Design considerations:

(A) *Sedimentation Chamber:*

It is rectangular in shape with the following specifications :

(*i*) Detention period = 2 to 4 hours (usually 2 hours).

(*ii*) Flowing through velocity should not be more than 0.3 m./minute.

(*iii*) Surface loading should not exceed 30, 000 litres/ m^2 of plan area/day.

(*iv*) $L \leq 30$ m, $\frac{L}{B} = 3$ to 5.

(*v*) Total depth = 9 to 11 m, The depth of sedimentation chamber is about 3 to 3.5 or so. Free-board =45 cm.

(B) *Digestion chamber:*

This chamber is generally designed for a minimum capacity of 57 litres per capita.

- The working conditions in imhoff tanks are **anaerobic in lower** compartment and **aerobic** in upper compartment.

Clarigesters

- Clarigesters are small patented circular imhoff type double storey tanks, without bottom hoppers, and fitted with mechanical sludge and scum breaking equipment.
- Chlorine is sometimes used in sewage treatment to help in *grease separation*.
- Dissolved organic solids in waste water treatment may be removed by *reverse osmosis*.

DISPOSAL OF SOLID WASTES AND REFUSE OF A SOCIETY

Definition, Classification, Quantity and Composition of Refuse

- All solid and semi-solid wastes of community, except human excreta and sullage is called *refuse*. It includes garbage, ashes, rubbish, dust, etc.
- Sullage includes the liquid wastes from the bath rooms, kitchen sinks, wash basins etc.

  (*a*) **Garbage:** Putrescible organic wastes includes food articles, vegetable peelings, fruits peelings etc. When it is scientifically processed and composted, then it is possible to obtain valuable products, like grease, hog wood, fertilizer, etc. Garbage normally weights 450 to 900 kg/m^3.

  (*b*) **Ashes:** Incombustible waste products (700 to 850 kg/m^5).

  (*c*) **Rubbish:** It includes all non-putrescible wastes except ashes. It includes paper, glass, rags, etc. (50 to 400 kg/m^3).

- The usual weight of refuse varies between 300 to 600 kg/m^3.
- In an average modern city, each citizen produces about 0.3 to 0.8 kg. of solid domestic waste per day.
- The density of Indian refuse is generally higher than that of the developed countries and hence the Indian refuse can be carried efficiently and economically by mechanical transport for land fillings. But the calorific value of Indian refuse is much smaller, and its moisture content is high, it cannot be easily burnt or incinerated, and hence incineration method of refuse disposal is not so suitable for India, as it is far developed countries like U.S.A.

Collection, Removal and carriage of Refuse

In India, the refuse is generally collected in individual houses in small containers, and from there, it is collected by sweepers in small hand driven lorries / carts, and then dumped into the masonry chambers constructed along roadsides, by municipalities. The refuse is finally carried away by municipal trucks, for further disposal during day times. The methods adopted here are highly unsatisfactory, and need tremendous improvements and changes.

Disposal of Refuse :

(*i*) Sanitary land filling

(*ii*) Burning or incineration

(*iii*) Burreing it out into the sea. (obselete method)

(*iv*) Pulverization; and

(*v*) Composting

(*i*) **Disposal of refuse by sanitary land filling:**

In low lying area, the refuse is filled up or dumped in layers of 1.5 m or so, and each such layer is covered by good earth of atleast 20 cm. thickness, so that the refuse is not directly exposed. If the thickness of land filling is large filling shall be done in layers, and each layer shall be left out for atleast seven days, and compacted by movement of bull, motors trucks etc. for its settlement, before starting filling the second layer of refuse.

- The land filling operation is essential a biological method of waste treatment, since the waste is stabilised by aerobic as well as anaerobic process.
- The refuse get stabilised, generally within a period of 2 to 12 months, and settles down by 20–40% of its original height.
- This method is widely adopted in our country. 90% of Indian refuse is disposed off in this manner.

(*ii*) **Disposal of Refuse by Composting :**

Composting of refuse is a biological method of decomposing solid wastes. This decomposition can be effected either under aerobic conditions, or under anaerobic conditions, both. The final end product is a manure, called compost or humus.

In India, two methods, which generally adopted are:

(*i*) Indore process; and

(*ii*) Bangalore process

(*iii*) Indore method uses manual turning of piled up mass (refuse and night soil) for its decomposition under aerobic conditions.

- The Bangalore method is primarily anaerobic in nature; This method is widely adopted by municipal authorities throughout the country. The refuse and night soil, in this method are therefore piled up in layers in an underground earthen trench (10m × 1.5 m × 1.5 m). This mass is covered at its top by layer of earth of about 15 cm depth, and is finally left over for decomposition.
- Sanitary land fills may cause troubles during peak *monsoons*.
- Leachate is a coloured liquid, that comes out of sanitary land fills.
- Quantity of refuse produced in an average Indian city or a town is of the order of $\frac{1}{4}$ to $\frac{1}{5}$ heefare/day.

SEWAGE COLLECTION FROM HOUSES AND BUILDINGS

- Traps being placed at suitable points for its efficient functioning or to avoid evolution of foul smells.
- The house sewer should be disconnected from the public sewer by the provision of an intercepting trap; so as not to allow foul gases from public server to enter the house sewer.
- The layout of house plumbing system should, as far as possible, be such as to permit easy cleaning in case of blockade, or repair in case of leakage, or additions if additional sanitary fittings are provided on a future date.

FUNCTIONS AND TYPES OF TRAPS BEING USED IN SANITARY PLUMBRING SYSTEM

- Traps may be defined as fittings, placed at the ends of the *soil pipes* or the *sullage pipes* (*waste pipes*) to prevent the passage of foul gases from the pipes to the outside. This is possible because traps do enclose or maintain water seal between the pipe and the outside.
- Soil pipes are the pipes which carry the night soil, and sullage pipes are the pipes which carry the sullage from bathrooms and kitchens.

