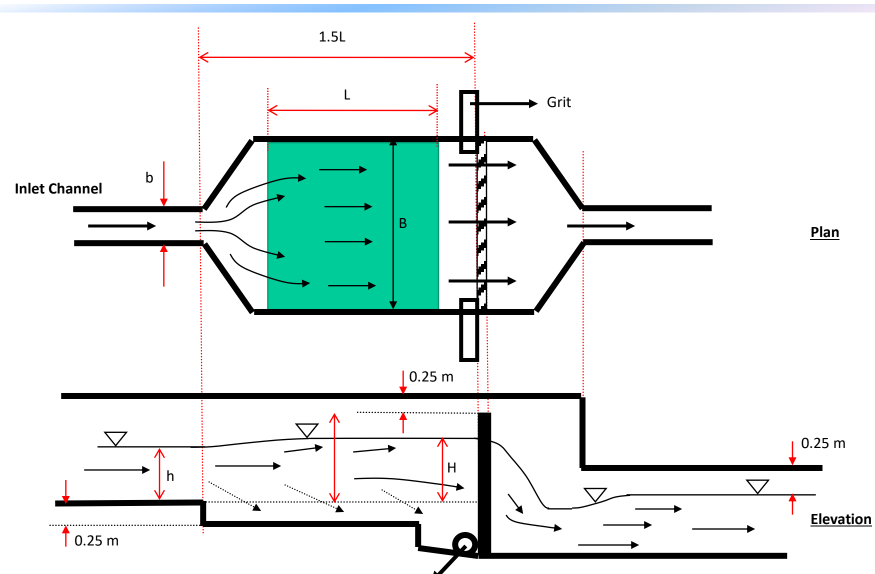
CE412 A
Water Supply & Wastewater Disposal Systems

Part II_Wastewater_Illustration

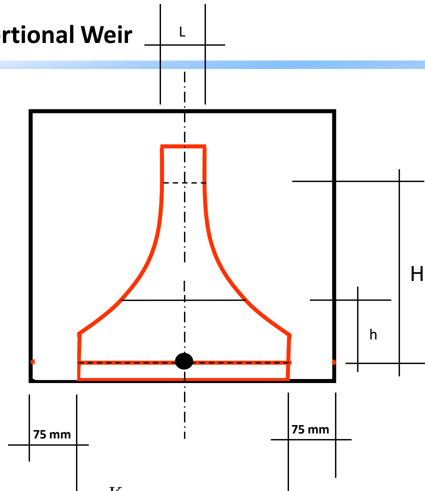
Instructor :
Dr Vinod Tare

Grit Chamber: To remove inorganic particles up to 0.25 mm in diameter

V_{sc} (0.25 mm particles) = 0.254 ms^{-1}; Thus, $V_H < V_{sc}$



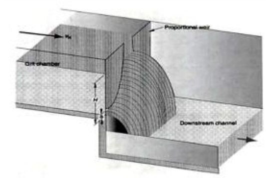
Proportional Weir



This for any h, $l = \frac{K}{h^{\frac{1}{2}}}$.

Thus the shape of the sides of the weir can be determined.

Flow through unit height of the weir is constant. Hence, velocity upstream of the weir (v_H) is constant irrespective of the flow.



$Q = 1.57.C_d.\sqrt{2g}.l.h^{\frac{3}{2}}$; $C_d = 0.6 - 0.9$

Assuming $C_d = 0.6$,

$Q = (4.173).[l.h^{\frac{1}{2}}].h$; $\frac{Q}{h} = (4.173).[l.h^{\frac{1}{2}}]$

The objective is to make $\frac{Q}{h}$ constant

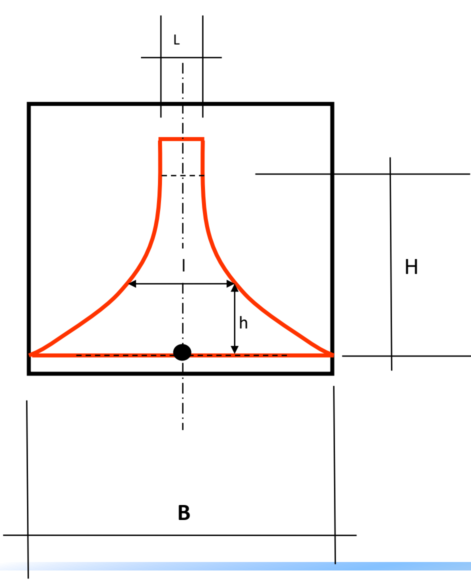
Thus, $[l.h^{\frac{1}{2}}]$ must be constant

If, Q and H are known,

$L = \frac{Q}{4.173.[H^{\frac{3}{2}}]}$;

Thus, $[L.H^{\frac{1}{2}}] = K$ can be calculated

Proportional Weir



Example Problem

Design a proportional weir to be fitted at the downstream end of rectangular grit chamber. The grit chamber is designed for a peak flow of 0.30 $m^{3/}s$ (Q).

Solution :

V_{sc} for 0.25 mm grit particles = 0.254 m/s
Let V_H (< V_{sc}) be 0.25 m/s
Let depth of flow (H) corresponding to Q be 0.5 m
Width of the grit chamber (B) = Q/(v_H.H) = 2.4 m

Flow across a proportional weir is given by the formula,

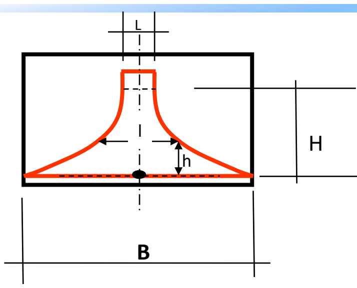
$$Q = (4.173).[L.H^{\frac{1}{2}}].H;$$

$$L = \frac{Q}{4.173.[H^{\frac{3}{2}}]} = 0.203 \text{ m}$$

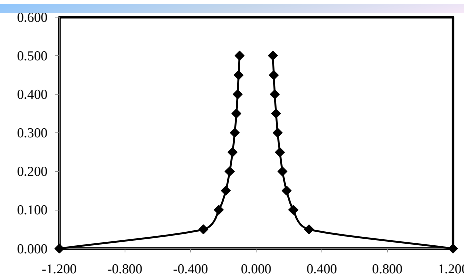
$$K = [LH^{\frac{1}{2}}] = 0.144$$

$$\text{Thus, } [l.h^{\frac{1}{2}}] = 0.144$$

Design Calculations:



