Chapter 3
Suspended growth treatment systems
(Aerobic & Anaerobic)

3.1 Introduction:-

Many treatment systems based on suspended microorganisms have been developed and still used until now. Some of these systems are aerobic and other are anaerobic. Some of the most commonly used systems are:-

* Activated sludge systems:-

- Conventional activated sludge system.
- Oxidation ditches.
- Sequencing batch reactor (SBR)
- Aerated lagoons.
- Waste stabilization ponds.
- Up flow anaerobic sludge blanket (UASB)

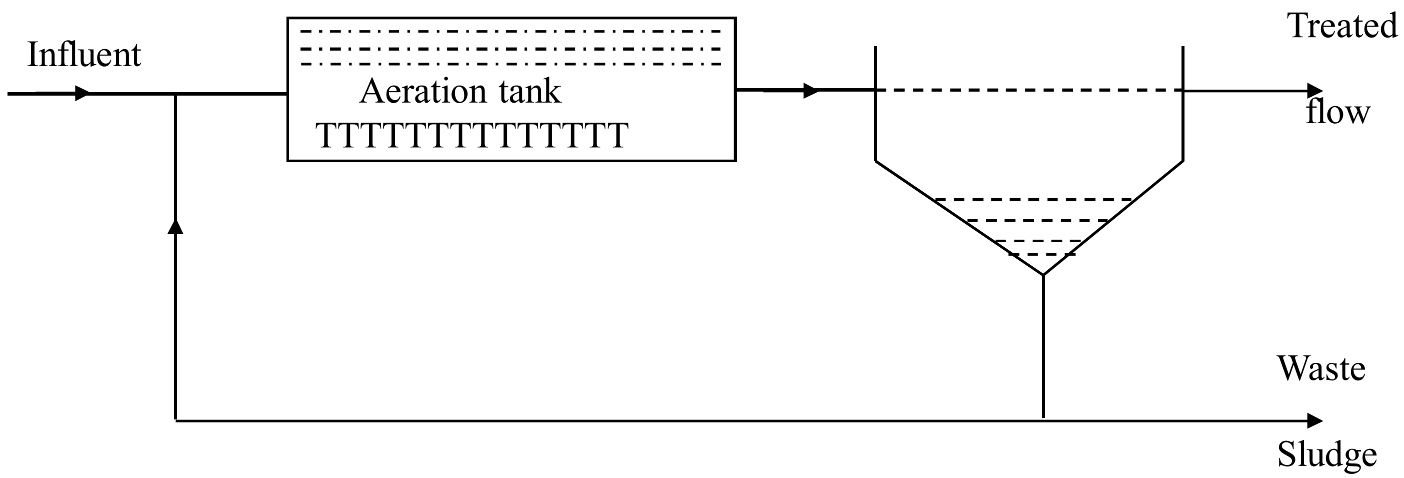
3.2 conventional activated sludge system:-

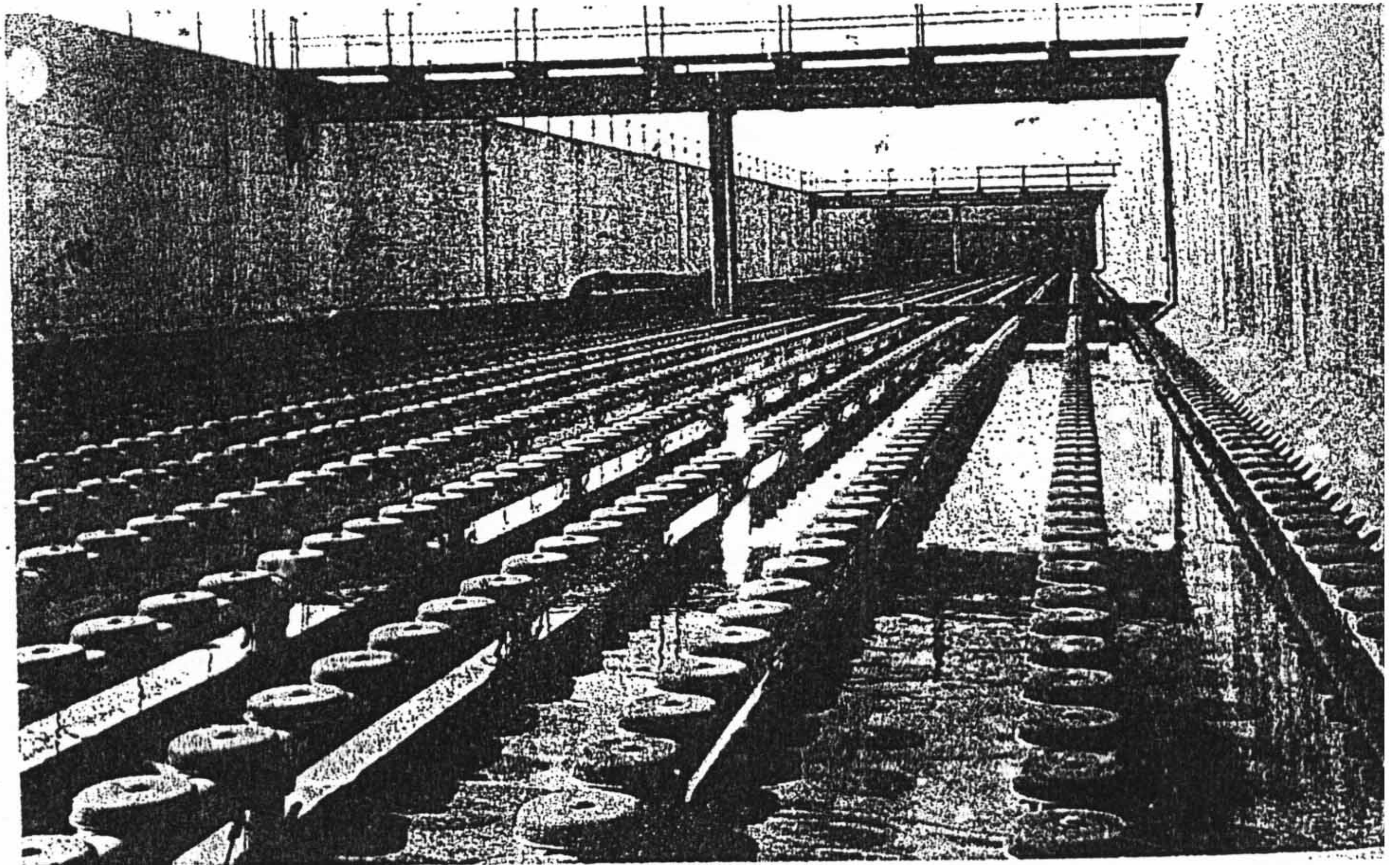
* The first version of activated sludge systems is called the conventional activated sludge system.

This system is composed of two parts:-

a. Aeration tank.
b. Final sedimentation tank.

•The aeration tank in this system can be designed either as a complete mixed flow reactor (CMFR) or as a plug flow reactor (PFR).





Plug-flow aeration tank equipped with dome aeration devices

A. Design of activated sludge system as a (CMFR).

a. design of the aeration tank:-

- To design the aeration tank we need to find:-
- Tank volume.
- Recycle ratio.
- Sludge wasting.
- Oxygen requirements.
- Check for some parameters such as θ_c ,θ, and S.
- **The aeration tank can be used to:-**
- Remove BOD only. (one sludge) or (separate stage)
- Nitrify only (convert NH_4^+ to NO_3^-) (one sludge or separate stage)
- Remove BOD + nitrify (two sludge or single stage).

Example 3.1:-
A completely mixed activated sludge system is to be used for organic matter removal only (one sludge system). Design this system knowing the following:-

*** microorganisms growth constants are:-**
$\mu_m = 2.5 d^{-1}$, $K_d = 0.05 d^{-1}$, Y = 0.5 mg VSS/mg BOD_5 removed
$K_S = 100$ mg BOD_5 / L

* Flow = 0.15 m^3/s = 12960 m^3/d
BOD_5 = 84 mg/L (Soluble)

* required effluent → $(BOD_5)_{total}$ = 30 mg/L
Suspended solids (SS) = 30 mg/L

Solution:-
1. Since we always deal with soluble substrate, first we need to find the effluent soluble BOD5 :-
(BOD5) soluble = S = (BOD5)total - BOD5 in suspended solids or paticulate
(BOD5) in suspended solids = 63% * SS
(BOD5)particulate = 0.63 * 30 = 18.9 mg BOD5 /L
(BOD5)soluble in effluent = 30 – 18.9 = 11.1 mg BOD5 /L
*Note: (BOD5)soluble in effluent = S = (BOD5) soluble in the aeration tank.

Follow Example 3.1:-

2. Calculate θ_C:-

$$S = \frac{K_s(1+K_d\theta_c)}{\theta_c(\mu_m - K_d)-1}$$ (this is equ. (18) for CMFR)

$$1.11 = \frac{100[1+0.05*\theta_c]}{\theta_c[2.5-0.05]-1} \Rightarrow \text{Solve for } \theta_C$$

θ_C = 5 days

- **Check for minimum sludge retention time θ_C^m:-**

$= \mu_m$ - K_d = 2.5 – 0.05 = 2.45

θ_c^m = 0.408 d

So $S.F = \frac{\theta c}{\theta_c^m} = \frac{5}{0.408}$ = 12.25 d (2<12.25<20) OK

- **Check for s_{min}:-**

$$S_{min} = K_s \frac{Kd}{\mu_m - K_d} = 100\left[\frac{0.05}{2.5-0.05}\right] = 2.04 \text{ mg } BOD_5/L$$

$S > S_{min} \rightarrow$ o.k $(11.1 > 2.04)$

So use $\theta_c = 5$ days, $S = 11.1$ mg BOD_5/L

3. Calculate the aeration tank volume:-

- Assume the concentration of biomass (X) is equal to 3000mg MLVSS /L:-

$$X = \frac{\theta_c}{\theta} \cdot \frac{Y(S_0 - S)}{(1 + K_d \theta_c)} \quad \text{(equ. 17)}$$

$$3000 = \frac{5 * 0.5[84 - 11.1]}{\theta[1 + 0.05 * 5]} \quad \text{(solve for } \Theta\text{)}$$

$\theta = 0.0486$ day

$\theta = 1.17$ hours

- Calculate the volume:-

$V = \theta Q = 0.0486*12960 \approx 630 \text{ m}^3$

• Check F/M ratio:-

$$F/M = \frac{12960 m^3}{d} \cdot \frac{1}{630 m^3} \cdot \frac{L}{3000 mg} \cdot \frac{84 mg}{L} \cdot \frac{10^3 L}{1 m^3} \cdot \frac{1 m^3}{10^3 L}$$

F/M = 0.576 mg BOD_5 /mg MLVSS .d (O.K)

Typical range for conventional activated sludge system is 0.1 – 0.6 mg BOD_5 /mg MLVSS .d.

this F/M is accepted. In case that we need to change F/M we can change the assumed X.

4. Calculate the amount of sludge to be wasted:

$P_x = Y_{obs}\, Q\, (S_o - S)$

$$Y_{obs} = \frac{Y}{1 + K_d \theta c} = \frac{0.5}{1 + 0.05 * 5} = 0.4 mgVss / mgBoD_5$$

$$P_x = 0.4 \frac{mg_{biomass}}{mgBoD_5} * 12960 \frac{m^3}{d} [84 - 11.1] \frac{mg.BOD_5}{L} \cdot \frac{10^3 L}{m^3} \cdot \frac{1 Kg}{10^6 mg_{biomass}}$$

$$P_x \cong 378 Kg / d$$

Another way to find P_X:-

$$P_x = Q_w X_r + Q_e X_e = \frac{XV}{\theta_c} = 3000 \ \frac{mg}{L} * \frac{630 \ m^3}{1 m^3} \cdot \frac{10^3 L}{5 d} \cdot \frac{kg}{10^6 mg}$$

$$P_x = 378 \ kg / d$$

• Calculate Q_w (waste sludge flow):-

Assume X_r = 10000 mg VSS/L (Typical range: 8000 – 12000 mg VSS/L)

$P_x = Q_W X_r + Q_e X_e$, (neglect X_e compared to X_r)

$$P_x = Q_w X_r \Rightarrow Q_w = \frac{P_x}{X_r} = \frac{378 * 10^6 \ mg / d}{10.000 \ mg / l} = 37800 \ \frac{L}{d}$$

$Q_W = 37.8 \ m^3/d$

5. Calculate the recycle flow Q_r:-

$$\frac{Q_r}{Q} = \frac{X}{X_r - X} = R$$ (Sometimes called $\propto$ or recycle ratio)

$$R = \frac{3000}{10000 - 3000} = 0.43$$

$$Q_r = 0.43 \ Q \cong 5573 \ m^3 / d$$

6. Calculate oxygen requirements:-

$$R_0 = Q(S_0 - S) - 1.42 \ P_x$$

$$= 12960 \ \frac{m^3}{d} [84 - 11.1] \frac{mgBoD_5}{L} \cdot \frac{10^3 L}{1 m^3} \cdot \frac{1 kg}{10^6 mg} - 1.42 * 378 \ \frac{kg}{d}$$

$$R_0 = 408 \ KgO_2 / d$$

Example 3.2:-

For example 3.1, we need to design the CMFR system for both organic matter removal and nitrification. The microorganisms growth constants for hetrotrophs are the same as in example 3.1, and for nitrifies (i.e. autotrophs) are:-

$\mu_{max} = 0.25 d^{-1}$, $Y_n = 0.2$ mg Vss/mg NH_4-N, $K_d = 0.04 d^{-1}$

$K_n = 0.4$ mg/L

It is also given that:-

TKN = 40 mg/L (in the influent of the reactor)

TKN = 1 mg/L (effluent nitrogen goal).

Solution:-

1. It was calculated in example 3.1 that θ_c required for BOD_5 removal was = 5 days.

- We need to check if this θ_c is enough for to achieve complete nitrification.

- **Find S_{min} for nitrogen:-**

$$S_{min} = K_n \frac{(k_d)_n}{(\mu_m)_n - (k_d)_n} = 0.4 \frac{0.04}{0.5 - 0.04} = 0.035 \text{ mg N/L} < 1 \text{ mg N/L}$$

(OK)

- Calculate θ_c for complete nitrification:-

$$N = \frac{K_n\left[1 + (K_d)_n \theta_c\right]}{\theta_c\left[(\mu_m)_n - (K_d)_n\right] - 1}$$

$$1 = \frac{0.4(1 + 0.04 * \theta_c)}{\theta_c(0.25 - 0.04) - 1}$$

Solve for $\theta_c \Rightarrow \theta_c = 7.2$ days

So $(\theta_c)_n > (\theta_c)_{BOD5} \Rightarrow$ so take $\theta_c = 7.2$ days for design purposes.

2. Cheek for $(\theta_c^{min})_n$:-

$$\frac{1}{\theta_c^m} = (\mu_m)_n - (K_d)_n = 0.25 - 0.04 \Rightarrow \theta_c^{min} = 4.76 days$$

$$S.F = \frac{7.2}{4.76} = 1.5 < 2 \quad \text{not OK}$$

So take S.F = 2.1 $\Rightarrow$ $\theta_c = 2.1 \text{ X } 4.76 \cong 10$ days

So take $\theta_c = 10$ days

3. Calculate the actual S and N in the effluent:-

$$N = \frac{0.4(1+0.04*10)}{10(0.25-0.04)-1} = \frac{0.56}{1.1} = 0.51\, mg\ N/L > 0.035\, mgN\ /L \quad \text{OK}$$

$$S = \frac{100\ (1+0.05*10)}{10\ (2.5-0.05)-1} = \frac{150}{23.5} = 6.38\ mg\ BOD\,5/L > 2.04\ mg\ BOD_5/L \quad \text{OK}$$

4. Calculate θ:-

Assume thjat $\frac{X_{nitrifiers}}{X_{Total}} = 0.10$ (this ratio is called nitrifies fraction f_n)

So $X_{nitrifiers}$ = 0.1 x 3000 = 300 mg Vss/L

$X_{heterotrophs}$ = 0.9 X 3000 = 2700 mg Vss/L

* θ for heterotrophs:-

$$\theta = \frac{\theta_c\, Y\,(S_o - S)}{X\,(1+K_d\,\theta_c)} = \frac{10*0.5(84-6.38)}{2700\,(1+0.04*10)} = \frac{388.1}{3780} = 0.103\ d$$

*θ for nitrifies:-

$$\theta = \frac{10*0.2\,(40-0.51)}{300\,(1+0.04*10)} = \frac{78.98}{420} = 0.188\ d$$

θ for nitrifiers > θ for heterotrophs, so take

θ = 0.188 d = 4.5 hours

5. Calculate the volume of the reactor:-

V = Q θ = 12960 * 0.188 = 2436 m^3

Compare this volume with the 630 m^3 needed for BOD_5 removal only.

<u>Note:-</u>

To find $\frac{X_{nitrifiers}}{X_{total}} = f_n$, use the following equation:

$$f_n = \frac{0.16(N_0 - N)}{0.6(S_0 - S) + 0.16(N_0 - N)}$$

, where N_0 = TKN in the influent, and N = TKN in the effluent.

6. Calculate the sludge to be wasted:-

* for hetaotrophs:-

$$Px = Y_{obs}\, Q(S_o - S) = 0.4 * 12960(84 - 6.38) \bullet \frac{10^6 L}{m^3} \bullet \frac{1kg}{10^6 mg}$$

P_x = 402 kg Vss/d

* for nitrifiers:-

$$Y_{obs} = \frac{Y}{1 + k_d \theta_c} = \frac{0.2}{1 + 0.04 * 10} \cong 0.143\ mg\ \ Vss\ / mgN$$

$$P_x = 0.143 * 12960\ (40 - 0.51) \frac{10^3 l}{m3} \cdot \frac{1kg}{10^6 mg}$$

$$p_x = 37.8\, kg\ Vss\ / d$$

$$(Q_W)_N = \frac{P_X}{(X_r)_N} = \frac{37.8 * 10^6\ mg\ / d}{(10.000) * 0.1 mg\ / l} = 37800\ \ L / d$$

$$(Q_w)_{BOD\,5} = \frac{402 * 10^6\ mg\ / d}{10.000 * 0.9\, mg\ / l} = 44667\ \ L / d$$

Total Q_w = 37800 + 44667 = 82467 L/d $\cong$ 83 m^3/d

6- Calculate oxygen requirement:-

$$R_o = Q(S_O - S) - 1.42\,P_X + 4.57\,Q\,(N_O - N)$$

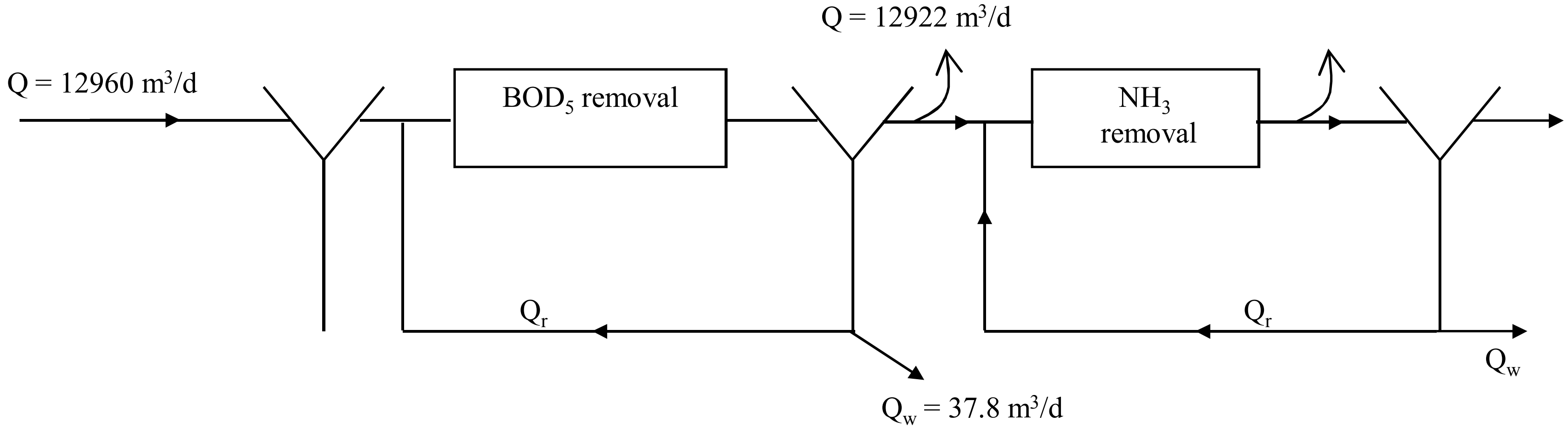
$$= 12960\,(84 - 6.38) * \frac{1kg}{10^6\,mg} \cdot \frac{10^3\,l}{1m^3} - 1.42\,(402 + 37.8) + 4.57 * 12960(40 - 0.51)\frac{10^3\,l}{m^3} \cdot \frac{1kg}{10^6\,mg}$$

$$R_O = 1005.96 - 624.5 + 2338.9$$

$$\cong 2720\,Kg\,O_2\,/\,d$$

Example 3.3:-

For example 3.2, we need to design a separate stage CMFR for nitrification only. The Data for the nitrifies and TKN are the same as in Example 3.2.



