Biochemical Oxygen Demand

- $L_t = L_0(1-10^{-k_1 t})$

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

where, L_0: Ultimate BOD, mg/L

L_t : BOD remaining at any time, mg/L

k_1: 1st order reaction rate constant, 1/d

t : time, d

- Rate constant k_1 is dependent on temperature, it can be calculated as,
- $k_{1t} = k_{1\,20^0}\Theta^{T-20}$

  where, $\Theta = 1.047$
- The value of Θ is temperature dependent

Example 7.9. *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°?*

$K = 0.23$ (given)

$\therefore \quad K_D = 0.434\,K = 0.434 \times 0.23 = 0.0998 \cong 0.1.$

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

Using equation (7.16), we have

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

$$\therefore \quad Y_5 = L\left[1-(10)^{-K_D \cdot 5}\right], \qquad \text{where } Y_5 = \text{BOD of 5 days}$$

or

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

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

$$= L\left[1-\frac{1}{3.16}\right] = L\,[1-0.316] = 0.684\,L$$

$$\therefore \quad L = \frac{150}{0.684} = 219.4 \qquad \ldots(i)$$

or $L = 219.4$ mg/l.

Now, let us find K_D value at 15°C

Using equation (7.18), we have

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

$$\therefore \quad K_{D(15°)} = 0.1[1.047]^{15-20°} = 0.1\,[1.047]^{-5}$$

$$= 0.1\left[\frac{1}{(1.047)^5}\right] = 0.1\left[\frac{1}{1.258}\right] = 0.079$$

Now, again using

$$Y_t = L\left[1-(10)^{-K_D \cdot t}\right]$$, where Y_t is BOD of t days

we have $$Y_8 = 219.4\left[1-(10)^{-0.079\times 8}\right]$$

$$= 219.4\left[1-\frac{1}{(10)^{0.632}}\right] = 219.4\left[1-\frac{1}{4.285}\right]$$

$$= 219.4\,[1-0.233] = 219.4 \times 0.766 = 168.2 \text{ mg/l}$$

Hence $BOD_8 = Y_8 = 168.2$ mg/l. **Ans.**

1. Determine 1-day BOD and ultimate first stage BOD for a w/w whose 5- day BOD is 200mg/L at 20°C. The reaction constant k (base e) = $0.23d^{-1}$. What would have been the 5-day BOD if the test have been conducted at 25°C?

Given : k_1 (to the base e)=0.23/d
BOD_5=200mg/L at 20°C

Calculate:

L_0

BOD_1 at 20°C

BOD_5 at 25°C

Ans: Formula: $Y = L_0(1 - e^{-k_1 t})$, $L_0 = \frac{Y}{(1-e^{-k_1 t})}$, $k_{1T^0} = k_{1\,20^0} 1.047^{(T-20)}$

1) Ultimate BOD: $\therefore L_0 = \frac{Y}{(1-e^{-k_1 t})} = \frac{200}{(1-e^{-0.23*5})} = 293 mg/L$

2)Determine 1-day BOD: $Y_1 = L_0\left(1 - e^{-k_1 * 1}\right)$

$$= 293(1 - e^{-0.23*1}) = 60.1 mg/L$$

3)Determine 5day BOD at 25°C:

$k_{125^0} = k_{1\,20^0} 1.047^{(25-20)} = 0.29 d^{-1}$

$Y_5 = L^0\left(1 - e^{-k_1 * 5}\right) = 293(1 - e^{-0.29*5}) = 224 mg/L$

$$BOD, mg/L = \frac{(D_1 - D_2) - (B_1 - B_2)f}{P}$$

D1= initial DO of diluted sample

D2= Final DO of diluted sample after 5 days

B1= initial DO of dilution water

B2= Final DO of the dilution water

f = (volume of diluted sample- volume of raw sample)/volume of diluted sample

P= dilution ratio= volume of raw sample/volume of diluted sample

2. A BOD test was conducted at 20°C in which 15mL of waste sample was diluted with dilution water to 300mL.

Given:

Initial DO of diluted sample D_1 =8.8mg/L

Final DO after 5 days D_2=1.9mg/L

Initial DO of seeded dilution water B_1=9.1mg/L

Final DO of seeded dilution water B_2=7.9mg/L

Calculate

5-day BOD at 20°C

- Ans:Forrmula, $BOD, mg/L = \frac{(D_1 - D_2) - (B_1 - B_2)f}{P}$

Where $f = \frac{300-15}{300} = 0.95$

$P = \frac{15}{300} = 0.05$

$BOD, mg/L = \frac{(8.8-1.9)-(9.1-7.9)0.95}{0.05} = 115.2\text{mg/L}$

$$BOD = \text{Depletion of oxygen} \times \text{Dilution factor}$$

Depletion of Oxygen = D1-D2

D1= initial DO of diluted sample

D2= Final DO of diluted sample after 5 days

- Dilution factor

$$= \frac{100}{\text{Per cent of solution}} = \frac{100}{2} = 50$$

Example 7.2. *If 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-day incubation at 20°C ; find the BOD of raw sewage.*

Solution. Volume of sample of sewage = 2.5 ml.

Volume of diluted sample = 250 ml.

$$\therefore \quad \text{Dilution ratio} = \frac{250}{2.5} = 100.$$

Loss of dissolved oxygen during the test

= D.O. before testing – D.O. after testing

= 8 – 5 = 3 mg/l.

Using equations (7.11), we have

∴ BOD of sewage = Loss of oxygen × Dilution factor

= 3 mg/l × 100 = 300 mg/l. **Ans.**

Example 7.3. *A 2% solution of a sewage sample is incubated for 5 days at 20°C. The depletion of oxygen was found to be 4 ppm. Determine the BOD of the sewage.*

Solution. Dilution factor

$$= \frac{100}{\text{Per cent of solution}} = \frac{100}{2} = 50$$

Depletion of oxygen = 4 ppm.

