The Islamic University of Gaza- Civil Engineering Department

Advanced Sanitary Engineering- ECIV 5325

Unit 3. Suspended growth treatment systems (Aerobic & Anaerobic)

Based on Dr. Fahid Rabah lecture notes

Many treatment systems based on suspended microorganisms have been developed and still used till now.

Some of these systems are aerobic and other are anaerobic.

Some of the most commonly used systems are:
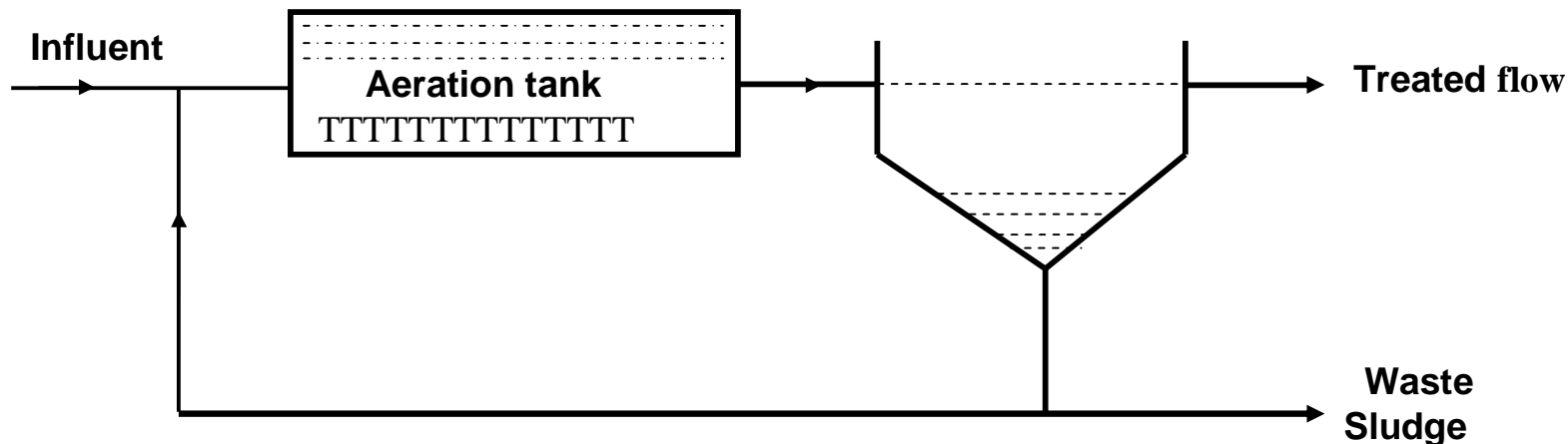
Activated sludge systems:

• Conventional activated sludge system

• Oxidation ditches

• Sequential batch reactor (SBR)

• Aerated lagoons

• Waste stabilization ponds

• Up flow anaerobic sludge blanket (UASB)
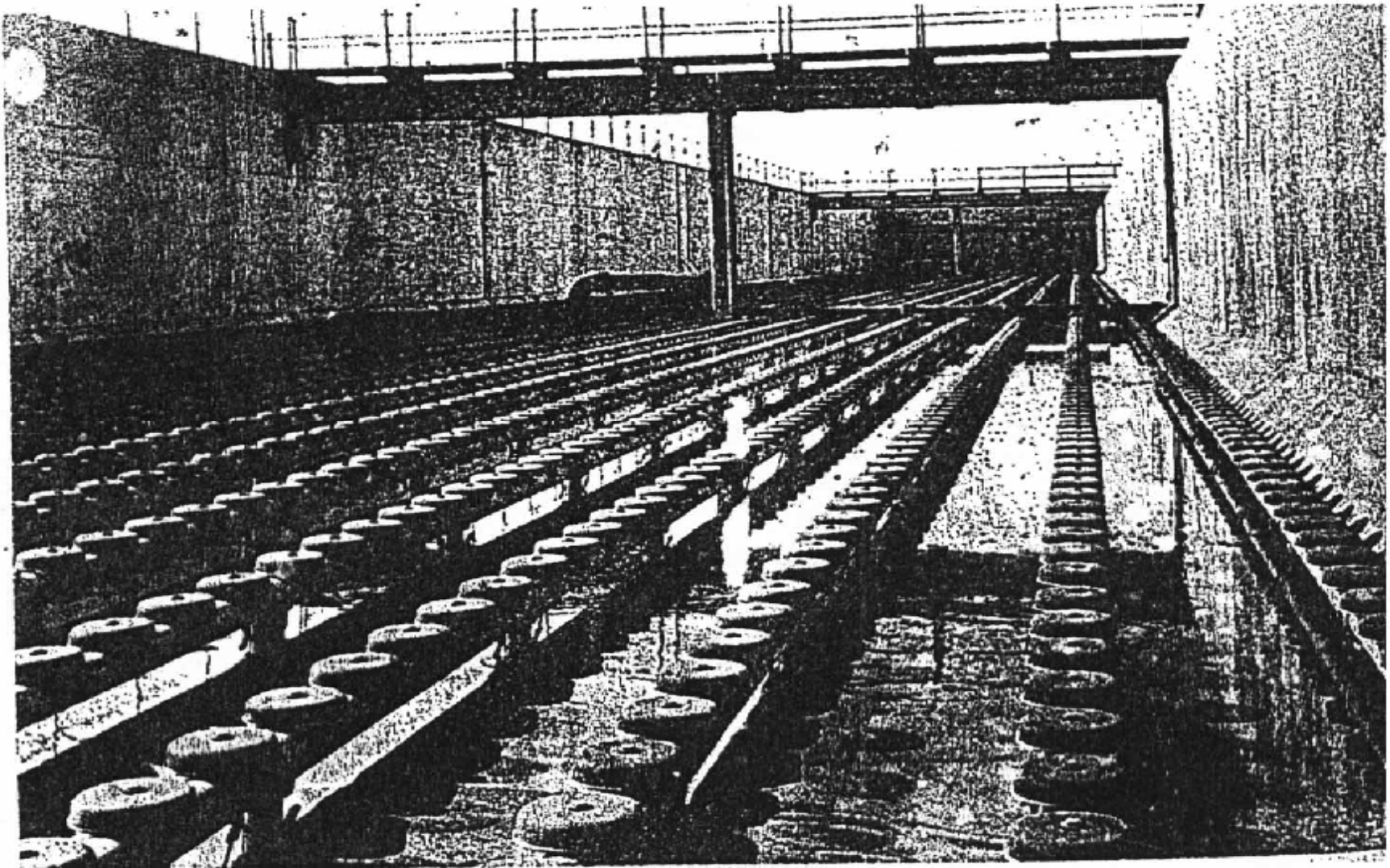
Conventional activated sludge system

The first version of activated sludge systems are called 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

1. Design of activated sludge system as a completely mixed flow reactor (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 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:-

m_m = 2.5d^{-1} , K$_d$ = 0.05d^{-1} Y = 0.5 mg VSS/mg BOD removed

K$_S$ = 100 mg BOD / L   Flow = 0.15 m^3/s = 12960 m^3/d

BOD = 84 mg/L (Soluble)   required effluent → (total BOD) = 30 mg/L

Solution:-

1. Since we always deal with soluble substrate, first we need to find the effluent soluble BOD :-

(BOD) soluble = S = (BOD) total - BOD in suspended solids or particulate

(BOD) in suspended solids = 63% * SS

(BOD) particulate = 0.63 * 30 = 18.9 mg BOD5 /L

(BOD) soluble in effluent = 30 – 18.9 = 11.1 mg BOD /L

Note: (BOD) soluble in effluent = S = (BOD) soluble in the aeration tank.

Follow Example 1:-

2. Calculate q_c:- $\boxed{S = \dfrac{K_s(1 + K_d q_c)}{q_c(m_m - K_d) - 1}}$ (this is equ. (18) for CMFR)

$$11.1 = \frac{100\left[1 + 0.05 * q_c\right]}{q_c\left[2.5 - 0.05\right] - 1}$$ $\Rightarrow$ Solve for $q_c \longrightarrow q_c = 5$ days (mean cell residence time)

$m_m - K_d = 2.5 - 0.05 = 2.45$ $\qquad \dfrac{1}{q_c m} = m_m - K_d$

$q_c^m = 0.408$ d (minimum sludge retention time)

So S.F $= \dfrac{qc}{q_c^m} = \dfrac{5}{0.408} = 12.25$ d (2<12.25<20) OK

Check for s$_{min}$:-

$$S_{min} = K_s \frac{K_d}{m_m - K_d} \qquad 100 \left[\frac{0.05}{2.5 - 0.05} \right] = 2.04 \text{ mg BOD/L}$$

$S > S_{min} \rightarrow$ o.k (11.1 > 2.04) So use q_c = 5 days, S = 11.1 mg BOD/L

3. Calculate the aeration tank volume:-

Assume the concentration of biomass (X) is equal to 3000 mg MLVSS (Mixed liquor volatile suspended solids)/L:-

$$X = \frac{q_c}{q} \cdot \frac{Y(S_0 - S)}{(1 + K_d q_c)} \qquad \text{(equ. 17)}$$

$$3000 = \frac{5 * 0.5[84 - 11.1]}{q[1 + 0.05 * 5]} \qquad \text{(solve for } \Theta\text{)} \qquad \theta = 0.0486 \text{ day} \qquad \theta = 1.17 \text{ hours}$$

Calculate the volume:- V = θ Q = 0.0486*12960 ≈ 630 m^3

Check F/M ratio:-

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

= 0.576 mg BOD/mg MLVSS .d   (O.K)

Typical range for conventional activated sludge system is 0.1 – 0.6 mg BOD/mg MLVSS .d. (Mixed liquor volatile suspended solids)

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:

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

$$Y_{obs} = \frac{Y}{1 + K_d qc} = \frac{0.5}{1 + 0.05 * 5} = 0.4 mgVss / mgBoD$$

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

$$P_x \cong 378 Kg / d$$

Another way to find P_x:-

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

$$P_x = 378kg/d$$

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 m³/d

5. Calculate the recycle flow Q_r:-

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

$$R = \frac{3000}{10000 - 3000} = 0.43 \qquad\qquad Q_r = 0.43Q \cong 5573 m^3/d$$

6. Calculate the Oxygen required:-

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

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

$$R_0 = 408 \, KgO_2/d$$

13

Example 2:-

For example 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 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 1 that θ_c required for BOD removal was = 5 days.