Solution:-

1.Calculations for $\theta_c^{\min}$, $S_{\min}$ for nitrifies:-

From example 3.2 it was found that:-

$\theta_c^{\min}$ = 4.76 days, N_{min} = 0.035 mg N/L

N = 1.0 mg-N/L.

2. The flow interring the nitrification CMER is:

$Q^{\backslash} = Q - Q_w = Q_e$ from the BOD_5 removal CMFR

$Q^{\backslash} = 12960 - 37.8 \cong 12922\ m^3/d$

3.Calculate for the nitrification CMFR:-

Since no BOD_5 removal occurs in this CMFR, only nitrifies are active in this reactor, this can be understood from this equation:-

$$f_n = \frac{0.16\,(N_O - N)}{0.6\,(S_O - S) + 0.16\,(N_O - N)}$$

$$\Rightarrow but\ (S_O - S) = 0.0\,(no\ BOD_5\ removal\ )$$

$$S_o \Rightarrow f_n = 1.0$$

$$\Rightarrow take\ \frac{F}{M} = 0.3\,\frac{mg\ TKN}{mg\ Vss\,.d}\,(Typical\ \ range\ \ 0.04 - 0.3)$$

$$\frac{F}{M} = \frac{QN_0}{VX} \Rightarrow VX = \frac{QNo}{F/M}$$

$$VX = \frac{12922 * \frac{m^3}{d} * \frac{10^3 L}{m^3} * 40\ mg\frac{N}{L}}{0.3\ mgN/mgvss.d} = 1.723 * 10^9\ mgvss$$

$$take\quad X = 1500\ mgvss/d$$

$$V = \frac{1.723 * 10^9}{1500} \cong 1149 * 10^3 L = 1149\ m^3$$

$$\theta = \frac{V}{Q} = \frac{1149}{12922} = 0.089\ d = 2.13\ hrs$$

__4. find θ_C:-__

$$\theta = \frac{\theta_c Y (N_0 - N)}{X(1 + K_d \theta_c)}$$

$$0.089 = \frac{\theta_c * 0.2[40 - 1]}{1500\ (1 + 0.04 * \theta_c)} \Rightarrow \theta_c = 54\ days \qquad (typical\ range\ is\ 10 - 100\ days$$

__5. Calculate the sludge to be wasted:-__

$$\underline{P_x = Y_{obs}\ Q(N_O - N)}$$

$$Y_{obs} = \frac{Y}{1+K_d\theta_c} = \frac{0.2}{1+0.04*54} = 0.06\,mgvss/mg-N$$

$$P_x = 0.06*12922\left[40-1\right]\frac{10^3 L}{m^3}.\frac{1kg}{10^6 mg} = 30\ kgvss/d$$

$$Q_w = \frac{P_x}{(X_r)_{Nitrifiers}} \Rightarrow assume\ \ X_r = 10000\ \ mg/L$$

$$Q_w = \frac{30*10^6 mgvss/d}{10.000\,mgvss/L} = 3000\,L/d = 3m^3/d$$

<u>6. Calculate oxygen requirements:-</u>

$R_o = 4.57\ Q\ (N_O + N)$ (Note:- this is the oxygen needed for nitrification only)

$$= 4.57*12922*10^3\frac{L}{d}(40-1)*\frac{1kg}{10^6 mg}$$

$R_o = 2303$ kg O_2/d

7. Calculate the volume of air to be supplied:-

- At standard conditions i.e → T = 20 °C, pressure = 1 atm, air density = 1.185 kg/m^3
- % oxygen by mass in air = 23.2%.
- Assuming 100% oxygen transfer efficiency:

$$Q_{air} = \frac{R_0}{\rho_{air} * [O_2 \%]} = \frac{2303 \ kgO_2 / d}{1.185 \ kg / m^3 * 0.232} = 8377 \ m^3_{air} \ / d$$

- Assume 8% oxygen transfer efficiency:-

$$Q_{air} = \frac{8377}{0.08} \cong 104713 \ \ m^3_{air} \ / d$$

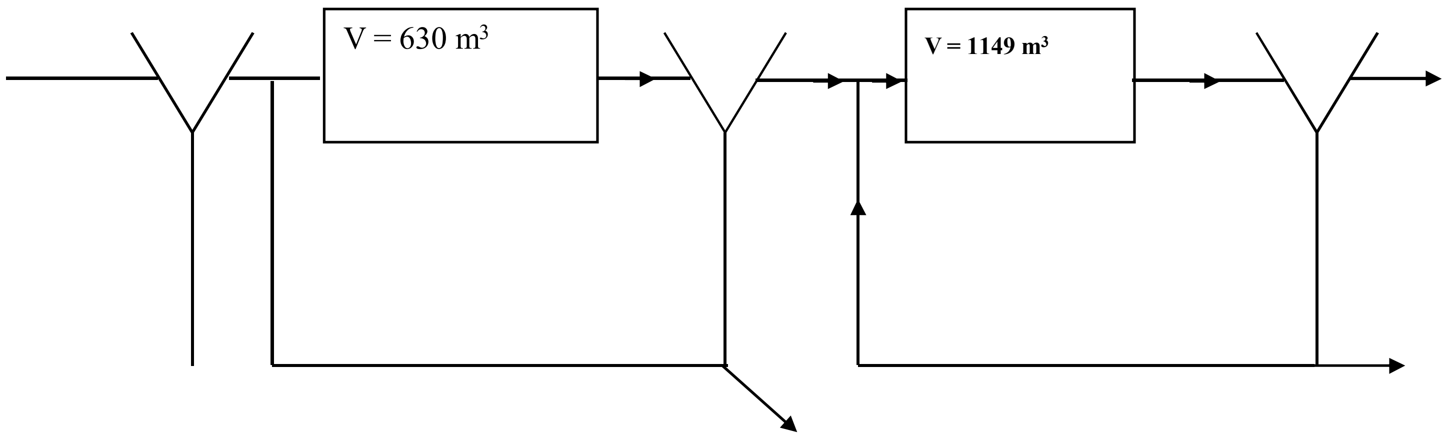
- If pure oxygen to be used:-

$$Q_{oxysen} = \frac{R_0}{\rho_{air}} = \frac{2303}{1.185} = 1943 \ \ m^3_{O2} \ / d$$

Assume 8% oxygen transfer efficiency:-

$$Q_{oxysen} = \frac{1943}{0.08} = 24288 \ \ m^3_{O_2} \ / d$$

•So the separate stage nitrification system will look as the following:-



Example 3.4:- Denitrification

For the system designed in Examples 3.1,3.2, and 3.3, design a separate stage denitrification completely mixed flow reactor (CMFR).

The denitrifying bacteria have the following growth constants:-

$$\mu_m^D = 0.4d^{-1}, Y_{Dn} = 0.9\frac{mgvss}{mgNo_3}, K_d = 0.04d^{-1}$$

$$K_{Dn} = 0.16mgNO_3^- - N/L$$

- Required → No_3^- - N in the effluent = 1 mg No_3^- - N/L

(S$_o$ = (No_3^- - N)$_o$ = D$_o$ = 39 mg No_3^- - N/L)

Solution:-

- The procedure is the same as that followed in example 3.1, except that we do not need oxygen for identification.
- We need to add organic matter, be cause denitrifies are heterotrophic bacteria.

1. Calculate θ_c^{min} S_{min} (or D_{min})

$$\frac{1}{\theta_c^{\min}} = \mu_m^{Dn} \frac{D_o}{K_{Dn} + D_o} - K_d = 0.4 * \frac{39}{0.16 + 39} - 0.04 \cong 0.36$$

$$\theta_c^{\min} = 2.78\, d$$

$$S_{\min} = D_{\min} = K_{Dn} \frac{K_d}{\mu_m^{Dn} - K_d} = 0.4 * \frac{0.04}{0.4 - 0.04} \cong 0.04 mgNo_3^- - N$$

2. Calculate θ_c:-

$$D = \frac{K_{Dn}(1 + K_d\theta_c)}{\theta_c(\mu_m^{Dn} - K_d) - 1}$$

Follow Example 3.4:-

$$1 = \frac{0.16[1 + 0.04\theta_c]}{\theta_c[0.4 - 0.04] - 1} \Rightarrow \theta_c = 3.28d$$

$\theta_c > \theta_c^{min}$ O.K

Cheek factor of safty:-

$$S.F = \frac{\theta_c}{\theta_c^{min}} = \frac{3.28}{2.78} = 1.18 < 2 \text{ not o.k}$$

Take S.F = 2.1

$\theta_c = 2.1\ \theta_c^{min} = 2.1*2.78 \cong 5.84$ days o.k

3. Calculate θ:-

Assume X = 3000 mg MLVSS/L

$$\theta = \frac{\theta_c}{X}\cdot\frac{Y_{Dn}[D_o - D]}{1 + K_d\theta_c}\frac{5.84}{3000}\cdot\frac{0.9[39 - 1]}{(1 + 0.04 * 5.84)} = 0.54d$$

$\theta \cong 1.3$ hrs o.k

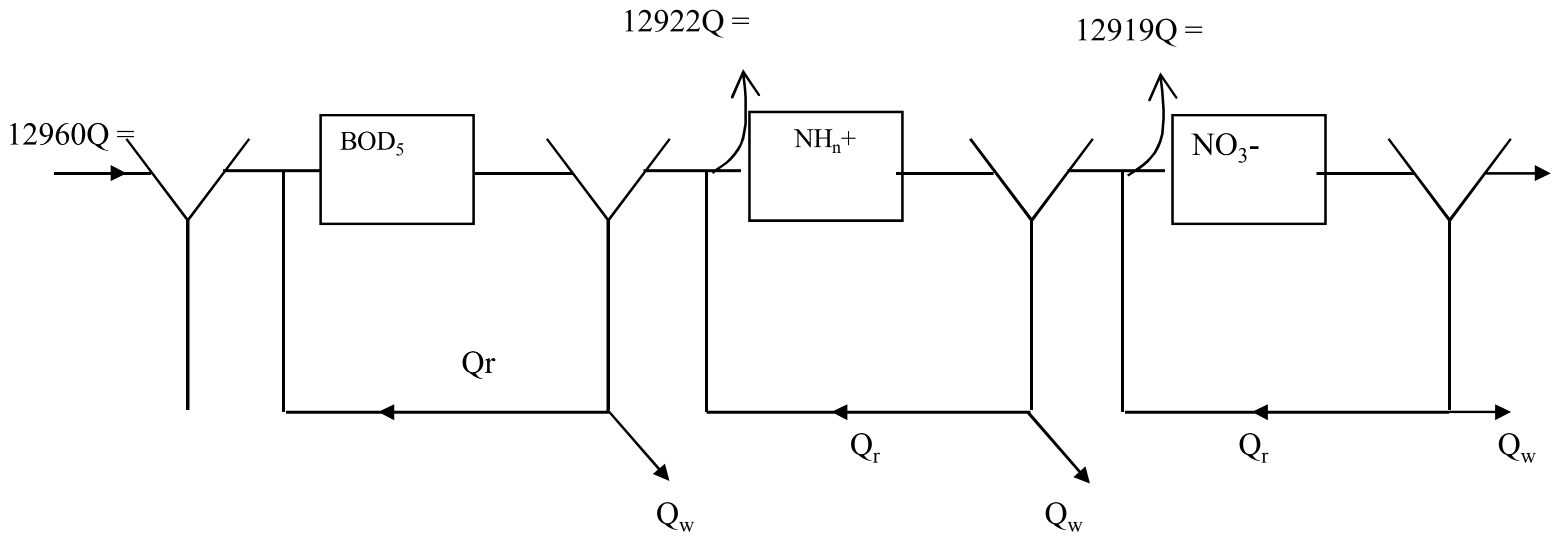
4. Calculate V:-

θ = 12922 – 3

= 12919 m3/d

V = Qθ = 12919 * 0.054 = 697.6 $\cong$ 698 m3

*<u>Find Px , QW, Qr by the same way as in example 3.1</u>



b. Design of the final (Secondary) clarifier:-

The final sedimentation tank or clarifier, is an essential part in the activated sludge system. It is needed for gathering (by settling) the sludge and returning part of it to the system. The following parameters are used to design this tank:-

1. The overflow rate:- (or hydraulic loading)

it is the amount of flow in m^3/d applied to the unit area (m^2) of the sedimentation tank and it's units are:-

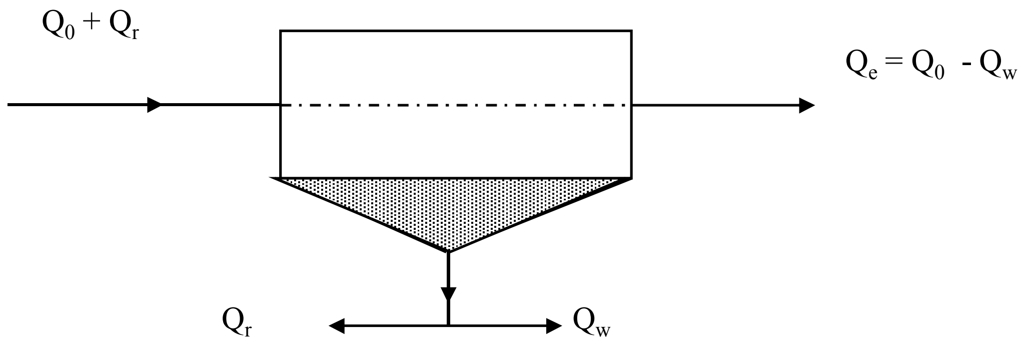
$$O/F = \frac{Q_o - Q_w}{A_s} \quad (A_s = \text{Surface area})$$

$$O/F = \frac{Q_e}{A_s} \quad \text{(note: } Q_r \text{ interring the settler is pumped from the bottom)}$$

- Some times we ignore Q_w
- The typical range of O/F is (20-34) $\frac{m^3}{m^2 \bullet d}$

the unit is $\frac{m^3}{m^2 \bullet d}$ or $\frac{m}{d}$

- In this range we expect good separation of solids from liquid in the final sedimentation tank.



2. The weir loading rate:-

It is the amount of flow in m^2/d applied to the unit length (m) of the effluent weir. The weir is the circumference of the sedimentation tank from which the wastewater leaves the tank.

- The typical range of weir loading (WL) is:-

$$WL \rightarrow 125 - 250 \ \frac{m^3}{m \bullet d} \ or \ (\frac{m^2}{d}) , (WL = \frac{Q_e}{weir \ \ length})$$

3. The solids loading rate (SL):-

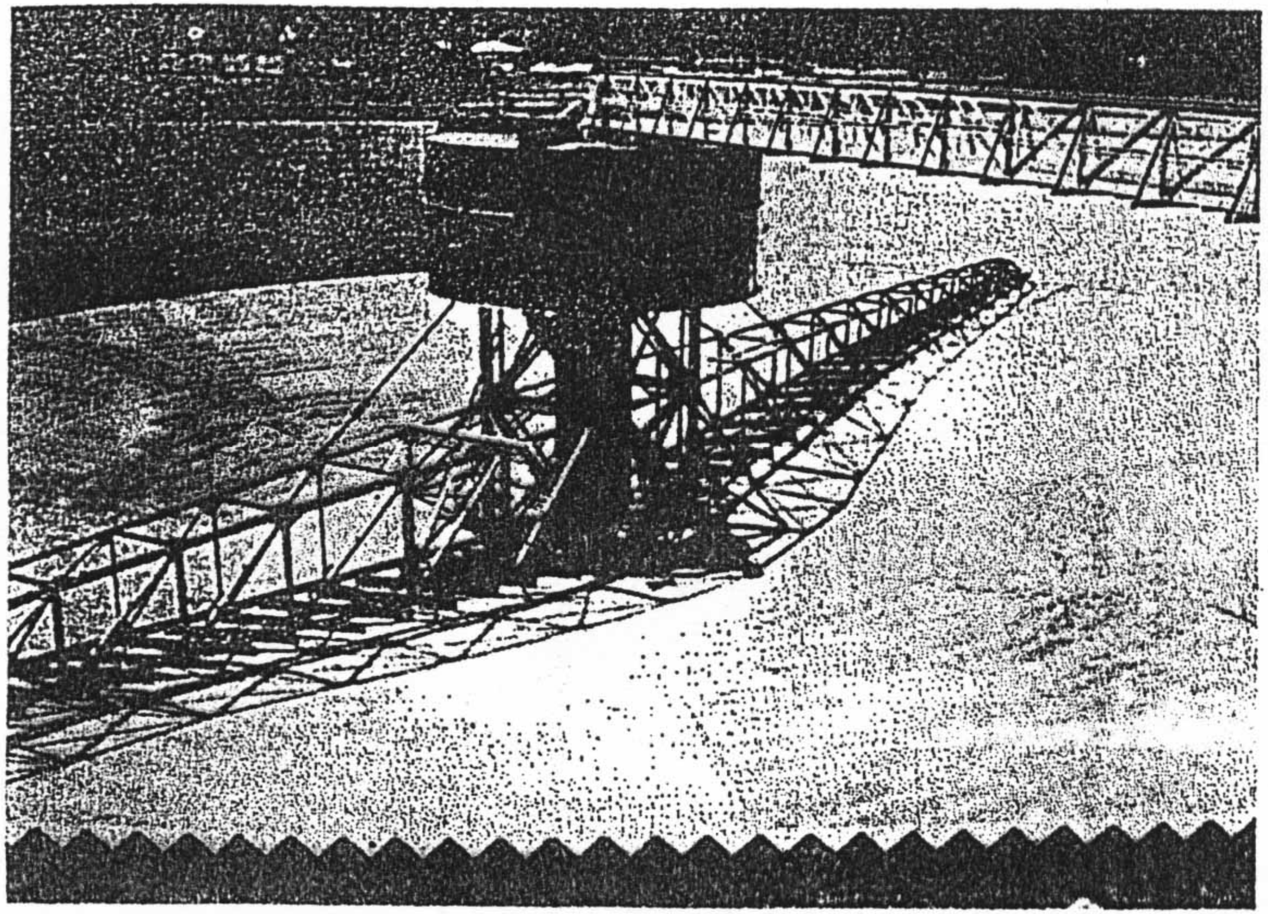
It is the amount of solids in (Kg) applied to the unit area of the settling tank per day. $(SL = \frac{(Q_o + Q_r)}{As} \bullet X)$

Typical range of SL is 130 – 300 Kg/ d.m^2. If SL is higher than 300 $\frac{Kg}{d.m^2}$ the suspended solids will increase in the effluent of the settling tank.