Using equation (7.11), we have

$$\text{BOD} = \text{Depletion of oxygen} \times \text{Dilution factor}$$

$$= 4 \text{ ppm} \times 50 = 200 \text{ ppm} \quad \textbf{Ans.}$$

Example 7.10. *The 5 day 30°C BOD of 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. $K_{D(20^\circ)} = 0.1$

Now, using equation (7.18)

$$K_{D(T^\circ)} = K_{D(20^\circ)} [1.047]^{T-20^\circ}, \text{ we get}$$

$$K_{D(30^\circ)} = 0.1 [1.047]^{30^\circ-20^\circ} = 0.1 [1.047]^{10} = 0.158 \quad \ldots(i)$$

Now, using $Y_t = L\left[1-(10)^{-K_D \cdot t}\right]$, we get

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

$$\therefore \quad Y_{5 \text{ at } 30^\circ} = L\left[1-(10)^{-K_D(30^\circ)\times 5}\right]$$

or

$$110 = L\left[1-(10)^{-0.158\times 5}\right] = L\left[1-(10)^{-0.79}\right]$$

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

or $$110 = L\,(0.838) \quad \text{or } L = \frac{110}{0.838} = 131.3$$

or $$L = 131.3 \text{ mg/l.}$$

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

$$= 131.3\left[1 - (10)^{-0.1 \times 5}\right] = 131.3\left[1 - \frac{1}{(10)^{0.5}}\right]$$

$$= 131.3 \times (1 - 0.316) = 131.3 \times 0.684$$

$$= 89.8 \text{ mg/l.}$$ **Ans.**

$= 89.8$ mg/l. **Ans.**

Example 7.11. *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 (given)

Now using Eq. (7.16), we have

The BOD at 20°C, say after $t = 5$ days, is given by

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

Using $Y_t = 100$ mg/l (given)

$$K_{D(20°)} = 0.1$$

we have $$100 = L\left[1-(10)^{-0.1\times 5}\right]$$

or $$100 = L\left[1-(10)^{-0.5}\right] = L\left[1-\frac{1}{3.16}\right]$$

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

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

Now let us work out K_D at 37°C, by using Eq. (7.18) as :

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

or $$K_{D(37^\circ)} = 0.1\,[1.047]^{37-20} = 0.1\,[1.047]^{17}$$

$$= 0.1 \times 2.4 = 0.24.$$

Now, we have to work out Y_t for one day *i.e.* Y_1 at 37°C, using

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

$$\therefore \quad Y_1 = L\left[1-(10)^{-K_D \cdot 1}\right]$$

or $$Y_1\,(\text{at } 37^\circ\text{C}) = 146.2\left[1-(10)^{-K_{D(\text{at } 37^\circ\text{C})} \times 1}\right]$$

$$= 146.2\left[1-(10)^{-0.24\times 1}\right] = 146.2\left[1-\frac{1}{(10)^{0.24}}\right]$$

$$= 146.2\left[1-\frac{1}{1.738}\right] = 146.2\,[1-0.575] = 62.07.$$

Hence, Y_1 at 37°C = **62.07 mg/l. Ans.**

Hence, Y_1 at [illegible] mg/L. Ans.

Example 7.12. *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. $Y_{1\,(at\,30°)} = 110$ mg/l. ; $Y_{5\,(at\,20°)} = ?$; $K_{D\,(20°)} = 0.1$.

First of all, let us calculate K_D at 30°C, by using Eq. (7.18) *i.e.*

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

or
$$K_{D\,(30°)} = 0.1\,[1.047]^{30-20°} = 0.1\,[1.047]^{10}$$
$$= 0.1 \times 1.583 = \mathbf{0.158.}$$

Now using Eq. (7.16), we have

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

At 30°C and for one day, we have

$$Y_{1\,(30^\circ)} = \left[1-(10)^{-K_{D\,(30^\circ)}\times 1}\right]L$$

or

$$110 = L\left[1-(10)^{-0.158\times 1}\right] = L\left[1-\frac{1}{1.438}\right]$$

$$= L\,[1-0.696] = L\,[0.304]$$

or

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

Now again using $Y_t = L\left[1-(10)^{-K_D \cdot t}\right]$, we have

$$Y_{5(20^\circ)} = L\left[1-(10)^{-K_{D(20^\circ)}\times 5}\right]$$

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

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

$$= 361.8\,[1-0.316] = 247.4 \text{ mg/l.} \quad \text{Ans.}$$

Example 7.13. *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. Use eqn. (7.16) as :

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

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

$$Y_5 = \text{BOD of 5 days}$$

$$= 600 \text{ mg/l} = L\left[1-(10)^{-0.1\times 5}\right]$$

or $$600 \text{ mg/l} = L\left[1-(10)^{-0.5}\right] = L\left[1-\frac{1}{(10)^{0.5}}\right]$$

$$= L\left[1-\frac{1}{3.16}\right] = L\,[1-0.316] = 0.684\ L$$

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

$$\therefore \qquad L = \frac{600}{0.684} \text{ mg/l} = 877.5 \text{ mg/l}.$$

Hence, the ultimate BOD = 877.5 mg/l. **Ans.**

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

$$= Y_u\,[1-0.01] = Y_u\,[0.99]$$

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

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. **Ans.**

Example 7.14. *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% wastewater 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

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

$= 0.018 + 2.91 = 2.928$ mg/l.

D.O. of the incubated sample after 5 days

$$= 0.8 \text{ mg/l.}$$

Thus, D.O. consumed in oxidising organic matter

$$= 2.928 - 0.8 = 2.128 \text{ mg/l.}$$

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

$$= 2.128 \times \frac{100}{3} = 70.93 \text{ mg/l.}$$

Ultimate B.O.D. is given by L.

Using Eq. (7.16), we have

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

or

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

The value of K_D at test temp. is given as 0.1. Substituting the known values in Eq. (*i*) above, we have

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

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

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

or $$L = \frac{70.93}{0.684} = 103.7 \text{ mg/l.}$$ **Ans.**

Example 7.8. *Calculate the population equivalent of a city given (i) the average sewage from the city is 95×10^6 l/day, and (ii) the average 5 day BOD is 300 mg/l.*

Solution. Average 5 day BOD = 300 mg/l.