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

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

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

- **Calculate q_c for complete nitrification:-**

$$N = \frac{K_n\left[1+(K_d)_n q_c\right]}{q_c\left[(m_m)_n - (K_d)_n\right]-1} \qquad\qquad 1 = \frac{0.4(1+0.04*q_c)}{q_c(0.25-0.04)-1}$$

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

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

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

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

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

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

16

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

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

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

4. Calculate θ:-

Assume that $\dfrac{X_{nitrifiers}}{X_{Total}} = 0.10$ (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

<u>θ for heterotrophs</u>: $q = \dfrac{q_c\, Y(S_o-S)}{X(1+K_d\, q_c)} = \dfrac{10*0.5(84-6.38)}{2700(1+0.04*10)} = \dfrac{388.1}{3780} = 0.103\, d$

<u>θ for nitrifies</u>: $q = \dfrac{10*0.2(40-0.51)}{300(1+0.04*10)} = \dfrac{78.98}{420} = 0.188\, d$

θ for nitrifiers > θ for heterotrophs, so take θ = 0.188 d = 4.5 hours

5. Calculate the volume of the reactor:-

$V = Q \ \theta = 12960 * 0.188 = 2436 \ m^3$
Compare this volume with the $630 \ m^3$ needed for BOD removal only.

Note:-

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

$$f_n = \frac{0.16(N_0 - N)}{0.6(S_0 - S) + 0.16(N_0 - N)} \ , \ \text{where} \quad N_0 = \text{TKN in the influent, and}$$

$N =$ TKN in the effluent.

---

6. Calculate the sludge to be wasted:-

* for hetaotrophs:-

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

$P_x = 402$ kg Vss/d

for nitrifiers:-

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

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

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

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

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

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

Calculate oxygen requirement:-

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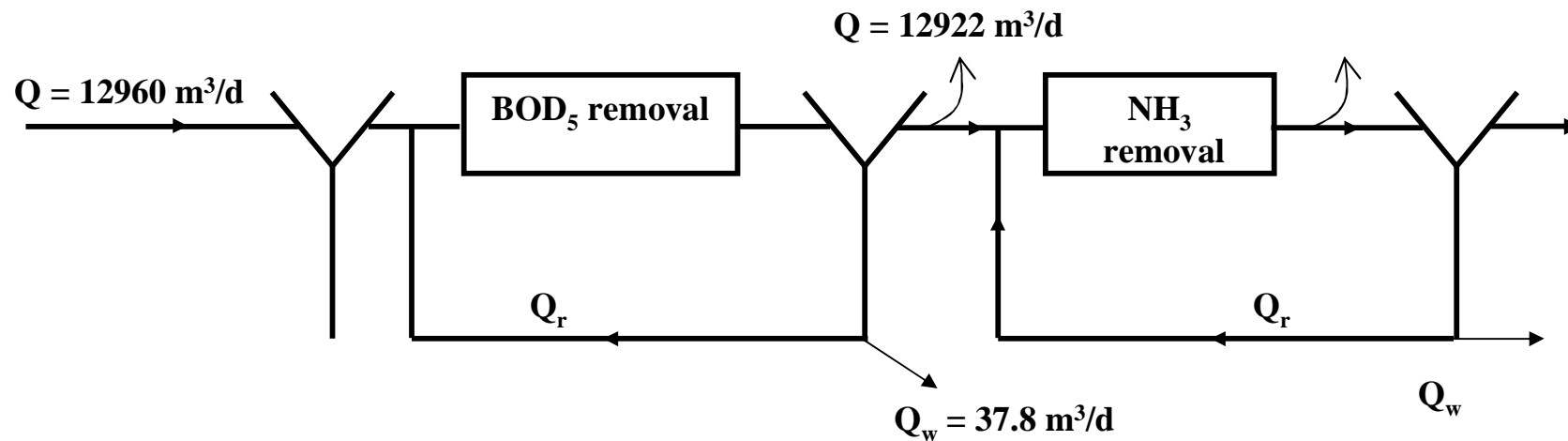
$$= 12960\,(84 - 6.38)* \frac{1kg}{10^6\,mg} \cdot \frac{10^3\,l}{1m^3} - 1.42\,(402 + 37.8) + 4.57*12960(40 - 0.51)\frac{10^3\,l}{m^3} \cdot \frac{1kg}{10^6\,mg}$$

$$R_O = 1005.96 - 624.5 + 2338.9$$

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

Example 3:-

For example 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 2.

Solution:-

1.Calculations for q_c^{min} , S_{min} for nitrifies:-

From example 2 it was found that:-

q_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' = Q – Q_w = Q_e from the BOD removal CMFR

Q' = 12960 – 37.8 $\cong$ 12922 m^3/d

3.Calculate for the nitrification CMFR:-

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

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

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

$$f_n = 1.0$$

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

22

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

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

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

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

$$q = \frac{V}{Q} = \frac{1149}{12922} = 0.089 \quad d = 2.13 \, hrs$$

4. find q_c:-

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

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

5. Calculate the sludge to be wasted:- $P_x = Y_{obs} Q(N_O - N)$

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

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

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

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

24

6. Calculate oxygen requirements:-

$R_o = 4.57\ Q\ (N_O - N)$

(Note:- this is the oxygen needed for nitrification only)

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

$R_o = 2303$ kg O_2/d

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

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

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

- Assume 8% oxygen transfer efficiency:-
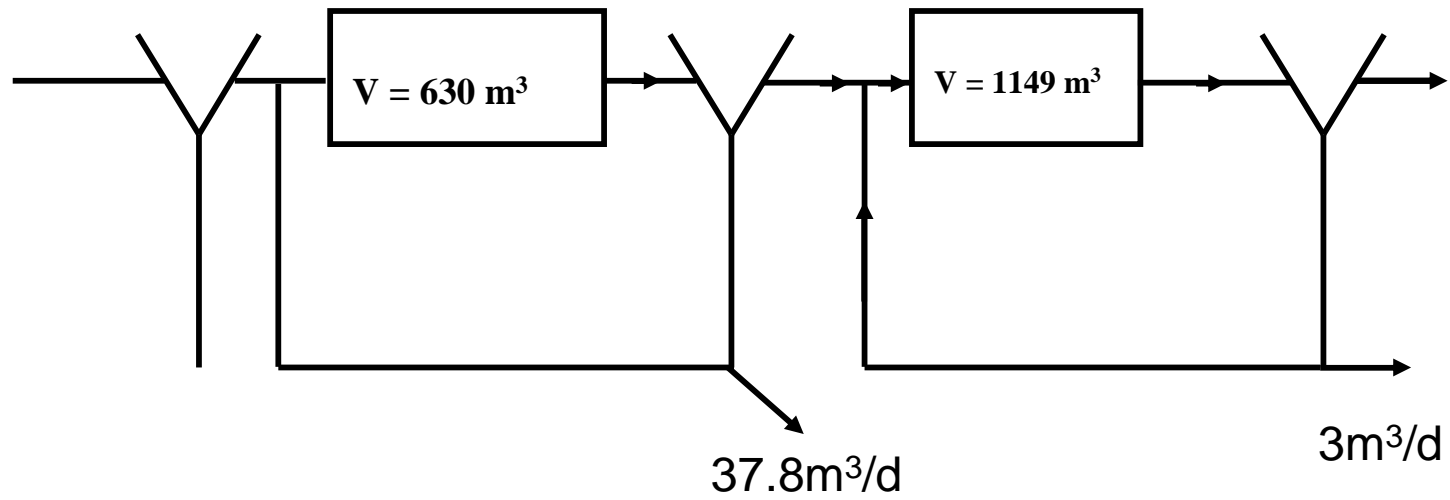
$$Q_{air} = \frac{8377}{0.08} \cong 104713 \, m_{air}^3 / d$$

- If pure oxygen to be used:-

$$Q_{oxysen} = \frac{R_0}{r_{air}} = \frac{2303}{1.185} = 1943 \, m_{O2}^3 / d$$

Assume 8% oxygen transfer efficiency:-

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

So the separate stage nitrification system will look as the following

V = 630 m³   V = 1149 m³

37.8m³/d   3m³/d

Example 4:- Denitrification

For the system designed in Examples 1, 2, and 3, design a separate stage
denitrification completely mixed flow reactor (CMFR). The denitrifying bacteria
have the following growth constants:-

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

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

Required $\rightarrow NO_3^-$ - N in the effluent = l mg NO_3^- - N/L
(So = $(NO_3^-$ - N$)_o$ = D_o = 39 mg NO_3^- - N/L)

Solution:-

- The procedure is the same as that followed in example 1, except that we do not need oxygen for denitrification.
- We need to add organic matter, because denitrifies are heterotrophic bacteria.