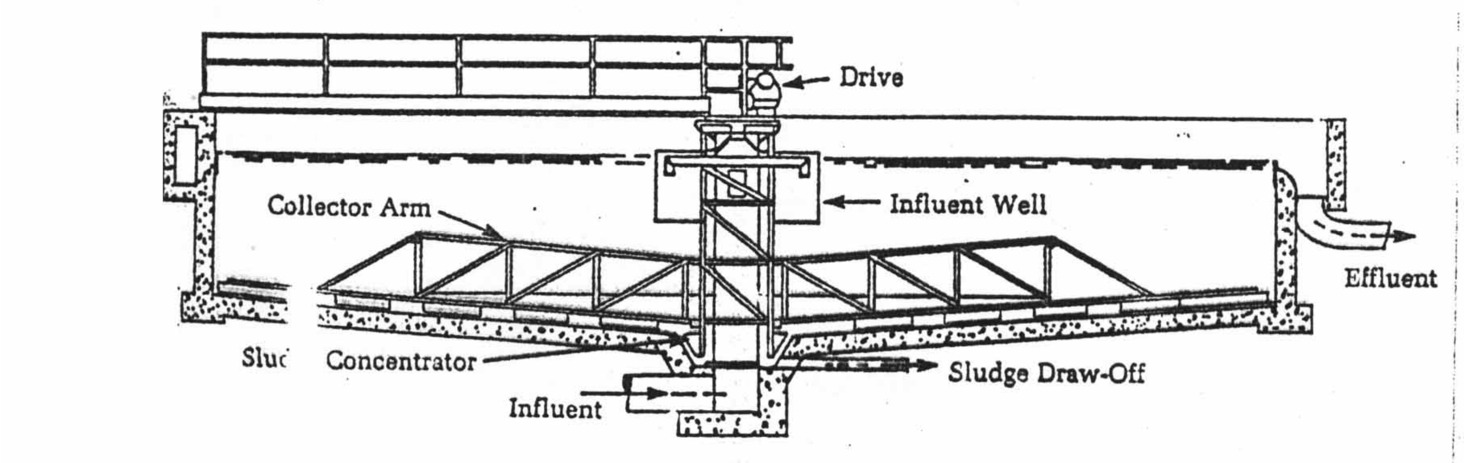
Final settling basin side water depth

	Side water depth, m	
Tank diameter, m	Minimum	Recommended
<12	3.0	3.4
12 to 20	3.4	3.7
20 to 30	3.7	4.0
30 to 42	4.0	4.3
>42	4.3	4.6

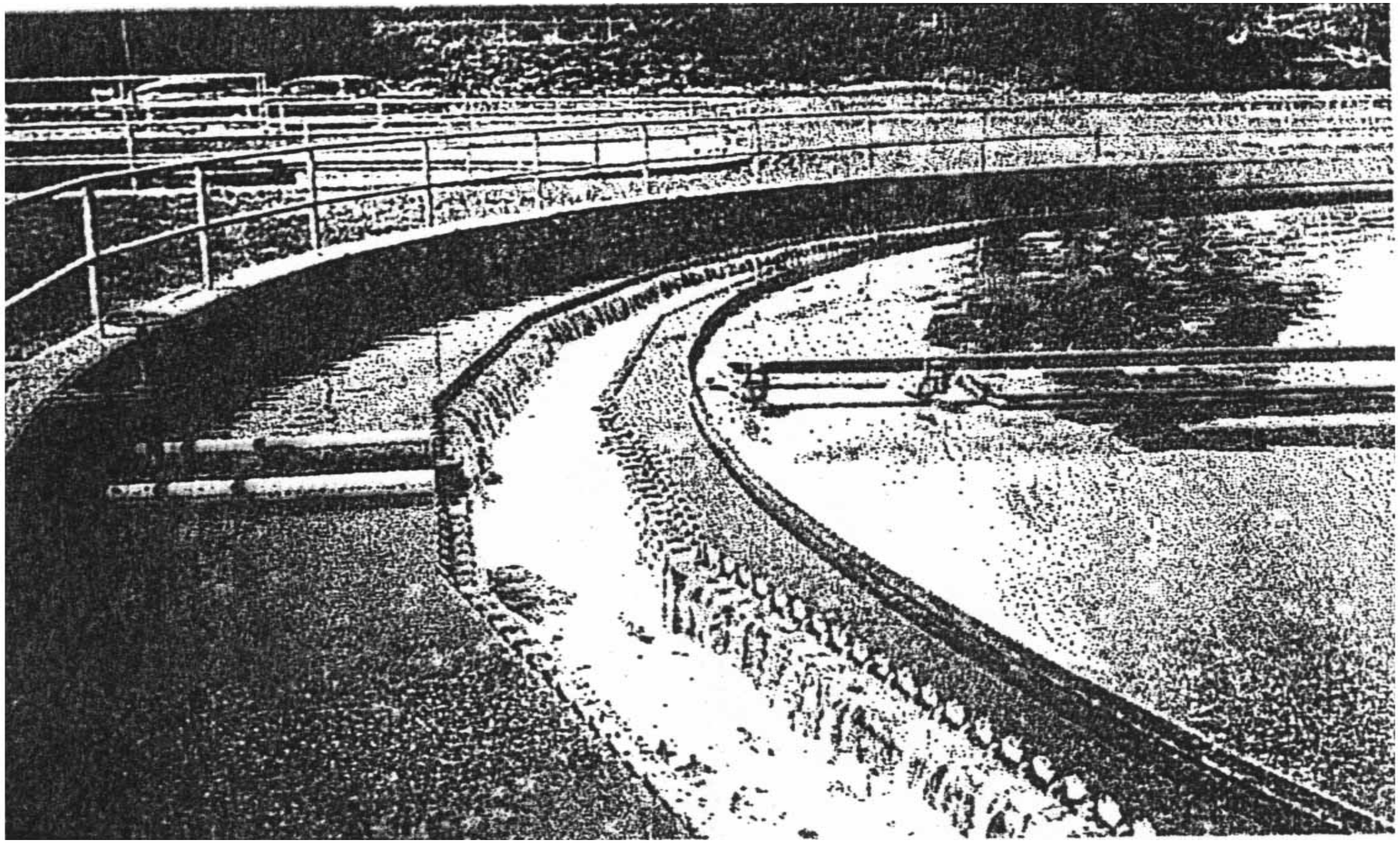
Source: Joint Task Force of the Water Environment Federation and the American Society of Civil Engineers, Design of Municipal Wastewater Treatment Plants Vol. I, Manual of Practice No. 8, Chapters 1-12, Alexandria, VA, 1992



Photograph of sludge collector for circular sedimentation basins.



Schematic diagram of sludge collector for circular sedimentation basins.



Effluent channel of circular sedimentation basins.

Example 3.5:-

Design the secondary clarifier (final settling tank) for the CMFR in Example 3.1.

Solution:-

(Q_o = 12960 m^3/d) , (Q_w = 37.8 m^3/d)

* find A_s:-

$$Take\, O/F = 33\frac{m}{d}$$

$$O/F = \frac{Qe}{As}$$

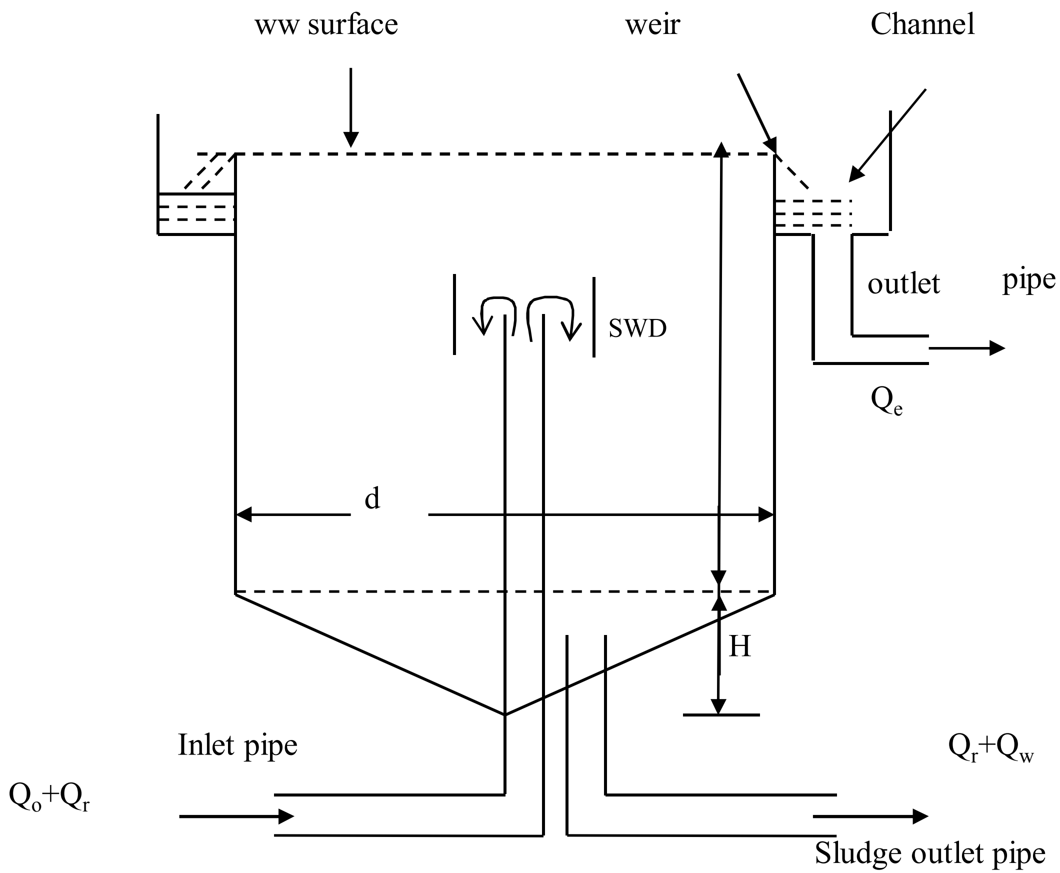
$$A_s = \frac{Q_e}{O/F} = 12922\frac{m^3}{d}.\frac{1}{33m/d} \cong 392m^2$$

(note: $Q_e = Q_o - Q_w$ = 12960 – 37.8 = 12922 m^3/d)

* find the diameter:-

$$As = \frac{\pi d^2}{4}$$

$$d = \sqrt{\frac{4 * 392}{II}} = 22.3 \text{ d}$$


ww surface
weir
Channel
SWD
outlet
pipe
Q_e
d
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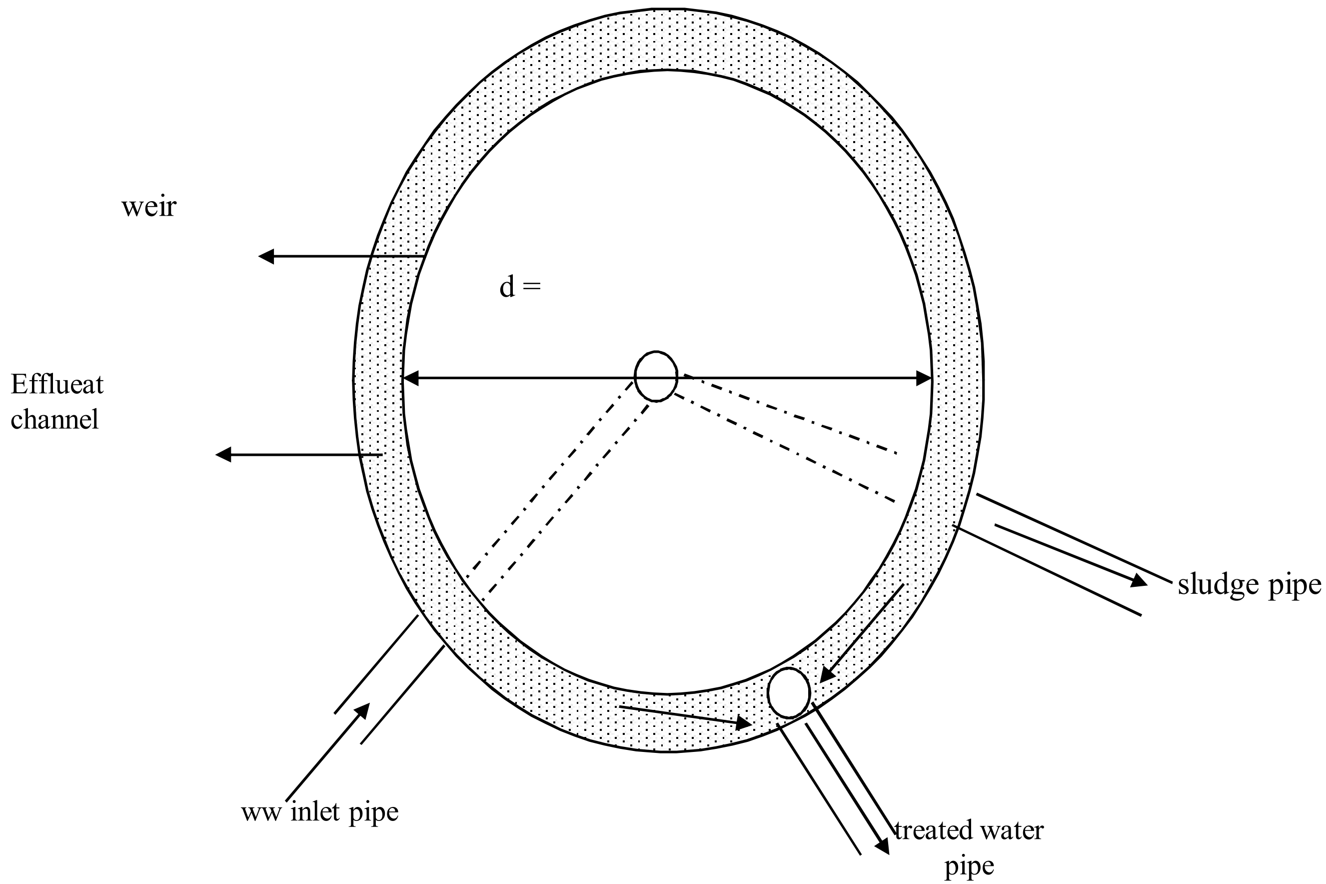
Inlet pipe
Q_o+Q_r
Q_r+Q_w
Sludge outlet pipe


weir
d =
Efflueat
channel
sludge pipe
ww inlet pipe
treated water
pipe

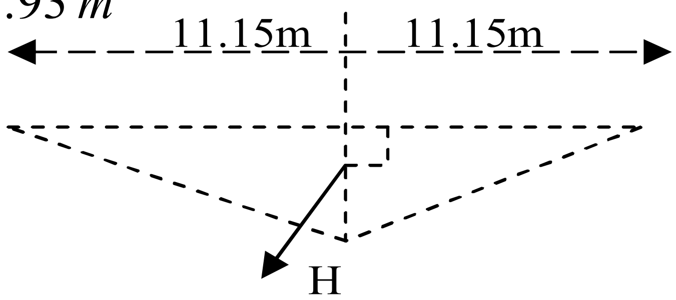
* Select a side wall depth (SWD):

From the final setting basin side water depth:-
For (d) in the range 20-30 m → SWD = 4.0 m

* find (H), the depth of the inclined bottom depth:-

The typical slope of the bottom is 1:12, So:-

$$\frac{H}{11.15} = \frac{1}{12} \Rightarrow H = 0.93\, m$$



Take H = 1.0 m

* Check for the solids loading rate SL:-

$$SL = \frac{[Q_o + Q_r] * X}{As} = \frac{(12960 + 5573)m^3}{392\, m^2} \cdot \frac{3000\, mgvss}{L} \cdot \frac{10^3\, L}{m^3} \cdot \frac{1kg}{10^6\, mg}$$

$$SL \cong 142\, kg / d.m^2 \; (in\ the\ range\ 130 - 300)\; o.k.$$

*Check for the weir loading rate (WL):-

$$WL = \frac{Qe}{weir\ length} = \frac{12422\ m^3/d}{\pi(22.3)m} \cong 184 \frac{m^3}{m \bullet d} \qquad \text{OK}$$

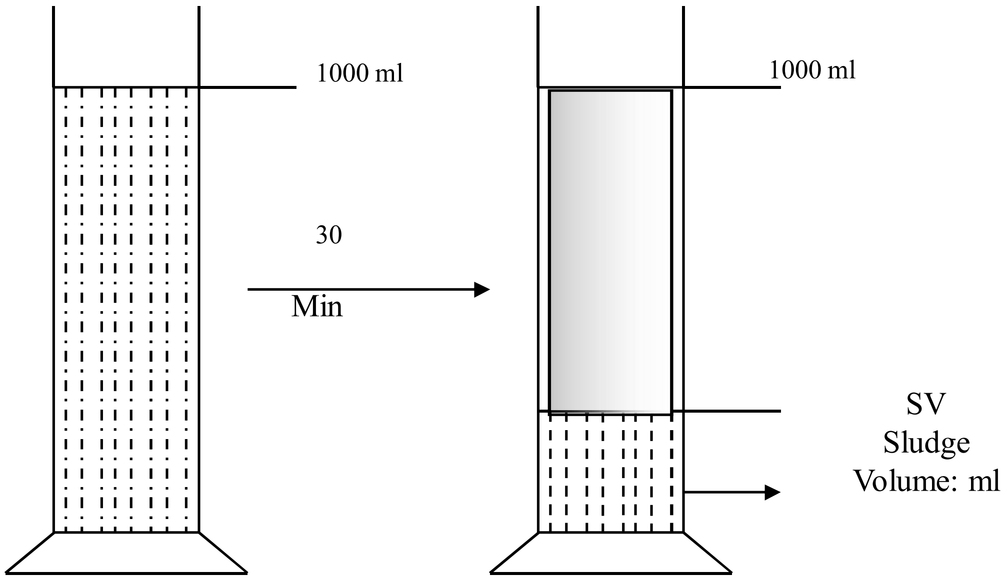
Typical range of WL is 125-250 m^3/m.d

* **Sludge volume index SVI:-**

* This parameter is used to check the quality of settling and the efficiency of the secondary settling tank in activated sludge systems. SVI is also used to control the concentration of the biomass in the biological reactor (X) and the concentration of the biomass in the return sludge (X_r).

