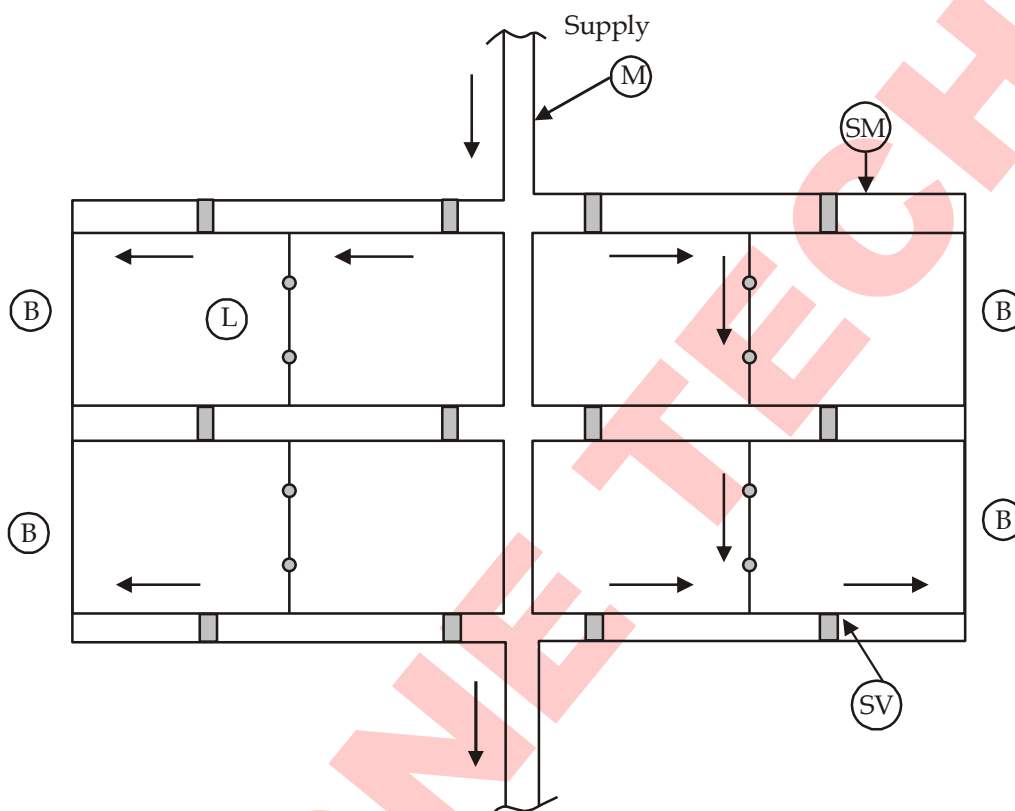
Hence, actual chlorine dosage, which has reacted in water, i.e. chlorine demand of water.

$$= 0.4 - 0.2 = 0.2 \text{ mg/l.}$$

35.

Sol.

Grid Iron System: This system is also known as reticular system. In this, one main pipe runs through centre and branches and laterals run in grid pattern which are interconnected. Since mains, branches and laterals are interconnected hence dead ends are eliminated and water reaches at different locations through more than one route.



(M) Main pipe

(L) Laterals

(SM) Sub Main

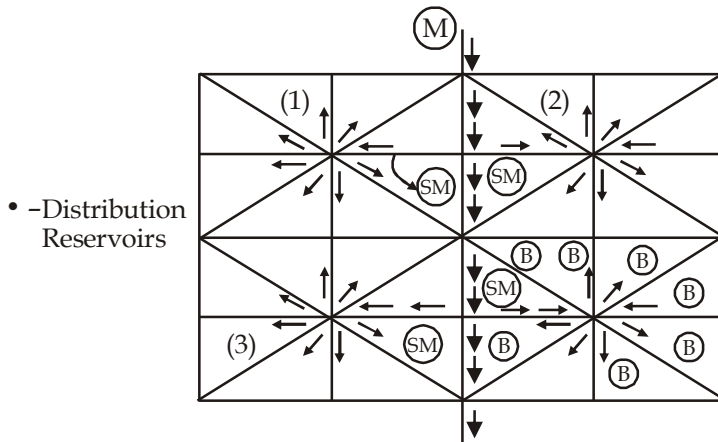
(SV) Sluice valve

(B) Branches

Radial System: In this a very big area is divided in several zones and at the centre of each zone a distribution reservoir is kept. This method gives higher service head and efficient water distribution. The supply pipes are laid radially ending towards the periphery. It gives quick service and calculation of pipe sizes are easy.

1, 2, 3 and 4 are distribution reservoirs.





36.

Sol.

- Average standard BOD of domestic sewage is 80 gms per person per day.
- The number of person which produce the amount of BOD at the rate of 80 gms per person per day equal to that produced by industrial sewage is called population equivalent of industrial sewage.
- Industrial wastewater are generally compared with per capita normal domestic wastewater, so as to rationally charge the industries for the population caused by them. The strength of the industrial sewage is, thus worked out as below.

$$\left[\begin{array}{c} \text{Standard BOD (5 days)} \\ \text{of industrial sewage} \end{array} \right] = \left[\begin{array}{c} \text{Standard BOD (5 days) of domestic} \\ \text{sewage per person per day} \end{array} \right] \times \left[\begin{array}{c} \text{Population} \\ \text{equivalent} \end{array} \right]$$

37.

Sol.

(i) Waste per 1000 kg of milk received

$$= \frac{246}{1,13,000} \times 1000 \text{ cum} = 2.177 \text{ m}^3$$

(ii) BOD per 1000 kg of milk

$$\begin{aligned} &= 1400 \text{ mg/l} \times 2.177 \text{ m}^3 \\ &= \frac{1400 \times 1000}{1000 \times 1000} \times 2.177 \text{ kg} \\ &= 30.478 \text{ kg.} \end{aligned}$$

38.

Sol.

$$K_{D(20^\circ)} = 0.12$$

$$Y_5 = 250 \text{ mg/l (at } 20^\circ\text{C)}$$

$$Y_{15} \text{ at } 25^\circ\text{C} = ?$$

$$\text{Using } Y_5 \text{ at } (20^\circ) = L \left[(1 - 10)^{-K_{D(20^\circ)} \cdot t} \right], \text{ we have}$$

$$250 = L \left[(1 - 10)^{-0.12 \times 5} \right] = L \left[1 - \frac{1}{(10)^{0.6}} \right]$$

$$0.7488 L = 260$$

$$L = 333.86 \text{ mg/l}$$

or

Now,

$$K_{D(25^\circ)} = K_{D(20^\circ)} [1.047]^{T - (20^\circ)}$$

$$= 0.12 [1.047]^{25 - 20}$$

$$= 0.12 \times 1.258 = 0.151$$

$$\begin{aligned}
 \text{Now, } Y_{15 \text{ at } 25^\circ} &= L \left[1 - (10)^{K_D(25^\circ) \cdot t} \right] \\
 &= 336.86 \left[1 - (10)^{-0.151 \times 15} \right] \\
 &= 333.86 \left[1 - \frac{1}{(10)^{2.265}} \right] \\
 &= 332.05 \text{ mg/l.}
 \end{aligned}$$

39.
Sol.

Given: $k_D = 0.434k$ (At 20°C); $k_D = 0.434 \times 0.20 = 0.0868$; $\text{BOD}_5 = 150 \text{ mg/l}$ (At 20°C)

$$L \left(1 - (10)^{-5 \times k_D} \right) = 150 \text{ mg/l}$$

Here L is the ultimate BOD,

$$L \left\{ 1 - (10)^{-5 \times 0.0868} \right\} = 150$$

$$\Rightarrow L = 237.39 \text{ mg/l}$$

We know,

$$k_{D_T} = k_{D(20^\circ)} [1.047]^{T-20^\circ}$$

$$\therefore k_{D(15^\circ)} = 0.0868 [1.047]^{15-20}$$

$$\Rightarrow k_{D(15^\circ)} = 0.069$$

8 days BOD at 15°C is given by,

$$\begin{aligned}
 \text{BOD}_8 &= L \left[1 - (10)^{-k_D \times 8} \right] \\
 &= 237.39 \left[1 - (10)^{-0.069 \times 8} \right] = 170.79 \text{ mg/l}
 \end{aligned}$$

40.

- Sol.**
- (i) **Sewage:** Mixture of water and liquid waste generated due to different activities in the community is called as sewage.
 - (ii) **Sewers:** Pipes or conduits required to carry the sewage from one point to another one are termed as sewers.
 - (iii) **Sewerage:** The entire method of collecting, treating and disposing the sewage is referred as sewerage system.