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

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

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

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

2. Calculate θ_c:-

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

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

$\theta_c > \theta_c^{min}$ O.K   Cheek factor of safety:-

$$S.F = \frac{q_c}{q_c^{min}} = \frac{3.28}{2.78} = 1.18 < 2 \text{ not o.k} \qquad \text{Take S.F} = 2.1$$

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

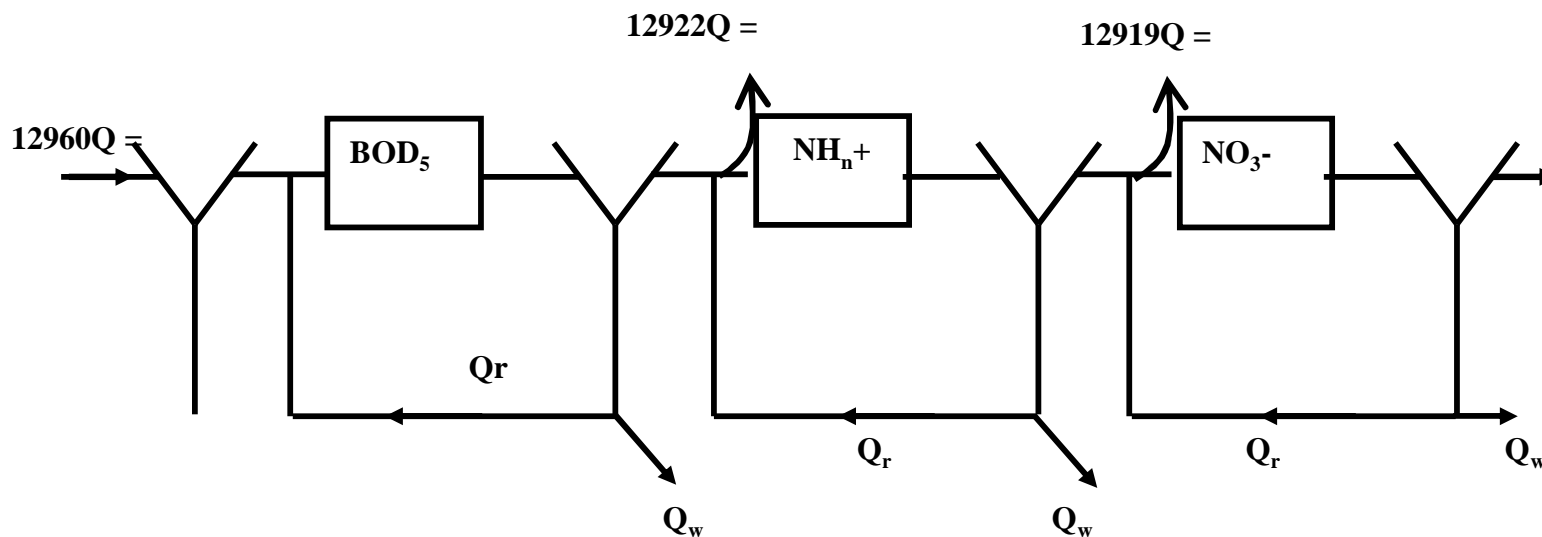## 3. Calculate θ:-

Assume X = 3000 mg MLVSS/L

$$q = \frac{q_c}{X} \cdot \frac{Y_{Dn}[D_o - D]}{1 + K_d q_c} \frac{5.84}{3000} \cdot \frac{0.9[39 - 1]}{(1 + 0.04 * 5.84)} = 0.054d \qquad \theta \cong 1.3 \text{ hrs o.k}$$

4. Calculate V:-

$Q = 12922 - 3 \quad = 12919 \ m^3/d$

$V = Q\theta = 12919 * 0.054 = 697.6 \cong 698 \ m^3$

<u>Find P_x , Q_W, Q_r by the same way as in example 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. <u>The overflow rate</u>:- (or hydraulic loading)
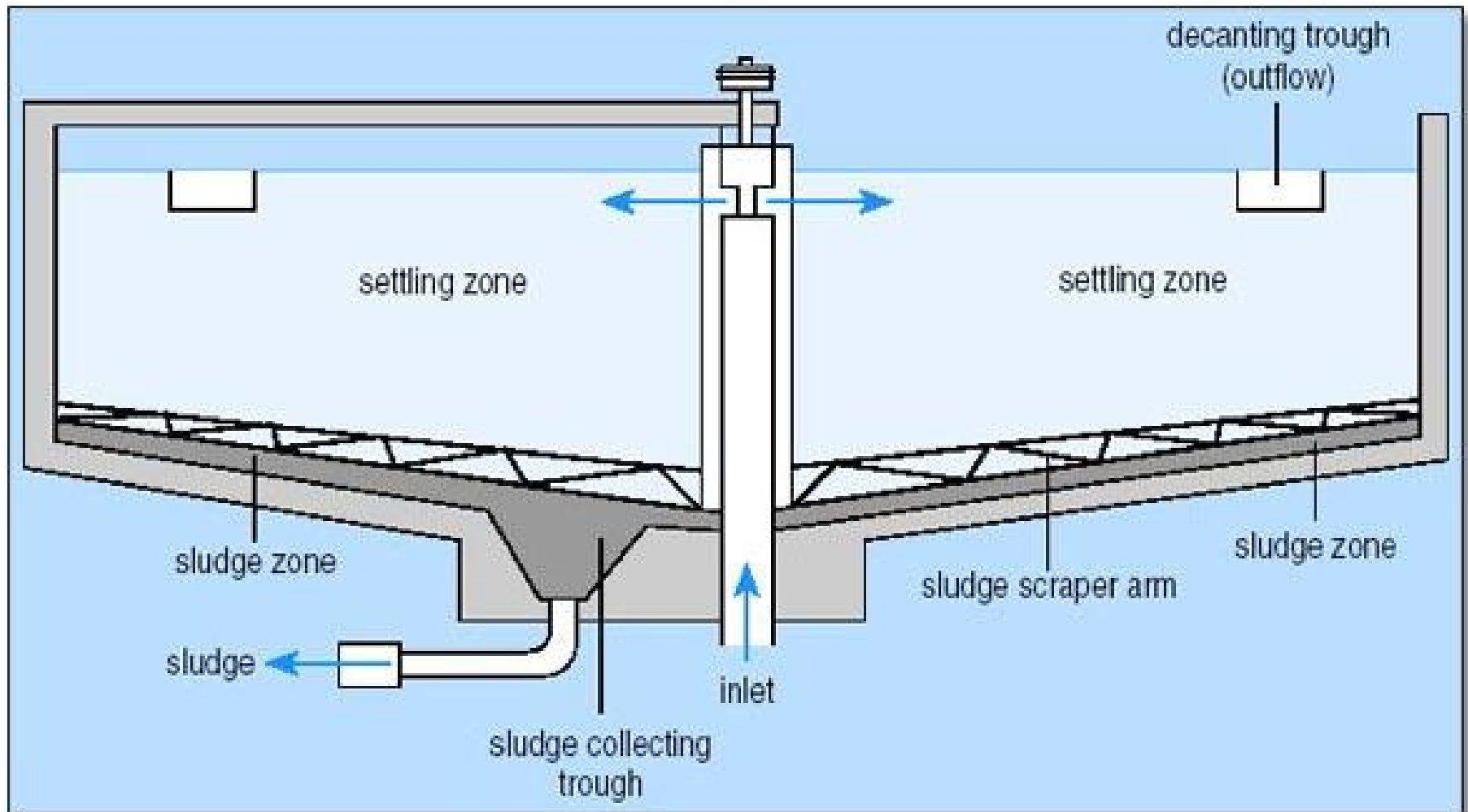it is the amount of flow in m^3/d applied to the unit area (m^2) of the sedimentation tank and it's units are:-

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

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

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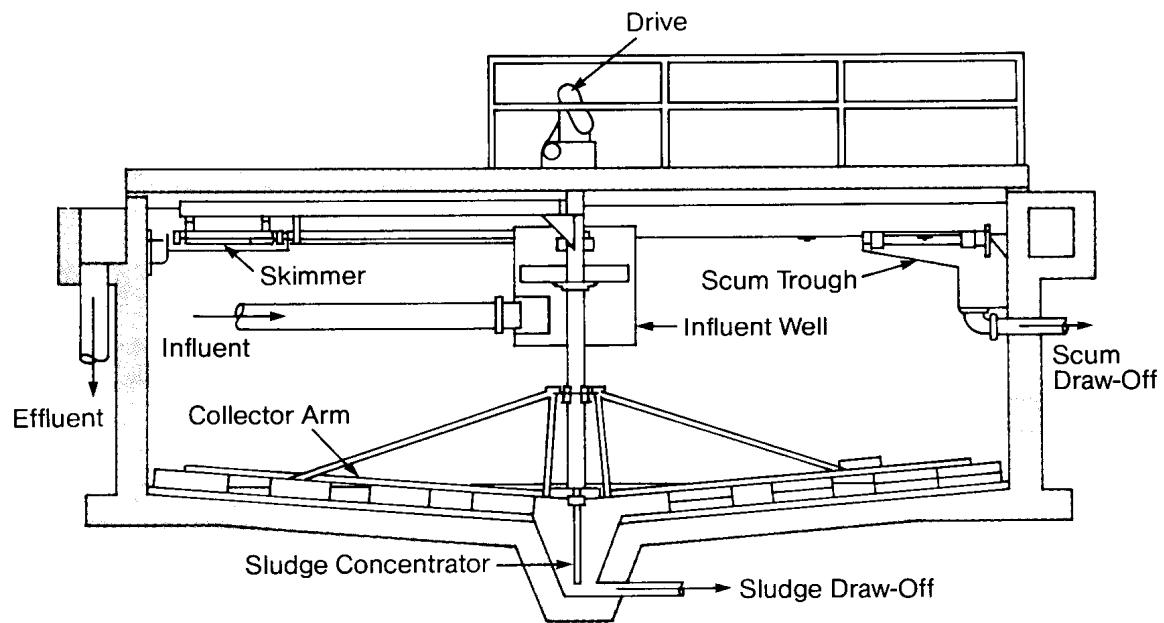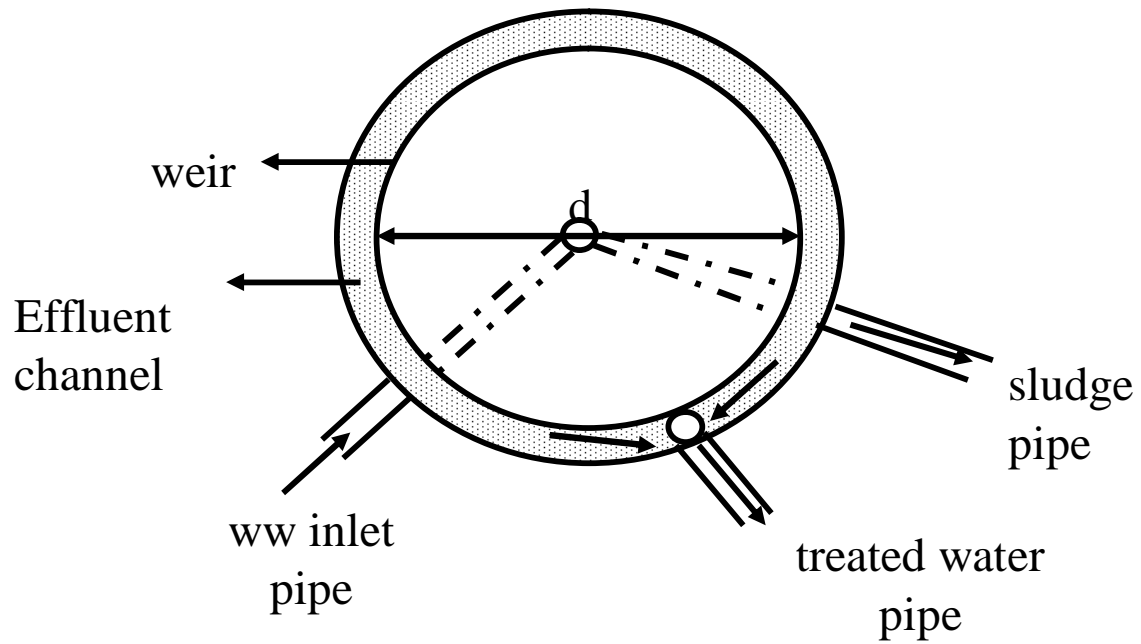
- The typical range of O/F is (20-34) $\dfrac{m^3}{m^2 \bullet d}$

