CE412 A
Water Supply & Wastewater Disposal Systems

Part II_Wastewater

Instructor :
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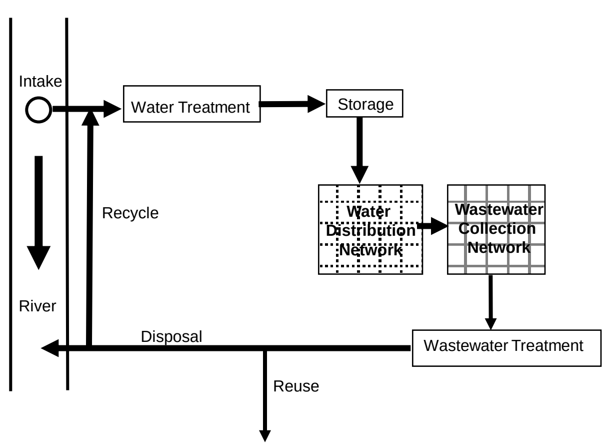
Urban Water Cycle



Wastewater Quantity

- Consumptive use of water is ~5% of the water supplied.
- Water used for horticultural uses and fire fighting does not become wastewater
- In areas with full sewer network, 80 % of remaining water supplied becomes wastewater

Sample Calculations

In 2012, Domestic Population = 9, 870; Temporary population = 1500
In 2032, Domestic Population = 12,300; Temporary population = 3000;

In 2012: Initial

Domestic population = 9870
Average domestic water demand =
9870. (180) = 1.78 MLD
Total temporary population = 1500
Temporary water demand = 1500.(40) =
0.06 MLD
Commercial water demand = 0.5.(1.78) =
0.89 MLD
Average wastewater production =
0.8.(1.78 + 0.06 + 0.89) = 2.18 MLD

Maximum wastewater production =
1.8 x Average = 1.8.(2.18)
= 3.92 MLD
Peak wastewater production =
3.(2.18) = **6.55 MLD**

Initial Peak wastewater production is used for checking scouring velocity in sewers

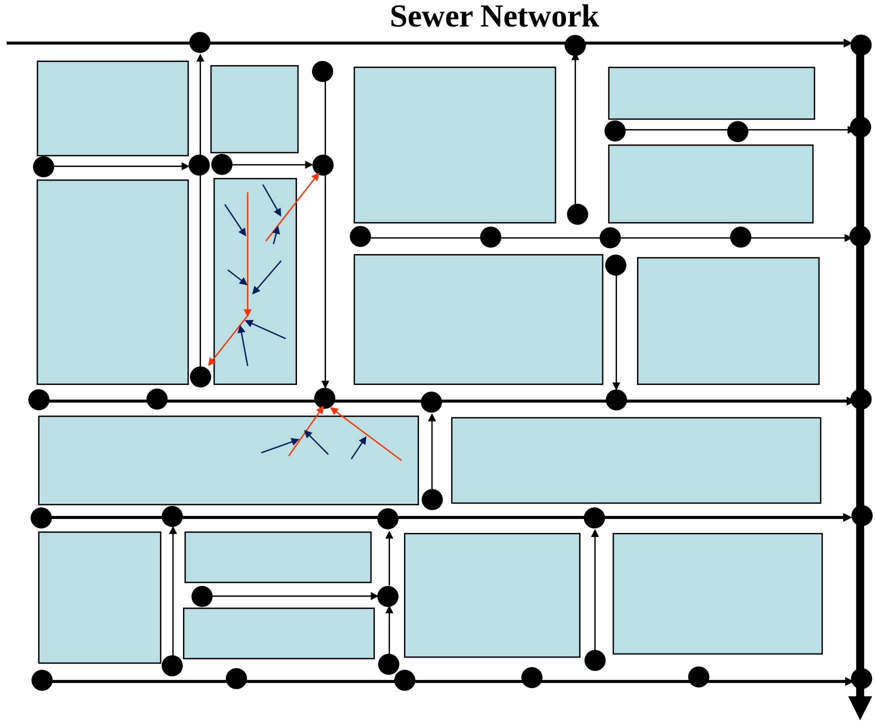
In 2032: Final

Domestic population = 12300
Average domestic water demand =
12300. (235) = 2.89 MLD
Total temporary population = 3000
Temporary water demand = 3000.(60) =
0.18 MLD
Commercial water demand = 0.5.(2.89) =
1.44 MLD
Average wastewater production =
0.8.(2.89 + 0.18 + 1.44) = 3.61 MLD