Average sewage flow = 95×10^6 l/day

$\therefore$ Total BOD in sewage

$$= 300 \times 95 \times 10^6 \text{ mg/day}$$

$$= 300 \times 95 \text{ kg/day} = 28500 \text{ kg/day}$$

Population equivalent

$$= \frac{\text{Total 5 day BOD in kg/day}}{0.08}$$

[assuming the domestic sewage quantity to be 0.08 kg/person/day]

$$= \frac{28500}{0.08} = 3{,}56{,}250. \textbf{ Ans.}$$

Example 9.1. *Estimate the screen requirement for a plant treating a peak flow of 60 million litres per day of sewage.*

Solution. Peak flow = 60 Ml/day

$$= \frac{60 \times 10^6}{1000} \text{ cu-m/day}$$

$$= \frac{60{,}000}{24 \times 60 \times 60} \text{ cu-m/sec} = 0.694 \text{ m}^3\text{/sec.}$$

Assuming that the velocity through the screens (at peak flow) is not allowed to exceed 0.8 m/sec, we have

The net area of screen openings required

$$= \frac{0.694}{0.8} \text{ m}^2 = 0.87 \text{ m}^2.$$

Using rectangular steel bars in the screen, having 1 cm width, and placed at 5 cm clear spacings, we have

The gross area of the screen required

$$= \frac{0.87 \times 6}{5} = 1.04 \text{ m}^2$$

Assuming that the screen bars are placed at 60° to the horizontal, we have

The gross area of the screen needed

$$= \frac{104}{\frac{\sqrt{3}}{2}} = \frac{104 \times 2}{\sqrt{3}} = 1.2 \text{ m}^2.$$

Hence, a coarse screen of 1.2 m^2 area is required. **Ans.**

While designing the screen, we have also to design its **cleaning frequency**. The cleaning frequency is governed by the head loss through the screen. The more the screen openings are clogged, more will be the head loss through the screen. Generally, not more than half the screen clogging is allowed. To know whether the screen has been clogged and needs cleaning, we can check or measure the head loss.

The head loss through the cleaned screen and half-cleaned screen, can be computed as follows :

Velocity through the screen = 0.8 m/sec.

Velocity above the screen

$$= \frac{0.8 \times 5}{6} \text{ m/sec} = 0.67 \text{ m/sec}$$

Head loss through the screen $= 0.0729\,(V^2 - v^2)$	...(9.1)

$$= 0.0729\,(0.8^2 - 0.67^2) = 0.0134 \text{ ; say } \mathbf{0.013\ m}.$$

When the screen openings get half clogged, then

The velocity through the screen

$$= v = 0.8 \times 2 = 1.6 \text{ m/sec}$$

$\therefore$ Head loss = $0.0729\,(1.6^2 - 0.67^2) = 0.1538$; **say 0.15 m.**

This shows that when the screens are totally clean, the head loss is negligible *i.e.* about 1.3 cm only ; whereas, the head loss shoots up to about 15 cm at half the clogging. The screens should therefore be cleaned frequently, as to keep the head loss within the allowable range. **Ans.**

GRIT CHAMBER

Example 9.2 (a) *A rectangular grit chamber is designed to remove particles with a diameter of 0.2 mm, specific gravity 2.65. Settling velocity for these particles has been found to range from 0.016 to 0.022 m/sec, depending on their shape factor. A flow through velocity of 0.3 m/sec will be maintained by proportioning weir. Determine the channel dimensions for a maximum wastewater flow of 10,000 cu m/day.*

Solution. Let us provide a rectangular channel section, since a proportional flow weir is provided for controlling velocity of flow.

Now,

Horizontal velocity of flow = V_h = 0.3 m/sec.

Settling velocity is between 0.016 to 0.022 m/sec, and hence let it be 0.020 m/sec.

Now, Q = velocity × cross-section

or $Q = V_h \times A$

where Q = 10,000 cu m/day

$$= \frac{10000}{24 \times 60 \times 60} \text{ m}^3/\text{s} = 0.116 \text{ m}^3/\text{s}$$

$\therefore \quad 0.116 = 0.3 . A$

$\therefore \quad 0.116 = 0.3.A$

$$A = \frac{0.116}{0.3} \quad \text{or} \quad A = 0.385 \text{ m}^2.$$

Assuming a water depth (H) of 1 m above the crest of the weir, which is kept at 0.3 m above the channel bottom, we have the width (B) of the basin as

$$1 \times B = 0.385$$

or $\quad B = 0.385$ m ; say **0.4 m.**

Overall depth of grit chamber (D)

= Water depth above the crest of weir + 0.3 m

+ Free board of 0.45 m

= 1.0 m + 0.3 m + 0.45 m = **1.75 m**

Now, settling velocity

$$V_s = 0.02 \text{ m/sec}$$

$$\therefore \quad \text{Detention time} = \frac{\text{Water depth in the basin}}{\text{Settling velocity}} = \frac{1}{0.02} = 50 \text{ secs}$$

$\therefore$ Length of the tank = V_h × Detention time = 0.3 × 50 m = **15 m.**

Example 9.3. *Design an aerated grit chamber for treating municipal waste water with average flow rate of 0.5 m^3/s (43.2 MLD). Assume the peak flow rate to be 3 times the average.*

Solution. Peak flow rate = 0.5 m^3/sec × 3 = 1.5 m^3/s

Assume average liquid detention time = 3 min = 180 sec.

∴ Aerator volume = 1.5 m^3/s × 180 s = 270 m^3

In order to drain the channel periodically for routine cleaning and maintenance, *use two chambers.*

$$\therefore \quad \text{Volume of one aerated channel} = \frac{270\ \text{m}^3}{2} = 135\ \text{m}^3$$

To determine the *dimensions of the aerated channel*, assume depth of 3 m and width-depth ratio of 2 : 1.

∴ Width of channel = Depth × 2 = 3 m × 2 = 6 m

$\therefore$ Length of channel $= \dfrac{135\,m^3}{3\,m \times 6\,m} = 7.5$ m

Increase the length by about 20% to account for inlet and outlet conditions.

$\therefore$ Provided length = 7.5 × 1.2 = **9 m.**

Hence, use 2 chambers, each of size 9 m × 6 m × 3 m depth. **Ans.**

Air supply requirement. Assume that 0.03 m^3/min per m length may be adequate.

$$\text{Air required} = 0.03\ \frac{m^3}{\text{min}\,.\,m} \times 9\ m = \mathbf{0.27\ m^3/min.}\quad \textbf{Ans.}$$

Volume of grit produced daily. Assume that 50 m^3/M.cum of sewage of grit is produced by the incoming sewage, the daily grit volume produced

$$= \text{Peak flow rate of sewage in } m^3/d \times \text{Grit produced per } m^3 \text{ of sewage}$$

$$= 1.5\ \frac{m^3}{s} \times \left(24 \times 3600\ \frac{s}{d}\right) \times \frac{50\ m^3}{10^6\ m^3} = \mathbf{6.48\ m^3/d.}\quad \textbf{Ans.}$$

[**Note.** The grit handling facilities must be based on sustained peak flow rate. Hence, the arrangement for removal of grit @ 6.48 m^3/d must be provided.]