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

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

Schematic diagram of sludge collector for circular sedimentation basins.

weir

Effluent
channel

ww inlet
pipe

d

sludge
pipe

treated water
pipe

Drive

Skimmer

Influent

Effluent

Collector Arm

Scum Trough

Influent Well

Scum
Draw-Off

Sludge Concentrator

Sludge Draw-Off

Effluent channel of circular sedimentation basins.

$Q_0 + Q_r$ $Q_e = Q_0 - Q_w$

Q_r Q_w
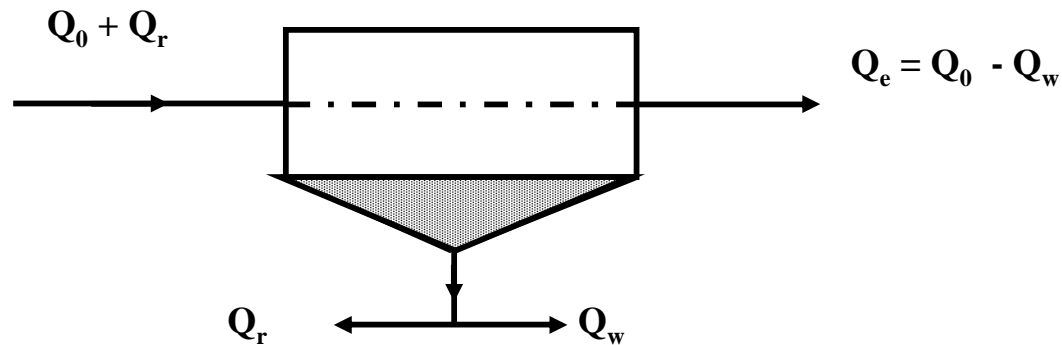
2. The weir loading rate:-

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:-

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

36

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.

$$(SL = \frac{(Q_o + Q_r)}{A_s} \bullet X) \quad \frac{Kg}{d.m^2}$$

Typical range of SL is 130 – 300 Kg/ d.m². If SL is higher than 300 the suspended solids will increase in the effluent of the settling tank.

Final settling basin side water depth (SWD)

Tank diameter, m	Side water depth, 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

Example 5:-

Design the secondary clarifier (final settling tank) for the CMFR in Example 1.

Solution:-

$(Q_o = 12960 \text{ m}^3/\text{d})$, $(Q_w = 37.8 \text{ m}^3/\text{d})$

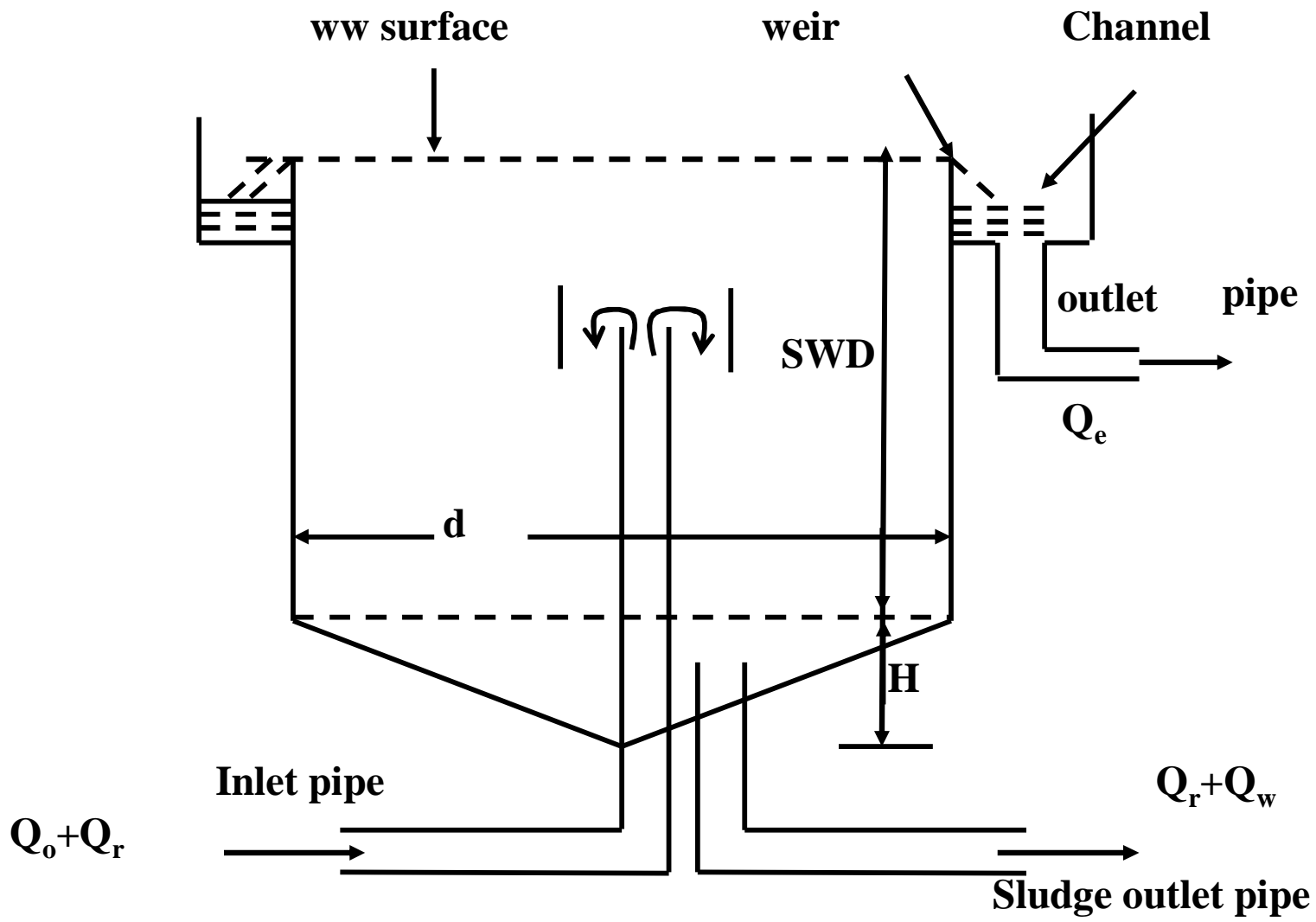**find A$_s$:-** $\quad Take\ O/F = 33\dfrac{m}{d} \quad\quad O/F = \dfrac{Qe}{As} \quad$ **The typical range of O/F is (20-34)** $\dfrac{m^3}{m^2 \bullet d}$

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

(note: Q$_e$ = Q$_o$ − Q$_w$ = 12960 − 37.8 = 12922 m^3/d)

find the diameter:- $\quad\quad\quad\quad A_s = \dfrac{p\,d^2}{4}$
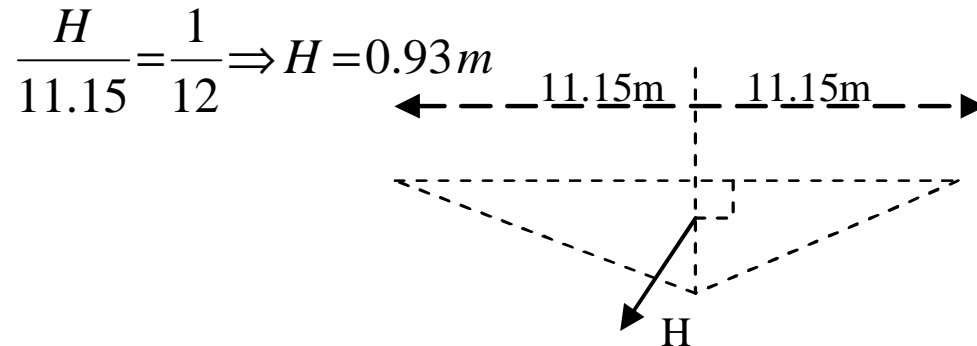
$$d = \sqrt{\frac{4*392}{II}} \quad = 22.3 \text{ m}$$

* 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:-

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

Take H = 1.0 m

* Check for the solids loading rate SL:-

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

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

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

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

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:-

$$SVI = \dfrac{SV}{X} * 1000 \dfrac{ml}{g}$$	SVI = sludge volume index SV = volume of settled solids in one liter graduated cylinder after 30 minutes settling,