Types of traps depending upon their Shapes.

(*i*) P–traps

(*ii*) Q–traps; and

(*iii*) S–traps

Depending upon their use:

Type of traps

(*i*) Floor traps or Nahani trap

(*ii*) Gully trap; and

(*iii*) Intercepting trap.

- An intercepting trap is provided at the junction of a house sewer and a municipal sewer.
- A gully trap is provided at the junction of an unfoul roof or room drain and a foul bath or a kitchen drain.
- A floor trap or Nahani trap is provided at the head of each house drain.

SOLVED EXAMPLES

1. The 5 day 30°C BOD of a sewage sample is 110 mg/l. Calculate its 5 days 20°C BOD. Assume the deoxygenation constant at 20°C, K_{20} as 0.1.

Solution.

Given, $k_{D(20°)} = 0.1$

We know, $k_{D(T°)} = k_{D(20°)}[1.047]^{T-20°}$

$\therefore$ $k_{D(30°)} = 0.1[1.047]^{30-20°}$

$= 0.1[1.047]^{10} = 0.158$

Now, $Y_t = L[1-(10)^{-k_D \cdot t}]$

$\therefore$ $Y_{5\text{ at }30°} = L[1-(10)^{-k_D \cdot 5}]$

$= L[1-(10)^{-k_{D(30°)} \times 5}]$

$= L[1-(10)^{-0.158\times 5}]$

$= L[1-(10)^{-0.79}]$

$= L\left[1-\dfrac{1}{(10)^{0.79}}\right] = L[1-0.162]$

or $110 = L(0.838)$

$\therefore$ $L = \dfrac{110}{0.838} = \mathbf{131.3\ mg/l.}$

Now $Y_{5\text{ at }20°C} = L\left[1-(10)^{-k_D(20°)\times 5}\right]$

$= 131.3[1-(10)^{-0.1\times 5}]$

$= 131.3\left[1-\dfrac{1}{(10)^{0.5}}\right]$

$= 131.3 \times (1-0.316)$

$= \mathbf{89.8\ mg/l.}$

2. The following observations were made on a 3% dilution of waste water.

Dissolved oxygen (D.O.) of aerated water used for dilution = 3.0 mg/l

Dissolved oxygen (D.O.) of diluted sample after 5 days incubation =0.8 mg/l

Dissolved oxygen (D.O.) of original sample = 0.6 mg/l.

Calculate the B.O.D. of 5 days and ultimate BOD of the sample assuming that the deoxygenation coefficient at test temp. is 0.1.

Solution. The 100% contents of the diluted sample consists of 3% waste water and 97% of aerated water used for dilution.

Hence its D.O. = D.O. of waste water × its content + D.O. of dilution water × its content

$= 0.6 \times 0.03 + 3.0 \times 0.97$

$= \mathbf{2.928\ mg/l.}$

D.O. of the incubated sample after 5 days = 0.8 mg/l.

$\therefore$ D.O. consumed in oxidising organic matter

$= 2.928 - 0.8 = 2.128$ mg/l.

$\therefore$ B.O.D. of 5 days = D.O. consumed × Dilution factor

$= 2.128 \times \dfrac{100}{3} = \mathbf{70.93\ mg/l.}$

Let ultimate B.O.D. is given by L therefore

$Y_t = L\left[1-(10)^{-k_D \cdot t}\right]$

$Y_5 = L\left[1-(10)^{-k_D \times 5}\right]$

The value of k_D at test temperature is given as 0.1.

$70.93 = L[1-(10)^{-0.1\times 5}] = L[1-(10)^{-0.5}]$

$= L\left[1-\dfrac{1}{(10)^{0.5}}\right] = L\left[1-\dfrac{1}{3.16}\right]$

$= L \times 0.684$

or $L = \dfrac{70.93}{0.684} = \mathbf{103.7\ mg/l.}$

3. The BOD_5 of a waste water is 150 mg/l at 20°C. The k value is known to be 0.23 per day. What would BOD_8 be, if the test was run at 15°?

Solution. Here, the given value of constant, k = 0.23 per day, cannot be the value of deoxygenation constant k_D (on base 10), but it must be the *rate constant (on base e)* which was symbolised by us as 'k', where, $k_D = 0.434\ k$. Because, the highest k_D value at 20°C for strong municipal sewage is of the order of 0.1 to 0.15, or slightly less. Hence, the given k value must be the value of k and not of k_D.

Hence, at 20°C, we use

$k = 0.23$ (given)

$\therefore$ $k_D = 0.434\,k = 0.434 \times 0.23$

$= 0.0998 \approx 0.1.$

Also BOD of 5 days = BOD_5 = 150 mg/l (at 20°C)

We know, $Y_t = L[1-(10)^{-k_D \cdot t}]$

$\therefore$ BOD of 5 days, $Y_5 = L[1-(10)^{-k_D \cdot 5}]$

or $150 = L[1-(10)^{-0.1\times 5}]$

$= L[1-(10)^{-0.5}]$

$= L\left[1-\dfrac{1}{(10)^{0.5}}\right]$

$= L\left[1-\dfrac{1}{3.16}\right] = 0.684\ L$

$\therefore$ $L = \dfrac{150}{0.684}$

Now, to determine value of k_D at 15°C, using the relation

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

$$\therefore \quad k_{D(15^\circ)} = 0.1[1.047]^{15-20^\circ}$$
$$= 0.1[1.047]^{-5}$$
$$= 0.1\left[\frac{1}{(1.047)^5}\right]$$
$$= 0.1\left[\frac{1}{1.258}\right] = 0.079$$

Now, again using $Y_t = L[1-(10)^{-k_D.t}]$,

we have,

$$BOD_{\text{of 8 days,}}\ Y_8 = 219.4\,[1-(10)^{-0.079\times 8}]$$
$$= 219.4\left[1-\frac{1}{(10)^{0.632}}\right]$$
$$= 219.4\left[1-\frac{1}{4.285}\right]$$
$$= \mathbf{168.2\ mg/l}$$

4. The BOD_5 of a waste has been measured as 600 mg/l. If k_1 = 0.23/day (base e), what is the ultimate BOD_u of the waste. What proportion of the BOD_u would remain unoxidised after 20 days.