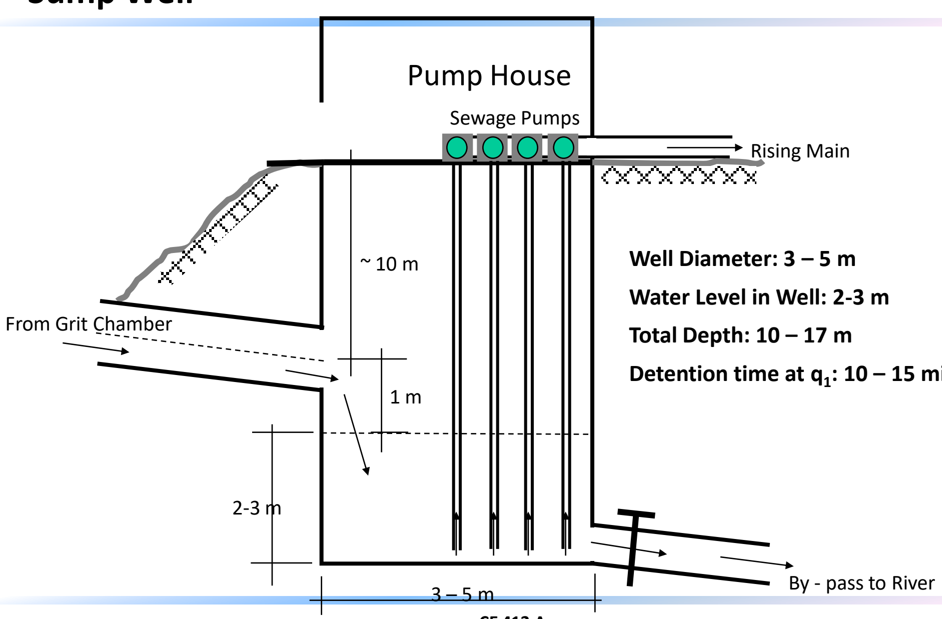
Weir Profile



h	$l=K/(h^{0.5})$	q	$B = Q/(v_H.H)$	$v_H = q/(B.h)$
0.050	0.643	0.030	2.400	0.25
0.100	0.455	0.060	2.400	0.25
0.150	0.371	0.090	2.400	0.25
0.200	0.322	0.120	2.400	0.25
0.250	0.288	0.150	2.400	0.25
0.300	0.263	0.180	2.400	0.25
0.350	0.243	0.210	2.400	0.25
0.400	0.227	0.240	2.400	0.25
0.450	0.214	0.270	2.400	0.25
0.500	0.203	0.300	2.400	0.25

H = 0.500
L = 0.203
K = 0.144

Sump Well



Well Diameter: 3 – 5 m

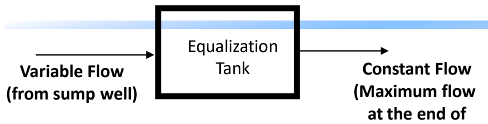
Water Level in Well: 2-3 m

Total Depth: 10 – 17 m

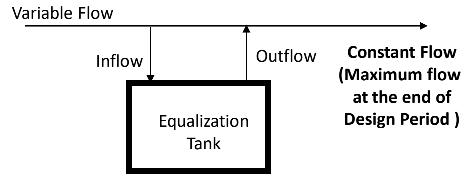
Detention time at q_1: 10 – 15 min

Equalization Tank (Flow Equalization)

Q = Maximum Flow, m^3/h ;
n = duration, hr

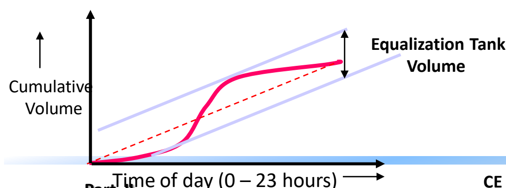


In- Line Equalization



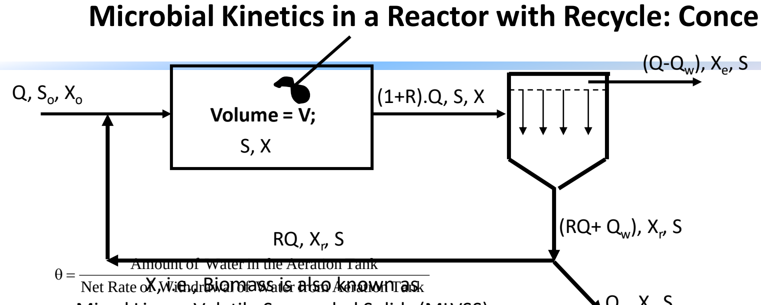
Off - Line Equalization

Determination of Equalization Tank Volume



Time of Day	Flow	Cumulative Volume
0-1	0.9Q	0.9Q..n
1-2	0.7Q	1.6Q.n
2-3	0.5Q	2.1Q.n
3-4	0.4Q	2.5Q.n
4-5	0.3Q	2.8Q.n
5-6	0.3Q	3.1Q.n
6-7	0.4Q	3.5Q.n
7-8	0.7Q	4.2Q.n
8-9	1.1Q	5.3Q.n
9-10	1.6Q	6.7Q.n
10-11	1.4Q	8.1Q.n
11-12	1.4Q	9.5Q.n
12-13	1.3Q	10.9Q.n
13-14	1.2Q	12.2Q.n
14-15	1.2Q	13.4Q.n
15-16	1.1Q	14.5Q.n
16-17	1.1Q	15.6Q.n
17-18	1.1Q	16.7Q.n
18-19	1.1Q	17.8Q.n
19-20	1.2Q	19.0Q.n
20-21	1.3Q	20.3Q.n
21-22	1.3Q	21.6Q.n
22-23	1.2Q	22.8Q.n
23-24	1.2Q	24Q.n
Total	24Q	

Microbial Kinetics in a Reactor with Recycle: Concept of q_c



$$\theta = \frac{\text{Amount of Water in the Aeration Tank}}{\text{Net Rate of Withdrawal of Water from Aeration Tank}}$$

X, i.e., Biomass is also known as Mixed Liquor Volatile Suspended Solids (MLVSS)

Sludge Wasted (DX) = $Q_w.X_r$

$$= \frac{V}{[(1+R)Q - RQ]} = \frac{V}{Q}$$

Hydraulic Retention Time (q):

$$\theta_c = \frac{\text{Amount of Biomass in the Aeration Tank}}{\text{Net Rate of Withdrawal of Biomass from Aeration Tank}} = \frac{X.V}{(1+R).Q.X - R.Q.X_r}$$