1000 ml

1000 ml

30 Minute

SV
Sludge
Volume: ml

43
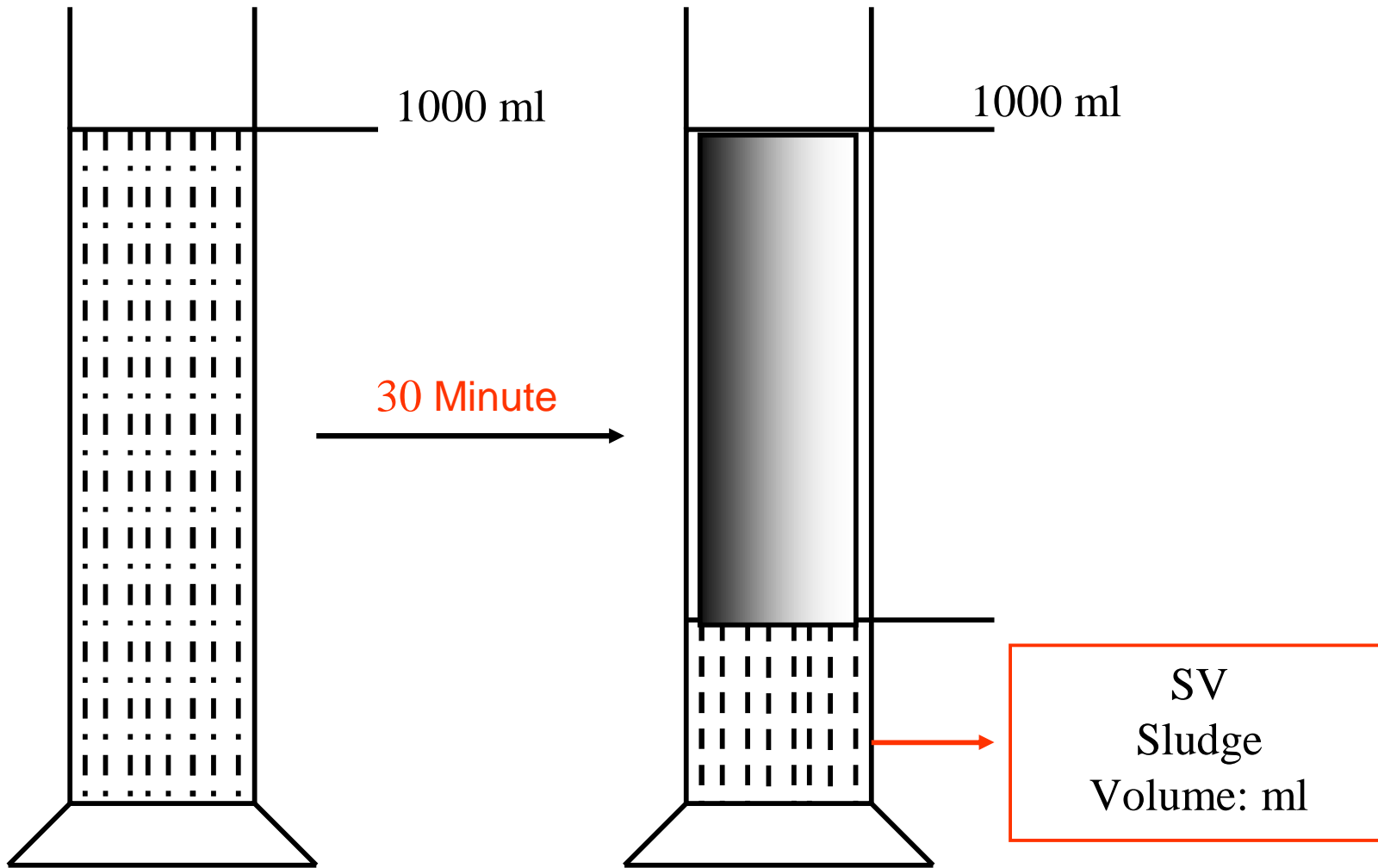
X = biomass concentration in the biological reactor such as (CMFR),

mgSS/L→ (MLSS). SVI is related to X_r by the following relation:-

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

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

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

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

* 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.

Example 6: For example 1 find the SVI, SV.

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

* MLVSS = Mixed Liquor Volatile Suspended Solids

Solution:-

$X_r = 10.000 \ mg \ vss / L \ (or \ 10.000 \ mg \ MLVSS / L) \ from \ example \ 1$

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

$= 12500 \ mg \ MLss$

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

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

* find sludge volume SV:-

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

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

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

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

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

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.
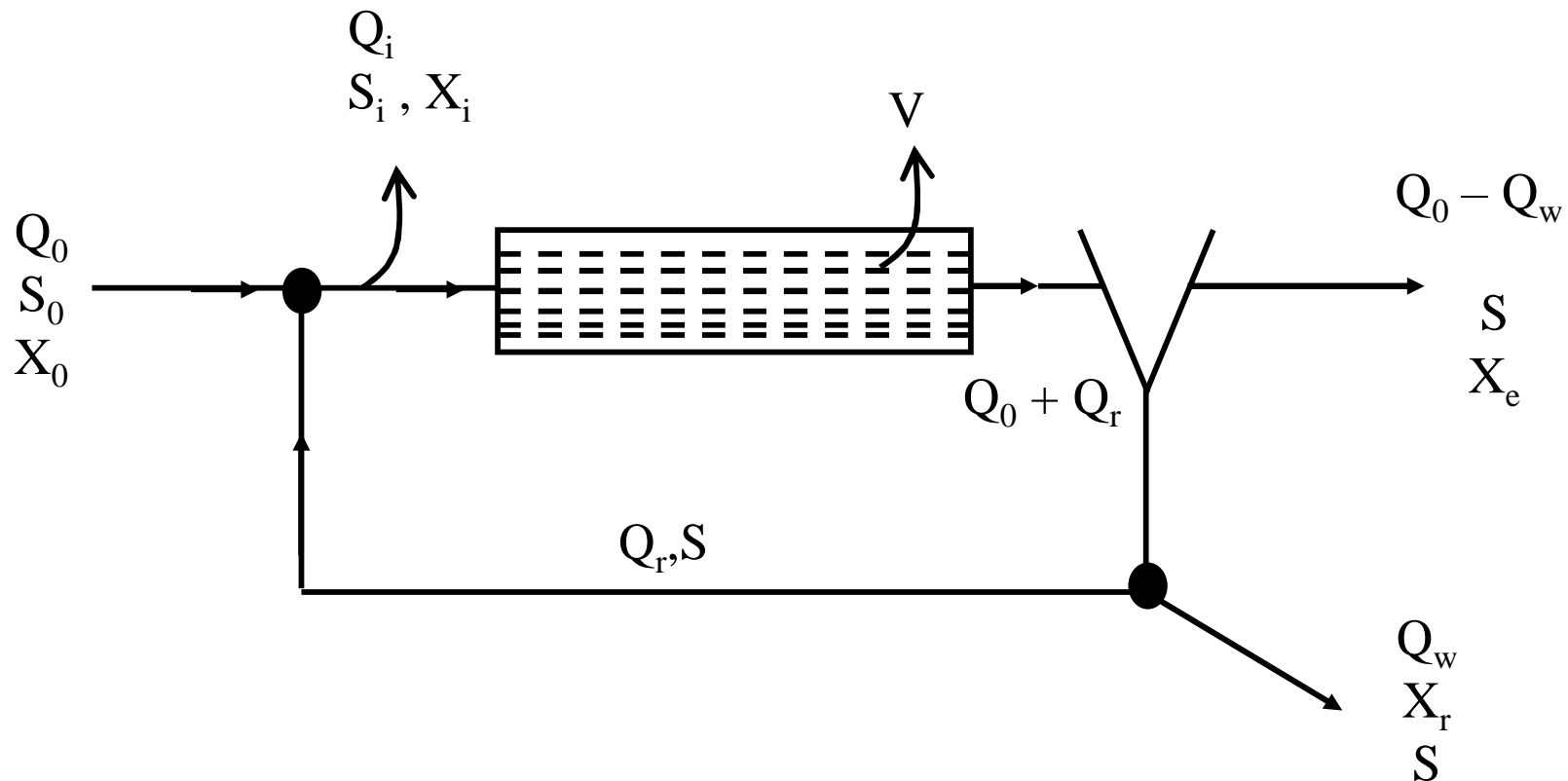
Example 7 Solve example 1 using a PFR.

a. find qc:-

q_c^{min} was calculated as 0.408 d ,   for PFR we find q_c from equation 20:-

$$\frac{1}{q_c} = \frac{m_m(S_o - S)}{(S_o - S) + (1+a)K_s \ln\left[\dfrac{S_i}{s}\right]} - k_d$$

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

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

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

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

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

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

$\theta_c = 1.92$ days

* Check for S.F:- $S.F = \dfrac{q_c}{q_c^m} = \dfrac{1.92}{0.408} = 4.71 > 2 \ O.K$

b. Calculate q from equation 19:-

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

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

* it is typically preferred to have a minimum q of 1.0 hr. To increase q we can either decrease X or increase q_c , or do both things. So, assume $q = 1 \, hr = 0.042 \, d, \ \overline{X} = 3000 \, mg/l$, and solve for q_c .

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

50

$$* \; for \; PFR \frac{q_C}{q} \; Should \; be > 5$$

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

C. Calculate the reactor volume:-

$$V = Qq = 12960 * 0.042 = 544 \, m^3$$

d. Calculate P_x, Q_w, R_o the same as in example 1.

Oxidation ditches:- (OD)

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

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 re-circulated 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.