Example 9.4. *Design a suitable grit chamber cum Detritus tank for a sewage treatment plant getting a dry weather flow from a separate sewerage system @ 400 l/s. Assume the flow velocity through the tank as 0.2 m/sec ; and detention period of 2 minutes. The max. flow may be assumed to be three times of dry weather flow.*

Solution. The length of the tank

$$= \text{Velocity} \times \text{Detention time} = 0.2 \times (2 \times 60) = 24 \text{ m.}$$

Since the peak flow is three times the DWF, let us provide three detritus tanks, each designed for passing D.W.F.

∴ The discharge passing through each tank = 400 l/s = 0.4 m^3/sec.

$$\therefore \quad \text{Cross-sectional area required} = \frac{\text{Discharge}}{\text{Velocity}} = \frac{0.4}{0.2} = 2 \text{ m}^2.$$

Assuming the water depth in the tank to be 1.2 m, we have the width of the tank

$$= \frac{\text{Area of } X\text{-section}}{\text{Depth}} = \frac{2}{1.2} = 1.67 \text{ m ; say } 1.7 \text{ m.}$$

Hence, use a Detritus tank with 24 m × 1.7 m × 1.2 m size.

At the top, a free-board of 0.3 m may be provided ; and at the bottom, a dead space depth of 0.45 m for collection of detritus may be provided.

Thus, the overall depth of the tank

$$= 1.2 + 0.3 + 0.45 = 1.95 \text{ m}.$$

The tank will be 1.7 m wide upto 1.5 m depth, and then the sides will slope down to form an elongated trough of 24 m length and 0.8 m width at the bottom with rounded corners, as shown in Fig. 9.10.

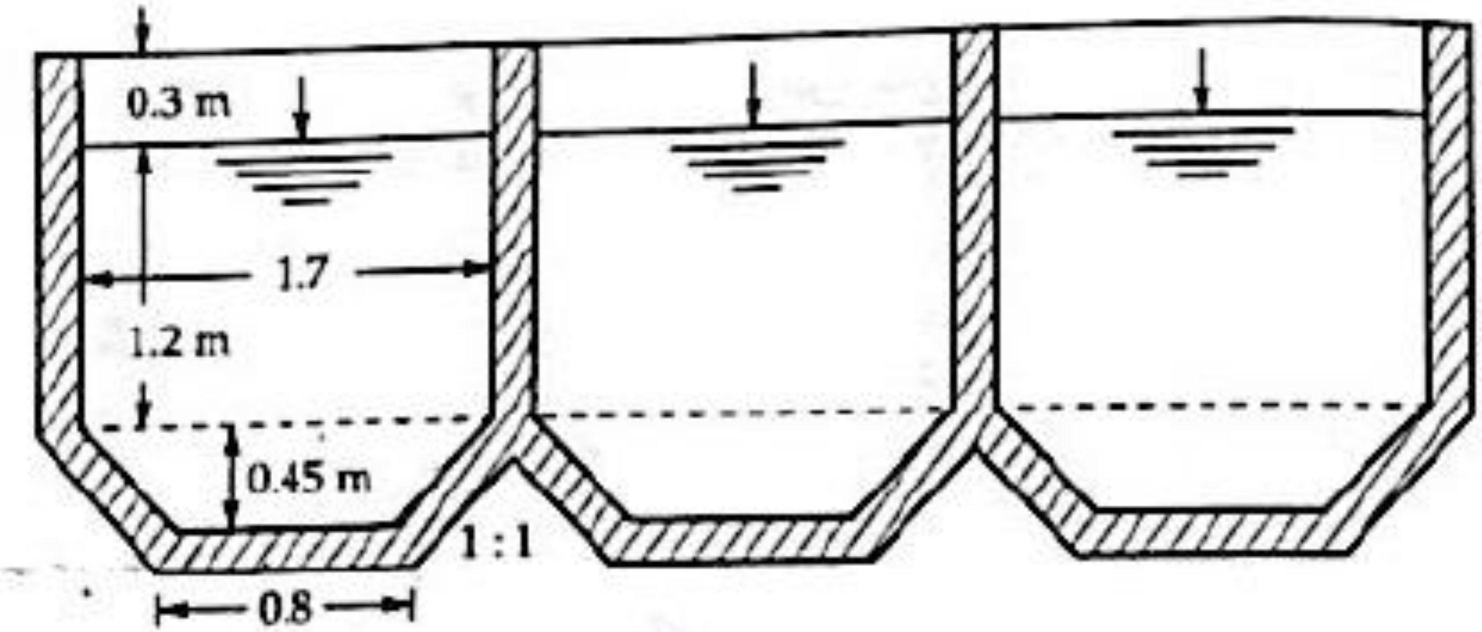

Fig. 9.10

SEDIMENTATION

called the *sludge zone.*

Example 9.5. *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 the quantity of sewage required to be treated per day (*i.e.* max. daily).

$$= 0.8 \times 12 \text{ million litres} = 9.6 \text{ M. litres}$$

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.* the capacity of the tank required

$$Q = \frac{9.6}{24} \times 2 \text{ M. litres} = 0.8 \text{ M litres} = 800 \text{ cu. m.}$$

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

The length of the tank required

$$= \text{Velocity of flow} \times \text{Detention period}$$

$$= 0.3 \times (2 \times 60) \text{ m} = 36 \text{ m.}$$

Cross-secticnal area of the tank required

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

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

The width of the tank required

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

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

The overall depth of the tank

$$= 3 + 0.5 = 3.5 \text{ m}.$$

Hence, a rectangular sedimentation tank with an overall size of 36 m × 7.4 m × 3.5 m can be used.

[Note. 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.]. **Ans.**

Alternatively,

Example 9.6. *Design a circular settling tank unit for a primary treatment of sewage at 12 million litres per day. Assume suitable values of detention period (presuming that trickling filters are to follow the sedimentation tank), and surface loading.*

Solution. Assuming the normal detention period for such cases as 2 hr, and surface loading as 40,000 litres/sq. m/day ; we have

The quantity of sewage to be treated per 2 hours

$$= 12 \text{ M. litres} \times \frac{2}{24} = 1 \text{ M. litres} = 1000 \text{ m}^3.$$

∴ Capacity of tank = 1000 m^3.