Rational Method: It has been evolved to estimate the peak drainage discharge and is used for design of sewers for small area,

$$Q = \frac{1}{36} k \cdot p_c \cdot A$$

$Q \rightarrow$ Peak drainage discharge in m^3/s

$k \rightarrow$ runoff coefficient

$P_c \rightarrow$ critical rainfall intensity of design frequency

$A \rightarrow$ Catchment area contributing to runoff in ha.

Disadvantage: Results given by this method for larger areas are generally erroneous and misleading as in reality run-off depends on many factors which are inter-related.

41.

Sol.

Significance of $\frac{\text{BOD}}{\text{COD}}$ ratio:

The typical values of ratio of $\frac{\text{BOD}}{\text{COD}}$ for untreated municipal wastewater are in the range from 0.3 to

0.80. If the $\frac{\text{BOD}}{\text{COD}}$ ratio for untreated wastewater is 0.50 or greater, the waste is considered to be easily treatable by biological means. If the ratio is below about 0.30, either the waste may have some toxic components or acclimated microorganisms may be required in its stabilization.

Type of wastewater	$\frac{\text{BOD}}{\text{COD}}$
--------------------	---------------------------------

- | | |
|---------------------------|-------------|
| 1. Untreated | 0.3 – 0.80 |
| 2. After primary settling | 0.4 – 0.60 |
| 3. Final effluent | 0.10 – 0.30 |

As BOD is predominantly a biochemical parameter, it generally reflects biodegradability of organic matter, thus making $\frac{\text{BOD}}{\text{COD}}$ ratio, a good indicator of the proportion of biochemically degradable

organic matter to total organic matter. Thus $\frac{\text{BOD}}{\text{COD}}$ is typically a measurement used to describe the organic composition of wastewater.

42.

Sol. **Given:** $P = 50,000$; $S = \frac{1}{700}$; $n = 0.01$

Let water demand be 200 lpcd and assuming 80% of water supplied emerges as sewage.

$$\text{Design discharge, } Q_D = \frac{3 \times (0.80 \times 200 \times 50,000 \times 10^{-3})}{24 \times 3600} \text{ m}^3 / \text{s} = 0.278 \text{ m}^3 / \text{s}$$

Let us assume sewer is running full,

$$\Rightarrow Q_D = \frac{1}{n} \times A \times R^{2/3} \times S^{1/2}$$

$$\text{or, } 0.278 = \frac{1}{0.01} \times \left(\frac{\pi}{4} \times D^2 \right) \times \left(\frac{D}{4} \right)^{2/3} \times \left(\frac{1}{700} \right)^{1/2}$$

$$\therefore D = 0.582 \text{ m}$$

$$\text{Velocity of flow, } v = \frac{1}{n} \times R^{2/3} \times S^{1/2} = \frac{1}{0.01} \times \left(\frac{0.582}{4} \right)^{2/3} \times \left(\frac{1}{700} \right)^{1/2} = 1.0456 \text{ m/s}$$

As velocity is greater than 0.8 -0.9 m/s at full depth flow condition, then self-cleansing velocity will be achieved. Hence design is fine.

Design Parameters: Slope = 1 in 700
Diameter, $D = 0.582 \text{ m}$

43.

Sol.

Given: $Q = 3.5 \text{ MLD}$; $L = 30 \text{ m}$; $B = 9 \text{ m}$; $H = 4 \text{ m}$; $\text{MLSS} = 1800 \text{ mg/l}$; $S_0 = 130 \text{ mg/l}$; $X_0 = 150 \text{ mg/l}$

$$\text{BOD loading i.e., } \text{OLR} = \frac{Q_0 S_0}{V} = \frac{3.5 \times 10^6 \times (130 \times 10^{-6})}{(30 \times 9 \times 4 \times 10^{-4})} \text{ kg / ha - m / day}$$

$$\text{OLR} = 4212.963 \text{ kg/ha-m/day}$$

$$\frac{F}{M} = \frac{Q_0 S_0}{VX} = \frac{3.5 \times 10^6 \times 130 \times 10^{-6}}{(30 \times 9 \times 4) \times (1800 \times 10^{-6}) \times 10^3} = 0.23$$

$$\text{Aeration period, } t = \frac{V}{Q} = \frac{30 \times 9 \times 4}{3.5 \times 10^6 \times 10^{-3}} \text{ day} = 7.41 \text{ hours}$$



Sludge Age:

$$\text{Volume, } V = 30 \times 9 \times 4 \text{ m}^3 = 1080 \text{ m}^3$$

$$X = 1800 \text{ mg/l}$$

$$Q = 3.5 \times 10^6 \text{ l/d}$$

$$X_0 = 150 \text{ mg/l}$$

$$\theta_c = \frac{V \cdot X}{Q X_0} = \frac{(1080 \times 10^3) \times 1800}{150 \times 3.5 \times 10^6} \text{ day}$$

$$\theta_c = 3.70 \text{ days}$$

44.

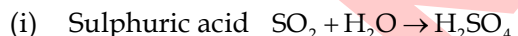
Sol. The activities involved with the management of solid wastes from the point of generation to final disposal has been grouped into six functional elements.

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

45.

Sol. Secondary air pollutants : The large no. of primary air pollutants present in the air often react with one-another aided by sun-light and water vapours to give rise to totally new types of pollutants called as secondary air pollutants.

Various acid :



Sulphur Dioxide (SO₂) : SO₂ when inhaled, affects the mucous membranes of human body. It is a primary air pollutant which in combination with water vapours forms a much hazardous secondary air pollutant viz. sulphuric acid (H₂SO₄). SO₂ increases the breathing rate and causes oxygen deficit in human body. SO₂ usually originates from oil refineries, chemical factories and combustion of fuels. Thermal power plants also emit SO₂ in huge proportions.

Carbon Monoxide (CO) : CO mainly originates from combustion of fuels in vehicles. Incomplete combustion of organic matter in limited supply of air gives rise to CO.

CO has a very high affinity towards blood hemoglobin (Hb) rather than with oxygen. Thus when inhaled, it replaces oxygen and forms carboxy-hemoglobin which is of no use for the human respiratory system. Thus when nearly half of the blood hemoglobin gets used up to form carboxy-hemoglobin, death results.

46.

Sol. In the natural environment, in the biosphere, there in fact, exists a perfect balance or equilibrium between the various organisms, and this is known as ecological balance.

When rivers get polluted, their ecological balance is disturbed but situation does not remain same forever due to the following self purification processes:

1. **Dilution and dispersion :** This results in diminishing the concentration of pollutant and thus reduces potential nuisance.
2. **Sedimentation:** The settleable solids if present will settle down on bed.



3. **Sunlight:** Due to sunlight in the process of photosynthesis, oxygen is released. This oxygen helps in oxidation of organic matter.
4. **Oxidation:** This is the most important action responsible for self purification of rivers. The deficiency of oxygen created is filled up by atmospheric oxygen.
5. **Reduction:** It occurs due to hydrolysis of organic matter settled at the bottom either chemically or biologically.

There is a limit to self purification and heavy pollution may cause unrepairable damage.

Management and Disposal of Radioactive Wastes:

(i) Initial Treatment of Waste:

- **Vitrification:** Long term storage of radioactive waste requires the stabilization of the waste into a form which will neither react nor degrade for extended periods of time. It is theorized that one way to do this might be through vitrification.
- **Ion Exchange:** It is common for medium active wastes in the nuclear industry to be treated with ion exchange or other means to concentrate the radioactivity into a small volume.

(ii) Long term management of waste:

- **Above-ground disposal:** It involves sealing waste in steel cylinder, which is placed in concrete cylinder which act as radiation shield.
- **Geologic Disposal:** The basic concept is to locate a large, stable geologic formation and permanently isolate nuclear waste from human environment.

PART - C

20 Marks Questions

47.