*SVI is the volume in milliliters (ml) occupied by 1g of activated sludge after the aerated liquor has settled 30 minutes and calculated as follows:-

$$SVI = \frac{SV}{X} * 1000 \frac{ml}{g}$$


1000 ml
30
Min
1000 ml
SV
Sludge
Volume: ml

Where,

SVI = sludge volume index, $\frac{mL}{g}$

SV = volume of settled solids in one liter graduated cylinder after 30 minuts settling, $\frac{mL}{L}$

X = biomass concentration in the biological reactor such as (CMFR), mgSS/L→ (MLSS).

SVI is related to X_r by the following relation:-

$$Xr = \frac{10^6}{SVI} mgss/L$$

SVI is related to Q_r (recycle flow) as follow:-

$$R = \frac{X}{X_r - X} = \frac{Q_r}{Q}$$

$$Qr = Q\left[\frac{X}{X_r - X}\right] \longrightarrow \quad Q_r = Q\left[\frac{X}{10^6/SVI - X}\right]$$

* Typical values of SVI:-

- Typical range of SVI for activated sludge operating at concentrations of MLSS (X) of 2000 to 3500 mg ss/L is 50 to 150 mL/g.
- Notice relation between SVI and X:-

$$SVI = \frac{SV}{X} * 1000 \frac{ml}{g}$$

When X is increased, SVI decrease, so if X is increased above 3500 mgSS/L to 5000 mgSS/L for example, SVI decrease below 50, ml which means bad settling. If X is decreased below 2000, then SVI increase above 150 $\frac{mL}{g}$ leading to bad settling.

<u>Example 3.6:-</u>

For example 3.1 find the SVI, SV.

Given that MLVSS = 0.8 MLSS $\Rightarrow$ MLSS = $\frac{MLVSS}{0.8}$

Solution:-

$$X_r = 10.000 \; mg\, vss / L \, (or\, 10.000\, mg\, MLVSS / L) \; from\ example \; 3.1$$

$$X_r = 10.000 \frac{mg\, Vss}{L} * \frac{1 mg\, ss}{0.8 mg\, vss} = 12500 mg\, ss / L$$

$$= 12500 mg\, MLss$$

$$X_r = \frac{10^6}{SVI} mg\, MLSS / L$$

$$SVI = \frac{10^6}{12500} 80 \frac{mL}{g} > 50 \frac{mL}{g} \; o.k\, , good\ settling$$

* find sludge volume SV:-

$$SVI = \frac{SV}{X} * 1000 \frac{mg}{g}$$

$$SV = \frac{X.SVI}{1000}$$

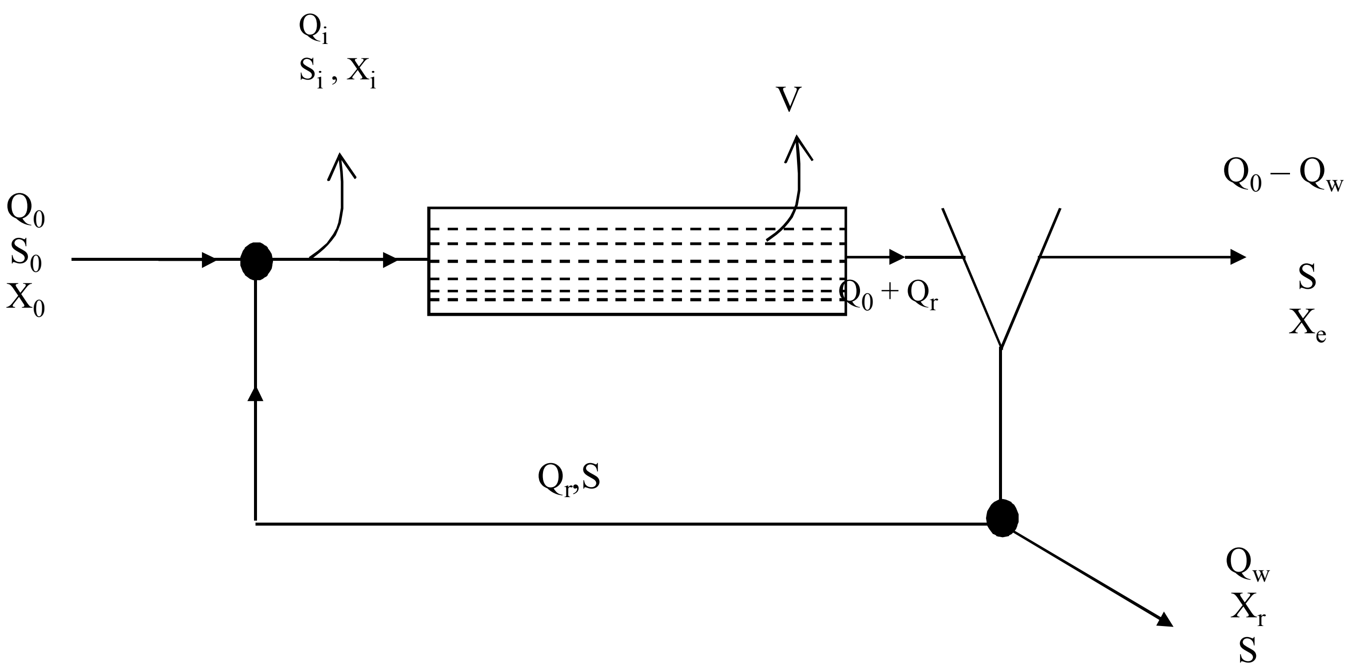
$$X = 3000 mg\, MLVSS / L = \frac{3000}{0.8} = 3750 mg\, MLSS / L$$

$$SV = \frac{3750 * 80}{1000} = 300 \frac{mL}{L}$$

$(this \quad is \quad the\ volume\ of\ \ sludge\ in\ one\ liter\ of\ \ ww \quad after\ 30 \min \quad settling)$

B. Design of activated sludge system as a plug flow (PFR):-

The conventional activated sludge system can be designed as a PFR. The following is an example to illustrate the procedure used.



<u>Example 3.7:-</u>

Solve example 3.1 using a PFR.

<u>a. find θc:-</u>

$\theta_c^{\min}$ was calculated as 0.408 d , for PFR we find θ_c from equation 20:-

$$\frac{1}{\theta_c} = \frac{\mu_m (S_o - S)}{(S_o - S) + (1+\alpha) K_s In\left[\frac{S_i}{s}\right]} - k_d$$

$$S_i = \frac{Q_o S_o + Q_r S}{Q_i}$$

$$\alpha = R = \frac{X}{X_r - X} = \frac{3000}{10000 - 3000} \cong 0.43$$

$$R = \frac{Q_r}{Q_o} \Rightarrow Q_r = 0.43 Q_o = 0.43 * 12960 = 5573\ m^3 / d$$

$$Q_i = Q_o + Q_r = 18533\ m^3 / d$$

$$S_i = \frac{12960 * 84 * 5573 * 11.1}{18533} = 62\, mgBoDS / L$$

$$\frac{1}{\theta_c} = \frac{2.5[84-11.1]}{[84-11.1]+[1+0.43]*100\ In\left[\frac{62}{11.1}\right]}0.05 = 0.522\ d^{-1}$$

$\theta_c = 1.92$ days

* Check for S.F:-

$$S.F = \frac{\theta_c}{\theta_c^m} = \frac{1.92}{0.408} = 4.71 > 2\ O.K$$

<u>b. Calculate θ from equation 19:-</u>

$$\overline{X} = \frac{\theta_c}{\theta}\frac{Y(S_o - S)}{1+K_d\theta_c}, let\ \ \overline{X} = 3000\ mg/l$$

$$\theta = \frac{1.92 * 0.5(84-11.1)}{3000\ (1+0.05*1.92)} = 0.0213\ d = 0.5\ hrs.$$

* it is typically preferred to have a minimum θ of 1.0 hr. To increase θ we can either decrease X or increase θ_c , or do both things. So, assume

$\theta = 1\ hr = 0.042\ d,\ \overline{X} = 3000\ mg/l$, and solve for θ_c .

$$3000 = \frac{\theta c}{0.042}.\frac{0.5)84-11.1)}{(1+0.05*\theta c)} \Rightarrow \theta c = 4.18\ days$$

$$* \ for \ PFR \ \frac{\theta_C}{\theta} Should \ \ be > 5$$

$$\frac{4.18}{0.042} = 99.5 >> 5 \ o.k$$

<u>C. Calculate the reactor volume:-</u>

$$V = Q\theta = 12960 * 0.042 = 544 \ m^3$$

<u>d. Calculate P_x, Q_w, R_o the same as in example 3.1.</u>

3.3 Oxidation ditches:- (OD)

Oxidation ditches are type of suspended growth systems. It is a modification of the conventional activated sludge system.

3.3.1 . Characteristics of oxidation ditches:

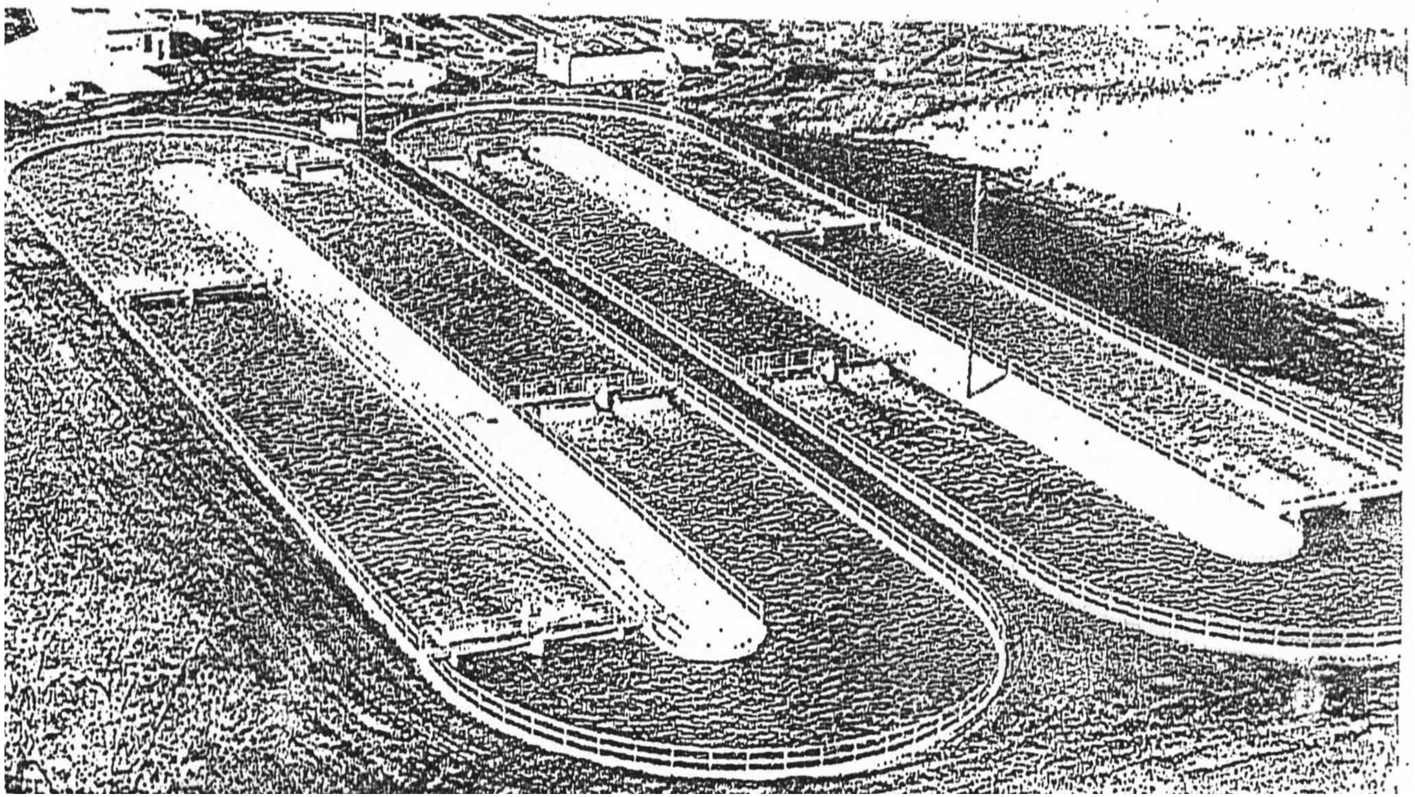
A. Configurations:-

The oxidation ditch consists of a ring or oval – shaped channel. It is some times called closed loop Reactor (CLR), and some times called Racetrack channel. The oxidation ditch may have a trapezoidal or rectangular cross section.

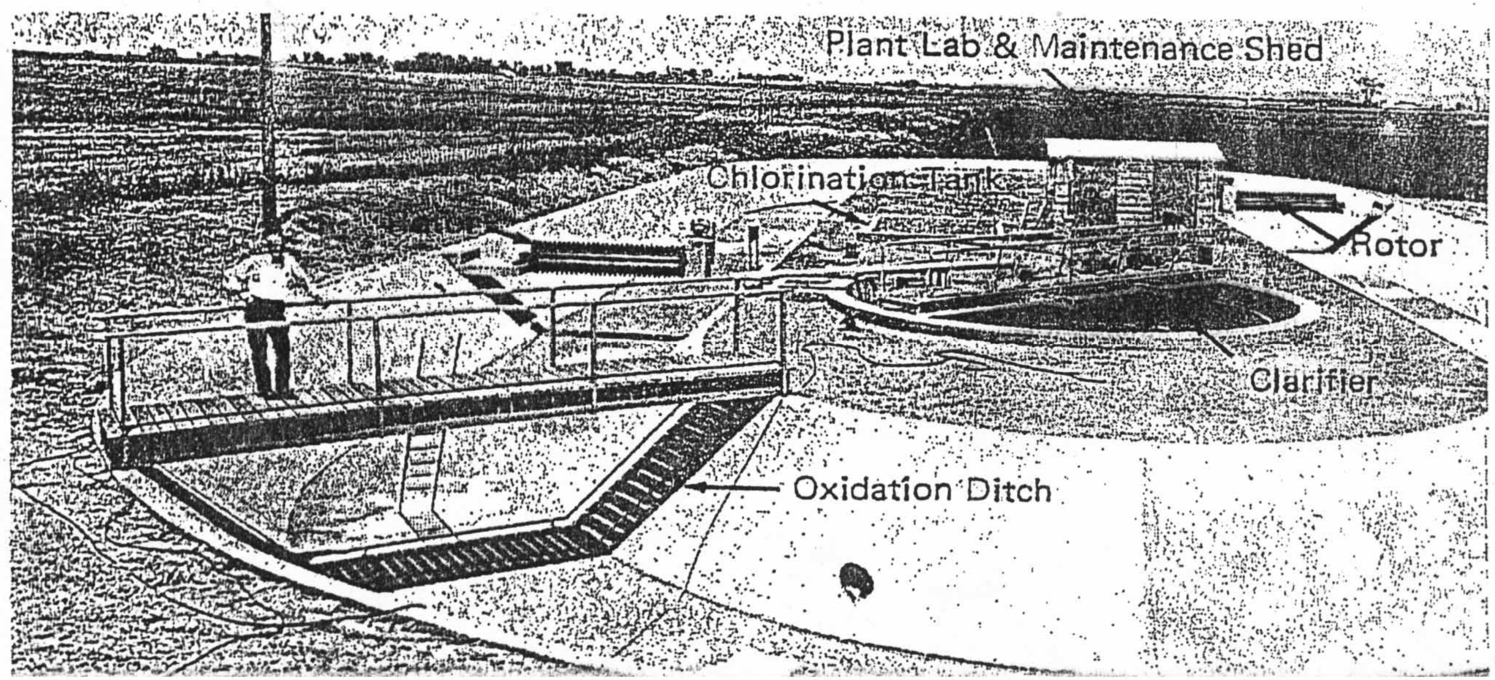
The wastewater is recirculated in the "CLR" using brush rotors (Kessners brush), which is also used for aeration.

There are many configurations of oxidation ditches as shown in the figures.

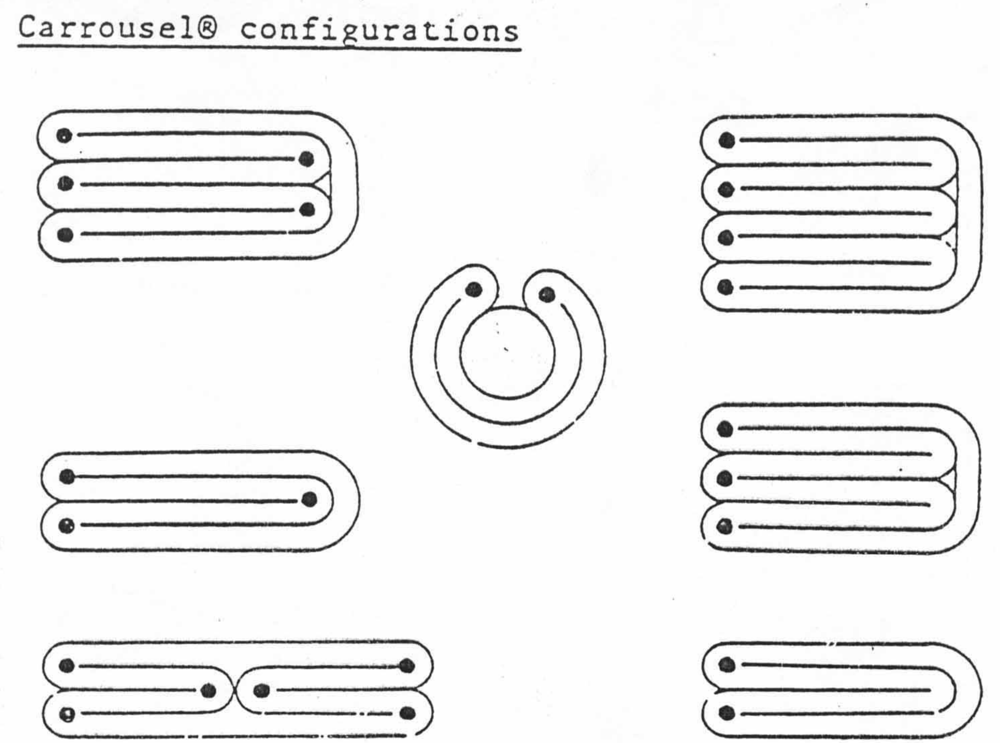
The velocity of flow in the OD is maintained at 0.25 – 0.3 m/s to keep the biomass in suspension. At this velocity, the mixed liquor completes a tank circulation in 5 – 15 min, leading to the dilution of the influent by 20-30 times. The influent of raw sewage is introduced just upstream of the aerator (Rotor). The effluent weir is located just upstream of the influent pipe.