Solution. We know,

$$Y_t = L\,[1-(10)^{-k_D.t}]$$

Here, $k = k_1 = 0.23$/day (given)

$$\therefore \quad k_D = 0.434\,k$$
$$= 0.434 \times 0.23 = 0.1.$$

Using t = 5 days, we have

$$\text{BOD of 5 days, } Y_s = L\,[1-(10)^{-0.1\times 5}]$$
$$= L\,[1-(10)^{-0.5}]$$
$$= L\left[1-\frac{1}{(10)^{0.5}}\right]$$
$$= L\left[1-\frac{1}{3.16}\right] = 0.684\ L$$

$$\therefore \quad 0.684\ L = 600\ \text{mg/l}$$

$$\therefore \quad \text{ultimate BOD, } L = \frac{600}{0.684} = \mathbf{877.5\ mg/l.}$$

Now
$$Y_{20} = L\,[1-(10)^{-0.1\times 20}]$$
$$= Y_u\left[1-\frac{1}{(10)^2}\right]$$
$$= Y_u[1-0.01] = Y_u[0.99]$$

$$\therefore \quad Y_{20} = 0.99\,Y_u$$

It means that 99% of BOD_u is utilised in 20 days, and hence only 1% of ultimate BOD would be left unoxidised after 20 days.

5. 2.5 ml of raw sewage has been diluted to 250 ml and the D.O. concentration of the diluted sample at the beginning of the BOD test was 8 mg/l and 5 mg/l after 5 days incubation at 20°C. Determine the BOD of raw sewage?

Solution.

$$\text{Dilution Factor} = \frac{250}{2.5} = 100$$

$$\text{D.O. consumed} = 8 - 5 = 3\ \text{mg/l.}$$

$$\text{B.O.D. of raw sewage} = \text{D.O. consumed} \times \text{D.F.}$$
$$= 3 \times 100 = \mathbf{300mg/l}$$

6. If a 3 day B.O.D. of sewage at 20°C is 400 mg/l. Find its 5 day B.O.D. at 20°C? Assume value of k_{20} = 0.1/day.

Solution. $Y_t = L\,[1-(10)^{-k_D t}]$

or $400 = L\,[1-(10)^{-.1\times 3}]$

or $L = 801.9$ mg/l

We know, $Y_t = L\,[1-(10)^{-k_D t}]$

Hence, 5 day B.O.D. at 20°C,

$$y_5 = 801.9\,[1-(10)^{-.1\times 5}]$$
$$= \mathbf{548.31\ mg/l}$$

7. A dairy processing about 1,33,000 kg of milk daily produced an average of 246 cubic metre per day of waste water with a BOD of 1400mg/l. The principal operations are bottling of milk, and making ice cream, with limited production of cheese. Compute the waste water flow and BOD per 1000 kg of milk received, and the equivalent population of the daily waste discharge.

Solution.

Daily milk processed = 1,33,000 kg

Daily waste water produced = 246 m^3

BOD of waste water = 1400 mg/l

Waste water produced per 1000 kg of milk

$$= \frac{246}{1,33,000} \times 1000\ m^3$$
$$= \mathbf{1.85\ m^3}$$

BOD of the waste water

$$= 1400\ \text{mg/l}$$
$$= 1400 \times 10^3\ \text{mg/cum}$$
$$= \frac{1400\times 10^3}{1000}\ \text{gm/cum}$$
$$= \frac{1400\times 10^3}{10^3\times 10^3}\ \text{gm/cum}$$
$$= 1.4\ \text{kg/cum}$$

Hence, BOD produced per 1000 kg of milk processed

$$= 1.4 \times 1.85\ \text{kg} = \mathbf{2.59\ kg.}$$

Daily BOD produced by 246 m^3 of waste water

$$= 1.4 \text{ kg/m}^3 \times 246 \text{ m}^3$$

$$= 344.4 \text{ kg/day.}$$

$$\text{Population equivalent} = \frac{\text{BOD of industry in kg/day}}{0.08}$$

$$= \frac{344.4}{0.08} = \mathbf{4305}.$$

8. The BOD of a sewage incubated for one day at 30°C has been found to be 110 mg/l. What will be the 5-day 20°C BOD? Assume $k_1 = 0.1$ at 20°C.

Solution.

Given, $Y_{1(\text{at } 30°)} = 110$ mg/l.

$k_{D(20°)} = 0.1$.