- Sol.** Per capita demand is the annual average amount of daily water required by one person and includes the domestic use, industrial use and commercial use, public use, water thefts etc. It is estimated as,

$$\text{Per capital demand (in l/c/d)} = \frac{\text{Total yearly water requirement of city}}{365 \times \text{Design population}}$$

$$\text{Arithmetic increase method: } P_n = P_0 + n.\bar{x}$$

$$\bar{x} = 4375 = \frac{P_{1970} + P_{1980} + P_{1990} + P_{2000} + P_{2010}}{5}$$

$$P_{2040} = P_{2010} + 3.\bar{x}$$

$$P_{2040} = [32.5 + 3 \times 4.375] \times 10^3 = 45625$$

Year	Population (in 10^3)	Increase in Pop. (in 10^3)	Growth rate	Incremental increase	% Decrease in growth rate
1970	15			—	—
1980	20	5	$\frac{5}{15} \times 100 = 33.3\%$	—	—
1990	24.5	4.5	22.5%	-0.5	10.8%
2000	29.5	5	20.41%	0.5	2.09%
2010	32.5	3	10.17%	-2	10.24%
		$\bar{x} = 4.375$	$r = \sqrt[4]{33.3 \times 22.5 \times 20.41 \times 10.17}$	$\bar{y} = -0.6667$	$r' = 7.71\%$

Geometric increase method: $r = (33.3 \times 22.5 \times 20.41 \times 10.17)^{1/4} = 19.86\%$

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

or,

$$P_{2040} = P_{2010} \left(1 + \frac{19.86}{100} \right)^3$$

or,

$$P_{2040} = 32.5 \times 10^3 (1 + 0.1986)^3$$

$$= 55963.67 \approx 55964$$

Incremental increase method:

$$P_n = P_0 + n\bar{x} + \frac{n(n+1)}{2} \cdot \bar{y}$$

$$= [32.5 + 3 \times 4.375 + 6 \times (-0.6667)] \times 10^3$$

$\therefore$

$$P_{2040} = 41624.8 \approx 41625$$

Rate of decrease method:

$$P_1 = P_0 + \left(\frac{r_0 - r'}{100} \right) \times P_0$$

$$P_{2020} = P_{2010} + \frac{(10.17 - 7.71)}{100} \times P_{2010} = 33299.5$$

$$P_{2030} = P_{2020} + \frac{(10.17 - 2 \times 7.71)}{100} \times P_{2020} = 32551.28$$

$$P_{2040} = P_{2030} + \frac{(10.17 - 3 \times 7.71)}{100} \times P_{2030} = 27462.23 \approx 27463$$

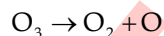
48.

Sol. **Various types of disinfectants:** The various methods of disinfectants can be categorized as:

- **Minor methods**
 - Boiling of water
 - Treatment with excess lime
 - Treatment with ozone
 - Treatment with iodine and bromine
 - Treatment with UV rays
 - Treatment with potassium permanganate
 - Treatment with silver i.e. Electro-Katadyn process
- **Major methods**
 - Chlorination

Boiling with water: Boiling destroys the bacteria present in water but it is not possible for public water supplies as boiling huge amount of water is not feasible.

Treatment with excess lime: Excess lime if added to water can kill bacteria also apart from removing the hardness. But excess lime makes the water too alkaline raising its pH. But this method requires removal of excess lime from water before it can be supplied to the general public. Thus recarbonation is done to remove the excess lime but this cannot protect the water from future recontamination. **Treatment with ozone:** Ozone is an allotrope of oxygen which is faintly blue gas of pungent odour. It is an excellent disinfectant. Because of its unstable nature, ozone readily breaks down releasing nascent oxygen as per the following reaction:



The nascent oxygen so produced is a powerful disinfectant and oxidises organic matter as well as bacteria from water.

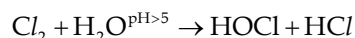
Treatment with iodine and bromine: Addition of iodine or bromine in water kills pathogenic bacteria. The quantity of these disinfectants is limited to 8 ppm and a contact period of 5 minutes is sufficient. This method is suitable for supplying water to small communities.

Treatment with UV rays: Ultra violet rays are highly effective in killing all types of bacteria. However the water to be treated with UV rays should be less turbid (< 15 mg/l) and low in colour. This method is suitable for supplying water to small communities.

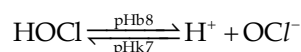
Treatment with potassium permanganate: Potassium permanganate (KMnO_4) is a powerful oxidizing agent which not only kills the bacteria but also oxidises the taste producing organic matter. The normal dose of this disinfectant varies from 1 to 2 mg/l with contact period of 4 to 6 hours. But with the passage of time, water treated with potassium permanganate produces a dark brown precipitate. This method is suitable for small communities particularly in rural areas.

Treatment with silver i.e. Electro-Katadyn process: In this method, metallic silver ions are introduced in water by passing it through a tube containing solid silver electrodes which are connected to a D.C. supply of about 1.5 V. The so introduced silver ions act as disinfectant. Its dose may vary from 0.05 to 0.1 mg/l with a contact period of 15 minutes to 3 hours. This method does not impart any colour or odour to water but is quite costly.

Chlorination: It is invariably the most adopted method of disinfection. This method is cheap, reliable, easy to handle and provides residual disinfection capabilities. However when used in large quantities, it imparts bitter taste to water. The reactions that take place are as follows:



The hypochlorous acid (HOCl) is unstable and breaks into hydrogen ions and hypochlorite ions as:



Thus disinfecting action of chlorine depends on pH of water.

(ii) Water required to be treated per day = $20000 \text{ m}^3 = 20 \times 10^6 \text{ l} = 20 \text{ MI}$

Chlorine usage per day = $8 \text{ kg} = 8000 \text{ g} = 8 \times 10^6 \text{ mg} = 8 \text{ M.mg}$

$\therefore$ Chlorine used per liter of water per day = $\frac{8 \text{ M.mg}}{20 \text{ MI}} = 0.4 \text{ mg/l}$



Hence, chlorine dosage required = 0.4 mg/l
 Now, residual chlorine left = 0.2 mg/l
 Thus, the actual chlorine which has reacted with water = 0.4 - 0.2 = 0.2 mg/l
 Hence, Chlorine demand of water = 0.2 mg/l

49.

Sol. Coagulation is a chemical technique which is directed towards the destabilization of the charged colloidal particles. Flocculation, on the other hand, is the slow mixing techniques which promotes the agglomeration of the destabilized particles.

Mechanisms of coagulation:

(i) **Double layer compression:** The negative colloid and its positively charged atmosphere produce an electrical potential across the diffused layer. It is highest at the surface and zero at the outside of the diffused layer, and is known as zeta potential.

- A cationic coagulant such as metal salt reduces the zeta potential of the particles by adding positive charge.
- Double layer compression involves adding salt to the solution. As the ionic concentration increases, the double layer and the repulsion energy curves are compressed until there is no longer energy barrier. Particle agglomeration occurs under these conditions.
- The thickness of the double layer depends on the concentration of ions in the solutions.

(ii) **Adsorption and charge neutralization:**

- When metal based coagulant (ferrous salt/alum) are added to water, it dissociates and $\text{Fe}^{2+}/\text{Al}^{3+}$ ions are formed which reacts to OH^- ions to form various monomeric and polymeric hydrolysed species.
- Adsorption of the metal hydrolysed products on the colloid surface caused charge neutralization and Vander Walls forces becoming dominant.
- Adsorption sites on the colloids can adsorb a polymer molecule and a bridge is formed when one or more particles are adsorb along the length of the polymer. Bridge particles intertwined with other bridged particles during the process.

(iii) **Sweep coagulation:** Addition of relatively large dosages of coagulant, usually aluminium or iron salts, which results in precipitation as hydrous metal oxides. Most of colloids and some of dissolved solids are literally swept from the bulk of water by becoming enmeshed in the settling hydrous oxide floc. Sweep floc is achieved by adding so much coagulant to the water that the water becomes saturated and the coagulant precipitate out. Then the particles get trapped in the precipitant as it settle down.

(iv) **Inter particle bridging:**

- Large molecules may be formed when aluminium or ferric salts get dissociated in water.
- Several colloids may become attached to one molecule or various molecule may get enmeshed resulting in settleable mass.
- Polymers may also be used alone or in combinations with $\text{Al}^{3+}/\text{Fe}^{+}$ salts.

50.

Sol.

Average daily demand = 4 MLD

Assume maximum daily demand = $1.5 \times$ Average daily demand

$$\Rightarrow Q_p = 1.5 \times 4 = 6 \text{ MLD}$$

Assume 5% of daily demand kept for backwashing and time of backwashing is 30 minutes.

$$\therefore \text{Effective design discharge} = \frac{6 \times 1.05}{\left(24 - \frac{30}{60}\right)} \times 24 = 6.43 \text{ MLD}$$

$$\text{Rate of filtration} = 5000 \text{ L/m}^2/\text{hr}$$

$$\therefore \text{Area of filter media} = \frac{6.43 \times 10^6}{(5000 \times 24)} = 53.62 \text{ m}^2$$

$$\text{Number of filter} = 1.22\sqrt{Q} = 1.22\sqrt{6.43} = 3.09 = 4 \text{ filters}$$

$$\therefore \text{Area of each filter} = \frac{53.62}{4} = 13.40 \text{ m}^2$$

$$\text{Taking } \frac{L}{B} = 2$$

$$\Rightarrow L \times B = 2B \times B = 13.405$$

$$\Rightarrow B = 2.5 \text{ m} \approx 2.60 \text{ m}$$

$$\Rightarrow L = 2B = 5.60 \text{ m}$$

$\therefore$ Adopt total 5 filters = 4+1 (stand by) filter each of size $5.60 \times 2.90 \text{ m}$

51.
Sol.

Water required per day = 4.5 ML

Assuming 4% of filtered water is required for washing of filter.