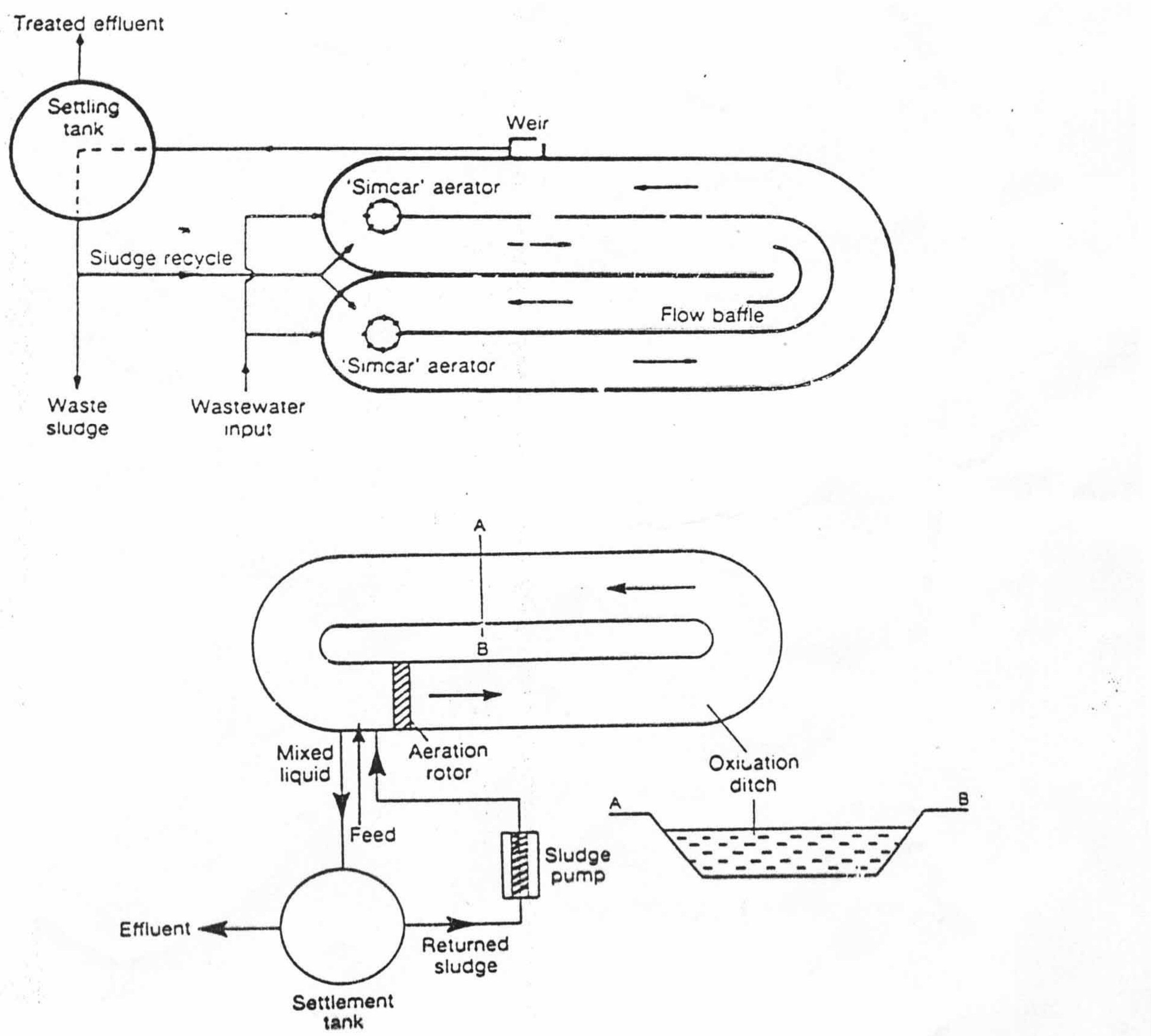
Oxidation Ditch



Oxidation Ditch (empty)



Oxidation Ditch: carrousel configurations



Typical Oxidation Ditch layout

B. Hydraulic model:-

Oxidation ditehs combine features of both PFR and CMFR models:

C. CMFR similarity:-

The rapid flow in the OD results in 20-30 dilutions which gives a considerable amount of mixing. The influent ww is mixed with the rotating ww at the inlet.

D. PFR similarity:-

The OD are long reactors, and thus they have some similarity with PFR a long the reactor.

E. Which model is used for OD design? CMFR or PFR?:-

Since PFR assumes no mixing, this case is not found in OD. So OD is designed as a CMFR. The error in this assumption leads to higher hydraulic detention time, which gives a safty factor in the design. Moreover, OD are designed at high (θ) anyway to achieve sludge stabilization so assuming that OD is CMFR is accepted

3.3.2. Difference between OD and conventional Activated sludge:-

- Oxidation ditches were developed to minimize the net sludge production compared to the conventional activated sludge system.
- Net sludge production minimization is achieved by using law F/M ratio (0.02 – 0.15 $\frac{mgBOD_5}{mgvss \bullet d}$). In this case the active biomass is forced to feed on the decaying biomass due to the shortage of food. This leads to lower sludge production, and the sludge to be wasted will be less and has lower organic content(i.e. more stabilized).
- OD are operated at high θc (15-30 days) and at high θ (15-36 hrs).
- It is theoretically possible in OD to minimize the net sludge production to zero. This can be achieved by making the produced biomass equal to the degraded biomass by endogenous decay (i.e. biomass feeding on dead biomass). This is presented mathematically as follows:-

Sludge produced = $YQ(S_o - S)$

Sludge decaying = $K_d XV$

Net production ⇒ $Px = YQ(S_o - S) - K_d XV$

Let net production $(P_x) = 0.0$

So ⇒ $YQ(S_o - S) = K_d XV$

And ⇒ $XV = \frac{Y}{K_d} Q(S_0 - S)$ or $X = \frac{Y}{K_d \theta}(S0 - S)$

This equation can be used to find X and V that can be used to a chive zero net sludge production.

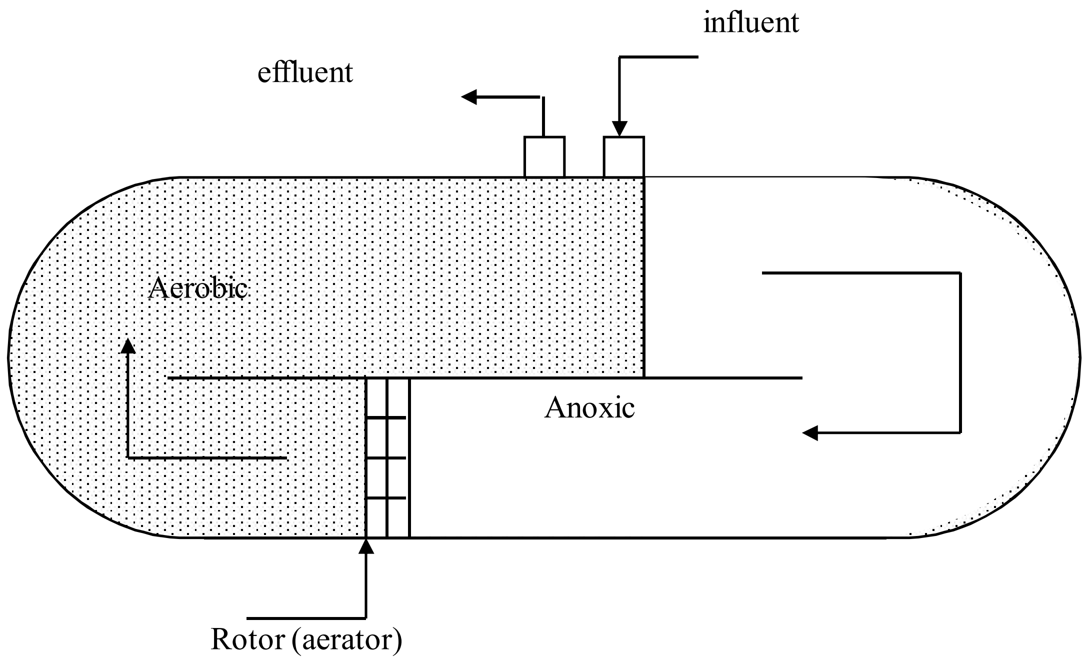
3.3.3 Processes that can be achived in oxidation ditches:-

Three processes can be achieved in oxidation ditches:-

- Organic matter removal (BOD_5 removal) in the aerobic zone.
- Nitrification (in the aerobic zone).
- Denitrification (in the anoxic zone).

- At the influent to the OD, we have organic matter in addition to nitrate (NO_3^-) coming from the aerobic zone and the dissolved O_2 is almost zero. This is called anoxic condition where denitrification occurs.

At the end of the anoxic zone and the beginning of the aerobic zone, we have the remaining organic matter that was not used for denitrification in coming in the influent in addition to O_2 (NH_4^+)addition to ammonium introduced by the aerator. In this condition both BOD_5 removal and nitrification occurs. At the end of the aerobic zone the dissolved oxygen becomes almost zero.


influent
effluent
Aerobic
Anoxic
Rotor (aerator)

Example 3.8:-

Design an oxidation ditch for BoD_5 removal only. The following Data are given:

- influent BOD_5= 300 mg BOD_5/L (soluble)
- effluent BOD_5=15 mg BOD/L (soluble)
- $Q_0 = 20.000\ m^3/d$
- Y= 0.5 mgvss/mgBOD_5 , $K_d = 0.03d^{-1}$, K_s = 30 mgBOD_5, $\mu_m = 2.5d^{-1}$

Assume that we want to operate the OD at Zero net sludge production solution:

1- Calculate X to achieve zero net production:

$$X\theta = \frac{Y}{K_d}(S_0 - S)$$

$$X\theta = 0.5\frac{mgvss}{mgBOD_s}\cdot\frac{d}{0.03}\cdot[300-15]\frac{mgBOD_5}{L} = 4750\frac{mgvss}{L}.d$$

1- check for $\frac{F}{M}$

$$\frac{F}{M}=\frac{S_0}{X\theta}=\frac{300}{4750}=0.063\frac{mgBOD_s}{mgvss.d}$$

3- Assume θ in the typical range (15-36 hrs),
take $\theta = 1$ day (24 hrs)

$$X\theta = 4750\frac{mgvss}{L}.d$$

X = 4750 mgvss/L , typical range for OD is (2500 – 6000), O.K.

4- In this example we do not need to check for θ_c because we assumed that no sludge wasting will take place, and theoriticaly $\theta_{c\rightarrow\infty}$

5- find the volume of the oxidation ditch

V = $Q\theta = 20000\frac{m^3}{d}*1d = 20000m^3$

Notice that the volume is very high due to the high θ

6- Calculate Q_r:

$$R = \frac{X}{X_r - X}$$, assume Xr = 10000 mgvss/L

$$R = \frac{4750}{10.000 - 4750} = 0.9$$

$Q_r = QR = 20.000*0.9 = 18000\ m^3/d$

7- find the oxygen requirements:

$$R_0 = Q_0(S_0 - S) - 1.42Px \quad ,(note\ that\ Px = 0.0)$$

$$R_0 = Q_0(S_0 - S) = 20000\frac{m^3}{d} \bullet \frac{10^3 L}{m^3}[300 - 15]\frac{mgBOD}{L} \bullet \frac{kg}{10^6 mg}$$

$$R_0 = 5700 kgO_2 / d$$

Example 3.9:

Repeat the Design in example 3.8, assuming that we want to allow for some sludge waste, by using a sludge age (θ_c) in the range 15 – 30 days.

Solution:

In this case design the oxidation ditch as a CMFR and use the equation of CMFR. The difference between the conventional CMFR and OD is the design parameters typical ranges $(\theta, \theta_c, X, F/M)$

a- Assume θ_c, = 30 days, assume θ = 15 hrs (0.625 days).

(note: in oxidation ditches we allow S.F above 20)

b- Calculate S:

$$S = \frac{30(1+0.03*30)}{30(2.5-0.03)-1} = 0.78 mgBOD_s / L$$

$$S_{\min} = K_s \frac{K_d}{\mu_m - K_d} = 30 * \frac{0.03}{2.5-0.03} = 0.36 mgBOD_5 / L < 0.78$$

c- calculate X:

$$X = \frac{\theta_c}{\theta} \frac{Y(S_0 - S)}{(1 + K_d \theta_c)}$$

$$X = \frac{30}{0.625} . 0.5 \frac{[300 - 0.78]}{1 + 0.03 * 30} = 3780 \ mgvss / L$$

The typical range of X is 2500 to 6000 mgVSS/L, O.k

d- check for $\frac{F}{M}$

$\frac{F}{M} = \frac{S_0}{\theta x} = \frac{300 \ mgBOD_5 / L}{0.625 * 3780 \ mgvss / L} = 0.121 \frac{mgBOD_5}{mgvss .d}$ within the range (0.02 – 0.15) O.K

e- calculate V:

V=Q = 20000 * 0.625 = 12500 m^3

F- calculate sludge production:

$$P_x = Y_{obs} Q (S_0 - S), Y_{obs} = \frac{Y}{1 + K_d \theta_c} = \frac{0.5}{1 + 0.03 * 30} = 0.26 \frac{mgvss}{mgBOD_5}$$

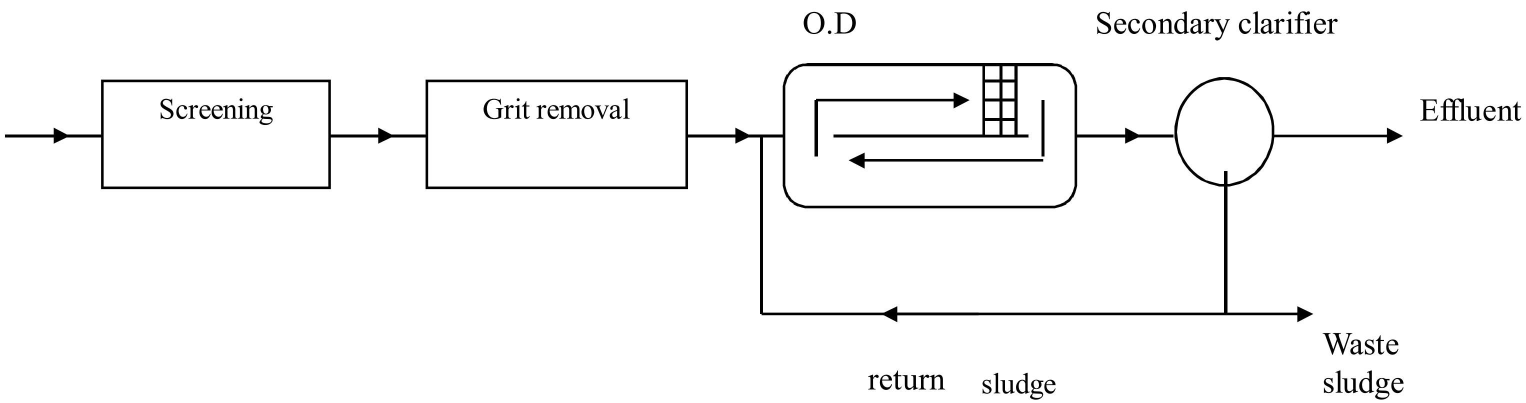
$$P_x = 0.26 * 20000 \frac{m^3}{d} (300 - 0.78) \frac{10^3 L}{m^3} . \frac{Kg}{10^6 mg} = 1556 \ kg / d$$

$$Q_w = \frac{P_x}{X_r} = \frac{1556 * 10^6}{10.000} = 155600 \frac{L}{d} = 155.6 m^3 / d$$

3.3.4 Advantages of oxidation ditches:

* low sludge production can be achieved due to using low F/M ratio
* The produced sludge, if any is stable and needs no further treatment. This means that no sludge treatment installations are needed.
* no need for primary sedimentation, because the high θ_c in the oxidation ditches is enough to digest the solids that is usually separated in the primary sedimentation tank.
* easy to operate and the operation and maintenance cost is much less than conventional activated sludge.
* Ability to nitrify and denitrify in one tank

3.3.5 OD process flow sheet:

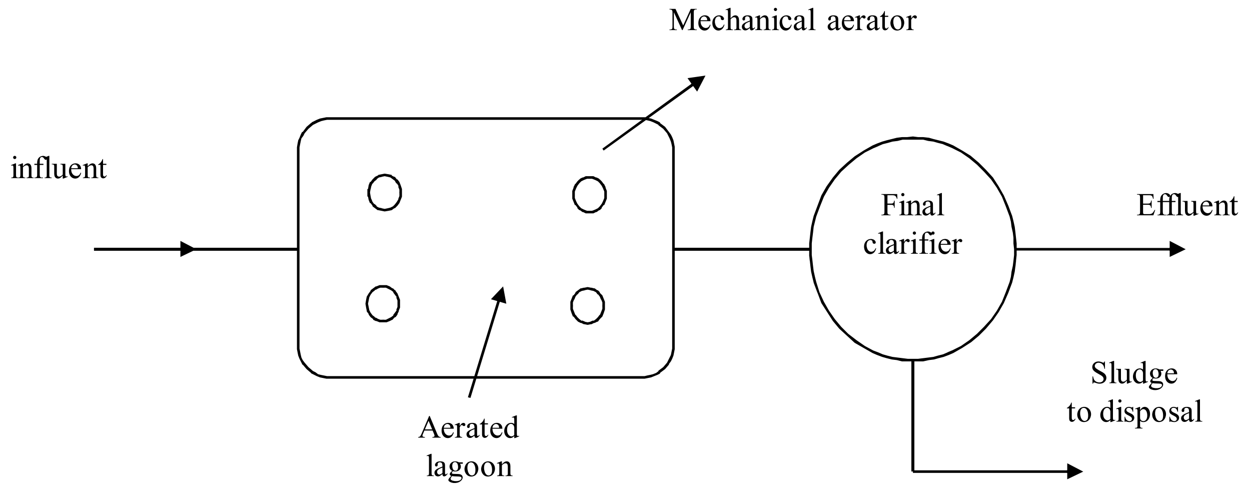


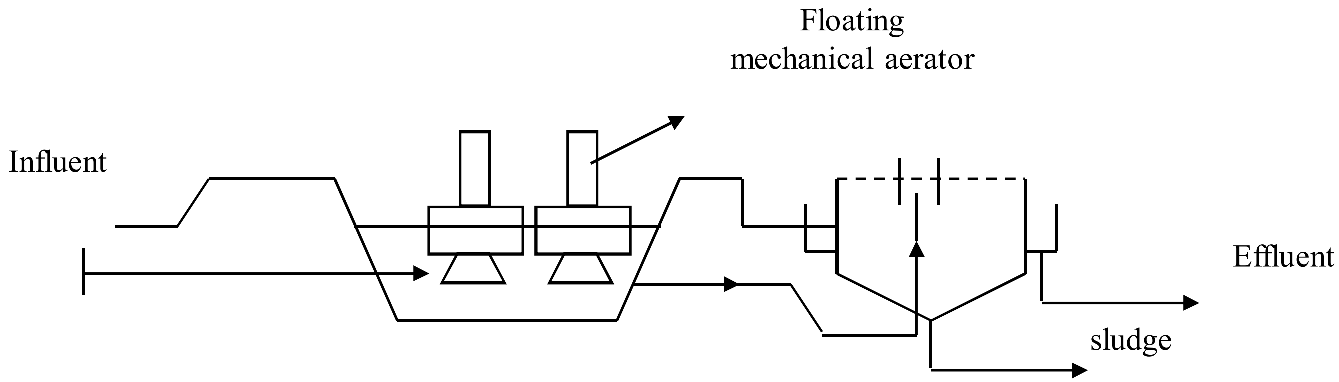
3.4 Aerated lagoons (AL):

Aerated lagoons are suspended growth waste water treatment system. They are not considered as an activated sludge system because no solids recycle is applied. This system (i.e AL) is a low cost low efficiency treatment system compaired to Activated sludge systems.