Now, surface loading

$$= \frac{Q}{\text{Surface area of tank}} = \frac{Q}{\frac{\pi}{4} . d^2}$$

4

or $$40{,}000 = \frac{12 \times 10^6}{\frac{\pi}{4} \cdot d^2}$$

where d is the dia. of the tank

or $$\frac{\pi}{4} \cdot d^2 = \frac{12 \times 10^6}{40{,}000}$$

or $$d = \sqrt{\frac{300 \times 4}{\pi}} = 19.55 \text{ m } \textbf{Say 19.6 m.}$$

Now, effective depth of tank

$$= \frac{\text{Capacity}}{\text{Area of X-section}} = \frac{1000}{\frac{\pi}{4} \times (19.6)^2} = \frac{1000}{302}$$

$$= 3.2 \text{ m. (Say).}$$

Hence, use a settling tank with 19.6 m dia. and 3.2 m water depth (with free board of 0.3 m extra depth). **Ans.**

The efficiency of such a conventional filter plant can be expressed by the equation evolved by National Research Council of U.S.A., and given as :

$$\eta\,(\%) = \frac{100}{1 + 0.0044\sqrt{u}} \qquad \ldots(9.32)$$

where η = Efficiency of the filter and its secondary clarifier, in terms of percentage of applied BOD removed.

u = Organic loading in kg/ha-m/ day applied to the filter (called unit organic loading).

of the filter media per day. The value of organic loading for conventional filters may vary between 900 to 2200 kg of BOD_5 per ha-m. This organic loading value can be further increased to about 6000—18000 kg of BOD_5 per ha-m in high rate trickling filters.

given in Eq. (9.5...).

Example 9.7. (*a*) *Design suitable dimensions of a circular trickling filter units for treating 5 million litres of sewage per day. The BOD of sewage is 150 mg/l.*

(*b*) *Also design suitable dimensions for its rotary distribution system, as well as the under-drainage system.*

Solution. Total BOD present in sewage to be treated per day

$$= 5\text{ ML} \times 150\text{ mg/L} = 5 \times 10^6 \times 150\text{ mg}$$

$$= 5 \times 150\text{ kg} = 750\text{ kg}.$$

Assuming the value of organic loading, say as, 1500 kg/hectare metre/day [*i.e.* between 900 to 2200 kg/ha-m/day], we have

The volume of filtering media required

$$= \frac{750}{1500}\text{ hectare-metre} = 0.5\text{ ha-m} = 5000\text{ m}^3.$$

Assuming the effective depth of filter, as, say 2 m, we have

The surface area of the filter required

$$= \frac{5000}{2}\text{ m}^2 = 2500\text{ m}^2.$$

Using a circular trickling filter of dia 40 m, we have the number of units required

$$= \frac{\text{Total area required}}{\text{Area of one unit}} = \frac{2500}{\frac{\pi}{4}(40)^2} \approx 2 \text{ Nos.}$$

can also be worked out by assuming the value of hydraulic loading, say as, 25 million litres per hectare per day [*i.e.* between 22 to 44 ML/ha/day]

∴ Surface area required

$$= \frac{\text{Total sewage to be treated per day}}{\text{Hydraulic loading per day}}$$

$$= \frac{5 \text{ ML/day}}{25 \text{ ML/ha/day}} \frac{5}{25} \text{ hectares}$$

$$= \frac{5}{25} \times 10{,}000 \text{ m}^2 = 2000 \text{ m}^2.$$

The surface area chosen is 2500 m^2, which is greater than 2000 sq. m, and hence safe.

Hence, 2 units each of 40 m dia and 2 m effective depth (*i.e.* 2.6 m overall depth), can be adopted. An extra third unit as stand-by may also be constructed. **Ans.**

Example 9.9. *A town having a population of 30,000 persons is producing the following sewages :*

(*i*) *Domestic sewage @ 120 l.p.c.d. having 200 mg/l of BOD.*

(*ii*) *Industrial sewage @ 3,00,000 l.p.d. having 800 mg/l of BOD.*

Design high rate single stage trickling filters for treating the above sewage. Assuming that the primary sedimentation removes 35% of BOD. Allow an organic loading of 10,000 kg/ha-m/day (excluding recirculated sewage). The recirculation ratio is 1.0 ; and the surface loading should not exceed 170 M.l./ha/day (including recirculated sewage). Also determine the efficiency of the filter and the BOD of the effluent.

Solution. Quantity of domestic sewage produced per day

$$= 120 \times 30,000 \text{ litres/day} = 3.6 \text{ M}.l\text{/day}.$$

BOD for domestic sewage = 200 mg/l.

∴ Total BOD of domestic sewage per day

$$= 3.6 \times 200 \text{ kg/day} = 720 \text{ kg/day} \quad ...(i)$$

Quantity of industrial sewage produced per day

$$= 3,00,000 \text{ litres.}$$ /d

BOD of industrial sewage = 800 mg/l

∴ Total BOD of industrial sewage

$$= \frac{3,00,000 \times 800}{10^6} \text{ kg/day}$$

$$= 240 \text{ kg} \qquad \ldots(ii)$$

Total BOD of domestic as well as industrial sewage per day

$$= 720 + 240 = 960 \text{ kg/day}$$

Out of this BOD, 35% is already removed in primary clarifier.

∴ BOD to be removed by filter unit

$$= 960 \times (0.65) = \mathbf{624\ kg/day.}$$

Volume of filter media required

$$= \frac{\text{Total BOD removed}}{\text{Organic loading}} = \frac{624}{10,000} \text{ ha-m}$$

$$= \frac{624}{10,000} \times 10^4 \text{ m}^3 = \mathbf{624\ m^3.}$$

Now, the total volume of sewage flowing

$$= (3.6 \times 10^6 + 3,00,000) \text{ litres/day}$$

$$= (3.9 \times 10^6) \text{ litres/day} = 3.9 \text{ M}l\text{/day}$$

A recirculation ratio of 1 means that the volume of recirculated sewage

$$(R) = \text{Original volume} = 3.9 \text{ M.}l\text{/day} \qquad \ldots(i)$$

Total volume (*i.e.*, original + recirculated)

$$= 2 \times 3.9 \text{ M.}l\text{/day} = 7.8 \text{ M.}l\text{/day}$$

$= 2 \times 3.9$ Ml/day = 7.8 Ml/day

∴ Filter area required

$$= \frac{\text{Total flow volume}}{\text{Surface loading}} = \frac{7.8 \text{ Ml/d}}{170 \text{ Ml/ha.d}}$$

$$= \frac{7.8}{170} \text{ hectares} = \frac{7.8}{170} \times 10^4 \text{ m}^2 = 458.8 \text{ m}^2.$$

∴ Dia of the filter tank required

$$= \sqrt{\frac{458.8 \times 4}{\pi}} = 24.17 \text{ m.}$$

Hence, use, say, 24 m dia tank with area as = 452.16 m^2. **Ans.**

∴ Depth of filter media required

$$= \frac{\text{Volume of filter media}}{\text{Surface area}} = \frac{624 \text{ m}^3}{452.16 \text{ m}^2} = 1.38 \text{ m.} \quad \textbf{Ans.}$$

Efficiency of this filter is given by Eq. (9.34) as

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

where Y = Total organic load, *i.e.*, total BOD applied to filter in kg/day = 624 kg per day.