Total filtered water required per day = $1.04 \times 4.5 = 4.68 \text{ M.L./day}$

Now, assuming 0.5 hr. is lost everyday in washing the filter.

$$\therefore \text{Filtered water required per hour} = \frac{4.68}{23.5} \text{ ML/hr} = 0.199 \text{ ML/hr}$$

Assuming rate of filtration to be 5000 l/hr/m^2

$$\text{The area of filter required} = \frac{0.199 \times 10^6}{5000} = 39.8 \text{ m}^2$$

Now, Assuming $\frac{L}{B} = 1.5$ and 2 filter beds are provided,

$$2 \times (L.B) = 39.8$$

or,

$$2 \times 1.5 \times B^2 = 39.8$$

$\Rightarrow$

$$B = 3.64 \text{ m}$$

Adopt,

$$B = 3.7 \text{ m}$$

Hence adopt 2 filter units, each of dimension $5.55 \text{ m} \times 3.7 \text{ m}$

Under-drainage system:

$$\text{Total area of perforation} = 0.2\% \times \text{filter area} = \frac{0.2}{100} \times (5.55 \times 3.7) = 0.0411 \text{ m}^2$$

Assuming, Area of each lateral = 2 times area of perforation in it (for 13 mm dia)

$$\text{Total area of laterals} = 2 \times \text{total area of perforations} = 2 \times 0.0411 = 0.0822 \text{ m}^2$$

$$\text{Area of manifold} = 2 \times \text{Area of laterals} = 0.1644 \text{ m}^2$$

$$\therefore \text{dia of manifold} = \sqrt{\frac{0.1644 \times 4}{\pi}}$$

$$d = 0.46 \text{ m}$$

Lets use 45 cm dia manifold pipe.

Let spacing between laterals be 15 cm.

$$\text{No. of laterals} = \frac{5.55 \times 100}{15} = 37 \text{ on either side of manifold.}$$

Hence use 74 laterals in all, in each unit.

$$\text{Length of each lateral} = \frac{\text{Width of filter}}{2} - \frac{\text{Dia of manifold}}{2} = \frac{3.7}{2} - \frac{0.46}{2} = 1.62 \text{ m}$$

Adopting 13 mm perforations in the laterals,

$$\text{Total area of perforations} = 0.0411 \text{ m}^2$$

$$\text{or, } x \times \frac{\pi}{4} \times (1.3)^2 = 411 \text{ cm}^2$$

$\Rightarrow$

$$x = 310$$

No. of perforation in each lateral = $\frac{310}{74} = 4.19$, say, 5.

$$\therefore \text{Area of perforations per lateral} = 5 \times \left[\frac{\pi}{4} \times (1.3)^2 \right] \text{cm}^2 = 6.64 \text{ cm}^2$$

$$\text{Area of each lateral} = 2 \times \text{Area of perforation} = 2 \times 6.64 = 13.28 \text{ cm}^2$$

$$\text{dia of each lateral} = \sqrt{\frac{13.28 \times 4}{\pi}} = 4.112 \text{ cm}$$

Hence O.K.

52.
Sol.

Sedimentation with Coagulation:

In order to increase the efficiency of treatment, coagulation aided sedimentation is adopted which is carried out in following 3 stages:

1. **Coagulation:** In this process certain chemicals like alum, known as coagulants are added in order to neutralize the negative protective charge present over the suspended impurities by imparting threshold energy induced in this process by fast mixing.
2. **Flocculation:** It is the process in which neutralised particles are brought in intimate contact with each other so as to promote their agglomeration resulting in increased size particles which can be easily settled. In order to achieve so, slow mixing is induced in flocculation tank.
3. **Sedimentation:** This process is carried in sedimentation tank which are designed in same way as plain sedimentation tank except here small detention time of 2-4 hours is sufficient as large sized particles settle easily.

Given: P = 1,00,000; Water supply = 120 l/c/d; $t_d = 6$ hrs; $v_t = 20$ cm/min = 0.2 m/min

$$Q_{\text{Design}} = 1.5 \times 120 \times 1,00,000 = 18 \text{ MLD}$$

$$\text{Volume} = Q_D \times t_d$$

$$= \frac{18 \times 10^6 \times 10^{-3}}{24} \times 6 = 4500 \text{ m}^3$$

$$\text{Length, } L = v_t \times t_d = 0.2 \times (6 \times 60) = 72 \text{ m}$$

$$\text{Let } \frac{L}{B}$$

$$\Rightarrow B = \frac{L}{4} = \frac{72}{4} = 18 \text{ m}$$

$$\text{Depth, } H = \frac{V}{L \times B} = \frac{4500}{72 \times 18} = 3.47 \text{ m, say } 3.5 \text{ m}$$

Assume freeboard = 0.50 m

- ∴ Depth of tank = $3.5 + 0.50 = 4.0$
 ∴ Dimensions of sedimentation tank is $72 \text{ m} \times 18 \text{ m} \times 4 \text{ m}$

Q.53

Sol.

S.No.	Item	Slow Sand Filters	Rapid Sand Filters
1	Rate of filtration	$3 \text{ m}^3 / \text{m}^2 / \text{d}$	$125 \text{ m}^3 / \text{m}^2 / \text{d}$
2	Size of bed	Large, 200 m^2	Small, $200\text{-}400 \text{ m}^2$
3	Depth of bed	0.3 m of gravel, 1.2 m of sand usually reduced to not less than 0.6 m by scraping.	0.5 m of gravel, 0.7 m of sand or less not, reduced by washing.
4	Size of sand	Effective size 0.25 to 0.3 to 0.35 mm, uniformity coefficient 2 to 2.5 to 3.	0.45 mm and higher uniformity coefficient 1.5 and lower depending on under-drainage system.
5	Grain size distribution of sand in filter	Unstratified	Stratified with smallest or lightest grains at top and coarsest or heaviest at bottom.
6	Under-drainage system	Split tile laterals laid in coarse stone and discharging into tile of concrete main drain	1. Perforated pipe laterals discharging into pipe mains. 2. Porous plates above inlet box 3. Porous blocks with included channels
7	Loss of head	0.15 m initial to 1 m final	0.3 m initial to 2.5 m final
8	Length of run between cleanings	20 to 60 days	12 to 72 hours
9	Penetration of suspended matter	Superficial	Deep
10	Method of cleaning	1. Scraping off surface layer of sand and washing and storing cleaned sand for periodic re-sanding of bed 2. Washing surface sand in Place by washer travelling over sand bed.	Dislodging and removing suspended matter by upward flow or backwashing which fluidises the bed. Possible use of water or air jets or mechanical rakers to improve scour.
11	Amount of wash water used in cleaning sand	0.2 to 6% of water filtered	1 to 4 to 6% of water filtered
12	Preparatory treatment of water	Generally none	Coagulation, flocculation and sedimentation

54.

Sol.

- Depth of tank = 3.5 m (given)
 Out of this depth, 0.5 m can be assumed as free-board
 Thus, effective depth of tank or water depth is given by
 $H = 3.5 - 0.5 = 3 \text{ m}$
 Length of tank, $L = 65 \text{ m}$
 Flow velocity, $v = 1.22 \text{ cm/s}$
 Specific gravity of particles = 2.65
 Temperature, $T = 25^\circ\text{C}$
 Kinematic viscosity, $\nu = 0.01 \text{ cm}^2/\text{s}$
 Let the particle size be 'd' and the settling velocity be v_s .



We know that for a particle to be removed in the settling tank it must satisfy the relation,

$$\frac{v}{v_s} = \frac{L}{H}$$

$$\Rightarrow v_s = \frac{vH}{L} = 1.22 \times \frac{3}{65} = 0.0563 \text{ cm/s}$$

But settling velocity v_s as per Stoke's equation is given as

$$v_s = \frac{g}{18}(S_s - 1) \frac{d^2}{\nu} \text{ ...for } d < 0.1 \text{ mm}$$

and

$$v_s = 418(S_s - 1)d^2 \left(\frac{3T + 70}{100} \right) \text{ ...for } d < 0.1 \text{ mm}$$

$$\therefore v_s = \frac{981}{18}(2.65 - 1) \frac{d^2}{0.01}$$

$$\Rightarrow 0.0563 \times 18 \times 0.01 = 1.65 \times 981 \times d^2 \Rightarrow d = 0.002502 \text{ cm}$$

$$\Rightarrow d = 0.025 \text{ mm } (< 0.1 \text{ mm}) \text{ Hence OK.}$$

$$\text{also } v_s = 418(2.65 - 1) \times d^2 \times \left(\frac{3 \times 25 + 70}{100} \right)$$

$$\Rightarrow 0.0563 \times 10 = 418 \times 1.65 \times d^2 \times \frac{145}{100}$$

$$\Rightarrow d = 0.024 \text{ mm } (< 0.1 \text{ mm}) \text{ Hence OK}$$

Thus the particles of size 0.024 mm and above shall be effectively removed.