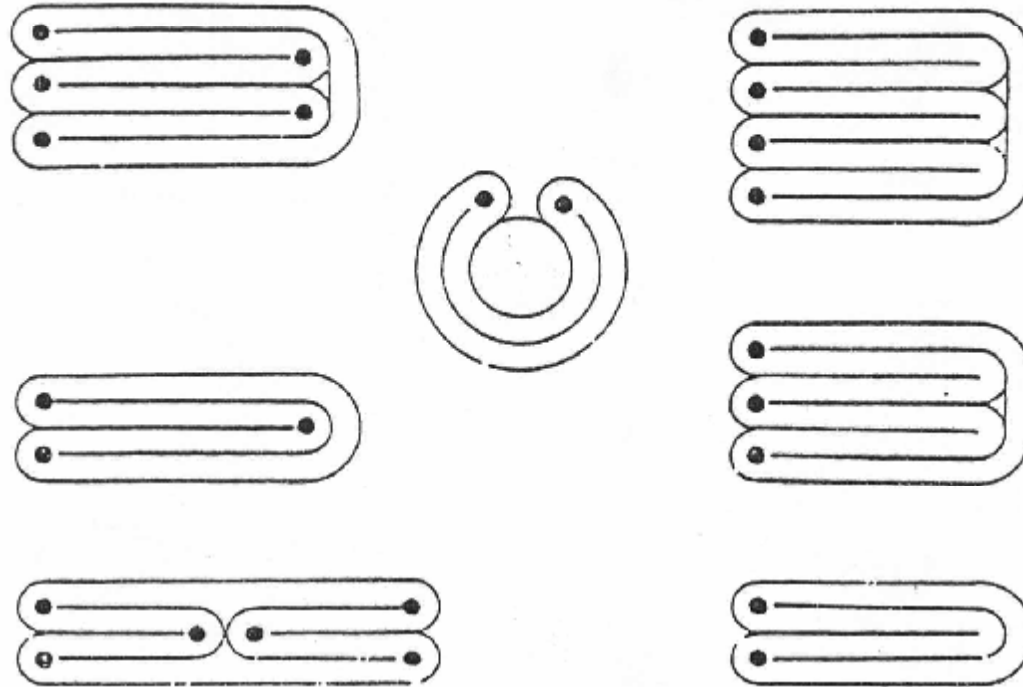The influent of raw sewage is introduced just upstream of the aerator (Rotor).

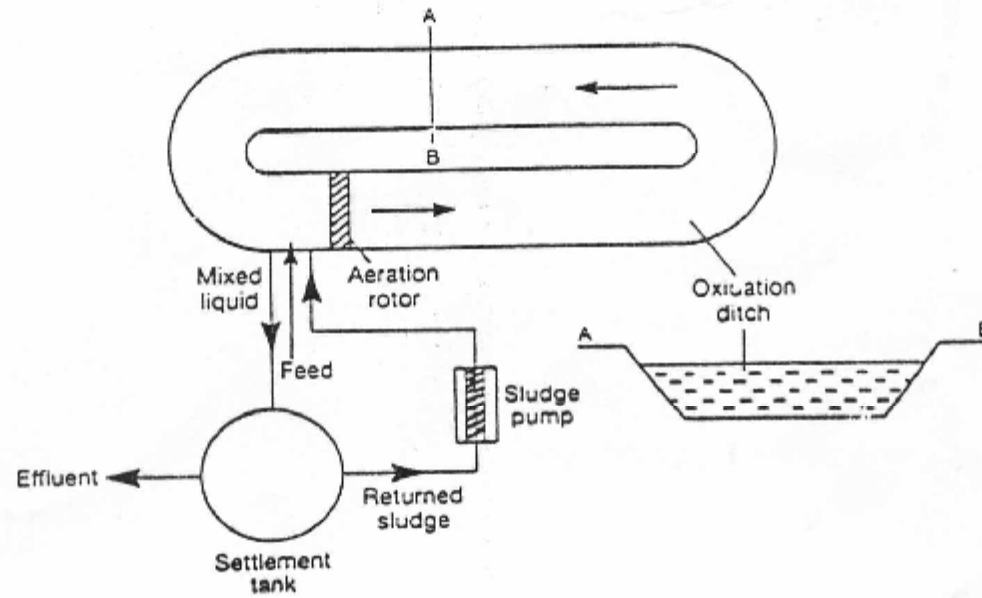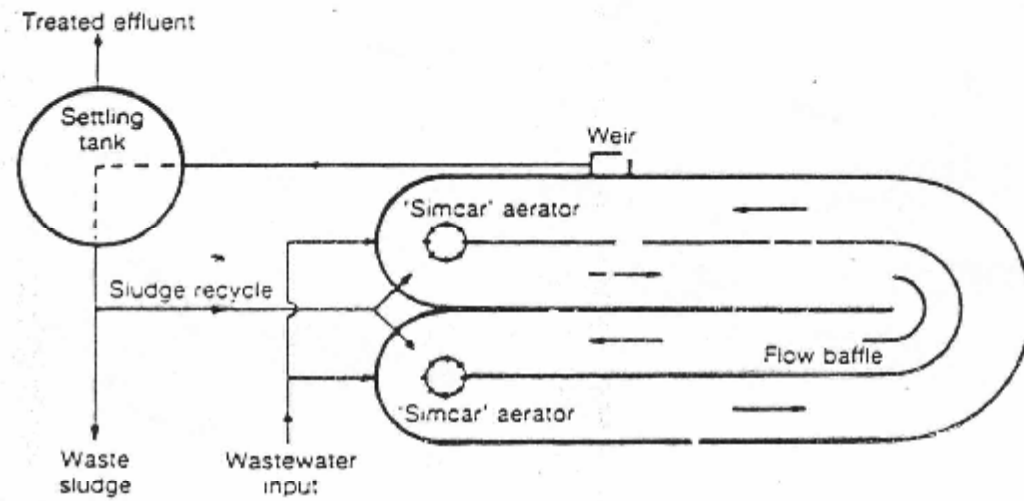The effluent weir is located just upstream of the influent
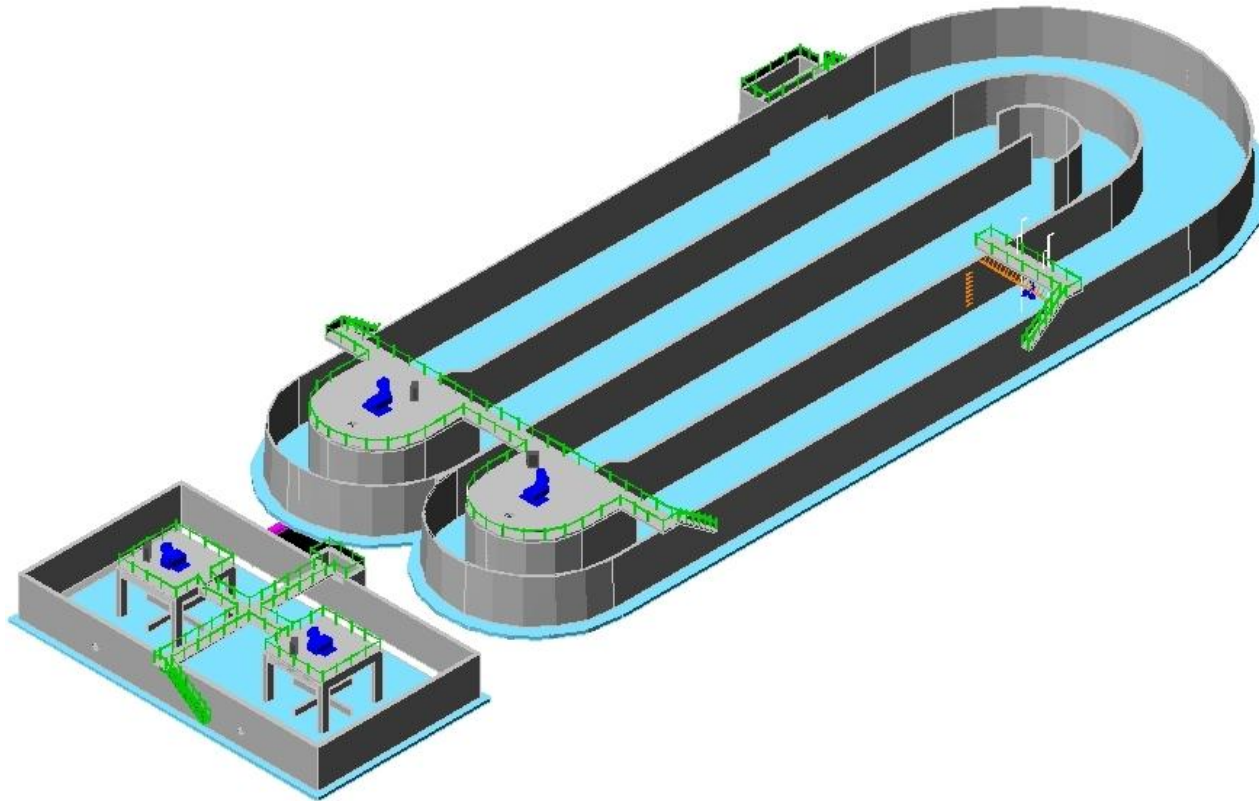
Oxidation Ditch

Carrousel® configurations

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

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 ($0.02 - 0.15 \dfrac{mgBOD_5}{mgvss \bullet d}$). 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 $\Rightarrow Px = YQ (S_o - S) - K_d XV$

Let net production $(P_x) = 0.0$

So $\Rightarrow YQ (S_o - S) = K_d XV$

And $\Rightarrow$ $\quad X = \dfrac{Y}{K_d q}(S_o - S)$ $\quad$ or $\quad XV = \dfrac{Y}{K_d}Q(S_0 - S)$

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

<u>Processes that can be achived in oxidation ditches:-</u>

Three processes can be achieved in oxidation ditches:-
  – Organic matter removal (BOD 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 addition to ammonium) NH_4^+ (coming in the influent in addition to O_2 introduced by the aerator.

- In this condition both BOD removal and nitrification occurs. At the end of the aerobic zone the dissolved oxygen becomes almost zero.