Now calculate k_D at 30°C, by using the relation

$$k_{D(T)} = k_{D(20)}[1.047]T^{-20°}$$

or $$k_{D(30°)} = 0.1[1.047]^{30-20°}$$

$$= 0.1[1.047]^{10}$$

$$= 0.1 \times 1.583 = \mathbf{0.158}.$$

Now, $$Y_t = L\,[1-(10)^{-k_D.t}]$$

At 30°C and for one day,

$$Y_{1(30°)} = L\,[1-(10)^{-k_{D(30°)}\times 1}]$$

or $$110 = L[1-(10)^{-0.156\times 1}]$$

$$= L\left[1-\frac{1}{1.438}\right]$$

$$= L[1-0.696]$$

or $$L = \frac{110}{0.304} = \mathbf{361.8\ mg/l.}$$

Again using $$Y_t = L\left[1-(10)^{-k_D.t}\right]$$

We have $$Y_{5(20°C)} = L\left[1-(10)^{-k_{D(20)}\times 5}\right]$$

$$= L\,[1-(10)^{-0.1\times 5}]$$

$$= L\left[1-\frac{1}{(10)^{0.5}}\right]$$

$$= 361.8\,[1-0.316]$$

$$= \mathbf{247.4\ mg/l.}$$

9. Change in concentration of organic matter, L, with time, t, is given by

$$\frac{dL}{dt} = -k.L.$$

Calculate the organic matter remaining after 3 days if the initial concentration was 200 mg/l, and k = 0.4 per day

Solution. $$\frac{dL}{dt} = -kL,$$

or $$\frac{dL}{L} = -k.dt$$

Integrating, we have,

$$\log_e L = -kt + C$$

or $$2.3 \log_{10} L = -\text{kt} + C$$

When $t = 0$ (at start),

$$L = 200 \text{ mg/l.}$$

$\therefore$ $$2.3 \log_{10} 200 = 0 + C$$

or $$C = 2.3 \times 2.301 = 5.28.$$

Now, the value of L after 3 days is given by

$$2.3 \log_{10} L = -0.4 \times 3 + C$$

$$= -1.2 + 5.28 = 4.08$$

or $$\log_{10} L = \frac{4.08}{2.3} = 1.773$$

or $$L = 59.3 \text{ mg/l.}$$

Hence the organic matter left after 3 days = 59.3 mg/l.

10. Calculate 1 day 37°C BOD of sewage sample whose 5 day 20°C BOD is 100 mg/l. Assume K_D at 20°C as 0.1.

Solution. 5 day 20°C BOD = 110 mg/l.

Now, we have the BOD at 20°C, say after 5 days,

$$Y_t = L\left[1-(10)^{-k_{D(20°)}.t}\right]$$

Given, $$Y_t = 100 \text{ mg/l., } k_{D(20°)} = 0.1$$

$$100 = L\,[1-(10)^{-0.1\times 5}]$$

$$= L\,[1-(10)^{-0.5}]$$

$$= L\left[1-\frac{1}{3.16}\right] = L\,[0.684]$$

or $$L = \frac{100}{0.684} = \mathbf{146.2\ mg/l.}$$

Now to calculate k_D at 37°C, using the relation

$$k_{D(T°)} = K_{D(20°)}\,[1.047]^{T-20°}$$

$\therefore$ $$k_{D(37°)} = 0.1[1.047]^{37-20}$$

$$= 0.1[1.047]^{17}$$

$$= 0.1 \times 2.18 = 0.218$$

Now, to calculate Y_t for one day *i.e.*, Y_1 at 37°C, using the relation

$$Y_t = L\left[1-(10)^{-k_D.t}\right]$$

$$= L\left[1-(10)^{-k_D.1}\right]$$

At 37°C $$Y_{1(37°)} = 146.2\left[1-(10)^{-k_{D(\text{at } 37°C)}\times 1}\right]$$

$$= 146.2\left[1-(10)^{-0.218\times 1}\right]$$

$$= 146.2\left[1-\frac{1}{(10)^{0.218}}\right]$$

$$= 146.2\left[1-\frac{1}{1.65}\right]$$

$$= \mathbf{59.70}$$

11. Data from an unseeded domestic waste water BOD test are :

5 ml of waste in 300 ml bottle, Initial D.O. of 7.8 mg/l, and 5 days D.O. equal to 4.3 mg/l.

Compute the BOD; and the ultimate BOD, assuming a k-rate of 0.10 per day.

Solution. Given, Initial D.O. = 7.8 mg/l

D.O. after 5 days of incubation =4.3 mg/l

$\therefore$ D.O. consumed in 5 days = 7.8 – 4.3 = 3.5 mg/l

BOD_5 of waste water

$$= \text{D.O. consumed by diluted sample} \times \left[\frac{\text{Volume of diluted sample}}{\text{Volume of uniluted sewage used}}\right]$$

$$= 3.5 \text{ mg/l} \times \left[\frac{300 \text{ ml}}{5 \text{ ml}}\right] = \mathbf{210\ mg/l}.$$

Now, using the relation

$$Y_t = L\left[1-(10)^{-k_D \cdot t}\right]$$

We have $$Y_5 = L\left[1-(10)^{-k_D \cdot 5}\right]$$

where $k_D = 0.1$ per day and $Y_5 = 210$ mg/l

$$\therefore \quad 210 = L[1-(10)^{-0.1\times 5}]$$

$$= L\left[1-\frac{1}{(10)^{0.5}}\right]$$

$$= L[1-0.316] = 0.684\ L$$

or $$L = \frac{210}{0.684} \text{ mg/l} = 307.1 \text{ mg/l}$$

Hence, ultimate BOD (Y_u) = L = **307.1 mg/l**

ESTIMATION OF DISCHARGE

12. A separate sewerage system has to be designed for a suburb near Delhi for a rainfall frequency of 2 years. This town is already provided with adequate water supply from water-works at per capita rate of 200 l/day/person. Compute the maximum storm drainage discharge for which the S.W. drain, of a pocket draining an area of 20 hectares will be designed. Also compute the peak design discharge for which the sewers of this pocket will be designed. Make use of hourly rainfall charts and assume the concentration time (or inlet time) as 20 minutes. The population of the pocket discharging sewage is 9000. Make suitable assumptions.