Check against the scour of deposited particles :

$$\text{Scour velocity, } v_d = \sqrt{\frac{8\beta}{f'} \times g(S_s - 1)d}$$

where $\beta = 0.04$ for unigranular sand and $f' = 0.03$ for settling tanks

$$\therefore v_d = \sqrt{\frac{8 \times 0.04}{0.03} \times 981 \times (2.65 - 1) \times 0.0024}$$

$$= 6.44 \text{ cm/sec} > 1.22 \text{ cm/sec (Hence OK)}$$

55.

Sol. Joints in water supply piping

Water supply pipes whether main or sub-mains are available in small lengths. Hence, joints have got an important part to play in piping system.

Following are the important types of joints in water mains :

- | | | |
|-----------------------------|---------------------|-----------------------|
| (i) Spigot and socket joint | (ii) Flanged joints | (iii) Expansion joint |
| (iv) Collared joint | (v) Flexible joint | (vi) Threaded joint |
| (vii) Simplex joint | | |

(i) **Spigot and Socket joint**

- The joint is also called bell and spigot joint.
- The enlarged end is called 'socket' or 'bell' whereas the normal end is called spigot.
- The normal end is fitted into the socket and proper alignment is done.
- The yarn of jute is wounded round the spigot end and a rubber gasket is placed tightly.

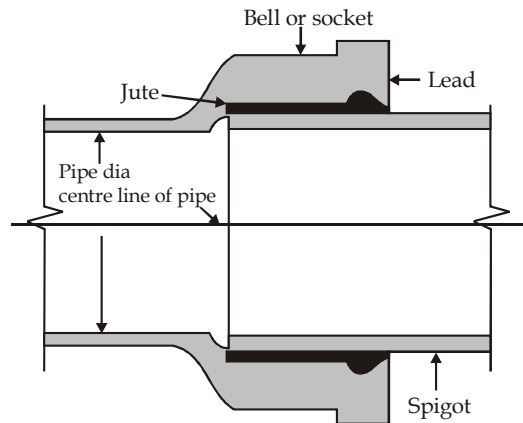


Fig. A socket and spigot joint

- The remaining space between the socket and the spigot is finally filled with molten lead. The molten lead solidifies quickly and makes the joints water tight.
- The joint is somewhat flexible and allows the pipe to be laid on flat curves without the use of special mountings.
- Mostly mains and submains of cast iron, spun iron or steel are joined with this type of joint.

(ii) Flanged Joint

- These joints are used for pumping stations filter plants and at other locations where the pipe line may be shifted.
- These joints are used in steel pipes and cast iron pipes.
- Two flanges are brought together, keeping a rubber washer (called gasket) in between them. They are then fixed by means of nuts and bolts.
- These joints are strong but rigid and hence cannot be used where deflections or vibration are expected.
- They are expensive and mostly used for indoor works.

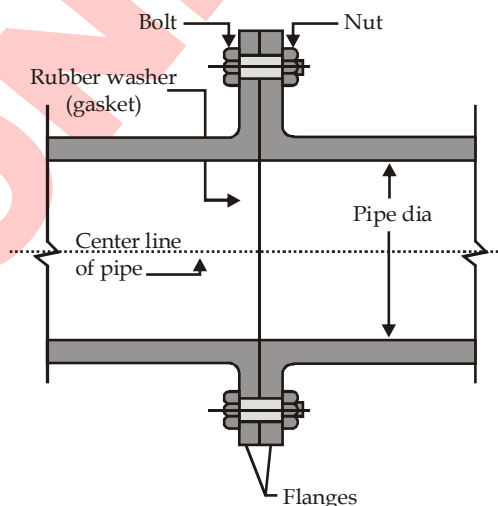


Fig. A Flanged Joint

(iii) Expansion Joint :

- These joints are provided at suitable intervals in the pipe lines, so as to counteract the thermal stresses produced due to temperature variations.
- For providing expansion joints in cast iron pipes, the socket end is cast flanged and the spigot end is plain.

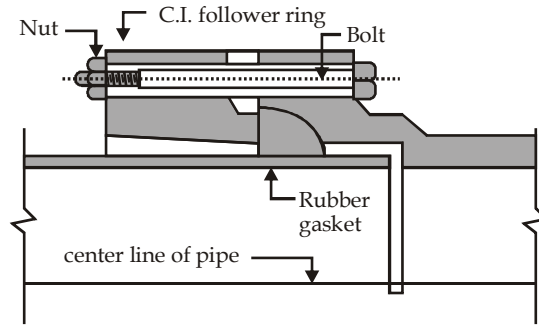


Fig. An expansion joint

(iv) Collared Joint :

- Concrete pipes are mostly joined by collared joints.
- Pipes are having spigot at one end and socket at the other and joined by a gasket and clean cement.

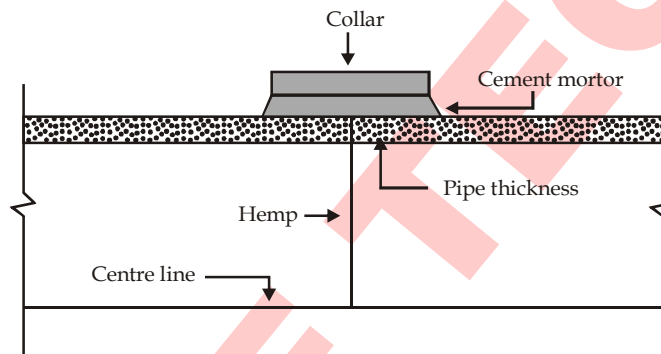


Fig. Collared Joint

(v) Flexible Joint :

- These are used where large seals, flexibilities are required.
- The pipes to be provided with such a joint are cast with special types of ends. The socket is spherical and the spigot having a bread at the end.

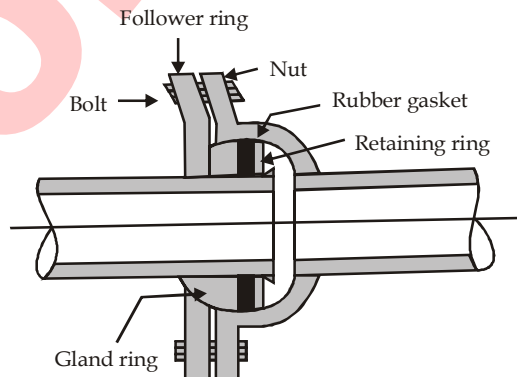


Fig. Flexible Joint

(vi) Threaded Joint :

Threaded joint are used for connecting G.I. pipes.

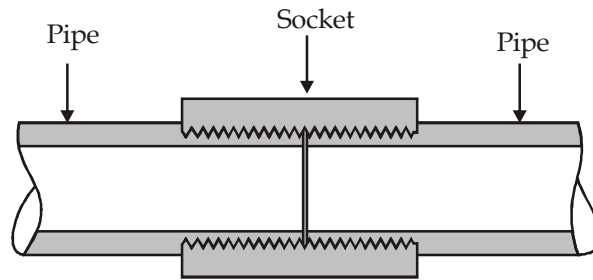


Fig. Threaded Joint

(vii) **Simplex Joint :**

- These joint consists of a pipe sleeve and two rubber gasket
- These joints are used in asbestos cement pipes.

56.
Sol.

(i) **Preliminary Treatment :**

- Preliminary treatment consists solely in separating the floating material (like dead animals, free branches, papers, pieces of rags, wood etc.), and also the heavy settleable inorganic solids.
- It also helps in removing the oils and greases, etc. from the sewage. This treatment reduces the BOD of the wastewater, by about 15 to 30%.
- The processes used as : Screening for removing floating papers, rags, clothes, etc. Grit chambers of Detritus tanks for removing grit and sand, and Skimming tanks for removing oils and greases.

(ii) **Primary Treatment System :**

- Primary treatment consists in removing large suspended organic solids. This is usually accomplished by sedimentation in Settling basins.
- The liquid effluent from preliminary treatment, often contains a large amount of suspended organic material and has a high BOD (about 60% of original).
- Sometimes, the preliminary as well as primary treatments are classified together, under primary treatment.
- The organic solids, which are separated out in the sedimentation tanks (in primary treatment) are often established by anaerobic decomposition in a digestion tank or are incinerated. The residue is used for land fills or soil conditioners.
- It will not remove the colloidal and soluble (dissolved) organic content of waste water.
- Primary treatment removes approx 60-70% of suspended solid.