60

influent

effluent

Aerobic

Anoxic

Rotor (aerator)
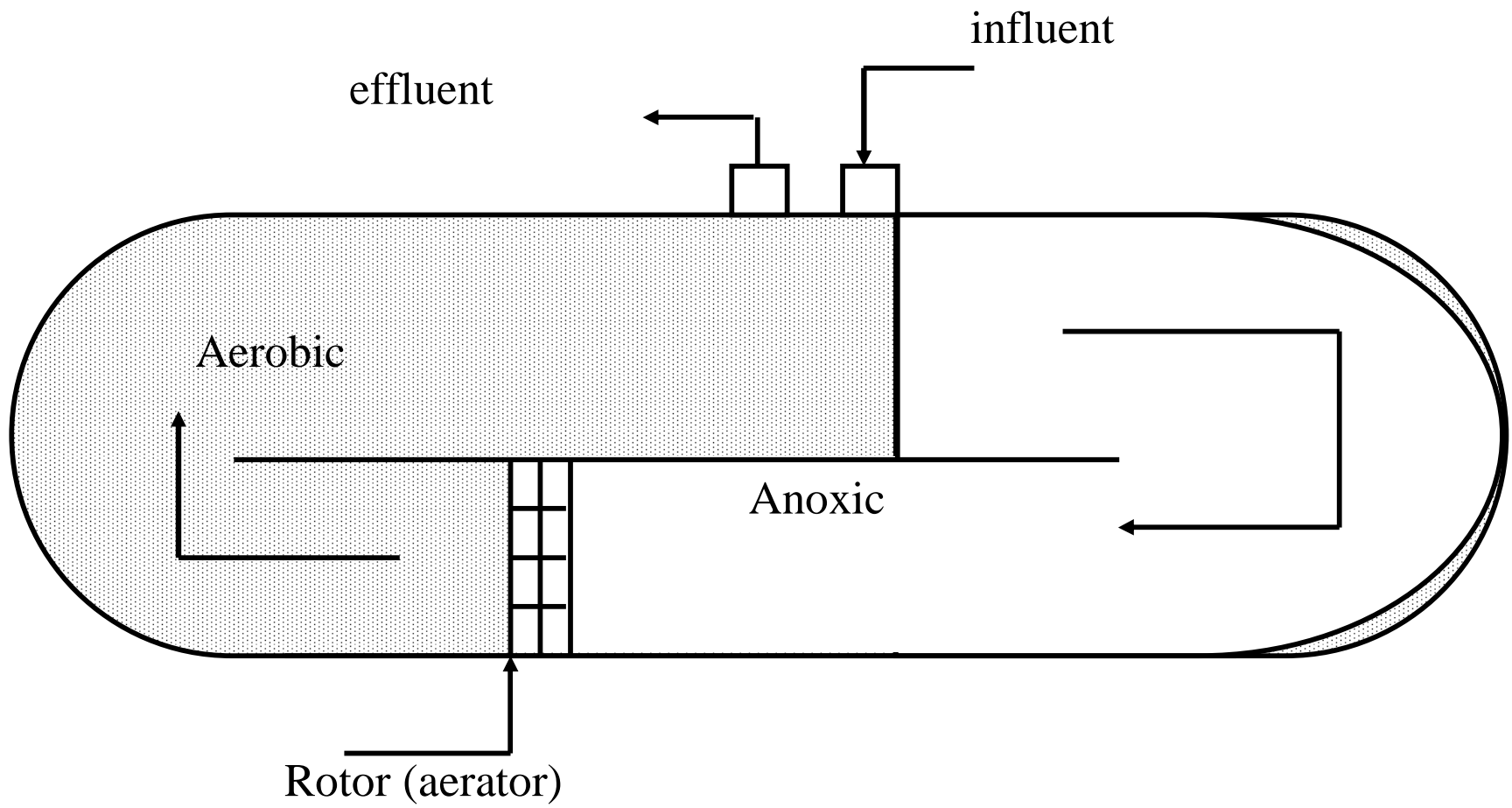
Example 8:-

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

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

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

1- Calculate X to achieve zero net production:

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

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

2- check for $\dfrac{F}{M}$

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

3- Assume q in the typical range (15-36 hrs),

take $q = 1$ day (24 hrs)

$$Xq = 4750\frac{mgvss}{L}.d$$

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

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

5- find the volume of the oxidation ditch

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

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

6- Calculate Q_r:

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

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

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

7- find the oxygen requirements:

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

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

$$R_0 = 5700 kgO_2 / d$$

Example 9:

Repeat the Design in example 8, assuming that we want to allow for some sludge waste, by using a sludge age (q_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 $(q, q_c, X, F/M)$

a- Assume q_c, = 30 days, assume q = 15 hrs (0.625 days).
(note: in oxidation ditches we allow S.F above 20)

b- Calculate S:

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

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

65

c- calculate X:

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

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

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

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

$$\frac{F}{M} = \frac{S_0}{qx} = \frac{300 \, mgBOD_5/L}{0.625 * 3780 \, mgvss/L} = 0.121 \frac{mgBOD_5}{mgvss.d} \quad \text{within the range}$$

$(0.02 - 0.15)$ O.K

e- calculate V:

V=Q q = 20000 * 0.625 = 12500 m³

F- calculate sludge production:

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

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

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

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 q_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

Oxidation Ditch process flow sheet:
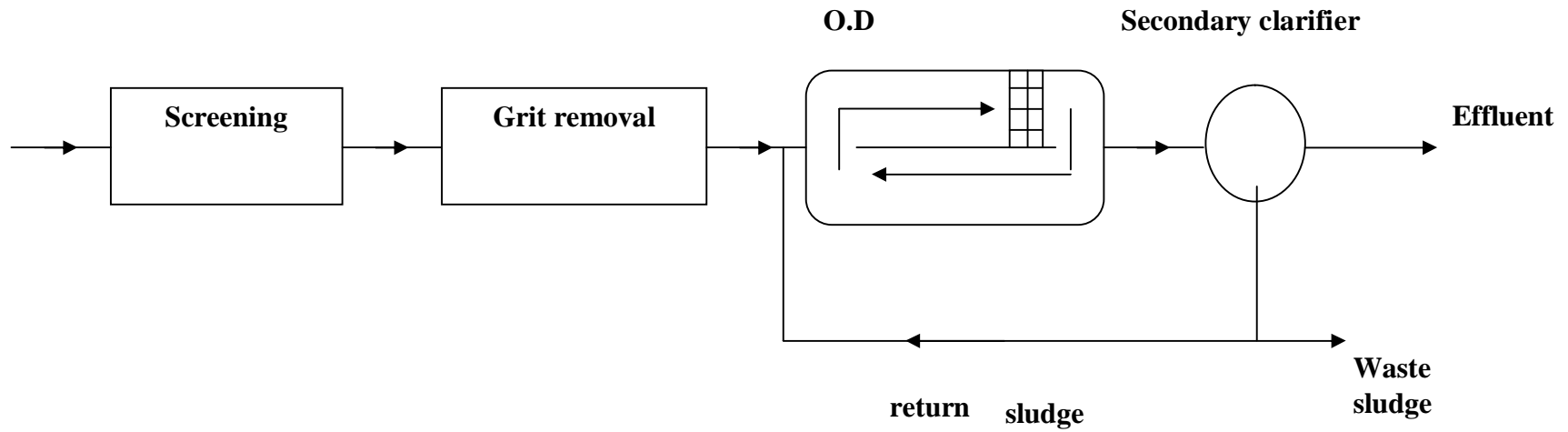
Aerated lagoons (AL):

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

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

Aerated lagoon

Sludge to disposal

Floating mechanical aerator

Influent

Effluent

sludge
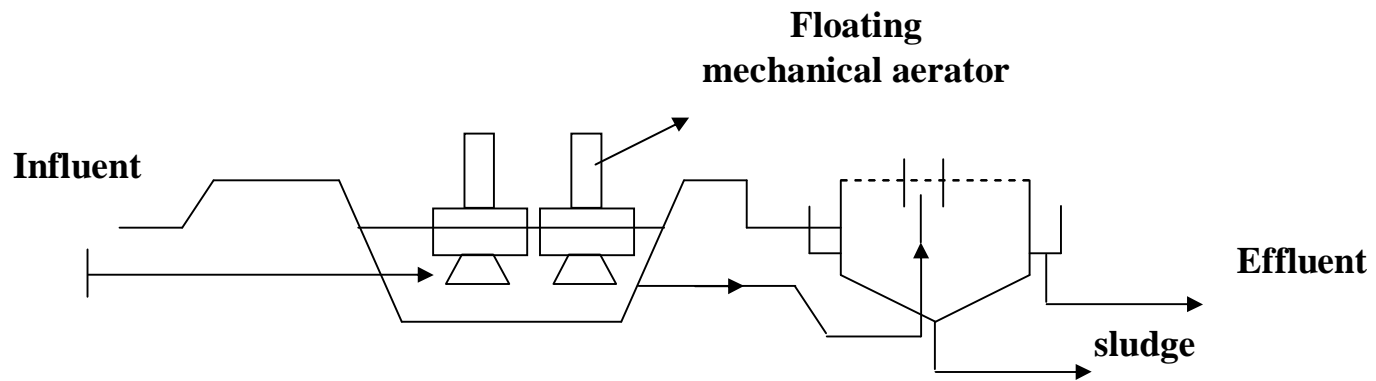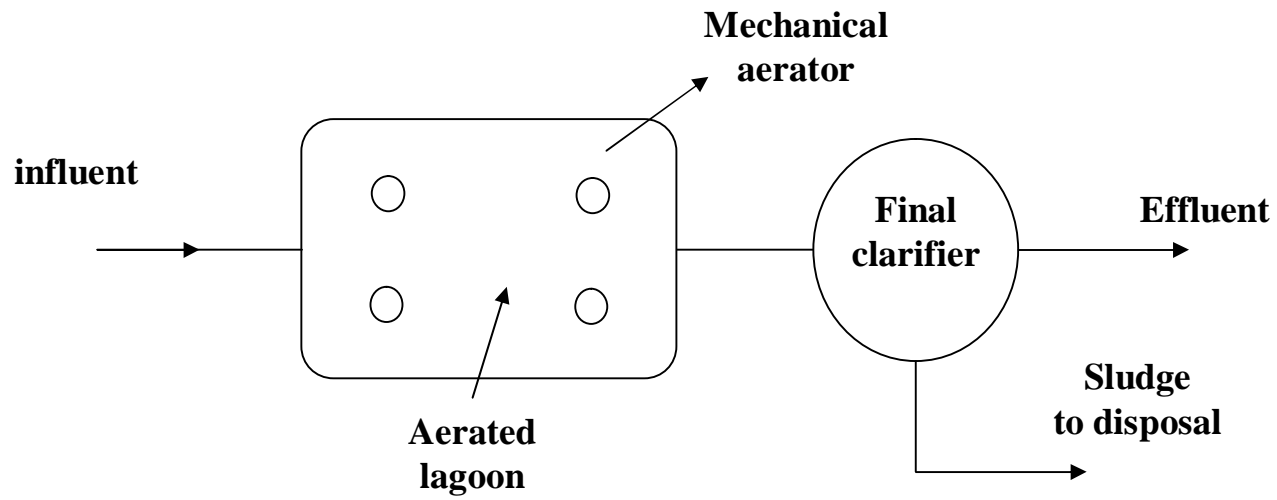
<u>hydraulic model of Aerated lagoons:</u>