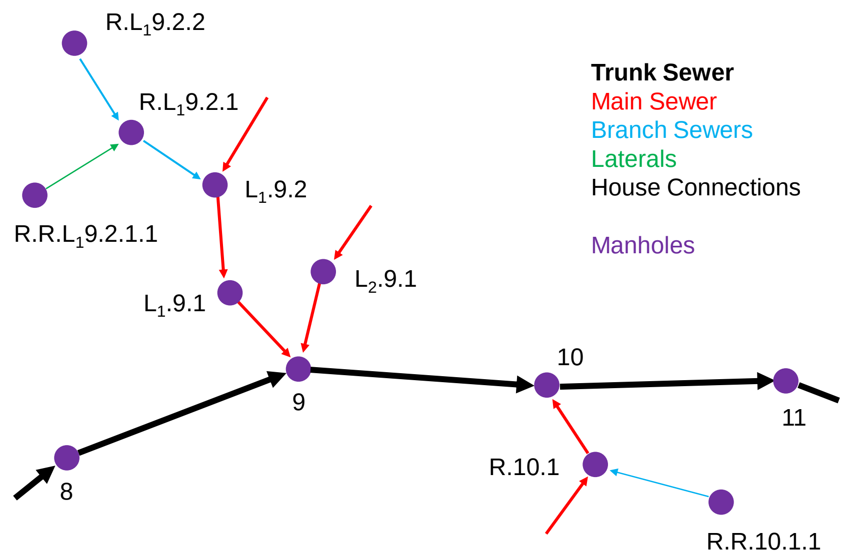
Maximum wastewater production =
1.8 x Average = 1.8.(3.61)
= 6.55 MLD
Peak wastewater production =
3.(3.61) = **10.83 MLD**

Final Peak wastewater production is used for determining sewer size

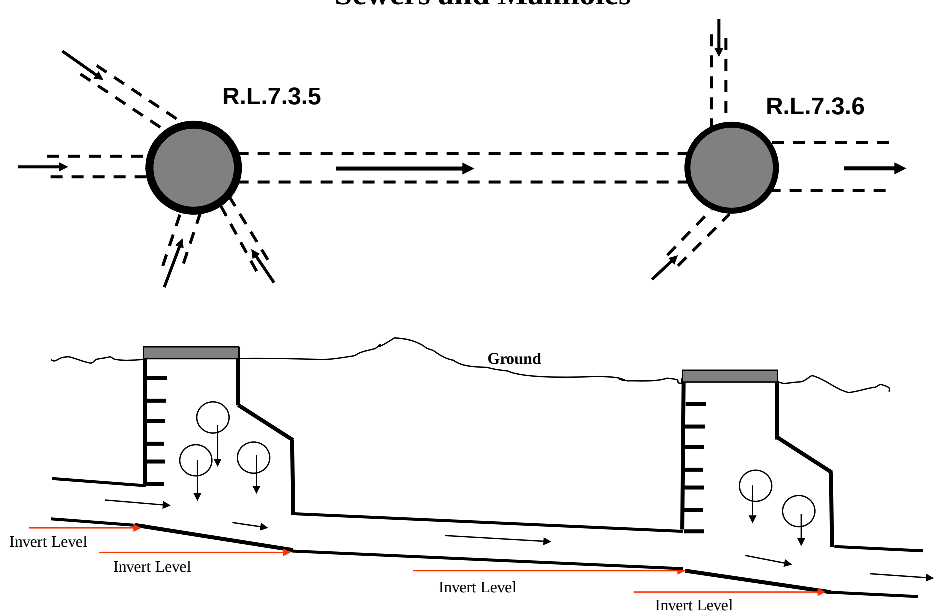
Sewer Network



The Sewer System



Sewers and Manholes



Design of Sewers

1. Manholes to be provided at the junction of sewers or at every change of alignment / gradient of the sewer.

2. Even for straight sewers of uniform diameter and constant slope, manholes must be provided every 30 m for cleaning purposes.

3. Branch sewers should connect to main sewers at angles ranging from 30 to 90 degrees

4. Minimum diameter of a sewer is 150 mm. Other diameters are 200 mm and higher at increments of 100 mm. Manning's coefficient 'n' for new concrete sewers is 0.013

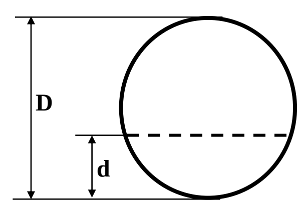
5. Sewers should not be more than 0.8 full at ultimate peak hourly flow. Corresponding flow velocity should be between 0.8 and 3.0 m/s

6. The velocity of flow in a sewer should be at least 0.6 m/s at the initial peak hourly flow. This is required to ensure that particles are not deposited in sewers on a permanent basis.