(iii) **Secondary Treatment System**

- Secondary treatment involves further treatment of the effluent, coming from the primary sedimentation tank.
- Secondary treatment usually consists of biological conversion of dissolved and colloidal organics into biomass that can subsequently be removed by sedimentation.
- Contact between microorganisms and the organics is optimized by suspending the biomass in the wastewater or by passing the wastewater over a film of biomass attached to solid surfaces. The most common suspended biomass system in the activated sludge process.
- Recirculating a portion of the biomass maintains a large number of organism in contact with the wastewater and speeds up the conversion process. The classical attached biomass system in the trickling filter.
- Secondary systems produce excess biomass that is biodegradable through endogenous catabolism and by other microorganisms.
- Secondary sludges are usually combined with primary sludge for further treatment by anaerobic biological process.

(iv) **The Final or Advanced Treatment**

- This treatment is sometimes called tertiary treatment. In most cases, secondary treatment of municipal wastewater is sufficient to meet effluent standards.
- In some instances, however additional treatment may be required. Tertiary treatment consists of



removing the organic load left the secondary treatment and particularly to kill the pathogenic bacteria.

- This treatment, which is normally carried out by chlorination, is generally not carried out for disposal of sewage in water, but is carried out, while using the river stream for collecting water for reuse or for water supplies.

57.

Sol. "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 BOD₅:

The standard 5 day BOD (BOD₅) 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 (DO) of this diluted sample. The diluted sample is then incubated for 5 days at 20°C. The dissolved (DO) of the diluted sample, after this period of incubation is again calculated. Then BOD₅ in mg/l is calculated as

$$\text{BOD}_5 = (\text{D.O}_i - \text{D.O}_f) \times \frac{\text{Vol. of the diluted sample}}{\text{Vol. of the undiluted sewage sample}} \text{ (or dilution factor)}$$

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

Initial DO of diluted sample,

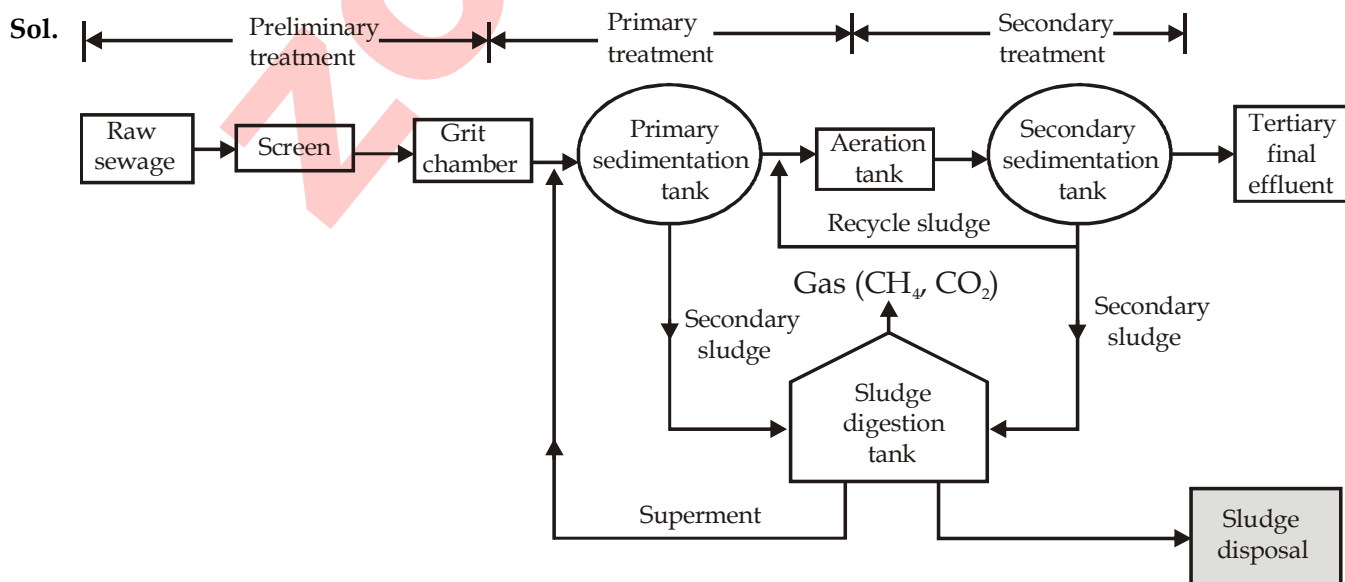
$$\text{DO}_i = \frac{V_{\text{waste}} \times \text{BOD}_5 + V_{\text{pure}} \times \text{BOD}_5}{V_{\text{waste}} + V_{\text{pure}}} = \frac{5 \times 0 + 295 \times 9.2}{300} = 9.05 \text{ mg/L}$$

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

DO_f = 5.0 mg/L

$$\begin{aligned} \text{BOD}_5 &= (\text{D.O}_i - \text{D.O}_f) \times \frac{\text{Vol. of the diluted sample}}{\text{Vol. of the undiluted sewage sample}} \\ &= (9.05 - 5.0) \times \frac{300}{5} = 243 \text{ mg/L} \end{aligned}$$

58.



S.No.	Type of treatment	Purification effected	Process or unit employed	Disposal of residuals
1	Preliminary treatment	(a) Removal of floating material like dead animals, free branches, pieces of rags, wood and other large sized floating materials. (b) Removal of heavy settleable inorganic solids. (c) Removal of fats and greases.	Coarse and fine screens of different designs Grit chambers or detritus tanks Skimming tanks or vacuators	Screening can be disposed of, easily, either by burials or burnings. The grit can be easily disposed of either by burials or burnings for raising low lying areas. The skimming contain unstable volatile organic materials, and have to disposed of by first stabilising them in digestion tanks by anaerobic process.
2	Primary treatment	Removal of suspended settleable organic solids	(i) Sedimentation tanks or (ii) Septic tanks (used only for small colonies or in rural areas). or (iii) Imhoff tanks (seldom used in modern days.)	Sludge containing organic material has to be stabilised first, in digestion tanks, and the digested material is then used as a manure or soil builder. The effluents are generally disposed of for sewage farming on land. These units combine sludge digestion along with sedimentation in these tanks.
3	Secondary or Biological Treatment	Removal of fine suspended non-settleable solids and colloids, including dissolved organic matter.	(i) Chemical flocculation (seldom used in sewage treatment) or (ii) Intermittent sand filters, followed and preceded by sedimentation (seldom used in modern days) or (iii) Conventional low rate trickling filters, followed and preceded by sedimentation. or	Sludge containing organic material has to be stabilised in digestion tanks, and the residue is used as manure or soil builder. Sludge containing organic material has to be stabilised in digestion tanks, and the residue is used as manure or soil builder. Sludge containing organic material has to be stabilised in digestion tanks, and the residue is used as manure or soil builder.

S.No.	Type of treatment	Purification effected	Process or unit employed	Disposal of residuals
			(iv) Modern high rate trickling filters, followed and preceded by sedimentation. or (vi) Activated sludge treatment in Aeration tank and secondary settling tank. or (v) Oxidation ponds.	Sludge containing organic materials has to be stabilised in digestion tanks and the residue is used as manure or soil builder. Sludge containing organic material has to be stabilised in digestion tanks, and the residue is used as manure or soil builder. Effluents are generally disposed of by using them for irrigation.
4	Final or Tertiary Treatment	Removal of Pathogens and the remaining very fine dissolved organic matter	Chlorination (seldom used for sewage treatment)	

59.

Sol.