Aerated lagoons are designed as completely mixed reactor without solids recycle. The derivations of the equations of such a system are presented in unit 2.
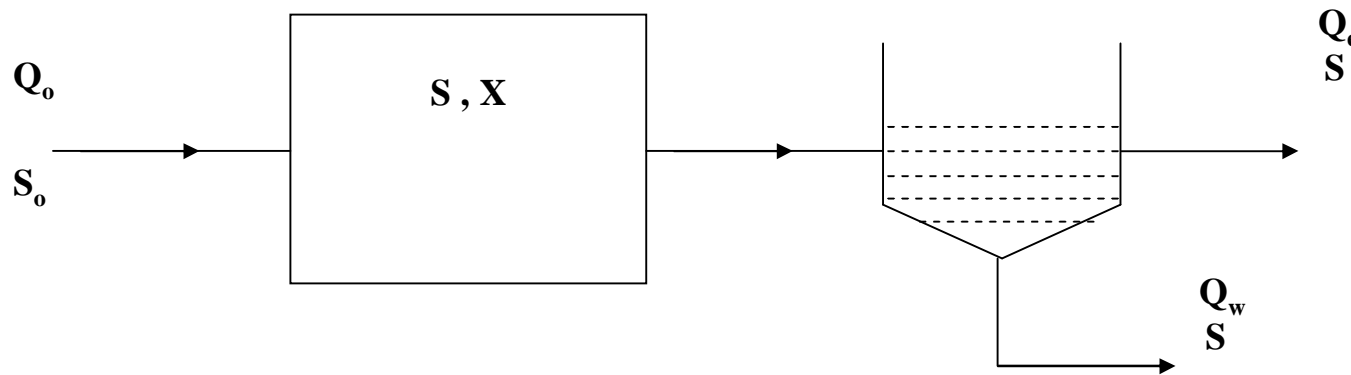
For "AL" the following design equations apply:

$$S = \frac{K_s(1+qK_d)}{q(m_m - K_d)-1}, \quad X = \frac{Y(S_0 - S)}{1+K_d q}$$

these equtions were derived previously (unit-2)

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 q_c is not enough to achieve nitrification (q_c, supply and sludge age and have a typical range of 3-10 days). Note that q AL is equal to in this q is equal to q_c dose not appear in the equations above , but q_c system.

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,

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

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

Example 10:

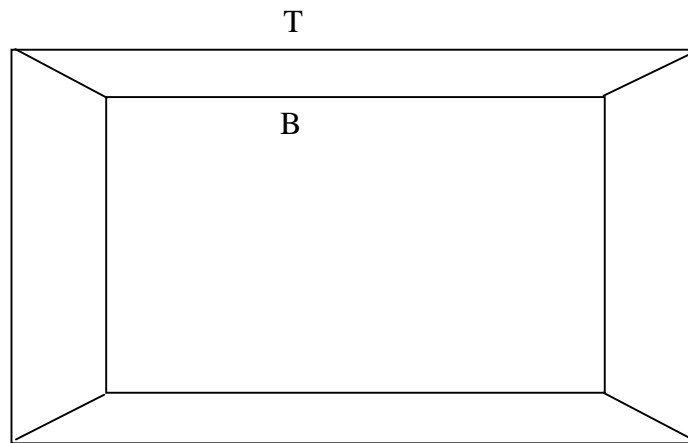Design an aerated lagoon to treat a domestic waste water with a total BOD of 400 mgBOD/L , a TSS of 130 mg/L and a daily flow of 8000 m^3/d. Heterotrophic bacteria growth constants are $m_m = 2.8d^{-1}$, K$_s$ = 60 mgBOD$_s$/L, K$_d$= 0.03d^{-1}, $Y = 0.5 \dfrac{mgvss}{mgBOD_s}$, assume q = 5 days.

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

T

h =2.5m

Slope = 1:3

h

= 3h

B

T

B
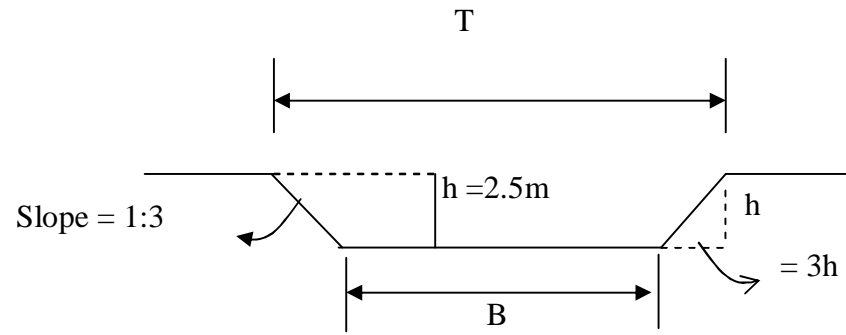
Solution:

1- determine (S_0) soluble:

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

2- calculate S:

$$S = \frac{K_s(1 + qK_d)}{q(m_m - K_d) - 1}$$

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

3. calculate X: typical range 100-400mgvss/L

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

4. Calculate the volume and surface area required:

* $V = Qq$ = 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→ $V = \left[\dfrac{T^2 + B^2}{2}\right] \bullet h$, and $T = B + 6h$

$40000 = \left[\dfrac{(B + 6 \bullet 2.56)^2 + B^2}{2}\right] \bullet 2.5$, solve for B→ B ≈ 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 q in this case: $q = \dfrac{V}{Q} = \dfrac{20000}{8000} = 2.5 \, days$

* calculate S in this case: $S = \dfrac{60(1 + 2.5 \bullet 0.03)}{2.5(2.8 - 0.03) - 1} \cong 10.9 \dfrac{mgBOD_5}{L}$

* calculate X in this case:

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

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

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

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

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

7. Calculate the oxygen requirements:

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

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

7. Calculate the power needed for oxygen transfer

assuming that 1.8 Kg O_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:

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

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

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

So mixing power controls the design.

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 (CMFR) while in the SBR it is interment.

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 Unit 2.

For SBR the following equation is applied:

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

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 an aerobic conditions

2. 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 .

3. 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 $\frac{1}{2} hr \ to \ 1 \ hr \left[\cong 20\% \ T \ Cycle \right]$.

During this step no mixing or aeration is applied.

3. Draw<u>:</u>

- 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 the $\cong 5\%$ Idle time is 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.

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.

Sludge Recycle:

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

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

n= number of SBR tanks used.

t_i = idle step time

Typical cycle time is 4-8hrs.

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

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

- At least two SBR tanks are needed
- No final sedimentation is needed
- No sludge recycle is needed
- If q_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.

<u>Advantages of SBR:</u>

- 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 q_c is >20 we do not need primary sedimentation tank, and the wasted sludge is stable.

(Note: $\dfrac{F}{M}$ ratio is similar to that of oxidation ditches, i.e. 0.02-

0.15 $mgBOD_5 / mgvss.d$)

Example 11:

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

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

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

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

$$Y = 0.5\dfrac{mgvss}{mgBOD}$$

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

$$K_s = 50\dfrac{mgBOD}{L}$$

$$m_m = 2.5d^{-1}$$

- Assume:

$$t_s = 0.5 \text{hrs} \quad \text{(typical)}$$
$$t_D = 0.5 \text{hs} \quad \text{(typical)}$$
$$t_r = 1.0 \text{hrs} \quad \text{(typically 1-2hrs)} \rightarrow \text{Should be checked by the}$$

batch reactor design equation.

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

$$\text{Tcycle} = t_f + t_r + t_s + t_D \qquad (t_i = 0.0, \text{ not needed})$$

$$t_f = \frac{t_r + t_s + t_D}{n-1} \quad \text{assume n =2 SBR tanks, if the dimension of SBR}$$

are within $5 \leq L \leq 30$ o.k., other wise more than 2 tanks are needed.

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

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

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

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

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

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

4. <u>Determine $\dfrac{V_F}{V_T}$ 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

$$V_F = V_T - 0.35 V_T = 0.65 V_T \qquad \text{wastewater disposed in each cycle.}$$

Since : $V_F = 625 \text{m}^3$

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

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

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

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

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

so two tanks are o.k $\quad 5 \leq 13.90 \leq 30m$

6. Determine the portion of heterotrophic and nitrifiers microorganisms and q_c :

X= X$_{nitrifiers}$ + X$_{hetrotrophs}$ = X$_n$+X$_h$

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

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

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

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

$q_c \cong 18.5 \; days$

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

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

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

93

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

Use the batch reactor design equation:

For nitrification:

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

N_o = 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).

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

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

94

- Check for BOD removal:

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

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

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

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

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

*   Note:

t_r for nitrification always control the design of SBR.

8. <u>Calculate sludge procedure:</u>

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

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

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

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

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

10. Check

$$\frac{F}{M} = \frac{QS_o}{Vx} = \frac{3750 \bullet 10^3 \bullet 150}{2800 \bullet 962 \bullet 10^3} = 0.21 \frac{mgBOD_5}{mgVss \bullet d}$$

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