Solution. Maximum one hour rainfall having 2 years frequency near Delhi = 4 cm

Dispersion factor for an area of 20 hectares =1.0.

$$\therefore \quad p_o = 4.0 \times 1.0 = 4 \text{ cm/hr.}$$

We know, $$p_c = p_o \times \left(\frac{2}{1+T_c}\right);$$

$$= 4 \times \left(\frac{2}{1+\frac{20}{60}}\right) = 6 \text{ cm/hr.}$$

Further, assume the coefficient of discharge for this residential pocket = 055

$$\text{Peak run off rate, } Q_p = \frac{1}{36}.K.p_c.A$$

$$= \frac{1}{36} \times 0.55 \times 6 \times 20 \text{ cumecs}$$

$$= 1.83 \text{ cumecs.}$$

Hence, drainage discharge for which the storm water drain of the pocket shall be designed = **1.83 cumecs**

Given, the town has been provided with a water supply from the water-works at a per capita rate of 200 litres/day/person.

$\therefore$ Water supplied per day = 200 × 9000 litres/day
= 1800 cu.m/day

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

Sewage produced per day = 0.8 × 1800 cu.m/day
=1440 cu.m/day

$\therefore$ Average sewage discharge per second

$$= \frac{1440}{24 \times 60 \times 60} \text{ cumecs}$$

$$= 0.0167$$

Assuming the peak sewage discharge as three times the average, we have

Maximum sewage discharge

$$= 3 \times 0.0167$$

$$= \mathbf{0.05\ cumecs}$$

Hence, the sewer of the locality will be designed for a discharge of 0.05 cumecs.

13. A population of 30,000 is residing in a town having an area of 60 hectares. If the average coefficient of run off for this area is 0.60, and the time of concentration of the design rain is 30 minutes, calculate the discharge for which the sewers of a proposed combined system will be designed for the town in question. Make suitable assumptions where needed.

Solution. First assume that the town is provided with a planned water supply from the water works at an average per capita rate equal to 120 litres/day/person. Also assume that 80% of this water supply will be reaching the sewers as sanitary sewage

$\therefore$ Quantity of sanitary sewage produced per day

$$= \left(\frac{80}{100}\right) 120 \times 30{,}000 \text{ litres}$$

$$= 0.8 \times 120 \times 30 \text{ cu. m}$$

$$= 2880 \text{ cu. m}$$

Quantity of sanitary sewage produced per second

$$= \frac{2880}{24 \times 60 \times 60} = 0.033 \text{ cumecs}$$

$\therefore$ Average sewage discharge = 0.033 cumecs.

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

Maximum sewage discharge

$$= 3 \times 0.033 = \mathbf{0.1 \text{ cumecs.}}$$

The storm water discharge can be computed by using Rational formula; *i.e.*,

$$Q_p = \frac{1}{36} k.p_c.A$$

where $$p_c = \frac{100}{T+20} = \frac{100}{30+20} = 2 \text{ cm/hr.}$$

$$\therefore \quad Q_p = \frac{1}{36} \times 0.60 \times 2 \times 60 \text{ cumecs}$$

$$= \mathbf{2 \text{ cumecs.}}$$

Hence, the total peak discharge for which the sewers of the combined system should be designed

= Maximum sewage discharge + Maximum storm runoff

$$= 0.1 + 2.0 = \mathbf{2.1 \text{ cumecs}}.$$

14. If in the above example, the density of population is 250 per hectare, and the quota of water supply per day is 225 litres; calculate the quantity of

(*i*) Sewage for which the sewers of a separate system, should be designed.

(*ii*) Storm water for which the sewers of a partially separate system should be designed.

Solution. Area of district = 36 hectares.

Population density = 250 persons per hectare

$\therefore$ Population = 36 × 250 = 9000

Average water supply per day = 225 litres/person

$\therefore$ Average quantity of water supplied to the district per day = 225 × 9000 litres = 20,25,000 litres = 2,025 cu. m.

$\therefore$ Rate of water supplied

$$= \frac{2025}{24 \times 60 \times 60} \text{ cumecs}$$

$$= 0.0234 \text{ cumecs.}$$

Assuming the sewage discharge as 0.8 times the water supplied, we have

Average rate of sewage produced = 0.8 × 0.0234 = 0.0187 cumecs.

Now assuming the peak rate of sewage as three times the average, we have peak rate of sewage flow = 3 × 0.0187 = **0.056 cumecs.**

HYDRAULIC DESIGN OF SEWERS

15. A 350 mm dia sewer is to flow at 0.35 depth on a grade ensuring a degree of self-cleansing equivalent to that obtained at full depth at a velocity of 0.8 m/sec. Find the required grade associated velocity the rate of discharge at this depth.

Given: Manning's rugosity coefficient = 0.014

Proportionate area = 0.315

Proportionate wetted perimeter = 0.472

Proportionate HMD (r/R) = 0.7705.

Solution. At full depth, V = 0.8 m/sec,

D = 350 mm = 0.35 m, N = 0.014.