7. Typical slopes in sewers vary from 1 in 1000 to 10 in 1000, which larger diameter sewers having less slope. Slope should in general be fixed at the smallest possible value, after other design considerations

8. A sewer should be at least 1 m below ground surface

Design of a Sewer Section: Procedure



Peak hourly flow (2032) = 0.30 $m^3/s = q_1$
Peak hourly flow (2012) = 0.15 $m^3/s = q_2$

$\boxed{\text{Design criteria: At } Q = q_1,\ d/D = 0.8}$

$$\boxed{v = \frac{1}{n}.R^{2/3}.S^{1/2}}$$

For a pipe flowing full, R = D/4

d/D	v/V_f	q/Q_f
1.0	1.000	1.000
0.9	1.124	1.066
0.8	1.140	0.968
0.7	1.120	0.838
0.6	1.072	0.671
0.5	1.000	0.500
0.4	0.902	0.337
0.3	0.776	0.196
0.2	0.615	0.088
0.1	0.401	0.021

$$\boxed{Q_f = \frac{1}{n}(0.3117).(D)^{8/3}.(S)^{1/2}}$$

$$\boxed{V_f = \frac{1}{n}(D/4)^{2/3}(S)^{1/2}}$$

Design of a Sewer Section: Procedure

Peak hourly flow (2032) = 0.30 $m^3/s = q_1$
Peak hourly flow (2012) = 0.15 $m^3/s = q_2$

$q_1 = 0.3\ m^3/s$; $d/D = 0.8$; $q_1/Q_f = 0.968$;

$Q_f = 0.3/0.968 = 0.3099\ m^3/s$

n = 0.013; Putting S = 0.002, D = 0.628 m, say, 0.700 m

Corresponding to D = 0.7 m, $Q_f = 0.4142\ m^3/s$

d/D	v/V_f	q/Q_f
1.0	1.000	1.000
0.9	1.124	1.066
0.8	1.140	0.968
0.7	1.120	0.838
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Therefore, q_1/Q_f provided = 0.3/0.4142 = 0.724; actual d/D = 0.65; $v/V_f = 1.100$

$V_f = 1.076$ m/s; v (in 2032) = 1.100.(1.076) = 1.184 m/s (okay)

$q_2/Q_f = 0.15/0.4142 = 0.338$; $v/V_f = 0.902$

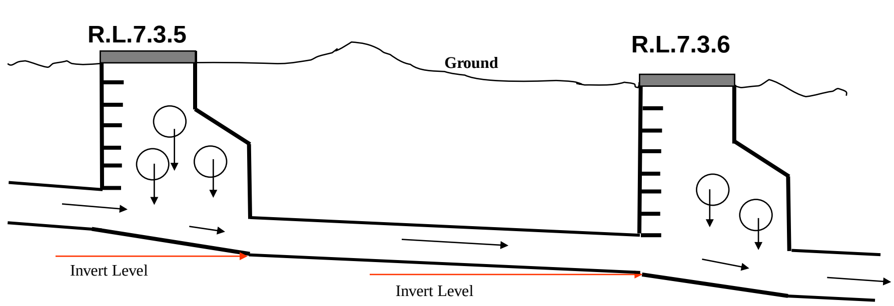
v (in 2012) = 0.902.(1.076) = 0.971 m/s (okay)

$$\boxed{Q_f = \frac{1}{n}(0.3117).(D)^{8/3}.(S)^{1/2}}$$

$$\boxed{V_f = \frac{1}{n}(D/4)^{2/3}(S)^{1/2}}$$

Tabular Calculations:

Manhole		Length (m)	Peak Flow (2032) m^3/s	D (mm)	S	Discharge, m^3/s (2032)		Velocity, m/s (2032)		Peak Flow (2012) m^3/s	Self-Cleansing Velocity (m/s)	Total Fall, m	Invert Elevation (m)	
From	To					Full	Actual	Full	Actual				Upper	Lower
1	**2**	**3**	**4**	**5**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**
R.L.7.3.5	R.L.7.3.6	88	0.30	700	0.002	0.4142	0.3000	1.076	1.184	0.15	0.971	0.176	97.276	97.100



Determination of Invert level of the Upper end of a Sewer, i.e., where the Sewer Exits the manhole

1. The center-line of the branch (or main) sewer should be at least 0.4 m above the center-line of the main (or trunk) sewer leaving the manhole. This will ensure dissipation of excess energy of water in the smaller sewer.

2. Manholes are connected over the center-line of the principal sewer. A minimum drop of 0.03m is provided between center-line of principal sewers entering and exiting manholes.

3. The diameter of a sewer exiting a manhole must always be greater than or equal to diameter of the principal sewer entering the manhole.

4. The top (crown level) of a principal sewer entering a manhole must never be at a lower elevation than that of a sewer exiting the manhole.