V = Volume of filter in ha-m

$$= \frac{624}{10{,}000} = 0.0624 \text{ ha-m}$$

F = Recirculation factor as given by Eq. (9.33) as :

$$= \frac{1+\frac{R}{I}}{\left[1+0.1\frac{R}{I}\right]^2} \text{ where } \frac{R}{I} = 1 \text{ (given)}$$

$$= \frac{1+1}{(1+0.1)^2} = \frac{2}{(1.1)^2} = \frac{2}{1.21} = 1.65$$

or $$\eta = \frac{100}{1+0.0044\sqrt{\frac{624}{0.0624\times1.65}}} = \frac{100}{1+0.0044\sqrt{\frac{10^4}{1.65}}} = 74.5\% \quad \textbf{Ans.}$$

BOD of the effluent left

$$= \frac{(100-74.5)}{100}\times 624 \text{ kg/day}$$

$$= \frac{25.5}{100}\times 624 = 159.12 \text{ kg}$$

Total volume of effluent = 3.9 M l/day

∴ BOD concentration in the effluent

$$= \frac{159.12\times10^6}{3.9\times10^6} \text{ mg}/l = 40.8 \text{ mg}/l. \quad \textbf{Ans.}$$

Example 9.10. *Determine the size of a high rate trickling filter for the following data :*

(i) *Sewage flow = 4.5 Mld ;*

(ii) *Recirculation ratio = 1.5 ;*

(iii) *BOD of raw sewage = 250 mg/l ;*

(iv) *BOD removal in primary tank = 30% ;*

(v) *Final effluent BOD desired = 30 mg/l.* (A.M.I.E. 1974)

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

$$= (1125)\ 0.7 = 787.5 \text{ kg.}$$

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, using equation (9.34), we have

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

where $\eta = 82.85\%$

Y = Total BOD in kg = 787.5 kg.

$$F = \frac{1 + \frac{R}{I}}{\left(1 + 0.1\frac{R}{I}\right)^2} \text{ ; where } = \frac{R}{I} = 1.5 \text{ (given)}$$

$$\therefore \quad F = \frac{1 + 1.5}{[1 + 0.1 \times 1.5]^2} = \frac{2.5}{(115)^2} = \frac{2.5}{1.322} = 1.89$$

$$\therefore \quad 82.85 = \frac{100}{1 + 0.0044\sqrt{\frac{787.5}{V \times 189}}}$$

or
$$1 + 0.0044 \,.\, \sqrt{\frac{416.6}{V}} = 1.207$$

or
$$\sqrt{\frac{416.6}{V}} = \frac{0.207}{0.0044} = 47.05$$

or $$\frac{416.6}{V} = 2213.3$$

or $$V = 0.188 \text{ hectare-m.} = 1880 \text{ m}^3$$

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

The surface area required

$$= \frac{1880}{1.5} \text{ m}^2 = 1253 \text{ m}^2$$

∴ Dia of the circular filter required

$$= \sqrt{1253 \times \frac{4}{\pi}} = 40 \text{ m.}$$

Hence, use a high rate trickling filter with 40 m dia., 1.5 m deep filter media, and with recirculation (single stage) ratio of 1.5. **Ans.**

Example 9.11. *Determine the size of a high rate trickling filter for the following data :*

Flow	*= 4.5 Mld*
Recirculation ratio	*= 1.4*
BOD of raw sewage	*= 250 mg/l*
BOD removed in primary clarifier	*= 25%.*
Final effluent BOD desired	*= 50 mg/l.*

Calculate also the size of the standard rate trickling filter to accomplish the above requirement. (Calcutta University 1967)

Solution. Total BOD present in raw sewage per day

$= 4.5 \text{ Ml} \times 250 \text{ mg/l} = 1125 \text{ kg}.$

BOD removed in the primary clarifier = 25%.

$\therefore$ BOD entering per day in the filter units

$= 0.75 \times 1125 \text{ kg} = 843.75 \text{ kg}.$

Permissible BOD concentration in the effluent = 50 mg/*l*.

$\therefore$ BOD allowed to go into the effluent

$= 50 \text{ mg/l} \times 4.5 \text{ Ml} = 225 \text{ kg}.$

$\therefore$ BOD removed by the filter per day = 843.75 − 225 = 618.75 kg.

$$\text{Efficiency of the filter} = \frac{\text{BOD removed}}{\text{Total BOD entering}} \times 100 = \frac{618.75}{843.75} \times 100 = 73.3\%.$$

Now, efficiency of the filter is given by Eq. (9.34) as

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

where Y = Total BOD applied to the filter per day in kg.

$= 843.75$ kg

F = Recirculation factor

V = Vol of filter in ha.m.

$$= \frac{1+\frac{R}{I}}{\left[1+0.1\frac{R}{I}\right]^2} \text{ ; where } \frac{R}{I} = 1.4 \text{ (given)}$$

$$= \frac{1+1.4}{(1+0.1\times 1.4)^2} = \frac{2.4}{(1.14)^2} = \frac{2.4}{1.3} = 1.85$$

$$\therefore \quad 73.3 = \frac{100}{1+0.0044\sqrt{\frac{843.75}{V\times 1.85}}}$$

or $$1+0.0044\sqrt{\frac{456}{V}} = \frac{100}{73.3} = 1.364$$

or $$\sqrt{\frac{456}{V}} = \frac{0.364}{0.0044} = 82.78$$

or $$\frac{456}{V} = 6853$$

or $$V = \frac{45.6}{6853} \text{ hectare-m.} = \frac{456}{6853}\times 10^4 \text{ m}^3 = 665.4 \text{ m}^3.$$

Using 1.5 m depth of the filter, we have

Area required $= \frac{665.4}{1.5} = 413.6 \text{ m}^2$ 443.6

$\therefore$ Dia of the filter tank required

$$= \sqrt{\frac{413.6\times 4}{\pi}} = 23.8 \text{ m.} \quad \textbf{Ans.}$$

For an equivalent standard rate filter ; F = 1.