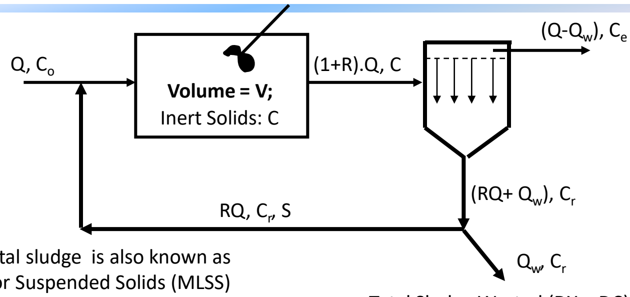
Biological Solids Retention Time (q_c):

$$\theta_c = \frac{X.V}{Q_w.X_r} = \frac{1}{\mu}$$

Please Note,

$$(1+R).Q.X = R.Q.X_r + Q_w.X_r$$

Inert Solids in a Reactor with Recycle: Fate of Inert Solids



X + C, i.e., total sludge is also known as Mixed Liquor Suspended Solids (MLSS)

Inert Solids Balance for the whole system:

$$Q.C_o - [(Q - Q_w).C_e + Q_w.C_r] = 0$$

$$Q.C_o = Q_w.C_r \qquad \text{(A)}$$

Retention Time of Inert Solids:

$$\theta_c = \frac{C.V}{Q_w.C_r}; \quad Q_W.C_r = \frac{C.V}{\theta_c} \qquad \text{(B)}$$

Total Sludge Wasted (DX + DC) = $Q_w.(X_r + C_r)$

$$Q.C_o.\theta_c = C.V; \quad C_o.\theta_c = C.\theta$$

$$C = C_o.\left(\frac{\theta_c}{\theta}\right) \qquad \text{(C)}$$

Design of Activated Sludge Process: Part I

- Given Q, assume q (4-12 hrs) and calculate volume of aeration tank (V)
- Given S and microbial kinetic constants (Y_T, k_d, K_s and q_{max}), calculate q
- Calculate m and hence q_c
- Given S_o, calculate biomass in aeration tank (X) and sludge wasting rate, DX = $Q_w.X_r$
- Given C_o, calculate inert solids concentration in aeration tank (C) and hence calculate (C+X)
- Given (C_r+X_r), calculate C_r and hence X_r. Then using DX and X_r values calculate Q_w
- Calculate D(C+X) and R
- Calculate solids input to the clarifier and also solids output from the clarifier and verify that these values are identical
- Assume a value for the underflow velocity (u) [~0.1 – 0.4 m/hr] for the clarifier and hence calculate A_s
- Calculate the limiting solids flux (SF_L) for the chosen u and hence calculate A_{req}. Verify that A_s > A_{Req}
- If not, reduce (C_r + X_r) and recalculate from step 6 onwards.

Design of Activated Sludge Process: Part I

Given: Q = 50 MLD; S_o = 350 mg/L; q = 6 hours

Aeration Tank Volume = Q.q = 12500 m^3; No. of Tanks Provided = 5
Volume of each tank = 12500/5 = 2500 m^3; Surface Area = 1250 m^3
Length = 40 m; B = 31.25 m; D = 2 m

Given: S = 4 mg/L; K_s = 25 mg/L; q_{max} = 4 /d; Y_T = 0.50; k_d = 0.05 /d

$$q = \frac{q_{max}.S}{K_s + S} = \frac{(4).(4)}{(25+4)} = 0.55\ /d \qquad \mu = Y_T.q - k_d = (0.5)(0.55) - 0.05 = 0.225\ /d$$

q_c = 1/m = 4.43 d;

For, S_o = 350 mg/L,

Given: C_o = 50 mg/L

$$X = \frac{(S_o - S)}{\theta.q} = \frac{(350-4)}{(6/24).(0.55)} = 2508\ mg/L$$

(C + X) = 3394 mg/L

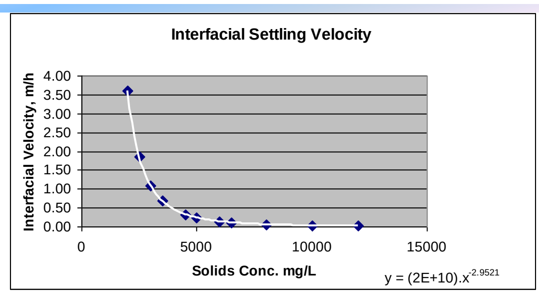
$$Q_w.X_r = \Delta X = \frac{X.V}{\theta_c} = \frac{2508.(12500)}{(4.43).(1000)} = 7082\ kg/d$$

$$C = C_o.\left(\frac{\theta_c}{\theta}\right) = 50.\frac{(4.43)}{(6/24)} = 885\ mg/L$$

Type III / Hindered / Zone Settling

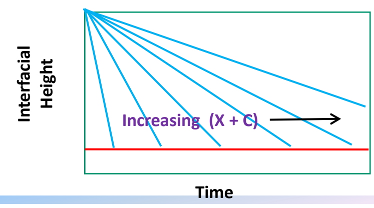
Interfacial Velocities of Sludge Solids

Solids (C+X) mg/L	V, m/hr
2000	*3.60*
2500	*1.86*
3000	*1.09*
3500	*0.69*
4500	*0.33*
5000	*0.24*
6000	*0.14*
6500	*0.11*
8000	*0.06*
10000	*0.03*
12000	*0.02*



Empirical Formula:

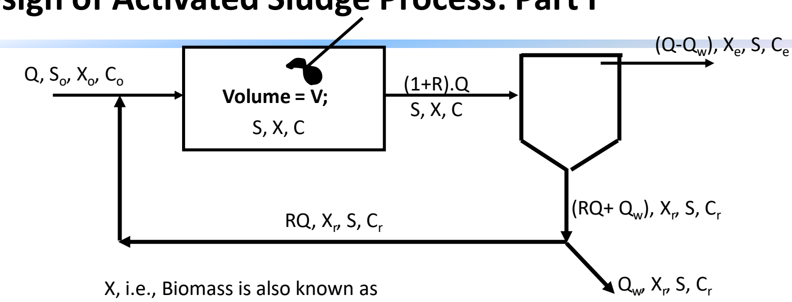
Interfacial settling velocity V (in m/hr) = $[2 \times 10^{10}]$. $[(X + C) \text{ in mg/L}]^{-2.9521}$



Measurement of Sludge Settling Characteristics

http://www.youtube.com/watch?v=GTLKoGGo0dM&feature=player_detailpage

Design of Activated Sludge Process: Part I



X, i.e., Biomass is also known as
Mixed Liquor Volatile Suspended Solids (MLVSS)

X + C, i.e., Sludge is also known as
Mixed Liquor Suspended Solids (MLSS)