- They are also called as percolating filters, consist of tanks of coarser filtering media, over which the sewage is allowed to sprinkle or trickle down, by means of spray nozzles or rotary distributors. The percolating sewage is collected at the bottom of the tank through a well designed under-drainage system.
- The decomposition of the organic matter and the resultant purification of the sewage is brought about by a population of micro-organisms. Micro-organisms and bacteria, which are naturally present in sewage, get attached to the filter media. Organic matter from the sewage influent is also absorbed on the biological film, formed by the micro-organisms around the filtering media particles (i.e. sand particles).
- In the outer portions of this film of biological mass or slime layer, the organic matter is degraded by the aerobic bacteria. As the micro-organisms grow, the thickness, of the slime layer increases, and the diffused oxygen is consumed by the upper portions of the slime layer, thereby creating an anaerobic environment near the surface of the media particles.
- The liquid sewage, exerting a shearing action, this breaks up some biological mass (slime) from the media, but the new slime layer continuous to grow. The continuing growth and break up of biological mass (slime) from the biofilm (slime layer), infact, creates a balance in the thickness of the formed biofilm. The break up or detachment of the biomass (biological solids) from the slime layer is known as sloughing.
- The sloughing, infact, imparts turbidity to the filter effluent, and the sloughed material (biological solids) is separated from the treated sewage in the secondary settling tank.
- The extent of sloughing is primarily a function of organic and hydraulic loading on the filter. The hydraulic loading accounts for shear velocities that are developed in the filter (which causes sloughing); while the organic loading accounts for the rate of metabolism in the slime layer (which loosens the biological mass in the slime layer, leading to sloughing).
- In order to ensure the large scale growth of the aerobic bacteria in the biofilm, sufficient quantity of oxygen is supplied by providing suitable ventilation facilities in the body of the filter; and also to some extent by the intermittent functioning of the filter.



60.
Sol.

Wastewater flow = 5 MLD

BOD of wastewater = 300 mg/l
= 1500 kg/day

BOD removal in PST = 25%

∴ BOD applied on filter, $y = 1500 \times (1 - 0.25) = 1125 \text{ kg/day}$

Desired BOD of treated wastewater, $y_e = 50 \text{ mg/l}$
= $50 \times 10^{-6} \times 5 \times 10^6 \text{ kg/day}$
= 250 kg/day

∴ Efficiency of filter required, $\eta = \left(\frac{y - y_e}{y} \right) \times 100 = \left(\frac{1125 - 250}{1125} \right) \times 1000 = 77.77\%$

(i) For high rate trickling filter

∴ Recirculation ratio, $\frac{R}{I} = 1.4$

∴ Recirculation factor, $F = \frac{\left(1 + \frac{R}{I} \right)}{\left(1 + 0.1 \frac{R}{I} \right)^2}$

$\Rightarrow F = \frac{(1 + 1.4)}{(1 + 0.1 \times 1.4)^2} = 1.846$

∴ Efficiency, $\eta = \frac{100}{\left(1 + 0.0044 \sqrt{\frac{y}{V.F.}} \right)}$

$\Rightarrow 77.77 = \frac{100}{1 + 0.0044 \sqrt{\frac{1125}{V \times 1.846}}}$

$\Rightarrow V = 0.1445 \text{ Ha-m} = 1445 \text{ m}^3$

Assume depth of filter media, $H = 1.5 \text{ m}$

∴ Area of circular filter, $A = \frac{V}{H}$

$\Rightarrow \frac{\pi}{4} D^2 = \frac{1445}{1.5}$

$\Rightarrow D = 35.02 \text{ m}$

∴ Diameter of high rate trickling filter should be 35 m and depth of filter 1.5 m

(ii) Low rate trickling filter

$\frac{R}{I} = 0$

∴ $F = 1$



$$\therefore \eta = \frac{100}{\left(1 + 0.0044 \sqrt{\frac{y}{V.F}}\right)}$$

$$\Rightarrow 77.77 = \frac{100}{1 + 0.0044 \sqrt{\frac{1125}{V \times 1}}}$$

$$\Rightarrow V = 0.2668 \text{ Ha-m} = 2666 \text{ m}^3$$

Assume depth of low rate trickling filter

$$H = 2.0 \text{ m} \quad (1.6 < H < 2.4 \text{ m})$$

$$\therefore \text{Area of the filter, } A = \frac{V}{H}$$

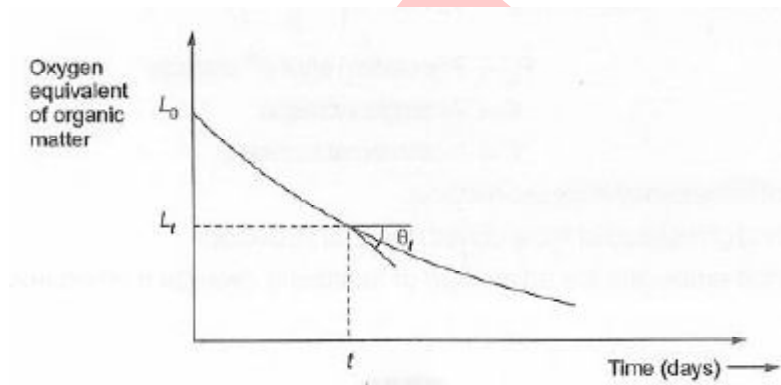
$$\Rightarrow \frac{\pi D^2}{4} = \frac{2666}{2}$$

$$\Rightarrow D = 41.19 \text{ m} \approx 42.0 \text{ m}$$

$\therefore$ Diameter of flow rate trickling filter should be 42 m and depth of filter 2.0 m.

61.

Sol. Let us consider the following curve,



$$\text{here, } \theta_t = \frac{dL_t}{dt} \alpha - L_t$$

$$\Rightarrow \frac{dL_t}{L_t} = -k \cdot dt$$

$$\text{Integrating both sides, } \int_{L_t=L_0}^{L_t=L_t} \frac{dL_t}{L_t} = k \int_{t=0}^{t=t} dt$$

$$\left| \ln(4) \right|_{L_0}^{L_t} = -k \left| t \right|_0^t$$

$$\ln\left(\frac{L_t}{L_0}\right) = -kt \text{ or } 2.303 \log_{10}\left(\frac{L_t}{L_0}\right) = -kt$$

$$\ln\left(\frac{L_t}{L_0}\right) = kt \text{ or } \log_{10}\left(\frac{L_t}{L_0}\right) = -0.434 k.t$$

$$\text{Let } 0.434k = k_D$$

$$\Rightarrow L_t = L_0 e^{-kt} \text{ or } L_t = L_0 \times (10)^{-k_D \cdot t}$$

$$\text{Now, } BOD_t = L_0 - L_t$$

$$\therefore \text{tday BOD} = L_0 (1 - e^{-kt}) \text{ or } L_0 (1 - 10^{-k_D \cdot t})$$

k and k_D are deoxygenation constant at base 'e' and '10' respectively.

L_0 is oxygen equivalent of organic matter just before the start of decomposition. ' L_0 ' is also equal to ultimate BOD.

$$\text{Given: } BOD \text{ at } 20^\circ\text{C} = 400 \text{ mg/l and } k = 0.1$$

$$\Rightarrow L_0 = 1543.3183 \text{ mg/l}$$

$$\begin{aligned} \text{5-day BOD at } 20^\circ\text{C, } BOD_5 &= L_0 (1 - e^{-0.1 \times 5}) \\ &= 1543.3183 (1 - e^{-0.5}) \\ BOD_5 &= 607.2484 \text{ mg/l} \end{aligned}$$

62.
Sol.

$$\begin{aligned} Q_{\text{peak}} &= 63 \text{ l/min} \\ P &= 50 \end{aligned}$$

$$\text{Design discharge, } Q_D = 63 \times 24 \times 60 = 90720 \text{ l/d}$$

Let us assume : Detention time of 1 day,

Cleaning period of 12 months,

Rate of accumulation of sludge = 50 l/c/year,

$$V_1 = Q_D \times t_d$$

$$\therefore V_1 = (90720 \times 10^{-3}) \times 1 = 90.72 \text{ m}^3$$

$$V_2 = \text{Rate of accumulation} \times \text{cleaning pd.}$$

$$\therefore V_2 = (50 \times 50 \times 10^{-3}) \times 1 = 2.5 \text{ m}^3$$

$$V = 90.72 + 2.5 = 93.22 \text{ m}^3$$

Let $\frac{L}{B}$ ratio be 2 and depth, $H = 1.5 \text{ m}$

$$\Rightarrow \text{Surface Area} = \frac{V}{H}$$

$$L \times B = \frac{93.22}{1.5}$$

$$2B^2 = 62.15 \Rightarrow B = 5.57 \approx 5.6 \text{ m}$$

as, $B > 0.9 \text{ m}$, design is okay.

$$\therefore L = 11.2 \text{ m}$$

$$\text{and } H = 1.5 \text{ m}$$

We provide free board = 0.3 m

Trench designing: Let us assume use of soak pit with percolation rate = 25 min

$$\text{From BIS equation, } q = \frac{204}{\sqrt{t}} = \frac{204}{\sqrt{25}} = 40.8 \text{ l/m}^2/\text{d}$$

$$\therefore \text{Surface Area of pit} = \frac{Q}{q} = \frac{90720}{40.8} \approx 2223.53 \text{ m}^2$$

$$L \times B = 2223.53$$

Let $\frac{L}{B}$ ratio be 2 for soak pit, $2B \times B = 2223.53$

$$B = 33.343 \approx 33.35 \text{ m, } L = 66.7 \text{ m}$$

$\Rightarrow$ Length of soak pit = 66.7 m
Breadth of soak pit = 33.35 m.