$$\therefore \quad 73.3 = \frac{100}{1 + 0.0044 \cdot \sqrt{\frac{843.75}{V}}}$$

or $$1 + 0.0044 \cdot \sqrt{\frac{843.75}{V}} = \frac{100}{73.3} = 1.364$$

or $$\sqrt{\frac{843.75}{V}} = \frac{0.364}{0.0044} = 82.73$$

or $$\frac{843.75}{V} = 6843$$

or $$V = \frac{843.75}{6843} \text{ ha-m} = 0.1233 \text{ ha-m.} = 1233 \text{ m}^3$$

$$\left[\because \begin{array}{l} \text{ha-m} = 10^4 \text{ sq. mm.} \\ \quad = 10^4 \text{ m}^3 \end{array}\right]$$

Using depth of filter as 1.5 m, we have

$$\text{Surface area required} = \sqrt{\frac{1233}{1.5}} = 822 \text{ m}^2$$

$\therefore$ Dia of the filter tank required

$$= \sqrt{\frac{822 \times 4}{\pi}} = 32.4 \text{ m.} \quad \textbf{Ans.}$$

Example 9.12. *A single stage filter is to treat a flow of 3.79 M.l.d. of raw sewage with BOD of 240 mg/l. It is to be designed for a loading of 11086 kg of BOD in raw sewage per hectare metre, and the recirculation ratio is to be 1. What will be the strength of the effluent, according to the recommendations of the National Research Council of U.S.A.*

Solution. Total BOD present in raw sewage

$$= 3.79 \text{ Ml} \times 240 \text{ mg/l} = 909.6 \text{ kg}$$

Now, filter volume required

$$= \frac{\text{Total BOD in raw sewage in kg}}{\text{Given BOD loading rate of 11,086 kg / ha-m}}$$

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

Now, assuming that 35% of BOD is removed in primary clarifier, we have

The amount of BOD applied to the filter

$$= 0.65 \times 909.6 \text{ kg} = 591.24 \text{ kg.}$$

Now, using equation (9.34), we have

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

where Y = Total BOD applied to the filter in kg

$= 591.24$ kg

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

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

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

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

∴ The amount of BOD left in the effluent

$$= 591.24\,[1 - 0.7747]\ \text{kg.} = 133.21\ \text{kg.}$$

∴ BOD concentration in the effluent

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

Example 9.13. *It is proposed to use a two stage plant instead of the single stage plant in example 9.12. The total volume of filter medium remains the same as was in one filter, i.e. 0.082 ha-m, and each filter is to contain half of this material, and the recirculation ratio is to be 1 for each filter. Determine the BOD of the plant effluent.*

Solution. For each filter $F = 1.65$.

For the first stage filter, the efficiency is given by

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

where Y = Total BOD applied to filter

= 591.24 kg (from previous example)

V = Volume of filter $= \dfrac{0.082}{2} = 0.041$ ha-m

$$\therefore \quad \eta = \frac{100}{1 + 0.0044\sqrt{\dfrac{591.24}{0.041 \times 1.65}}} = 70.85\%$$

∴ Percentage of BOD removed in first stage filter = 70.85%.

∴ Amount of BOD left in the effluent from that filter

$= 591.24\,[1 - 0.7085] = 172.32$ kg.

For the second stage filter, the efficiency is given by equati...

For the second stage filter, the efficiency is give[n]

The efficiency $\eta' = \dfrac{100}{1 + \dfrac{0.0044}{1-\eta}\sqrt{\dfrac{Y'}{V'.F'}}}$

where $Y' = 172.32$ kg

$V' = 0.041$ ha-m.

$F' = 1.65$.

$\eta = 0.7085$

$$\eta' = \frac{100}{1 + \dfrac{0.0044}{1-0.7085}\sqrt{\dfrac{172.32}{0.041 \times 1.65}}} = \frac{100}{1 + \dfrac{0.0044}{0.2915}\sqrt{\dfrac{172.32}{0.041 \times 1.65}}}$$

$= 56.76\%$

The amount of BOD left in the effluent from the plant

$$= 171.9\left[\frac{100 - 56.76}{100}\right] \text{kg.} = 74.33 \text{ kg.}$$

BOD concentration in the effluent

$$= \frac{\text{Total BOD}}{\text{Sewage volume}} = \frac{74.33 \times 10^6}{3.79 \times 10^6} \text{ mg}/l = 19.61 \text{ mg}/l. \quad \textbf{Ans.}$$

Example 9.14. *The design flow of sewage is 3.8 million litres per day, and the BOD of the raw sewage is 300 mg/l. Design a single stage Bio filter to produce an effluent having a BOD of 45 mg/l or less.*

Solution. Total BOD present in raw sewage per day

$$= 3.8 \times 300 \text{ kg.} = 1140 \text{ kg.}$$

Assuming that 35% of this BOD is removed in the primary sedimentation tank, we have

The total daily BOD applied to the filter = 0.65 × 1140 kg = 741 kg.

Now, the total daily BOD present in the effluent (permissible maximum)

$$= 3.8 \times 45 \text{ kg.} = 171 \text{ kg.}$$

∴ Total daily BOD to be removed by the filter = 741 – 171 = 570 kg.

$$\therefore \quad \text{Efficiency of the filter} = \frac{570}{741} \times 100 = 76.92\%.$$

Assuming an organic loading of say 10,000 kg/ha-m/day [*i.e.*, between 9,000 to 14,000], we have

$$\text{Volume of filter required} = \frac{\text{Total daily BOD removed}}{\text{Organic loading}}$$

$$= \frac{570}{10,000} \text{ ha-m.} = 0.057 \text{ ha-m.} = 570 \text{ m}^3$$

Organic loading

$$= \frac{570}{10,000} \text{ ha-m.} = 0.057 \text{ ha-m.} = 570 \text{ m}^3$$

Now, using equation (9.34), we have

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

where Y = Total daily BOD applied to filter in kg
= 570 kg.
V = Volume of filter = 0.057 ha-m.
and η = 76.92% (worked out earlier)

$$\therefore \quad 76.92 = \frac{100}{1 + 0.0044 \, . \sqrt{\frac{570}{0.057\,F}}}$$

or

$$76.92 = \frac{100}{1 + 0.44 \frac{1}{\sqrt{F}}}$$

or $$1 + \frac{0.44}{\sqrt{F}} = \frac{100}{76.92} = 1.3$$

or $$\frac{0.44}{\sqrt{F}} = 0.3$$

or $$F = \left(\frac{0.44}{0.3}\right)^2 = 2.15$$

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

or $$2.15 = \frac{1+\frac{R}{I}}{\left(1+0.1\frac{R}{I}\right)^2}$$

This gives $\frac{R}{I} = 1.47$ (Solving by trial)

$$\text{Detention period } (t) = \frac{\text{Volume of the tank}}{\text{Rate of sewage flow in the tank}}$$

$$= \frac{V \text{ in m}^3}{Q \text{ in m}^3/\text{day}}$$

$$= \frac{V}{Q} \text{ day} \quad \text{...(9.41}a\text{)}$$

$$t = \frac{V}{Q} \cdot 24 \text{ hour} \quad \text{...(9.41)}$$

where t = aeration period in hours

V = Volume of aeration tank

Q = Quantity of wastewater flow into the aeration tank, *excluding the quantity of recycled sludge.*