Biomass Wasted (DX) = $Q_w.X_r$
Sludge Wasted (DX + DC) = $Q_w.(X_r + C_r)$

$$\mu = Y_T.q - k_d \qquad q = \frac{q_{max}.S}{K_s + S} \qquad Q.(S_o - S)/V = q.X$$

$$m.X.V = Q_w.X_r \qquad m.V = Q.[1 + R - R(X_r/X)] \qquad \theta = \frac{V}{Q}$$

$$\theta_c = \frac{X.V}{Q_w.X_r} = \frac{1}{\mu} \qquad C = C_o.\left(\frac{\theta_c}{\theta}\right) \qquad \frac{C}{(C+X)} = \frac{C_r}{(C_r + X_r)}$$

Design of Activated Sludge Process: Part I

Given: **Q = 50 MLD;** **S_o = 350 mg/L;** **q = 6 hours**
Aeration Tank Volume = Q.q = 12500 m^3; No. of Tanks Provided = 5
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Length = 40 m; B = 31.25 m; D = 2 m

Given: **S = 4 mg/L; K_s = 25 mg/L; q_{max} = 4 /d; Y_T = 0.50; k_d = 0.05 /d**

$$q = \frac{q_{max}.S}{K_s + S} = \frac{(4).(4)}{(25+4)} = 0.55\ /d \qquad \mu = Y_T.q - k_d = (0.5)(0.55) - 0.05 = 0.225\ /d$$

q_c = 1/m = 4.43 d;

For, S_o = 350 mg/L,

$$X = \frac{(S_o - S)}{\theta.q} = \frac{(350-4)}{(6/24).(0.55)} = 2508\ mg/L$$

Given: **C_o = 50 mg/L**

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(C + X) = 3394 mg/L

$$C = C_o.\left(\frac{\theta_c}{\theta}\right) = 50.\frac{(4.43)}{(6/24)} = 885\,mg/L$$

Given: $(C_r + X_r)$ = 12500 mg/L

$$\text{Assuming, } \frac{C}{(C+X)} = \frac{C_r}{(C_r + X_r)} \qquad C_r = \frac{C.(C_r + X_r)}{(C+X)} = \frac{885.(12500)}{3394} = 3261\ mg/L$$

X_r = 12500 – 3261 = 9239 mg/L;

$$Q_w = \frac{\Delta X}{X_r} = \frac{7082}{9239} = 0.766\ MLD$$

$$\mu.\theta = (1 + R - R.\frac{X_r}{X}) \qquad R = \frac{(1-\mu.\theta)}{(X_r/X - 1)} = \frac{[1-(0.219).(6/24)]}{[(9239/2508)-1]} = 0.352$$

$$\Delta(C+X) = Q_w.C_r + \Delta X = (0.766).(3261) + 7082 = 9582\ Kg/d$$

Solids Input to the Clarifier = Q(1+R).(X+C) = 9557 Kg/hr

(RQ + Q_w) = (0.352).(50) + 0.766 = 18.35 MLD

Solids Output from the Clarifier = (RQ + Q_w).(X_r + C_r) = (18.35).(1000/24).12500/1000) = 9557 Kg/hr

Settleability of Activated Sludge Solids

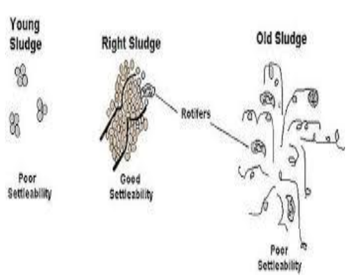
One of the factors essential to the performance of the activated sludge process is effective flocculation of the sludge, with subsequent rapid settling and compaction. Two types of bacteria are found in ASP sludge: 1) floc-forming, and 2) filamentous bacteria

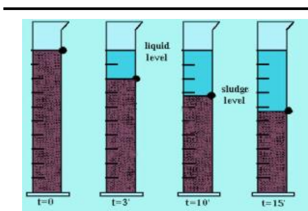
Normal flocs: A balance between floc-forming and filamentous bacteria results in strong flocs that keep their integrity in the aeration basin and settle well in the sedimentation tank (High BSRT).

Pin-point flocs: In these flocs, filamentous bacteria are absent or occur in low numbers. This results insmall flocs that do not settle well (Low BSRT).

Filamentous bulking: Filamentous bulking is caused by the predominance of filamentous organisms. The filaments interfere with sludge compaction. (very high BSRT)

Sludge Volume Index (SVI):





Sludge settleability is determined by measuring the sludge volume index (SVI), which is given by:

$$SVI\ (mL/g) = \frac{SV \times 1000}{MLSS}$$

where SV = volume of settled sludge after 30 min (mL/L);
and MLSS = mixed liquor suspended solids (mg/L).

Low SVI (< 100) means good sludge settleability
High SVI (> 150) means bad settleability

Type III / Hindered / Zone Settling

Interfacial Velocities of Sludge Solids

Solids (C+X) mg/L	V, m/hr
2000	*3.60*
2500	*1.86*
3000	*1.09*
3500	*0.69*
4500	*0.33*
5000	*0.24*
6000	*0.14*
6500	*0.11*
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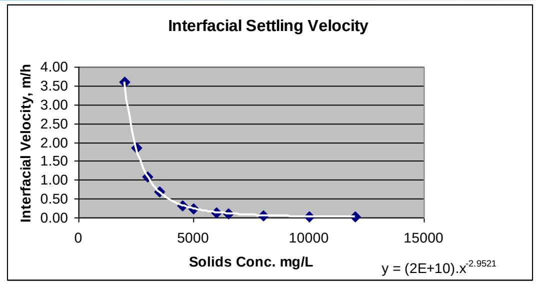
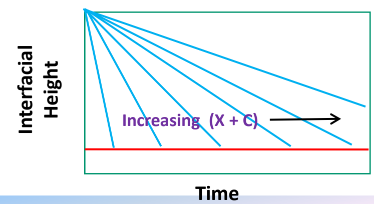


Empirical Formula:

Interfacial settling velocity V (in m/hr) = $[2\times10^{10}].[(X + C) \text{ in mg/L}]^{-2.9521}$



Measurement of Sludge Settling Characteristics

http://www.youtube.com/watch?v=GTLKoGGo0dM&feature=player_detailpage

Concept of Solid Flux

Solid Flux (SF) = Mass of solids passing through unit surface area of the clarifier in unit time kg/m²/hr