63.

Sol.

Given: $P = 10,000$; $Q = 135 \times 10,000 \text{ l/d} = 1.35 \text{ MLD}$; $\text{BOD}_5 = 300 \text{ mg/l}$; $G = 1$

It is given that pond is in Southern state.

$\therefore$ Latitude is between 24° N and 8° N approximately.

For 20° N , Organic Loading Rate, $\text{OLR} = 250 \text{ kg/ha/day}$.

For 16° N , $\text{OLR} = 275 \text{ kg/ha/day}$

So, Let us assume, $\text{OLR} = 270 \text{ kg/ha/day}$

$$\therefore \text{Surface Area required} = \frac{Q_0 S_0}{\text{OLR}} = \frac{1.35 \times 10^6 \times 300 \times 10^{-6}}{270} = 1.5 \text{ ha}$$

as area of oxidation pond is between 0.5 – 1 ha.

Let us provide two oxidation pond of 0.75 ha each.

Let $\frac{L}{B} = 2$ and Depth, $H = 1.5 \text{ m}$,

$$\therefore \text{S.A.} = 0.75 \times 10^4 \text{ m}^2$$

$$\text{or, } 2B \times B = 0.75 \times 10^4$$

$$\Rightarrow B = 61.24 \text{ m}$$

$$\therefore L = 122.48 \text{ m}$$

$$\text{and } H = 1.5 \text{ m}$$

$$\therefore \text{Volume, } V = L \times B \times H$$

$$\therefore V = 122.48 \times 61.24 \times 1.5 = 11251.01 \text{ m}^3$$

As there are 2 oxidation ponds,

$$\text{total volume, } V_t = 2 \times V = 22502.02 \text{ m}^3$$

$$\text{Detention time, } t_d = \frac{V}{Q} = \frac{22502.02}{1.35 \times 10^6 \times 10^{-3}} = 16.67 \text{ days}$$

$$= 2.38 \text{ weeks}$$

as $t_d > 2$ weeks

hence design is okay.

Design parameters $\rightarrow$ 2 oxidation ponds of 0.75 ha

$$\text{Length, } L = 122.48 \text{ m}$$

$$\text{Breadth, } B = 61.24 \text{ m}$$

$$\text{Depth, } H = 1.5 \text{ m}$$

and we provide free board = 0.3 m.

64.

Sol.

Given: $Q_0 = 20 \text{ MLD}$; $S = 200 \text{ mg/l}$; $S = 15 \text{ mg/l}$; $X = 3000 \text{ mg/l}$

$$\text{BOD removal efficiency, } \eta = \frac{S_0 - S}{S_0} \times 100 = \frac{200 - 15}{200} \times 100 = 92.5\%$$

$$\frac{F}{M} = \frac{Q_0 S_0}{V \cdot X}$$

$$0.2 = \frac{20 \times 10^6 \times 200}{V \times 3000}$$

$$V = 6666666.67 \text{ l}$$

$$\therefore \text{Volume, } V = 6666.67 \text{ m}^3$$

$$\text{Hydraulic Retention time, HRT} = \frac{V}{Q_0} = \frac{6666.67 \times 10^3}{20 \times 10^6} \text{ days} = 0.333 \text{ days} = 0.33 \times 24 = 8 \text{ hr}$$

$$\text{Volumetric loading, or OLR} = \frac{Q_0 S_0}{V} = \frac{20 \times 10^6 \times 200 \times 10^{-6}}{6666.67} = 0.6 \text{ kg / m}^3 / \text{ day}$$

65.

Sol. When sewage is discharged into a natural body of water, the receiving water gets polluted due to waste products, present in sewage effluents. But the conditions do not remain so forever, because the natural forces of purification, such as dilution, sedimentation, oxidation reduction in sun light, etc., go on acting upon the pollution elements and bring back, the water into its original condition. This automatic purification of polluted water, in due course, is called the self purification phenomena. The various natural forces of purification which help in effecting self-purification process are as below:

1. Physical forces are :

(i) Dilution and dispersion (ii) Sedimentation (iii) Sunlight

2. Chemical forces aided by biological forces are

(i) Oxidation (Bio) (ii) Reduction

1. Dilution and Dispersion

- This result in diminishing the concentration of organic matter, and thus reduces the potential nuisance of sewage.
- When sewage of concentration C_s flow at a rate Q_s into a river stream with concentration C_R flowing at a rate Q_R , the concentration C of the resulting mixture given by.

$$C_s Q_s + C_R Q_R = C(Q_s + Q_R)$$

or

$$C = \frac{C_s Q_s + C_R Q_R}{Q_s + Q_R}$$

2. Sedimentation

- The settleable solids, if present in sewage effluents, will settle down into the bed of the river, near the outfall of sewage, thus helping in the self purification process.
- Primarily the settled organic solid at bottom will be stabilized by anaerobic bacteria.

3. Sunlight

- Due to sunlight in the process of photosynthesis, oxygen is released. This oxygen helps in the oxidation of organic matter, thereby forming a stable product which is not a potential hazard.

4. Oxidation

- The oxidation of the organic matter will start due to dissolved oxygen in river water.
- The deficiency of oxygen created will be filled up by atmospheric oxygen.
- This is the most important action responsible for affecting self purification of rivers.

5. Reduction

- Reduction occurs due to hydrolysis of organic matter settled at the bottom either chemically or biologically.
- Anaerobic bacteria helps in splitting the complex organic matters of sewage into simple compounds and gases.
- These simple compounds can come in contact with oxygen during thorough mixing of river water, hence they will be stabilized eventually.

Various Factors on which natural forces of purification depends

1. Temperature

- The temperature affects the rate of biological and chemical activities, which are enhanced at higher temperature and depressed at lower temperature.
- Increase in temperature leads to decrease in D.O. and increase in rate of reaction. This is likely to lead to anaerobic condition.

2. Turbulence

- Turbulence will lead to increase in the oxygen mixed in water
- Wind and undercurrents in lakes and oceans causes turbulence which affect their selfpurification.

3. **Hydrography**

- It affects the velocity and surface expanse of the river stream.
- Higher velocity and larger surface area leads to greater turbulence and greater dilution of sewage added. This helps in self purification.

4. **Amount and type of organic matter:**

- Some compounds can be easily oxidised and some will take time thereby purification will be slow or fast depending on the type of organic matter.
- Algae which absorbs carbon dioxide and gives out oxygen is thus, very helpful in the self-purification process.

5. **Rate of reaeration :**

- More is the rate of reaeration faster will be the self purification.

66.

Sol. The principal methods used for landfilling in dry areas may be classified as, Area method, Trench method and Depression method.

(i) **Area method :** The area method is used when the terrain is unsuitable for the excavation of trenches in which to place the solid wastes. The filling operation usually is started by building an earthen levee against which wastes are placed in thin layers and compacted. Each layer is compacted as the filling progresses until the thickness of the compacted wastes reaches a height varying from 2 to 3 m. At that time, and at the end of each day's operation, a 150 to 300 mm layer of cover material is placed over the completed fill. The cover material must be hauled in the truck or earth-moving equipment from adjacent land or from borrow pit areas. In some newer landfill operations, the daily cover material is omitted.

A completed lift, including the cover material, is called a cell. Successive lifts are placed on the top of one another until the final grade called for in the ultimate development plane is reached. A final layer of cover materials is used when the fill reaches the final design height.

(ii) **Trench method :** The trench method of landfilling is ideally suited to areas where an adequate depth of cover material is available at the site and where the water table is well below the surface. To start the process (for a small landfill), a portion of the trench is dug with a bulldozer and the dirt is stockpiled to form an embankment behind the first trench.

Wastes are then placed in the trench, spread into thin layers and compacted. The operation continues until the desired height is reached. Cover material is obtained by excavating an adjacent trench or continuing the trench that is being filled. In large landfills, a dragline and one or more scrapers are used to excavate a deep rectangular pit.

(iii) **Depression method :** At locations where natural or artificial depressions exist, it is often possible to use them effectively for landfilling operations. Canyons, ravines, dry borrow pits and quarries have all been used for this purpose. The techniques to place and compact solid wastes in depression landfills vary with the geometry of the site, the characteristics of the cover material, the hydrology and geology of the site, and the access to the site. In a canyon site, filling starts at the head end of the canyon and ends at the mouth. This practice prevents the accumulation of water behind the landfill. Wastes usually are deposited on the canyon floor and from there are pushed up against the canyon face at a slope of about 2 to 1. In this way, a high degree of compaction can be achieved.