At 0.35 depth, $\frac{d}{D} = 0.35$,

$$\frac{a}{A} = 0.315,$$

$$\frac{p}{P} = 0.472, \quad \frac{r}{R} = 0.7705.$$

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

or $$0.8 = \frac{1}{0.014} \cdot \left(\frac{0.35}{4}\right)^{2/3} \cdot \sqrt{S}$$

or $$\sqrt{S} = 0.0568.$$

or $$S = 3.234 \times 10^{-3}$$

Now, for a sewer to have the same self-cleansing at 0.35 depth as it will be at full depth, we have

required gradient, $s_s = \left(\frac{R}{r}\right) S$

where, $\frac{r}{R} = 0.7705$ (given)

$$\therefore \quad s_s = \frac{1}{0.7705} \times 3.234 \times 10^{-3}$$

$$= 4.2 \times 10^{-3}, \textit{ i.e. } 4.2\%$$

Velocity generated at this gradient at 0.35 depth,

$$v_s = \frac{N}{n} \cdot \left(\frac{r}{R}\right)^{1/6} \times V$$

$$= 1 \times (0.7705)^{1/6} \times 0.8$$

$$= \mathbf{0.765 \text{ m/sec}}.$$

Discharge, $q_s = a.v_s$

$$= 0.315 \times \frac{\pi}{4} \times (0.35)^2 \times 0.765$$

$$= \mathbf{0.023 \text{ cumecs}}.$$

16. Calculate diameter and discharge of a circular sewer laid at a slope of 1 in 400 when it is running half full, and with a velocity of 1.9m/sec. (n in Manning's formula = 0.012).

Solution. When pipe is running half full,

Area of the section, $a = \dfrac{\pi D^2}{8}$;

Wetted perimeter $p = \dfrac{\pi D}{2}$

H.M.D., $r = \dfrac{D}{4}$

From Manning's formula,

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

or $$1.9 = \frac{1}{0.012}\left(\frac{D}{4}\right)^{2/3} \cdot \frac{1}{\sqrt{400}}$$

or $$D^{2/3} = 1.9 \times 0.012 \times 2.52 \times 20 = 1.15$$

or $$D = (1.15)^{\frac{3}{2}=1.5} = 1.23 \text{ m}$$

Hence, use 1.23 m diameter sewer.

Discharge, $$Q = a.v = \frac{\pi(1.23)^2}{8} \times 1.9 \text{ m}^3/\text{sec},$$

$$= 1.13 \text{ cumecs.}$$

17. Design a sewer running 0.7 times full at maximum discharge for a town provided with the separate system, serving a population of 80,000 persons. The water supplied from the water-works to the town is at a rate of 190 litres/person/day. The sewer is made up of brick work plastered smooth with cement mortar (n = 0.013) and the permissible slope is 1 in 600. The variations of n with depth may be neglected. Assume any other data not given and needed.

Solution. Population = 80,000

Rate of water supplied = 190 litres/person/day

Average rate of daily water supplied to the town

= 80,000 × 190 litres/day

Average water supplied (in cumecs)

$$= \frac{80,000 \times 190}{24 \times 60 \times 60 \times 1000}$$

= 0.176 cumecs

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

Average discharge of sewage produced

= 0.176 × 0.8 cumecs

= **0.14 cumecs.**

Assuming the maximum flow to be three times the average, we have

Maximum sewage discharge = 3 × 0.14 = 0.42 cumecs.

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

hence for a value of $\dfrac{d}{D} = 0.7$, $\dfrac{q}{Q} = 0.838$

Here q = 0.42 cumecs therefore,

$$Q = \frac{0.42}{0.838} = 0.5 \text{ cumecs.}$$

Now using Manning's nomogram at full flow and n = 0.013, for Q = 0.5 cumecs (500 litres/sec.),

and $S = \dfrac{1}{600}$;

we read the other unknown factors, as

D = 0.78 m and; V = 1.04 m/sec.

For $$\frac{d}{D} = 0.7; \text{ we have } \frac{v}{V} = 1.12$$

$$\therefore \quad v = 1.12\,V = 1.12 \times 1.04$$

= **1.17 m/sec.**

which is more than the self-cleansing velocity, and hence satisfactory.

Check for minimum flow.

Assuming the minimum flow in the sewer to be 1/3 time the average flow, we have

Minimum flow,

$$q_{min} = \frac{0.14}{3} = 0.047 \text{ cumecs.}$$

For a discharge ratio of $\dfrac{q_{min}}{Q} = \dfrac{0.42}{0.047} = 0.11$,

and n = N as given, we have

$$\text{Depth ratio} = \frac{d_{min}}{D} = 0.23$$

For this depth ratio, Velocity ratio = $\dfrac{v_{min}}{V} = 0.65$

∴ Velocity at minimum flow = 0.65 × 1.04 m/sec.

= 0.68 m/sec.

which is more than the minimum required of 0.45 m/sec, and hence, satisfactory.

Note. When the velocity at minimum flow (i.e., D.W.F.) is not satisfactory, we have either to increase the slope or try with increased diameter of the sewer.

18. A main combined sewer was designed to serve an area of 60 sq. km with an average population of 185 persons/hectare. The average rate of sewage flow is 350 litres/capita/ day. The maximum flow is 50% in excess of the average together with the rainfall equivalent of 12mm in 24 hours, all of which are run off. What should be the capacity of the sewer in cu. m/sec.?

Solution.

Total population of the area

= Population density × area

= 185 p/ha × (60 × 10^2) ha

= 11.1 × 10^5 persons

Average sewage flow

= 350 litres/capita/day

= 350 × 11.1 × 10^5

= 388.5 × 10^6 litres/day

$$= \frac{388.5 \times 10^6}{10^3} \times \frac{1}{24 \times 60 \times 60}$$

= 4.5 cumecs

Storm water flow

$$= \frac{\text{area in } m^2 \times \text{depth of S.R.O. in } m}{\text{Time of 24 hr. in sec.}}$$

$$= 60 \times 10^6 \times \left(\frac{12}{1000}\right)\frac{1}{24 \times 60 \times 60}$$

= **8.33 cumecs.**

Maximum sewage flow

= 1.5 × average sewage flow

= 1.5 × 4.5 cumecs

= **6.75 cumecs.**

∴ Total maximum flow of the combined sewer

= maximum sewage flow + storm flow

= 6.75 + 8.33

= 15.08 cumecs.