At any depth ,

Gravity Flux (SF_G) = $(C_i + X_i).v_i$

v_i is the interfacial velocity

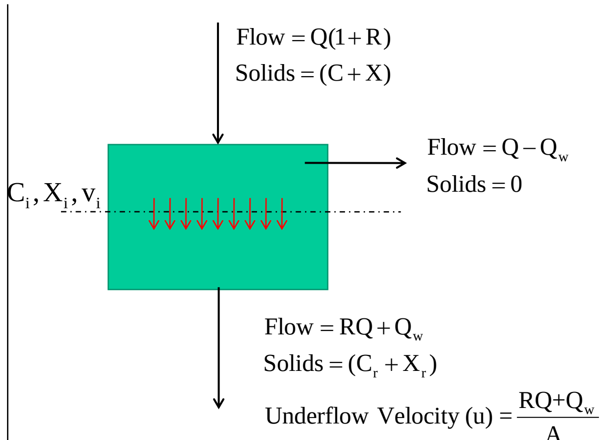
Underflow Flux (SF_u)= $(C_i + X_i).u$

Total Flux (SF_T) = $SF_G + SF_u$

Under steady state conditions,

SF_T is same at all depths

$(C_i + X_i)$ increases with depth



Single Stage Nitrification

Nitrification along with the ASP process itself

- High q_c system; extended aeration
- Oxygen provision must be made for nitrifying microorganisms
- BOD_5 is removed and residual ammonia is converted to nitrate in the aeration tank

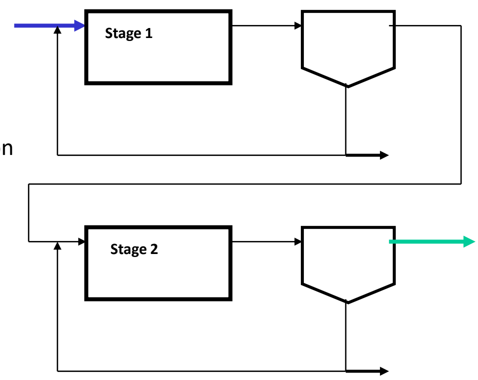
Two Stage Nitrification

Stage 1:

Low q_c
Only BOD removal and no nitrification

Stage 2:

High q_c
Removal of residual BOD
Nitrification



Example Problem: Single Stage Nitrification

Q = 10 MLD; S_o = 300 mg/L; $[TKN]_o$ = 20 mg/L (as N).

Require nitrified effluent.

K_s = 40 mg/L	q_m = 4 /d	Y_T = 0.50	k_d = 0.05 /d
$(K_s)_N$ = 2.0 mg/L	$(q_m)_N$ = 2 /d	$(Y_T)_N$ = 0.20	$(k_d)_N$ = 0.05 /d

Carbon Oxidation Calculations:

q = 8 hours; V = 3333 m^3
(q_c) = 10 days; m = $1/q_c$ = 0.1 /d; q = 0.3 /d; S = 3.24 mg/L
X = $(S_o – S)/(q.q)$ = 2968 mg/L; DX = m.X.V = 990 kg/d
Oxygen Requirement = $1.5Q(S_o – S) – 1.42(DX)$ = 3046 kg/d
Nitrogen Requirement = (14/113).(DX) = 123 kg/d
Influent N = 200 kg/d; Available for nitrification = 200 – 123 = 77 kg/d = 7.70 mg/L (as N)

Nitrification Calculations:

$(m)_N$ = 0.10 /d; $(q)_N$ = .75 / d [TKN] = 1.2 mg/L

X_N = ([TKN]$_i$ – [TKN])/(q.q_N) =(7.70 – 1.20)/(8/24)/0.75 = 57 mg/L
(neglecting N incorporation into X_N)

Ratio of microorganisms = (m_N.X_N)/(m_N.X_N + m.X) = 0.009
(this should be ~ 1 percent)

DX_N = m_N.X_N.V = 19 kg/d

$$NH_4^+ + 2.O_2 \rightarrow NO_3^- + H_2O + 2H^+$$

<u>Stoichiometry:</u>

Additional O_2 Reqd = 4.57.Q.{[TKN]$_i$ – [TKN]} – 1.42.(DX_N) = 270 kg/d
Total O_2 Requirement = 3046 + 270 = 3316 kg/d

SOT = 2.0 kg O_2/h/KW; Find AOT at 30^{o}C

$$[O_2^F]_l^{s_{20}} = 9.1\,\text{mg/L} \qquad [O_2^F]_l^{s_{30}} = 7.5\,\text{mg/L} \qquad [O_2]_l = 3.0\,\text{mg/L}$$

$$AOT = (K_L.a)_{20}^F.\left([0.8].[1.02]^{30-20}\right)\{[0.9].[7.5] - 3\}.(V)$$

$$AOT = (K_L.a)_{20}^F.(V).(5.607) = \frac{2}{9.1}.(3.657) = 0.81\ \text{kg}\,O_2/\text{h}/\text{KW}$$

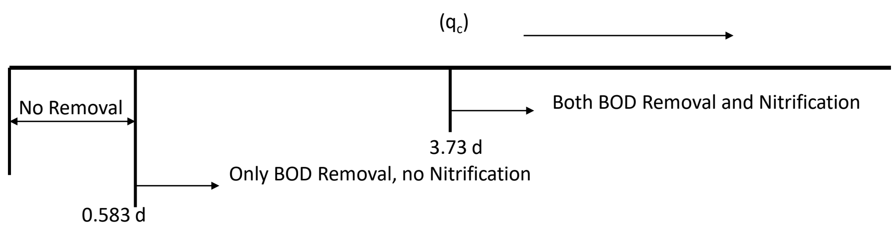
Concept of $(q_c)_{min}$
What is the minimum value of q_c below which, no removal of a substrate shall occur.

<u>**For Carbon Oxidation:**</u>
Corresponding q = q_m.S_o/(K_s + S_o) = 300.(4)/(40+300) = 3.53 / d;
m = 1.715 /d (q_c)min = 0.583 days

For Nitrification:

Corresponding $q_N = (q_m)_N.[TKN]_i/[\{K_s\}_N + [TKN]_i\} = 2.(7.7)/(2+7.7) = 1.59$ /d

$(m)_N = (0.2).(1.59)-0.05 = 0.27$ / d; $(q_c)_{min} = 3.73$ days



Example Problem: Dual Stage Nitrification

Q = 10 MLD; Influent BOD_5: 350 mg/L; Influent TKN = 40 mg/L (as N).

K_S = 40 mg/L;	q_m = 4 /d;	Y_T = 0.5;	K_d = 0.05 /d;
$(K_S)_N$ = 2.0 mg/L;	$(q_m)_N$ = 2 /d	$(Y_T)_N$ = 0.2 ;	$(K_d)_N$ = 0.05/d

Biomass may be represented as $C_5H_7O_2N$
Nitrogen incorporation in heterotrophic biomass must be accounted for.
Nitrogen incorporation in autoprophic biomass may be neglected.