Volumetric BOD loading or Organic loading

$$= \frac{\text{Mass of BOD applied per day to the aeration tank through influent sewage in gm}}{\text{Volume of the aeration tank in m}^3}$$

$$= \frac{Q \cdot Y_0 (\text{gm})}{V (\text{m}^3)} \quad \text{...(9.42)}$$

where Q = Sewage flow into the aeration tank in m^3.

Y_0 = BOD_5 in mg/l (or gm/m^3) of the influent sewage.

V = Aeration tank volume in m^3

∴ F/M ratio

$$= \frac{\text{Daily BOD load applied to the Aerator System in gm}}{\text{Total Microbial mass in the system in gm}} \quad ...(9.43)$$

If Y_0 (mg/l) represents the 5 day BOD of the influent sewage flow of Q m^3/day, then eventually,

The BOD applied to the Aeration system = Y_0 mg/l or gm/m^3

∴ *BOD load applied to the aeration system* = $F = Q \,.\, Y_0$ gm/day ...(i)

$$M = \text{MLSS} \times V$$
$$= X_T \,.\, V$$

where X_T is MLSS in mg/l

ng (i) by (ii), we get

$$\text{F/M ratio} = \frac{F}{M} = \frac{Q \,.\, Y_0}{V \,.\, X_t}$$

∴ Sludge age (θ_c)

$$= \frac{\text{Mass of suspended solid (MLSS*) in the system } (M)}{\text{Mass of solids leaving the system per day}}$$

$$\text{Sludge age} = \theta_c = \frac{V \cdot X_T}{Q_W \cdot X_R + (Q - Q_W) X_E} \quad ...(9.48)$$

where X_T = Concentration of solids in the influent of the Aeration Tank, called the MLSS, *i.e. Mixed Liquor Suspended Solids*, in mg/*l*.

V = Volume of Aerator

Q_W = Volume of wasted sludge per day.

X_R = Concentration of solids in the returned sludge or in the wasted sludge (both being equal) in mg/*l*

Q = Sewage inflow per day

X_E = Concentration of solids in the effluent in mg/*l*

Example 9.29. *An average operating data for conventional activated sludge treatment plant is as follows :*

(1) *Wastewater flow* $= 35000\ m^3/d$

(2) *Volume of aeration tank* $= 10900\ m^3$

(3) *Influent BOD* $= 250\ mg/l$

(4) *Effluent BOD* $= 20\ mg/l$

(5) *Mixed liquor suspended solids (MLSS)* $= 2500\ mg/l$

(6) *Effluent suspended solids* $= 30\ mg/l$

(7) *Waste sludge suspended solids* $= 9700\ mg/l$

(8) *Quantity of waste sludge* $= 220\ m^3/d.$

Based on the information above, determine:

(a) *Aeration period (hrs)*

(b) *Food to microorganism ratio (F/M) (kg BOD per day/kg MLSS)*

(c) *Percentage efficiency of BOD removal*

(d) *Sludge age (days).*

(G.A.T.E., 1993)

(a) Sludge age (days).

Solution. Given values are symbolised as :

$Q = 35000\ m^3/d$; $V = 10900\ m^3$

$Y_0 = 250$ mg/l ; $Y_E = 20$ mg/l

$X_T = 2500$ mg/l ; $X_E = 30$ mg/l

$X_R = 9700$ mg/l ; $Q_W = 220\ m^3/d$

These values are now used to calculate the desired factors, as below :

(a) Aeration period (t) in hr is given by Eq. (9.41) as

$$t = \frac{V}{Q} \cdot 24 = \frac{10,900}{35,000} \times 24 = 7.47 \text{ h} ; \text{ say } 7.5 \text{ h.} \quad \textbf{Ans.}$$

(b) F/M ratio F = Mass of BOD applied to aeration system

$= Q \cdot Y_0 = 35000 \times 250$ gm/day

$$= \frac{35000 \times 250}{1000} \text{ kg/day} = \mathbf{8750 \text{ kg/day}}$$

M = Mass of MLSS

$$= V . X_T = 10900 \text{ m}^3 \times 2500 \text{ mg/l } (\textit{i.e. } \text{gm/m}^3)$$

$$= \frac{10900 \times 2500}{1000} \text{ kg} = \mathbf{27{,}250 \text{ kg}}$$

$$\therefore \quad F/M \text{ ratio} = \frac{8750}{27{,}250}$$

$$= \mathbf{0.32 \text{ kg BOD per day/kg of MLSS.}} \quad \textbf{Ans.}$$

(c) *Percentage efficiency of BOD removal*

$$= \frac{\text{Incoming BOD} - \text{Outgoing BOD}}{\text{Incoming BOD}}$$

$$= \frac{250 - 20}{250} \times 100\% = \frac{230}{250} \times 100\% = \mathbf{92\%.} \quad \textbf{Ans.}$$

250 250

(*d*) Sludge age in days (θ_c) is given by Eq. (9.48) as

$$\theta_c = \frac{V \cdot X_T}{Q_W \cdot X_R + (Q - Q_W) \cdot X_E}$$

$$= \frac{27250 \text{ kg}}{(220 \text{ m}^3/\text{d} \times 9700 \text{ mg/l}) + (35000 \text{ m}^3/\text{d} - 220 \text{ m}^3/\text{d})\, 30 \text{ mg/l}}$$

$$= \frac{27250 \text{ kg}}{\frac{220 \times 9700}{1000} \text{ kg/d} + (35000 - 220)\frac{30}{1000} \text{ kg/d}}$$

$$= \frac{27250}{2134 + 1043.4} = \frac{27250}{3177.4} = 8.58 \text{ days.}$$ **Ans.**