∴ Capacity of the sewer = **15.08 cumec**.

19. A combined sewer of a circular section is to be laid to serve a particular area. Calculate the size of this sewer from the following data:

Area to be served =120 hectares.

Population =1,00,000

Maximum permissible flow velocity = 3 m/sec.

Time of entry for storm water = 10 minutes.

Time of flow in channel = 20 minutes.

Per capita water supply = 250 litres/day/person.

Coefficient of run-off for the area = 0.45.

Hourly, Maximum rainfall for the area at the design frequency = 5 cm

Assume any other data not given, and if needed.

Solution. Sewage Discharge (*i.e.*, D.W.F.) Computations.

Average water supplied

= 250 × 1,00,000 litres/day

$$= \frac{250 \times 1,00,000}{1000 \times 24 \times 60 \times 60} \text{ cumecs}$$

= 0.289 cumecs

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

Average sewage discharge

= 0.8 × 0.289

= 0.23 cumecs

Assuming the maximum sewage discharge to be 3 times the average discharge, we have

Maximum sewage discharge

= 3 × 0.23

= **0.69 cumecs.**

Storm water discharge computations

Time of concentration,

T_c = Time of entry + Time of flow

= (10 + 20) minutes

= 30 minutes.

Now, maximum hourly rainfall for the area

= p_o = 5 cm/hr.

$$\therefore \quad p_c = p_o\left(\frac{2}{1+T_c}\right)$$

where T_c = concentration time in hours

$$= \frac{30}{60} = 0.5 \text{ hour}$$

$$\therefore \quad p_c = 5\left(\frac{2}{1+0.5}\right) = 6.67 \text{ cm/hr.}$$

Now using rational formula, we have

Maximum storm run off,

$$Q_p = \frac{1}{36} \text{K}.p_c.\text{A}$$

$$= \frac{1}{36} \times 0.45 \times 6.67 \times 120$$

= 10 cumecs.

∴ Combined maximum discharge

= Storm run-off + Sewage discharge

= 10 + 0.69 = **10.69 cumecs**.

Now assuming the sewer to be running full at the maximum velocity of 3 m/sec at the time of maximum flow, we have

$$\text{Area required} = \frac{Q}{V} = \frac{10.69}{3} = 3.56 \text{ m}^2$$

∴ Diameter of sewer pipe required

$$= \sqrt{\frac{4}{\pi} \times 3.56} = \sqrt{4.53} = 2.13 \text{ m.}$$

Hence, use a sewer pipe of **2.13 m diameter**

20. Design a sewer to serve a population of 36,000; the daily per capita water supply allowance being 135 litres, of which 80 per cent finds its way into the sewer. The slope available for the sewer to be laid is 1 in 625 and the sewer should be designed to carry four times the dry weather flow when running full. What would be the velocity of flow in the sewer when running full?

Assume n =0.012 in Manning's formula.

Solution. Population = 36,000

Per capita water supply = 135 litres/person/day

$\therefore$ Average water supplied daily

$= 36{,}000 \times 135$ litres/day

Average water supplied in cumecs

$$= \frac{36{,}000 \times 135}{1000 \times 24 \times 60 \times 60} = 0.0562 \text{ cumecs.}$$

Average sewage discharge

$= 80\%$ of water supplied

$= 0.8 \times 0.0562$

$= 0.045$ cumecs.

$\therefore$ D.W.F. $= 0.045$ cumecs.

Maximum discharge for which sewer should be designed running full

$= 4 \times 0.045 = 0.18$ cumecs.

Now, using Manning's formula (and assuming that its Nomogram is not available) we have

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

[Capital letters being used for running full]

$$\therefore \quad 0.18 = \frac{1}{0.012}\left(\frac{\pi}{4}D^2\right)\left(\frac{D}{4}\right)^{2/3}\frac{1}{\sqrt{625}}$$

or $$\frac{0.18 \times 0.012 \times 4 \times 2.52 \times 25}{\pi} = D^{8/3}$$

or $$D^{8/3} = 0.173$$

or $$D = (0.173)^{\frac{3}{8}=0.375} = 0.31 \text{ m}$$

Hence, use 0.31 m diameter sewer pipe.

Velocity of flow when running full,

$$V = \frac{Q}{A} = \frac{0.18}{\frac{\pi}{4}(0.31)^2} = 2.39 \textbf{ m/sec.}$$

21. A 25 cm diameter sewer with an invert slope of 1 in 400 is running full. Calculate the velocity and rate of flow in the sewer. Is it self cleansing? Take $n = 0.015$.

Solution. Diameter of sewer, d = 25 cm = 0.25 m

Area of sewer when running full,

$$A = \frac{\pi}{4} \cdot d^2 = \frac{\pi}{4} \times (0.25)^2 \text{ m}^2 = 0.049 \text{ m}^2.$$

$$R = \frac{A}{P} = \frac{\frac{\pi}{4} \cdot d^2}{\pi d} = \frac{d}{4} = \frac{0.25}{4} = 0.0625 \text{ m}$$

$$S = \frac{1}{400}, \text{ and } N = 0.015$$

Manning's equation,

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

$$= \frac{1}{0.015} \times (0.0625)^{2/3} \cdot \frac{1}{\sqrt{400}}$$

$= 0.525$ m/s

$Q = V.\, A = 0.525 \times 0.049 \text{ m}^3\text{/s}$

$= 0.0257 \text{ m}^3\text{/s}.$

The velocity in the above sewer is 0.525 m/s, which is slightly more than 0.45 m/s, which is the numerical theoretical value of the self-cleansing velocity. Hence, the sewer can be termed as self-cleansing at full flow, although such self-cleansing should be obtained at partial flow also, such as at half full. In this case, the velocity at partial flow is likely to be lesser than 0.45 m/s or so, and hence the sewer will no longer remain self-cleansing at partial flow.