Stage 1 Calculations

$\theta = 8$ hr; $\theta_c = 1$ day; $\mu = \frac{1}{\theta_c} = 1.00$ /d;$q = \frac{\mu + K_d}{Y_T} = \frac{(1.00 + 0.05)}{0.5} = 2.10$ /d

$q = \frac{q_m.S}{K_S + S} = \frac{4.S}{40 + S} = 2.10$; Therefore, S = 44.21 mg/L $V = \theta.Q = \frac{8}{24}.10000 = 3333\,m^3$

$X = \frac{(S_o - S)}{\theta.q} = \frac{(350 - 44.21)}{\frac{8}{24}.(2.10)} = 436.8$ mg/L $\Delta X = \mu.X.V = \frac{(1).(436.8).(3333).(1000)}{10^6} = 1455$ Kg/d

Oxygen Requirement (in Kg/d) = 1.5.Q.(S_o – S) –1.42.(DX)

$$(1.5).10^7.\frac{(350 - 44.21)}{10^6} - (1.42).(1455) = 2520 \text{ Kg/d}$$

Nitrogen Requirement = $\frac{14}{113}.1455 = 180.2$ Kg/d Nitrogen

Influent Nitrogen = $\frac{40.10^7}{10^6} = 400$ Kg/d

Nitrogen Available for Nitrification =
400 – 180.2 = 219.73 kg/d = 21.97 mg/L

$[TKN]_i$ = 21.97 mg/L (as N) $(q_L)_N = \frac{2.(21.97)}{2 + 21.97} = 1.83$ /d $(\theta_c)_{min} = \frac{1}{(0.20).(1.83) - 0.05} = 3.16$ d

Since, $\theta_c < (\theta_c)_{min}$ no nitrification will occur in this tank

Stage II Calculations: For Carbon Oxidation

$\theta = 2$ hr; $\theta_c = 12$ days; $\mu = \frac{1}{\theta_c} = 0.083$ /d; $q = \frac{\mu + K_d}{Y_T} = \frac{(0.083 + 0.05)}{0.5} = 0.267$ /d

$q = \frac{q_m.S}{K_S + S} = \frac{4.S}{40 + S} = 0.267$; Therefore, S = 2.85 mg/L; $V = \theta.Q = \frac{2}{24}.10000 = 833.33\,m^3$

$X = \frac{(S_o - S)}{\theta.q} = \frac{(44.21 - 2.85)}{\frac{2}{24}.(0.267)} = 1858$ mg/L $\Delta X = \mu.X.V = \frac{(0.083).(1858).(833).(1000)}{10^6} = 128.52$ Kg/d

Oxygen Requirement (Kg/d) =1.5.Q.(S_o – S) –1.42.(DX)

$$(1.5).10^7.\frac{(44.21 - 2.85)}{10^6} - (1.42).(128.52) = 437.9 \text{ Kg/d}$$

Nitrogen Requirement = $\frac{14}{113}.(128.52) = 15.92 \text{ Kg/d Nitrogen}$

Nitrogen Available for Nitrification = 21.97 – 1.59 = 20.38 mg/L

[TKN] = 0.99 mg/L;

For Nitrification:

$$(q)_N = \frac{(\mu)_N + (K_d)_N}{(Y_T)_N} = \frac{0.083 + 0.05}{0.2} = 0.665/\text{d} \quad (q)_N = 0.665 = \frac{(q_m)_N.[TKN]}{(K_S)_N + [TKN]} = \frac{2.[TKN]}{2 + [TKN]}$$

$$(q)_N.X_N = \frac{[TKN]_o - [TKN]}{\theta} = \frac{20.38 - 0.99}{2/24} = 232.68 \quad X_N = \frac{236.16}{0.665} = 350 \text{ mg/L}$$

Ratio of microorganisms = $\frac{\mu_N.X_N}{\mu_N.X_N + \mu.X} = \frac{350}{350 + 1858} = 0.158$

(This ratio can be up to 20 percent)

$$\Delta X_N = \mu_N.X_N.V = 0.083.(350).\frac{(833).10^3}{10^6} = 24.19 \text{ Kg/d}$$

Additional Oxygen Requirement (Kg/d) =4.57.Q. $\{[TKN]_o - [TKN]\} \quad -1.42.(\Delta X_N)$

$$4.57.10^7.\frac{(20.38 - 0.99)}{10^6} - 1.42.(24.19) = 851.77 \text{ Kg/d}$$

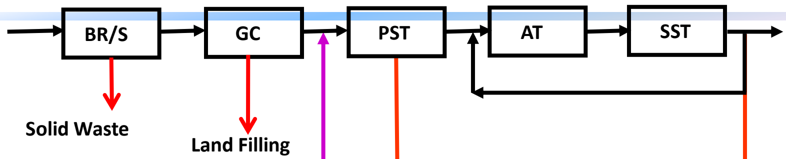
Total Oxygen Requirement (Stage II) = 438 + 852 = 1290 Kg/d

Gross Oxygen Requirement (Stage I + Stage II) = 2520 + 1290 = 3810

Gross Reactor Volume = 3333 + 834 = 4167 m^3 (First and Second Tank)

Gross Sludge Production = 2658 + 39 + 6.95 = 2704 Kg/d (First and Second Tank)

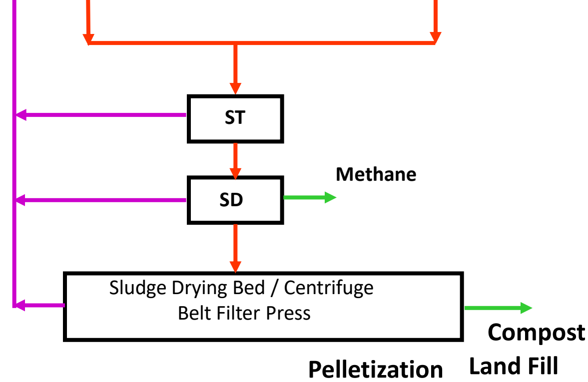
Sludge Management in Wastewater Treatment



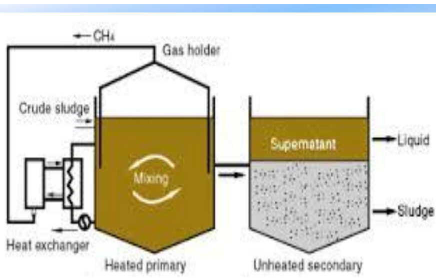
SD: Anaerobic Sludge Digestion

Anaerobic sludge digestion is useful only for organic sludge obtained (from PST and SST) during wastewater treatment. Anaerobic sludge digestion is a suspended growth anaerobic biodegradation process with no sludge recycle.