3.4.1 Configuration of "AL"

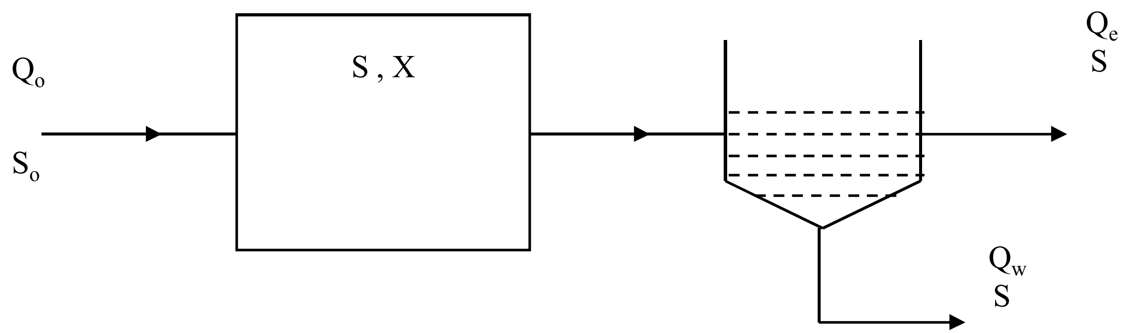
Aerated lagoons consist mainly of an earthen basin that has a large surface area and a shallow depth (1-3m). The sides slopes are generally 1:3 (some times (1:2)). The face area is usually square to achieve the best power transfer applied by the mechanical aerators. Surface mechanical aerators are used for both oxygen transfer and complete mixing of the lagoons.


Mechanical aerator
influent
Final
clarifier
Effluent
Sludge
to disposal
Aerated
lagoon


Floating
mechanical aerator
Influent
Effluent
sludge

3.4.2 hydraulic model of Aerated lagoons:

Aerated lagoons are designed as completely mixed reactor without solids recycle. The derivations of the equations of such a system are presented in chapter 2.



For "AL" the following design equations apply:

$$S = \frac{K_s(1+\theta K_d)}{\theta(\mu_m - K_d)-1}, X = \frac{Y(S_0 - S)}{1+K_d\theta}$$

these equtions were derived previously (Ch-2)

3.4.3 Differences between Aerated lagoons and Activated sludge:

* Since no solids are recycled in AL, the biomass concentration X is in the range of 100 – 400 mgvss/L , which is too low compared to 1500-6000 mgvss/L in activated sludge, As a result, we need much more reactor volume (V) to achieve similar treatment efficiency to that of Activated sludge systems.

* Only BOD removal is achieved in Aerated lagoons because the oxygen in θ_c is not enough to achieve nitrification (θ_c, supply and sludge age and have a typical range of 3-10 days). Note that θ AL is equal to in this θ is equal to θ_c dose not appear in the equations above , but θ_c system.

3.4.4 mixing power requirements:

The power needed for mixing is usually more than power needed for aeration in aerated lagoons. So we always need to check for mixing requirements using the following equation:

P= 0.004 X + 5

Where,

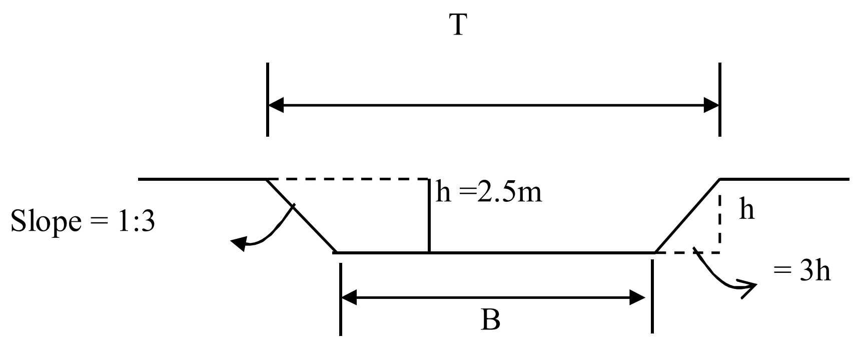
P = power input , $\frac{Kw}{10^3 m^3}$

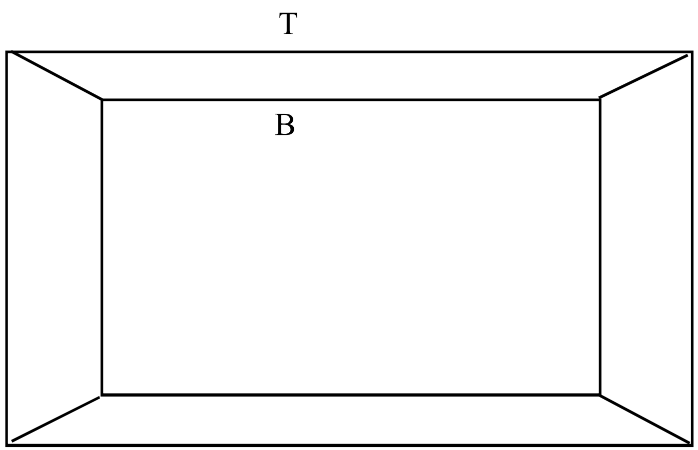
X= MLSS in the "AL" , mgss/L

Example 3.8:

Design an aerated lagoon to treat a domestic waste water with a total BOD_5 of 400 $mgBOD_5/L$, a TSS of 130 mg/L and a daily flow of 8000 m^3/d. Heterotrophic bacteria growth constants are $\mu_m = 2.8d^{-1}$, K_s = 60 $mgBOD_s/L$, K_d= $0.03d^{-1}$, $Y = 0.5\frac{mgvss}{mgBOD_s}$, assume θ = 5 days.

Assume that (BOD_5)/ Tss=0.63, and MLVSS = 0.8 MLSS ,


T
h =2.5m
Slope = 1:3
h
= 3h
B


T
B

Solution:

1- determine (S_0) soluble:

$(S_0)_{soluble}=(BOD_5)_{total} - (BOD_5)_{particulate}= 400 - 0.63*130 \cong 318$ mgBOD$_5$/L

2- calculate S:

$$S = \frac{K_s(1+\theta K_d)}{\theta(\mu_m - K_d) - 1}$$

$$S = \frac{60(1+5 \bullet 0.03)}{5(2.8-0.03)-1} = 5.37 \frac{mgBOD_5}{L}$$

3- calculate X:

$$X = \frac{0.5(318-5.37)}{1+0.03 \bullet 5} \cong 136 \frac{mgvss}{L}$$

, typical range 100-400mgvss/L, OK.

1- Calculate the volume and surface area required:

* $V = Q\theta = 5*8000\ m^3/d = 40000\ m^3/d$
* Assume the depth of the lagoon as 2.5m, and that the lagoon is square, find the surface area:

From the geometry of the AL$\rightarrow V = \left[\dfrac{T^2 + B^2}{2}\right] \bullet h$, and $T = B + 6h$

$40000 = \left[\dfrac{(B + 6 \bullet 2.56)^2 + B^2}{2}\right] \bullet 2.5$, solve for B$\rightarrow$ B $\approx$ 118.8 m

Then, T= 118.8+6*2.5 = 133.8 m

$A_{surface}$ or $A_s = (133.8)^2 \approx 17902\ m^2$
Note that this is a very large area.

5. Assume that only 9000 m^2 available, what changes should we do?

, so the proposed lagoon $\dfrac{9000}{17902} \cong 0.50$ We can reduce (V) by the ratio
volume (V) = 0.5*40000 = 20000 m^3

* calculate θ in this case:

$$\theta = \frac{V}{Q} = \frac{20000}{8000} = 2.5\ days$$

* calculate S in this case:

$$S = \frac{60(1 + 2.5 \bullet 0.03)}{2.5(2.8 - 0.03) - 1} \cong 10.9 \frac{mgBOD_5}{L}$$

* calculate X in this case:

$$X = \frac{0.5(318 - 10.9)}{(1 + 0.03 \bullet 2.5)} = 143 \frac{mgvss}{L}$$, typical range 100-400, OK

6. Calculate the sludge production for the first case, when $\theta = 5\ days$:

$P_X = Y_{obs} Q(S_0 - S)$,

$$Y_{obs} = \frac{Y}{1 + k_d \theta} = \frac{0.5}{1 + 0.03 \bullet 5} = 0.43 \frac{mgvss}{mgBOD_5},$$

$$P_X = 0.43 \bullet 8000 \frac{m^3}{d} \bullet \frac{10^3 L}{m^3} [318 - 5.37] \frac{1Kg}{10^6 mg} = 1075 \frac{Kg}{d}$$

7. Calculate the oxygen requirements:

$R_O = Q(S_0 - S) - 1.42 P_X$

$$R_O = 8000 \frac{m^3}{d} \bullet \frac{10^3 L}{m^3} (318 - 5.3) \frac{1Kg}{10^6 mg} - 1.42 \bullet 1075 \frac{Kg}{d} = 975 \frac{KgO_2}{d}$$

7. Calculate the power needed for oxygen transfer assuming that 1.8 KgO_2 requires 1KWh:

$$Power = 975\frac{KgO_2}{d} \bullet \frac{1}{1.8(KgO2/KWh)} \bullet \frac{1d}{24h} \cong 23\ Kw$$

8. Check power requirements for mixing:

$$P = 0.004X_{ss} + 5\,, \qquad Xss = \frac{Xvss}{0.80} = \frac{136}{0.80} = 170\frac{mgss}{L}$$

$$P = 0.004 \bullet 170 + 5 = 5.68\frac{Kw}{10^3 m_3}$$, but V= 40000 m^3

So the total power needed $P_{total} = 40000 \bullet \frac{5.68}{10^3} = 227Kw$

So mixing power controls the design.

1.5 Sequencing Batch Reactors (SBR):

Sequencing Batch reactors are suspended growth activated sludge system. The main difference between SBR and conventional sludge system is that in the later process in continuous (CHFR) while in the SBR it is interment.

1.5.1 Hydraulic model of SBR:

SBR are designed as batch reactors. The reactor is filled, then time is allowed for reaction to occur. During the reaction the reactor is completely mixed. The design equation of this system is presented in chapter 2.

For SBR the following equation is applied:

$$K_s \ln \frac{S_o}{S_t} - (S_o - S_t) = X\left(\frac{\mu_m}{Y}\right)t$$

1.5.1 **SBR process:**

The SBR process is a fill and draw process. This process has five steps as shown in the figure:

- Fill
- React (Aeration)
- Settle (sedimentation)
- Draw (decant)تفريق الماء الرائق
- Idle (فترة انتظار بعد انتهاء دورة المعالجة)

The following is a description of the five steps:

1. Fill:

- It is the process of adding raw sewage to the SBR tank.
- The fill volume is determined so that the added "Q" rises the volume from 25% V_T to 100%V_T.
- Typical time needed for the fill step is 25% of the cycle time.
- The volume addition is controlled by automatic valves or timers.

Interment aeration is needed in this step is needed to prevent aerobic conditions

1. React:

    - The purpose of this step is to start the aerobic reactions by applying oxygen and complete mixing. In this step both organic matter removed and nitrifications achieved.
    - The volume of the tank during this step is100% full with wastewater .
    - The time needed for this step is typically 35% of the cycle time. This time should be checked using the batched reactor design equation .

2. Settle:-

    - The purpose of settle step is to allow solids separation to occur providing a clarified supernatant to be discharged as treated effluent . It is a sedimentation step .
    - The settle step is controlled by [using automatic timers] , it takes $\frac{1}{2}\, hr\ to\ 1\ hr\ [\cong 20\%\, T\ Cycle]$.

During this step no mixing or aeration is applied.

3. Draw:

- The purpose of this step is to remove the clarified treated wastewater from the reactor.
- Draw (تفريغ الماء) is achieved by floating decants or automatic adjustable weirs.
- Draw time is 15 % T cycle (typically 45 minutes).
- The volume is reduced to 35% V_T
- No aeration or mixing is applied during this step.

To prevent solids from leaving with the effluent, it is usually preferred to add an extra volume above the sludge blanket.

4. Idle:

- The purpose of idle step in a multi-tank system(i.e. more than 2 tanks)is to provide time for one reactor to complete its fill cycle before switching to another tank. Idle is not a necessary step, and can be eliminated.
- Aeration and mixing can be applied to prevent anaerobic conditions, depending on the idle time.

of cycle time or longer in some cases. For example if $\cong 5\ \%$ Idle time is the flow "Q_{in}" is minimum and the other tank is in the fill phase is not receiving it's design "Q_{in}", then the tank in the idle step has to wait until the first tank completes the fill step.

3.5.3 Sludge Wasting:

Sludge wasting is not a separate step, it can be done in the idle step, or during the react step if the idle step is eliminated.

3.5.4 Sludge Recycle:

No sludge recycle is needed since sedimentation occurs in the biological reactor, so sludge is already there.

1.5.5 Cycle time in SBR:

The cycle time is the total time needed to complete the five steps mentioned above:

$T_{cycle} = t_f + t_r + t_s + t_d + t_i$

t_f = fill step time

t_r = react step time

t_s = settle step time

t_d = draw step time

t_i = idle step time

Note: there is a relation between t_f and t_r, t_s and t_d:

$$t_f = \frac{t_r + t_s + t_d}{n-1}$$

n= number of SBR tanks used.

Typical cycle time is 4-8hrs.

4. Typical flow chart of SBR treatment plant:

……………………………………………………………………………
……………………………………………………………………………
………………………………

- At least two SBR tanks are needed
- No final sedimentation is needed
- No sludge recycle is needed
- If θ_c is >20 days, no primary sedimentation tank is needed
- SBR tanks are square tanks in which $5 \leq L \leq 30m$
- Typical depth = 5m.

3.5.6 **Advantages of SBR:**

- Biological reactions and final sedimentation is achieved in one tank, so we do not need final sedimentation tank.
- No need for sludge recycle pumping station.
- If θ_c is >20 we do not need primary sedimentation tank, and the wasted sludge is stable.

(Note: $\frac{F}{M}$ ratio is similar to that of oxidation ditches, i.e. 0.02-0.15 $mgBOD_5/mgvssd$)

Example 3.9:

Design an SBR system to achieve both BOD_5 removal and nitrification. The following Data is available:

- $(BOD_5)_{\text{soluble}} = 150\dfrac{mg}{L}, Q = 7500\dfrac{m^3}{d}, X = 3500\dfrac{mgss}{L},$

$$X_s\,(i.e\ X_r) = 10{,}000\frac{mgss}{L}, K_n = 0.5\frac{mgN}{L}$$

$$Y_n = 0.12\frac{mgvss}{mgBOD_5}, \mu_m^n = 0.44d^{-1}, K_{dn} = 0.05d^{-1}$$

$$Y = 0.5\frac{mgvss}{mgBOD_5}$$

$$K_d = 0.05d^{-1}$$

$$K_s = 50\frac{mgBOD_5}{L}$$

$$\mu_m = 2.5d^{-1}$$

- Assume:

  t_s= 0.5hrs (typical)

  t_D= 0.5hs (typical)

  t_r = 1.0hrs (typically 1-2hrs)→Should be checked by the batch reactor design equation.

1. <u>Determine SBR operating cycle:</u>

Tcycle = $t_f + t_r + t_s + t_D$ (t_i = 0.0, not needed)

$t_f = \frac{t_r + t_s + t_D}{n-1}$ assume n =2 SBR tanks, if the dimension of SBR are within $5 \leq L \leq 30$ o.k., other wise more than 2 tanks are needed.

$$t_f = \frac{1.0+0.5+0.5}{2-1} = 2hrs$$

$$T_c = 2+1.0+0.5+0.5 = 4hrs$$

1. <u>Determine number of cycles per tank per day:</u>

$$\text{No cycles} = \frac{24\,hrs}{4\,hrs} = 6\,\frac{cycles}{\tan k \bullet d}$$

2. <u>Determine fill volume per cycle per tank:</u>

$$V_F = \frac{Q_0 / n}{No\ cycles} = \frac{7500/2}{6} = 625\,\frac{m^3}{fill}$$

3. <u>Determine $\frac{V_F}{V_T}$ fraction:</u>

V_{Fill} = fill volume
V_s = settle volume
$V_T = V_F + V_s$

$$V_T \bullet X = V_s \bullet X_s \rightarrow V_s = \frac{V_T X}{X_s}$$

Note: $V_{Fill} = V_{decant}$

$$V_s = \frac{3500}{10{,}000} \bullet V_T = 0.35\,V_T$$

V_{decant} = volume of treated

$V_F = V_T - 0.35V_T = 0.65V_T$ wastewater disposed in each cycle.

Since : $V_F = 625m^3$

$$V_T = \frac{V_F}{0.65} = \frac{625}{0.65} \cong 962m^3$$

1. <u>Determine the surface area of each SBR:</u>

   Assume the depth of each tank is 5m (typical depth)

$$A_{surface} = \frac{V_T}{depth} = \frac{962m^3}{5m} = 192m^2$$

$$L = \sqrt{192} \cong 13.90m$$

so two tanks are o.k $S \le 13.90 \le 30m$

6. <u>Determine the portion of heterotrophic and nitrifiers microorganisms and</u> θ_c:

X= $X_{nitrifiers} + X_{hetrotrophs} = X_n + X_h$

$$X = 3500 \bullet 0.8 = 2800 \ \frac{mgVss}{L}$$

$$X_n = \frac{QY_n(N - Ne)\theta_c}{[1 + K_{dn}\theta_c]} \quad , X_h = \frac{QY(S - Se)\theta_c}{[1 + K_d\theta_c]V_T}$$

$$2800 = \frac{3750 \bullet 10^3 \bullet 0.12(40 - 50)\theta_c}{[1 + 0.05\theta_c]962 \bullet 10^3} + \frac{3750 \bullet 10^3 \bullet 0.5(150 - 10)\theta_c}{[1 + 0.05\theta_c]962 \bullet 10^3}$$

$$2800 = \frac{18.5\theta_c}{[1 + 0.05\theta_c]} + \frac{272.9\theta_c}{(1 + 0.05\theta_c)} \cong 178 \ \frac{mgVss}{L}$$

$$\theta_c \cong 18.5 \ days$$

$$X_n = \frac{18.5\theta_c}{1 + 0.05\theta_c} + \frac{18.5 \bullet 18.5}{1 + 0.05 \bullet 18.5} \cong 178 \ \frac{mgVss}{L}$$

$$X_h = \frac{272.9\theta_c}{1 + 0.05 \bullet \theta_c} + \frac{272.9 \bullet 18.5}{1 + 0.05 \bullet 18.5} = 2623 \ \frac{mgVss}{L}$$

Or $\quad X_h = 2800 - 178 \cong 2622 \ \frac{mgVss}{L}$

1. <u>Check for the reaction time(t_r):</u>

Use the batch reactor design equation:

For nitrification:

$$K_n \ln\frac{N_o}{N_t} + (N_o - N_t) = X_n\left(\frac{\mu_m^n}{Y_n}\right)t$$

No = concentration of nitrogen in the SBR after dilution resulting from mixing V_{Fill} in V_{Total}:

$$N \bullet V_{Fill} = No \bullet V_T \rightarrow No = \frac{N \bullet V_{Fill}}{V_T} = \frac{40 \bullet 625}{962} \cong 26\frac{mgN}{L}$$

N_t = 0.5 mgN/L (the required nitrogen influent).

$$0.5 \ln\frac{2.6}{0.5} + (26 - 0.5) = 178\left(\frac{0.44}{0.12}\right)t$$

$t = 0.042\ day \cong 1hr$ (so t_r = 1.0 is o.k. as assumed)

- Check for BOD_5 removal:

$$K_s \ln\frac{S_o}{S_t} + (S_o - S_t) = X_h\left(\frac{\mu_m}{Y}\right)t$$

$$S_o = \frac{S \bullet V_{Fill}}{V_T} = \frac{150 \bullet 625}{962} \cong 97.5\frac{mgBOD_5}{L}$$

$$S_t = 10\frac{mgBOD_5}{L}$$ (the required BOD_5 in the effluent)

$$50\ln\frac{97.5}{10} + (97.5 - 10) = 2623\left[\frac{2.5}{0.5}\right]t$$

$$\mathrm{t} = 0.0154d \cong 0.37hrs < 1.0hr(t_r = 1.0hr)$$

* Note:

t_r for nitrification always control the design of SBR.