SEWAGE TREATMENT

22. Design a suitable rectangular sedimentation tank (provided with mechanical cleaning equipment) for treating the sewage from a city, provided with an assured public water supply system, with a max. daily demand of 12 million litres per day. Assume suitable values of detention period and velocity of flow in the tank. Make any other assumptions, wherever needed.

Solution. Assuming that 80% of water supplied to the city becomes sewage, we have

quantity of sewage required to be treated per day (*i.e.*, max. daily).

$= 0.8 \times 12$ million litres

$= 9.6$ M. litres

Now assuming the detention period in the sewage sedimentation tank as 2 hours, we have

The quantity of sewage to be treated in 2 hours *i.e.*, capacity of the tank required,

$$Q = \frac{9.6}{24} \times 2 \text{ M. litre}$$

$= 0.8$ M litres = 800 cu. m.

Now, assuming that the flow velocity through the tank is maintained at 0.3 m/minute; we have

Length of the tank required

= Velocity of flow × Detention period

= 0.3 × (2 × 60) m = 36 m.

Cross-sectional area of the tank required

$$= \frac{\text{Capacity of the tank}}{\text{Length of the tank}} = \frac{800}{36}\,\text{m}^2 = \mathbf{22.2\ m^2}.$$

Assuming the water depth in the tank (*i.e.*, effective depth of tank) as 3 m,

Width of the tank required

$$= \frac{\text{Area of X-section}}{\text{Depth}} = \frac{22.2}{3} = \mathbf{7.4\ m.}$$

Since the tank is provided with mechanical cleaning arrangement, no extra space at bottom is required for sludge zone.

Now, assuming a free-board of 0.5 m, we have

Overall depth of the tank = 3 + 0.5 = 3.5 m.

Hence, a rectangular sedimentation tank with an overall size of 36 m × 7.4 m × 3.5 m can be used. This satisfies the requirements like;

Length $\not>$ 4 to 5 times the width; and

The width not more than 7.5 m or so; the depth between 2.4 to 3.6 m, etc..

23. Estimate efficiency of a 30 m diameter and 1 m deep single stage high rate trickling filter for the following data:

Sewage flow = 4.5 mid (million litres/day)

Recirculation ratio = 1.4

BOD of raw sewage = 250 mg/l

BOD removed in primary clarifier = 25%

Solution. Total B.O.D. of raw sewage per day

$= 250 \times 4.5 \times 10^6$ mg/day

$= 1125 \times 10^6$ mg/day = 1125 kg/day

BOD removed in primary clarifier = 25%

BOD left in sewage entering filter unit

= 1125 × 0.75 = 843.75 kg/day

$$\text{Efficiency, } \eta = \frac{100}{1+0.0044\sqrt{Y/V.F.}}$$

where, Y = total B.O.D. in kg = 843.75 kg

$$\therefore \quad F = \frac{1+R/I}{\left(1+0.1\frac{R}{I}\right)^2} = \frac{1+1.4}{(1+0.1\times 1.4)^2} = 1.85$$

Volume of filter unit in hectare-metre,

$$V = \frac{\pi}{4}\times(30)^2\times 1\times 10^{-4}$$

= 0.0707 hectare metre

$$\text{Efficiency, } \eta = \frac{100}{1+0.0044\sqrt{843.75/0.0707\times 1.85}}$$

= 73.9%

24. Determine the size of a high rate trickling filter for the following data:

Sewage flow = 4.5 Mld; Recirculation ratio = 1.5

BOD of raw sewage = 250 mg/l; BOD removal in primary tank = 30%

Final effluent BOD desired =30 mg/l.

Solution.

Quantity of sewage flowing into the filter per day = 4.5 M.*l*/day.

BOD concentration in raw sewage = 250 mg/*l*.

∴ Total BOD present in raw sewage

= 4.5 Ml × 250 mg/l = 1125 kg.

BOD removed in primary tank = 30%

BOD left in the sewage entering per day in the filter unit = (1125) × 0.7 = 787.5 kg.

BOD concentration desired in final effluent = 30 mg/l.

∴ Total BOD left in the effluent per day

= 4.5 × 30 kg = 135 kg.

∴ BOD removed by the filter

= 787.5 – 135 = **652.5 kg.**

∴ Efficiency of the filter

$$= \frac{\text{BOD removed}}{\text{Total BOD}}\times 100$$

$$= \frac{652.5}{787.5}\times 100 = 82.85\%$$

Now, $\quad \text{Efficiency, } \eta = \dfrac{100}{1+0.0044\sqrt{\dfrac{Y}{V.F.}}}$

where $\eta = 82.85\%$

Y = total BOD in kg = 787.5 kg

$$\therefore \quad F = \frac{1+R/I}{\left(1+0.1\frac{R}{I}\right)^2}$$

$$= \frac{1+1.5}{[1+0.1\times 1.5]^2} = 1.89 \quad \text{...[since } \frac{R}{I} = 1.5$$

$$\therefore \quad 82.85 = \frac{100}{1+0.0044\sqrt{\dfrac{787.5}{V\times 1.89}}}$$

or $\quad 1+0.0044.\sqrt{\dfrac{416.6}{V}} = 1.2$

or $\quad \sqrt{\dfrac{416.6}{V}} = \dfrac{0.2}{0.0044} = 45.45$

or $\quad \dfrac{416.6}{V} = 2066.1$

or $\quad V = 0.2$ hectare-m = $\mathbf{2000\ m^3}$.