The objective is to reduce solids concentration in thickened sludge so that the load on sludge drying beds / centrifuge / belt filter press is reduced. The methane gas obtained as a product from the sludge digestion process may have economic value.



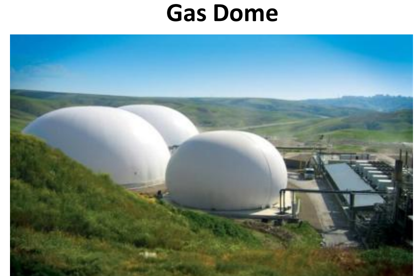
Anaerobic Sludge Digestion



Anaerobic Sludge Digestion Process

Gas Dome

Anaerobic Sludge Digester



Gas Storage

Some Important Parameters:

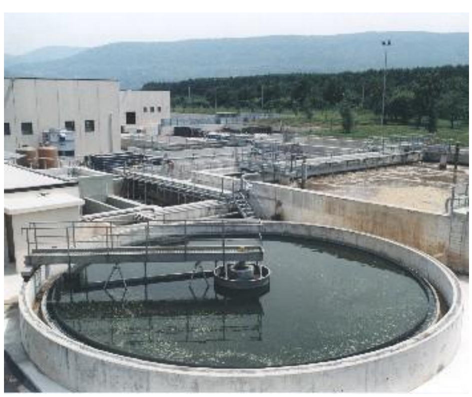
- Density of sludge solids (dry basis); 2200 kg/m^3 (organic sludge);
  2500 kg/m^3 (inorganic sludge from SST in water treatment)
  2650 kg/m^3(inorganic sludge from PST in water treatment)
- Percent sludge solids (weight basis) generally 1 – 2 percent
- Density of sludge Slightly more than density of water

If two of the above parameter are given or assumed, the third may be calculated.

Design of Sludge Thickener

0.12 MLD of sludge from the secondary sedimentation tank in a water treatment plant must be disposed. The solids concentration of the sludge is 20250 mg/L. Design a sludge thickener (diameter and depth) for thickening this sludge. Assume that the solids concentration (weight basis) in the thickened sludge is 4 percent. Calculated the quantity of thickened sludge (in MLD).
Density of water = 1000 kg/m^3; Density of sludge solids = 2514 kg/m^3
Solids loading (dry basis) to the thickener = 80 $kg/m^2/d$
Sludge retention time in thickener = 2 days.

Sludge Thickener



Similar to a sedimentation tank. Influent is either primary or secondary sludge (solids content 1-2 percent. Effluent is the thickened sludge (solids content ~ 4 percent).

Thickening occurs by Type IV settling, i.e., compression settling, where the mechanism of settling is the forcing out of water from the solids due to compressive force of solids on top.

Solution

Sludge solids applied per day (DX) = $Q_w.X_r$ = (0.12).(20250) = 2430 Kg/d

Thickener surface area = 2430/80 = 30.375 m^2; Diameter = 6.2 m

$$\text{Density of thickened sludge} = \rho_{sl} = \frac{1}{\frac{\text{fraction solids(wt. basis)}}{\text{Solids density}} + \frac{\text{fraction water(wt. basis)}}{\text{Water density}}}$$

$$= \frac{1}{\frac{0.04}{2514} + \frac{0.96}{1000}} = 1024 \text{ kg/m}^3$$

Weight of 1 m^3 of thickened sludge = (0.04).(1).(1024) = 40.96 kg;

Solids concentration = 40960 mg/L

Assuming 100 percents solids capture, (20250).(0.12) = 40960.(Q_{sl});

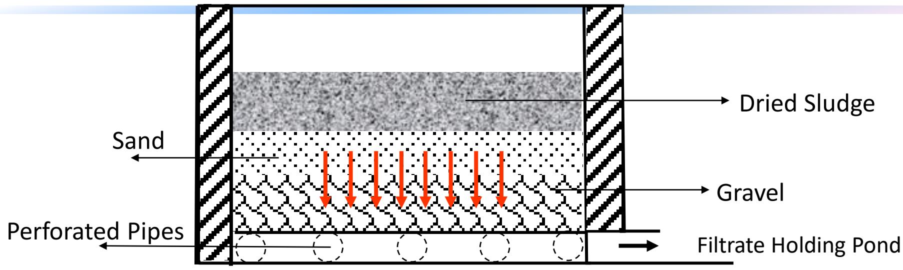
Q_{sl} = 59.4 m^3/d

Volume of sludge thickener = (0.12).(1000).(2) = 240 m^3;

Depth = 240/30.375 = 7.9 m

Design of Sludge Drying Beds

The thickened sludge is applied to sludge drying bed at a loading rate of 2433 kg/d (dry basis). Assuming the water content of the sludge after drying to be 70 percent, determine the total weight of dried sludge per day and its density. Also calculate the area requirement for sludge drying beds. Drying time = 2 weeks; Solids loading to sludge drying bed = 1.5 kg solids (dry basis) /m^2 / cycle



$$\text{Density of Dried sludge} = \rho_d = \frac{1}{\frac{\text{fraction solids(weight basis)}}{\text{Solids density}} + \frac{\text{fraction water(wt. basis)}}{\text{Water density}}} = \frac{1}{\frac{0.30}{2514} + \frac{0.70}{1000}} = 1220 \text{ kg/m}^3$$

Solution

Area requirement per day = 2433/1.5 = 1622 m^2;
A sludge drying bed can be reused every 2 weeks, i.e., for 26 days/year
Total sludge drying bed area = 1622.(365/26) = 22770 m^2 (say 6 acres)
30 percent weight of sludge = 2433 kg/d
Full weight of sludge = 2433/0.3 = 8110 kg/d

Design of Anaerobic Sludge Digester

0.05 MLD of thickened organic sludge with solids concentration of 45000 mg/L (dry basis) is treated in an anaerobic sludge digester. Calculate the solids content (weight basis) of the digested sludge.