8. Calculate sludge procedure:

$$P_x = \frac{XV}{\theta_c} = \frac{2800 \bullet 962 \bullet 10^3}{18.5 \bullet 10^6} \cong 146 \frac{KgVss}{day}$$

$$Q_w = \frac{P_x}{X_s} = \frac{146 \bullet 10^6}{8000 \bullet 10^3} \cong 18 \frac{m^3}{d}$$

If Q_w is taken during the react step:

$$Q_w = \frac{P_x}{X} = \frac{146 \bullet 10^6}{2800 \bullet 10^3} \cong 52 \frac{m^3}{d}$$

9. Calculate oxygen requirements:

$$R_o = Q(S_{in} - S_e) - 1.42 P_x + 4.57 Q(N_{in} - N_e)$$

$$-(150-10) - 1.42 \bullet 146 + 4.57 \bullet 3750 \bullet \frac{10^3}{10^6}(40-0.5) = 995 \frac{Kg^{o2}}{d} \; per \tan k$$

10. <u>Check</u> $\frac{F}{M}=\frac{QS_o}{Vx}=\frac{3750\bullet 40^3\bullet 150}{2800\bullet 962\bullet 10^3}=0.21\frac{mgBOD_5}{mgVss\bullet d}$

Typical range is(0.02-0.15), this is not in the range.

CHAPTER 4
Attached Growth Biological Treatment System

4.1 Introduction:

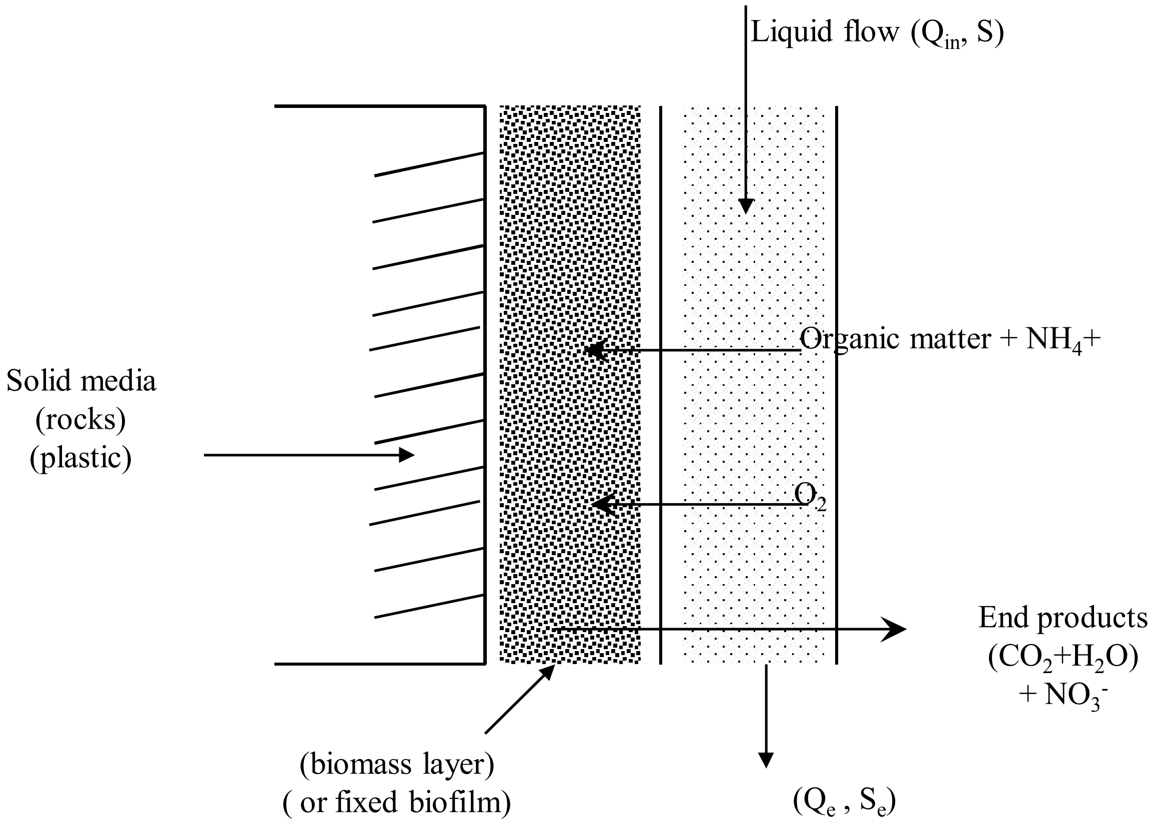
As mentioned previously, Attached Growth is a biological treatment process in which microorganisms responsible for conversion of organic matter or other constituents in wastewater are attached to some inert material such as: rocks, sand or specially ceramic or plastic materials. This process is also called fixed film process.

4.2 Examples of Attached growth system:

Many types of this system has been developed:

- Trickling filters(biological tower).
- Rotating biological contactors(RBC).
- Packed bed reactors
- Fluidized bed biofilm reactors.

4.3 Theory of attached growth treatment:



According to the figure shown, a biomass layer(bacteria) stick to the solid media and grow there. The liquid wastewater pass adjacent to the biomass layer forming a liquid layer. During the passage of the wastewater in the liquid layer and its contacts with the biofilm layer the organic matter, ammonia and dissolved oxygen in addition to other dissolved materials penetrate into the biomass layer by diffusion. The biochemical reactions such as organic matter oxidation, nitrification occur inside the biofilm layer. The end products such as CO_2, H_2O ,and NO_3^- Leave the biofilm layer back to the Liquid Layer and move out with the liquid flow to the effluent stream . Denitrification can be achieved in attached growth system in the lower parts of these systems where anoxic conditions exist . The bacteria in the biomass layer grow and some of it die . The dead bacteria lose its sticky characteristics and its removed from the biomass layer by the action of the moving liquid . The removed bacteria is then removed by sedimentation in a final sedimentation tank .

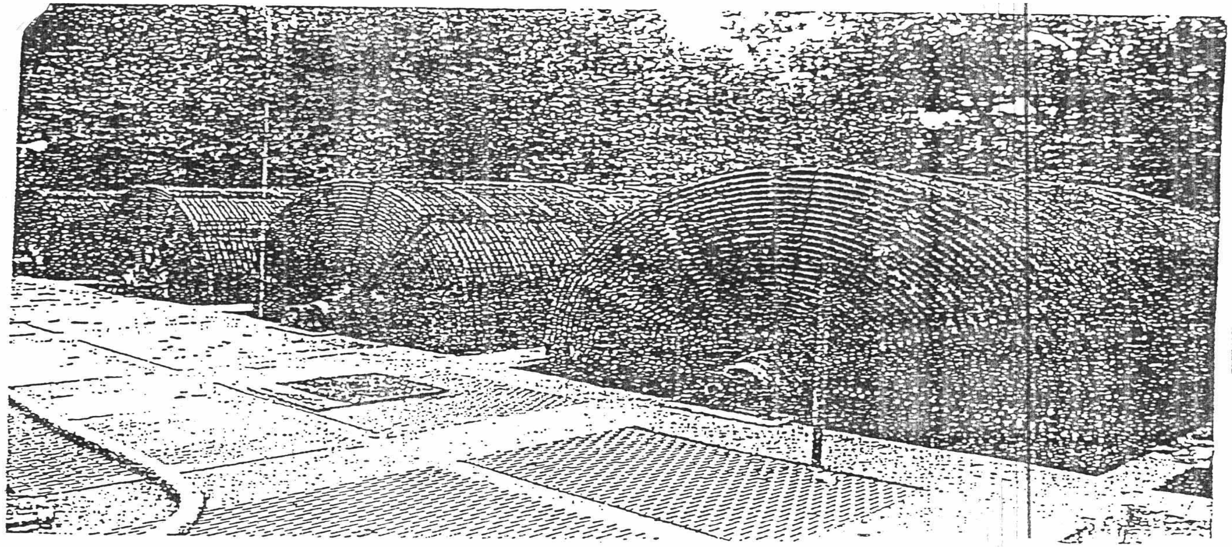
The biochemical reaction for BOD_5 removed nitrification and denitrification are the same of those in the suspended growth system.

4.4 applications of attached growth systems

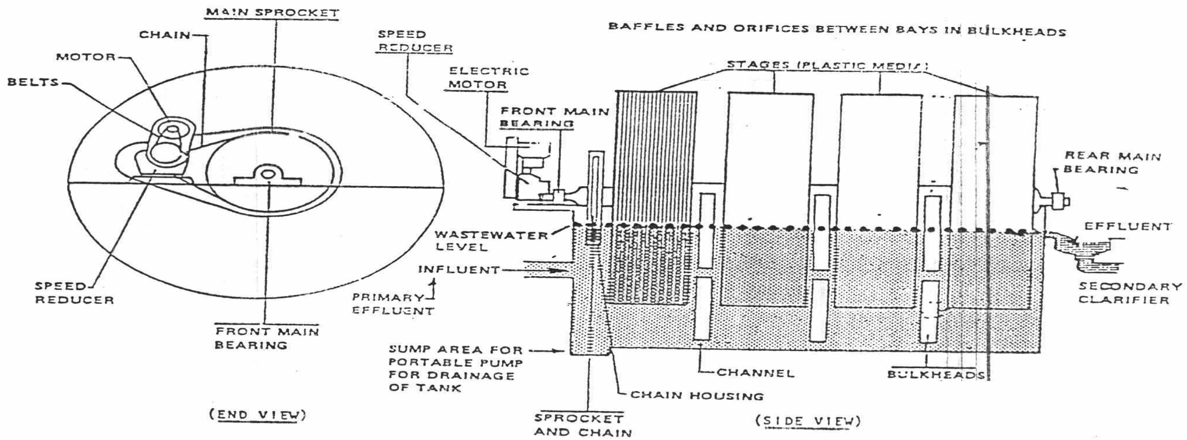
A rotating biological contactors{RBC}

4.4.1 Introduction

- Rotating biological contactors consist of a series of closely spaced circular disks of polyvinyl chloride (PVC) that are submerged in wastewater and rotated through it (see figure 1) .The cylindrical disk are attached to a horizontal shaft and are provided at standard unit sizes of approximately 3.5m in diameter and 7.5m in length . The surface area of disks for a standard unit is about 9300m^2 ,and 13900m^2 for high density units
- The RBC unit is partially submerged (typically 40%) in a tank containing wastewater , and the disks rotate slowly at about 1.0 to 1.6 revolutions per minute .
- As the RBC disks rotate out of the wastewater , aeration is accomplished by exposure to the atmosphere .



Fig(1) A full scale rotating biological contactor (Autotrol Corporation)



Equipment for inspection in a RBC unit.

4.4.2 RBC process design considerations:-

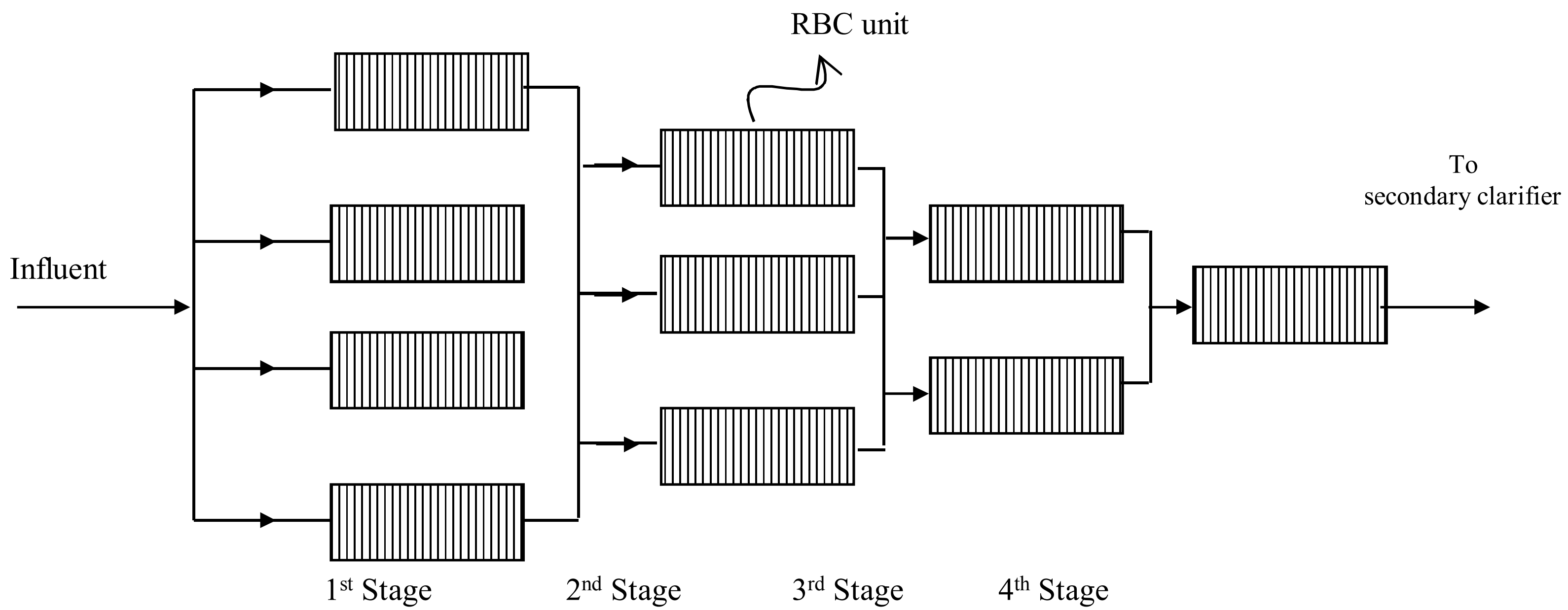
The following are the main design parameters needed to design the RBC System :-

1. Staging of the RBC units
2. Organic loading rate
3. Hydraulic loading rate

4.4.3 RBC staging:-

- The RBC process application typically consists of a number of units operated in series .
- For this purpose , RPC is divided in to stages . Number of stages depends on the treatment goals . For BOD removal " 2 " to " 4" stages are needed ,and " 6" or more stages for nitrification .

- NOTE :- the number of shafts in each stage depends on the treatment efficiency required .
- The separation between stages is accomplished by using baffles in a single tank or by a series of separate tanks .
- As the wastewater flow through the system , each subsequent stage receives an influent with a lower organic matter concentration than the previous stage.
- The RBC units may be arranged parallel or normal to the direction of wastewater flow.



4.4.4 Organic loading rate:

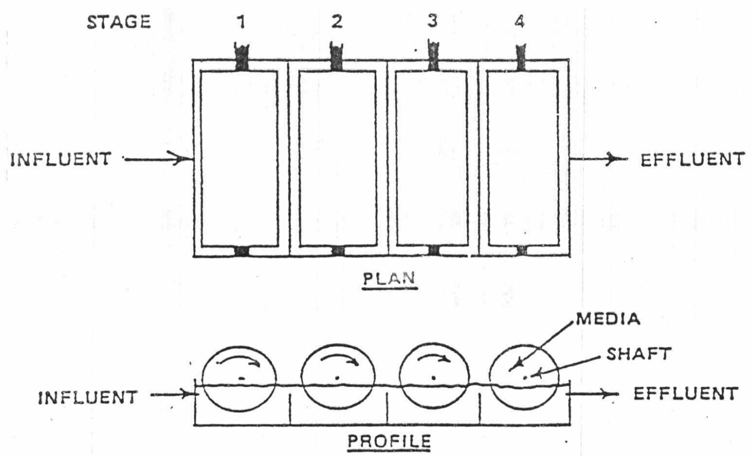
The organic loading rate for RBC in typically in the range 4-10 g(BOD) $\frac{soluble}{m^2 \bullet d}$ for BOD removal only. If both BOD removal and nitrification, the range is 2.5-4 $\frac{g(BOD_5)}{m^2 \bullet d}$

Nitrifying bacteria can not develop in RBC until(BOD_5) drops to less than 15mglL. The maximum nitrogen surface removal rate that has been observed to be about 1.5 $\frac{gN}{m^2 \bullet d}$

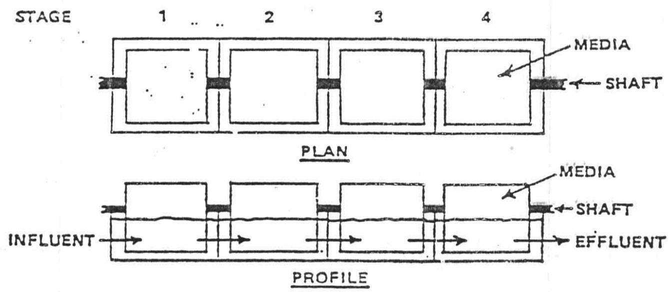
The maximum 1st stage organic loading is 12-15 $\frac{(BOD_5)_s}{m^2 \bullet d}$.

4.4.5 Hydraulic loading rate:

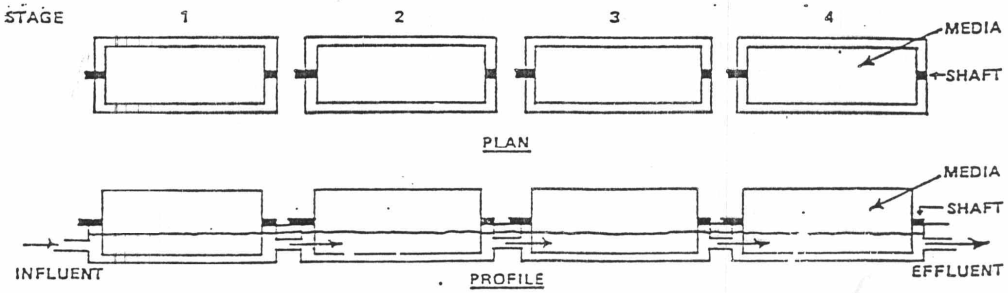
The typical hydraulic loading rate of 0.08-0.16 m^3/m^2 for BOD_5 removal and 0.03-0.08 m^3/m^2 for both BOD_5 removal and nitrification. The hydraulic detention time (θ)is 0.7-1.5hrs for BOD_5 removal and 1.5-4hrs for both BOD_5 removal and nitrification. The volume of RBC tank has been optimized at 0.0049 m^3/m^2 for one shaft of 9300 m^2.
A tank volume of 45 m^3 is needed. Based on this volume and a hydraulic loading rate of 0.08 $m^3/m^2.d$ the detention time is 1.44hrs. Atypical side wall depth is 1.5m to achieve 40% submergence.



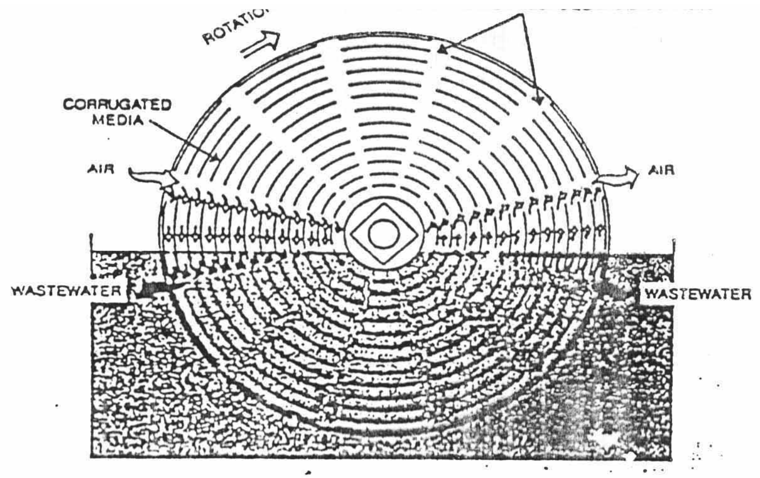
layout with flow perpendicular to shaft



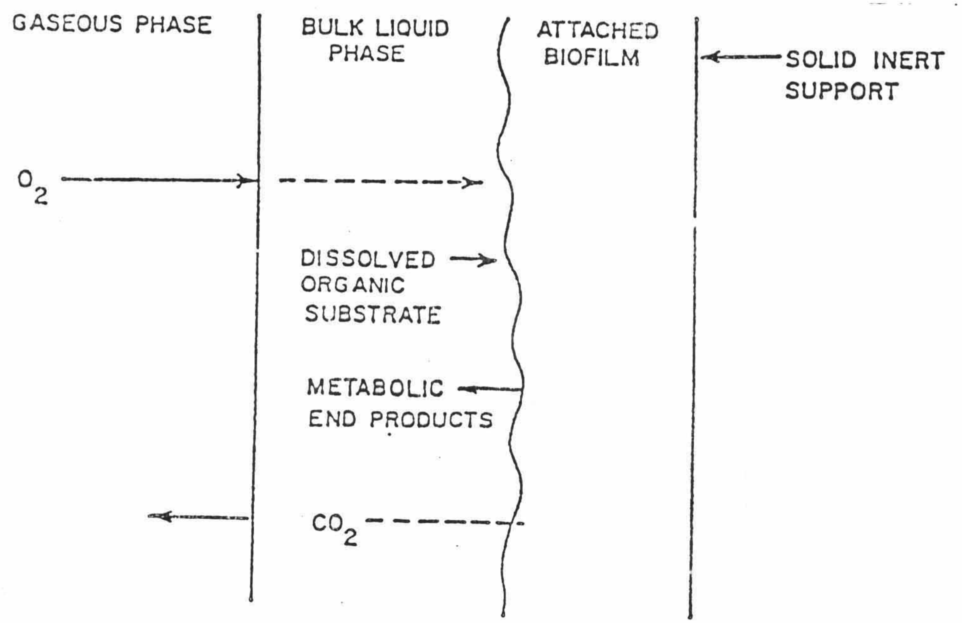
RBC with one shaft, flow parallel to shaft



RBC with four shaft, flow parallel to shaft

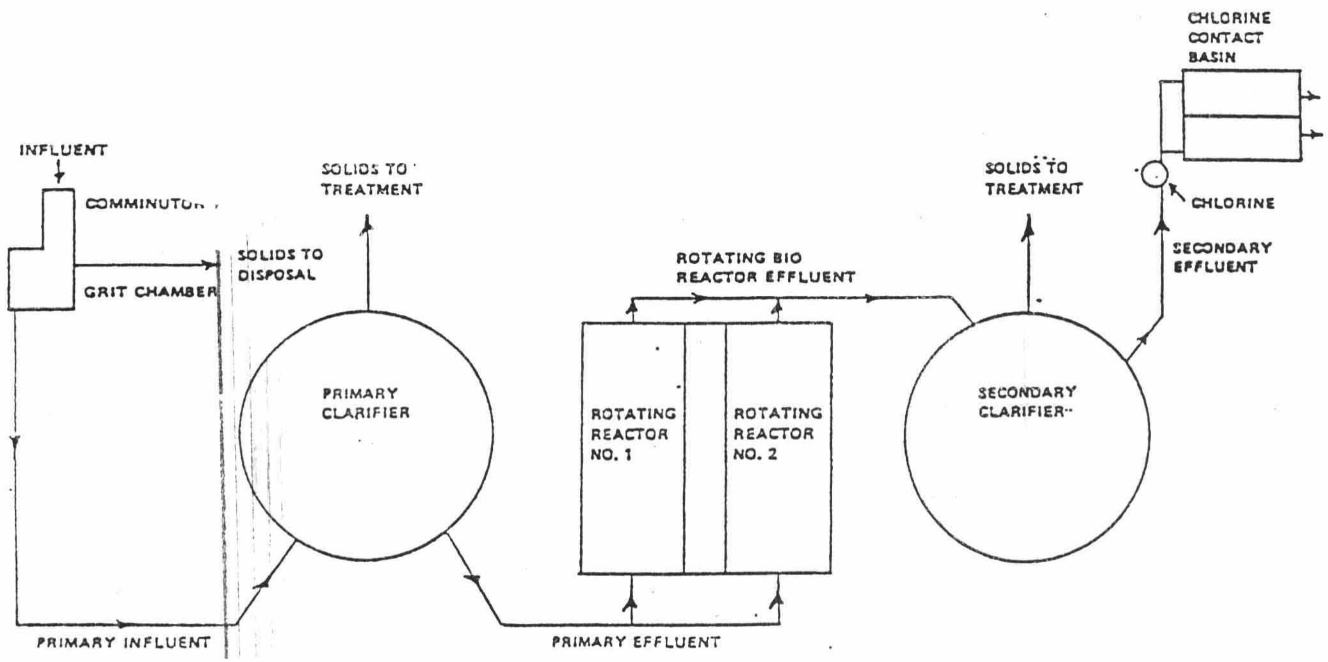


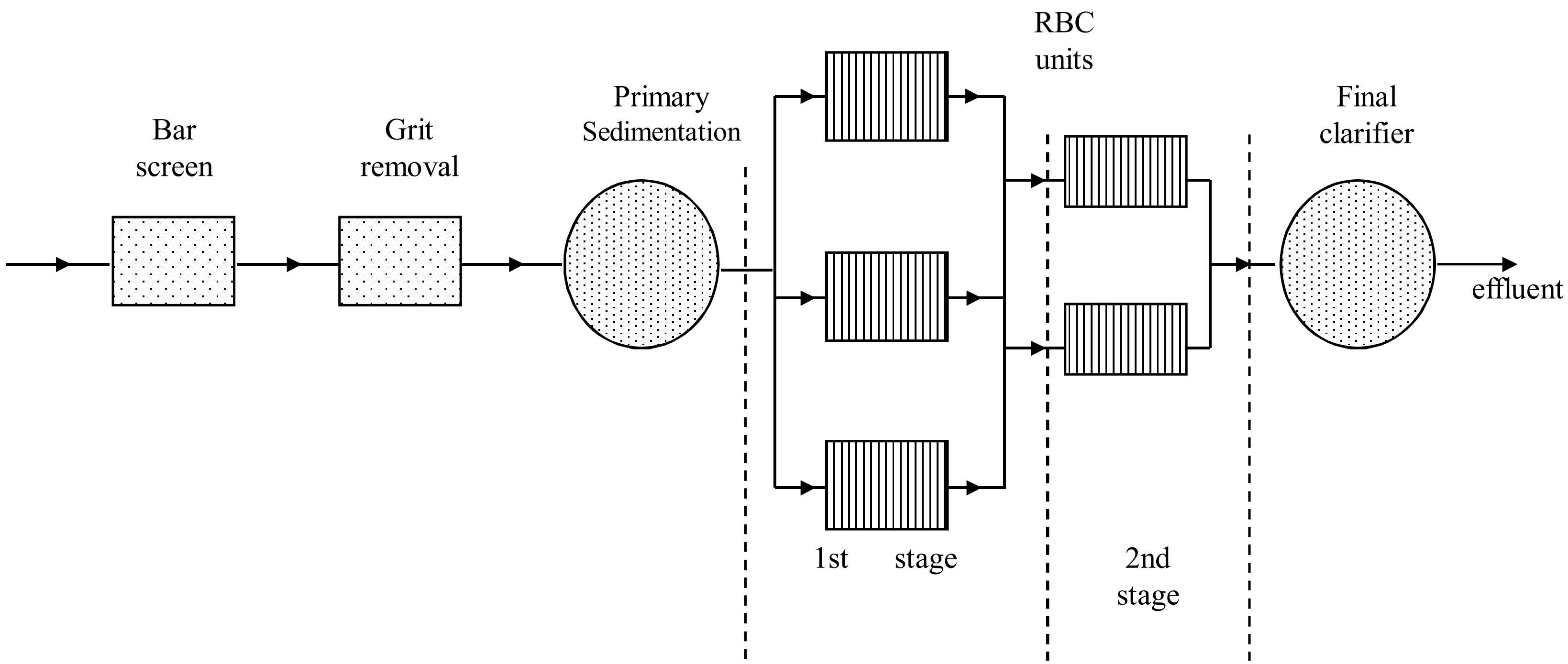
Schematic illustration of exchange of air and water in RBC reactor



Simplified model of biofilm

4.4.6 Typical treatment plan configuration:




Bar
screen
Grit
removal
Primary
Sedimentation
RBC
units
Final
clarifier
effluent
1st stage
2nd
stage

4.4.7 Design equation of RBC:

The following empirical equation developed by Optaken(US EPA,1985):

$$S_n = \frac{-1 + \sqrt{1 + 0.039(\frac{A_s}{Q})S_{n-1}}}{0.0195(\frac{A_s}{Q})}$$

Where S_n = soluble BOD_5 concentration in stage(n), (mg/L)

A_s = disk surface area on stage(n), m^2

Q= flow rate, m^3/d

Example 4.1:

Design a rotating biological contractor to treat an influent soluble BOD_5 of 90 mg BOD_5/L.

The flow(Q)= 4000 m^3/d

Solution:

- Assume 1st stage (BOD_5) organic loading= 15g/m^2.d

BOD_5 (loading)= $(BOD_5)_{\text{concentration}} \bullet Q_{in}$

$$=90\,\frac{mg}{L} \bullet 4000\,\frac{m^3}{d} \bullet \frac{10^3\,L}{m^3} \bullet \frac{g}{10^3\,mg} = 360{,}000\,\frac{g}{d}$$

Disk area= $\dfrac{360{,}000\,g/d}{15\,g/m^2.d} = 2400\,m^2$ (first area stage)

Use $9300\,\dfrac{m^2}{shaft}$ so number of shafts needed for the first stage:

$$N=\frac{2400\,m^2}{9300\,m^2/shaft} \cong 2.6 \qquad \text{say 3 shafts}$$

- <u>Calculate S_1, the BOD concentration after the first stage:</u>

$$S_n=\frac{-1+\sqrt{1+0.039(\frac{As}{Q})S_{n-1}}}{0.0195(As/Q)}$$

For the first stage n=1, $S_n=S_1$, $S_{n-1} = S_0$

$S_0 = 90$ mg/L, $A_S = 3\times 9300 = 27900 \quad m^2, Q = 4000 \; m^3/d$

$$\frac{A_S}{Q} = \frac{27900\,m^2}{4000\,m^3/d} = 6.98\, d/m$$

$$S_1=\frac{-1+\sqrt{1+0.039\bullet 6.98\bullet 90}}{0.0195\bullet 6.98} = 29.75\frac{mg}{L} > 14\frac{mg}{L}$$

So we need one more stage.

- <u>Add another stage and calculate S_2:</u>

Assume two shafts in the second stage:

$A_S = 2 \times 9300 m^2 = 18600\, m^2$

$$\frac{A_S}{Q} = \frac{18600}{4000} = 4.65 \frac{m}{d}$$

$S_n = S_2$, $S_{n-1} = S_{2-1} = S_1 = 29.75$mg/L

$$S_2 = \frac{-1 + \sqrt{1 + 0.039 \bullet 4.65 \bullet 29.75}}{0.0195 \bullet 4.65} = 16.86 \frac{mg}{L} > 14 \frac{mg}{L}$$

So we need one more stage.

- <u>Add another stage and calculate S_3:</u>

Assume one shaft in the third stage:

$A_S = 1 \times 9300\, m^2 = 9300\, m^2$

$$\frac{A_S}{Q} = \frac{9300\ m^2}{4000 \frac{m^3}{d}} = 2.33 \frac{d}{m}$$

Follow Example 4.1:

$S_n=S_3, S_{n-1}=S_{3-1}=S_2=16.86 mg/L$

$$S_3=\frac{-1+\sqrt{1+0.039\bullet 2.33\bullet 16.86}}{0.0195\bullet 2.33}\cong 13\frac{mg}{L}<14\frac{mg}{L} \quad \text{OK}$$

So three stages are enough.

- Check for the hydraulic loading:

$$HLR=\frac{Q}{total\ number\ of\ shafts\bullet Area\ of\ each\ shaft}$$

$N_{shafts}=3+2+1=6$ shafts

$$HLR=\frac{4000\frac{m^3}{d}}{6\times 9300}=0.072\frac{m^3}{m^2\bullet d},$$ typical range(0.08-0.16), which is a little bit lower than the range.

- Is nitrification possible in any of the three stages?:

*Nitrification is only possible when soluble BOD_5 loading is less than

$10g\ \frac{BOD}{m^2 \bullet d}$

- 1^{st} stage = $4000\frac{m^3}{d} \bullet 90\frac{g}{m^3} \bullet \frac{1}{3 \bullet 9300} = 12.9\ \frac{gBOD_5}{m^2 \bullet d}$ >10(no nitrification)
- 2^{nd} stage = $4000\frac{m^3}{d} \bullet 29.75\frac{g}{m^3} \bullet \frac{1}{2 \bullet 9300} = 6.4\ \frac{gBOD_5}{m^2 \bullet d}$ (nitrification occurs)
- 3^{rd} stage = $4000\ \frac{m^3}{d} \bullet 16.86\ \frac{g}{m^3} \bullet \frac{1}{1 \bullet 9300} = 7.25\ \frac{gBOD_5}{m^2 \bullet d}$ (nitrification occurs)

*Rate of nitrification is related to the soluble BOD_5 loading by the following equation:

$$r_n = 1.5\left[1 - 0.1(BOD_5)\right] \quad \frac{g\ N}{m^2 \bullet d}$$

* So for 2nd stage→ $r_n = 1.5\left[1 - 0.1 \bullet 6.4\right] = 0.54 \quad \frac{g\ N}{m^2 \bullet d}$

* And for 3rd stage→ $r_n = 1.5\left[1 - 0.1 \bullet 7.25\right] = 0.413 \quad \frac{g\ N}{m^2 \bullet d}$

*If the ammonia concentration in the influent to the 2nd stage is 30 mgN/L, find the effluent ammonia concentration.

$$r_n = 0.54 \quad \frac{g\ N}{m^2 \bullet d} \quad \text{for } 2^{nd} \text{ stage}$$

$$\text{Nitrogen removal} = 0.54 \quad \frac{g\ N}{m^2 \bullet d} \bullet (2 \bullet 9300 m^2) = 10044 \frac{g}{d}$$

$$\text{Concentration} = \frac{10044 g}{d} \bullet \frac{d}{4000 m^3} = 2.51 \frac{mg}{L}$$

$$\text{So} \rightarrow N_2 = 30 - 2.51 \cong 27.5 \frac{mg}{L}$$

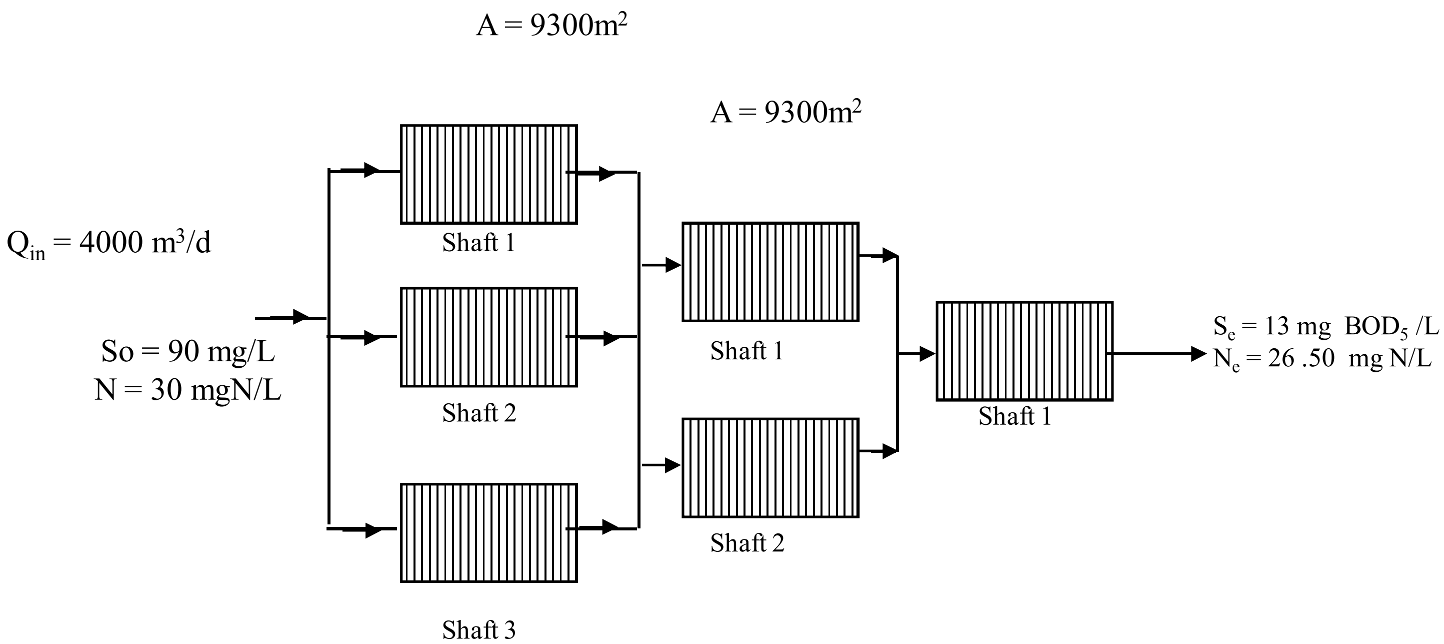
$$\rightarrow r_n = 0.413 \ \frac{g\ N}{m^2 \bullet d} \ \text{for } 3^{rd} \text{ stage,}$$

$$\text{Nitrogen removal} = 0.66 \ \frac{gN}{m^2 \bullet d} \times (1 \bullet 9300 m^2) = 3841 \frac{g}{d}$$

$$\text{Concentration} = 3841 \frac{g}{d} \bullet \frac{d}{4000 m^3} = 0.96 \frac{mg}{L}$$

$$\text{So} \rightarrow N_3 = 27.5 \ \frac{mgN}{L} - 0.96 \cong 26.5 \frac{mgN}{L}$$

If complete nitrification is needed a separate nitrification stage should be added after these stages.



Proposed design

Note: each shaft has a tank volume of 45m3.