COD reduction in digester = 60 percent; Y_T = 0.06; K_d = 0.03 /day.
Dry density of sludge solids = 2200 kg/m^3

Solution

Solids loading to the digester = $Q_{sl}.X_o$ = (0.05).(45000) = 2250 kg/d; q = q_c = 60 days

Digester volume (V_d) = $q.Q_{sl}$ = (60).(0.05).1000 = 3000 m^3; m = $1/q_c$ = 0.0167 /d; q = 0.778 /d

Assuming 75 percent of the sludge is organic (rest is inert),

S_o = (1.42).(0.75).(X_o) = 47925 mg/L (COD basis); Assuming 60% treatment efficiency, S = 19170 mg/L

$q.X_{sl} = (S_o – S)/q$, where X_{sl} = anaerobic biomass; X_{sl} = 616 mg/L; $DX_{sl} = m.X_{sl}.V_d$ = 30.9 kg/d

Reduction in solids = $Q_{sl}.(S_o – S)/1.42 – DX_{sl}$ = 0.05.(47925-19170)/1.42 – 30.9 = 981.6 kg/d

Solids effluent from the digester = (2250 – 981.6) = 1268.4 kg/d;

i.e., 1268.4 kg of solids in 50 m^3 of digested sludge of dry density 2200 kg/m^3

Solids content = 2.50 percent; 4 kg COD = 1 Kg Methane;

COD Consumed = (47925 – 19170).(0.05) = 1437.75 kg/d; Methane Produced = (1437.75)/4
= 359.4 Kg/d

Consequences of Pollutant Loading to Natural Water Bodies

Consequences of Oxygen Depletion In Rivers/Lakes

High BOD demand in rivers/lakes causes dissolved oxygen depletion and suffocation / death of fish.

High nutrient loading in lakes and slow flowing rivers causes eutrophication, i.e., excessive growth of aquatic algae/plants.

Consequences of Nutrient Addition in Lakes

Dead algae causes Oxygen depletion

Algae:
- Present as a suspension in water;
- Increases dissolved oxygen concentration of water
- Important fish food
- Dead Algae may cause DO depletion

Aquatic Plants:
- The leaves are outside the water
- Causes DO depletion
- Dead aquatic plants in increase DO depletion

Algae: Present as a suspension in water;
Increases dissolved oxygen concentration of water
Important fish food
Dead Algae may cause DO depletion

Aquatic Plants: The leaves are outside the water
Causes DO depletion
Dead aquatic plants in increase DO depletion

Hydraulic Considerations during Water / Wastewater Plant Design

Vertical leveling of the various treatment plant units are of extreme importance during water / wastewater plant design.

The line showing the water levels through various units of a treatment plant is known as the hydraulic grade line.

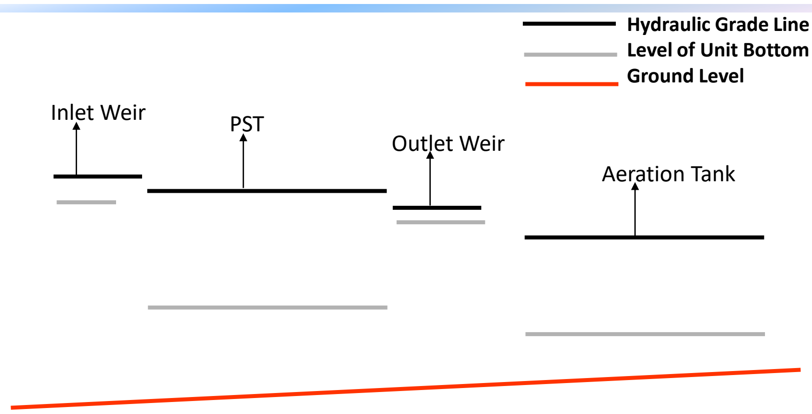
A treatment plant is generally designed to keep pumping to a minimum. Pumping is generally done at the beginning of the treatment train. Water is then expected to flow through various treatment units by gravity.

Hence the hydraulic grade line in a treatment plants goes down as the water passes through the treatment plant.

The hydraulic grade line of a treatment unit, along with the designed depth of water in that unit determine the bottom level of the unit.

Providing the required hydraulic grade line in a treatment plant will require that the upstream treatment units in a treatment plant be at a higher level. This is easy to achieve if the site of the treatment plant is naturally sloping. Otherwise, earthwork is required to re-contour the ground at the treatment plant site or the upstream units of the treatment plants may have to be built on stilts.

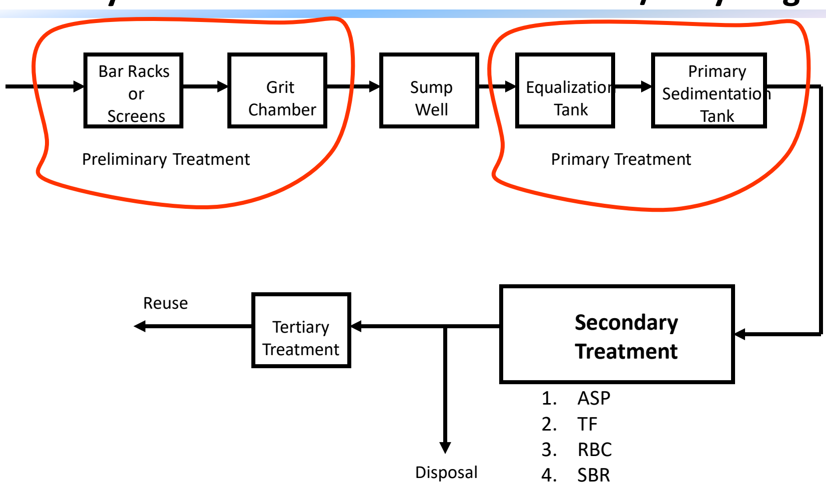
Hydraulic Grade Line, Level of the Unit Bottom and Ground Level



The hydraulic grade line of the last unit of a water treatment must be approximately at the ground level, such that the treated water is conveyed to the underground storage tank by gravity.

The hydraulic grade line of the last unit of a wastewater treatment plant should be at least 1m above the HFL of the receiving water body.

Tertiary Wastewater Treatment for Reuse/Recycling



1. ASP
2. TF
3. RBC
4. SBR
5. MBBR
6. MBR
7. Anaerobic + Polishing Treatment
8. Natural Systems (Ponds / Wetlands)

Tertiary Treatment of Sewage

- Nutrient Removal
    a. Nitrogen Removal: Denitrification
    b. Phosphorus Removal: Precipitation as Calcium Phosphate
- Suspended Solids Removal
    a. Rapid Sand Filtration/Pressure Filtration
    b. Membrane Filtration: Microfiltration
- Micro-pollutants Removal
    a. Activated Carbon Adsorption
    b. Ozonation/other Advanced Oxidation processes (AOPs)
- Dissolved Inorganic Solids Removal
    a. Ion Exchange
    b. Reverse Osmosis
- Disinfection
    a. Chlorination/other disinfectants
    b. UV disinfection