Wastewater Quality

Wastewater produced in domestic setting,

1. Black water: Toilet waste
   (mainly Organic Carbon, Nitrogen, Phosphorus, microorganisms)

2. Grey water: Kitchen waste, bathing and cleaning waste
   (mainly organic C, N and P, surfactants, salts, dirt, grit, other solid waste)

Domestic Wastewater = Black water + grey water

Organic Carbon = BOD_5
BOD_5 added by permanent population = 50 g /capita/d
BOD_5 added by temporary population = 25 g /capita/d

In 2012:

BOD_5 added = 50(9870) + 25(1500) = 531 kg/d

Average Flow: 531 kg in 1.84 ML = 288 mg/L BOD_5

Assume: BOD_5 : N (as N) : P (as P) on wt. basis) = 100: 10: 2

Av: BOD_5 = 288 mg/L; TKN = 28.8 mg/L (as N); Total-P = 5.76 mg/L (as P)

Commercial wastewater is assumed to have the same characteristics as domestic wastewater

In 2032:

BOD5 added = 50(12300) + 25(3000) = 690 kg/d

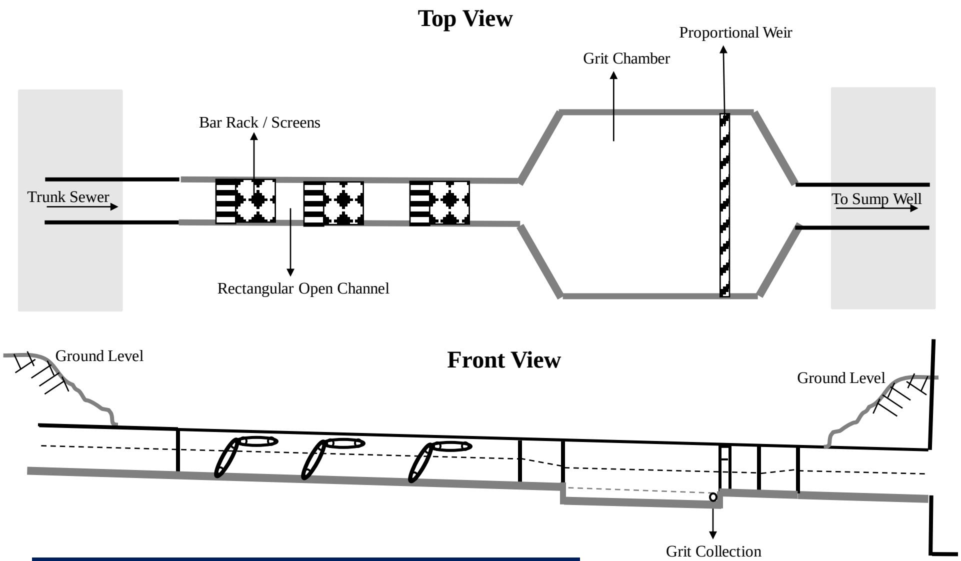
Average Flow: 690 kg in 3.07 ML = 224 mg/L BOD5

Assume: BOD_5 : N (as N) : P (as P) on wt. basis) = 100: 10: 2

Av: BOD_5 = 224 mg/L; TKN = 22.4 mg/L (as N); Total-P = 4.48 mg/L (as P)

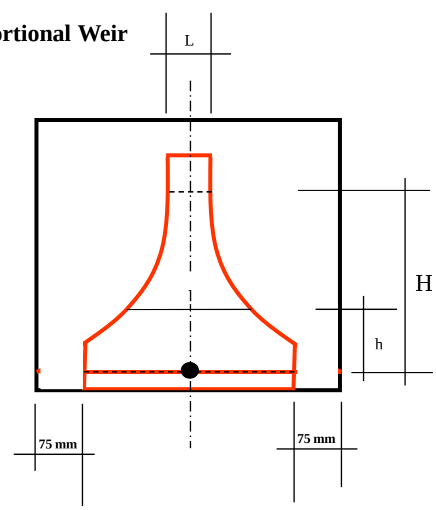
Commercial wastewater is assumed to have the same characteristics as domestic wastewater

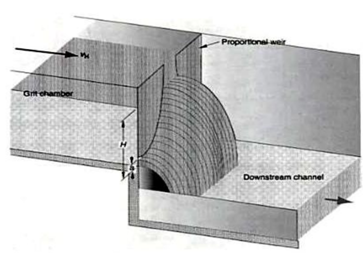
Preliminary Wastewater Treatment



http://www.youtube.com/watch?feature=player_detailpage&v=FMWNMNrfzC4

Proportional Weir





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

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) is constant irrespective of the flow.

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

Assuming $C_d = 0.6$,

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

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

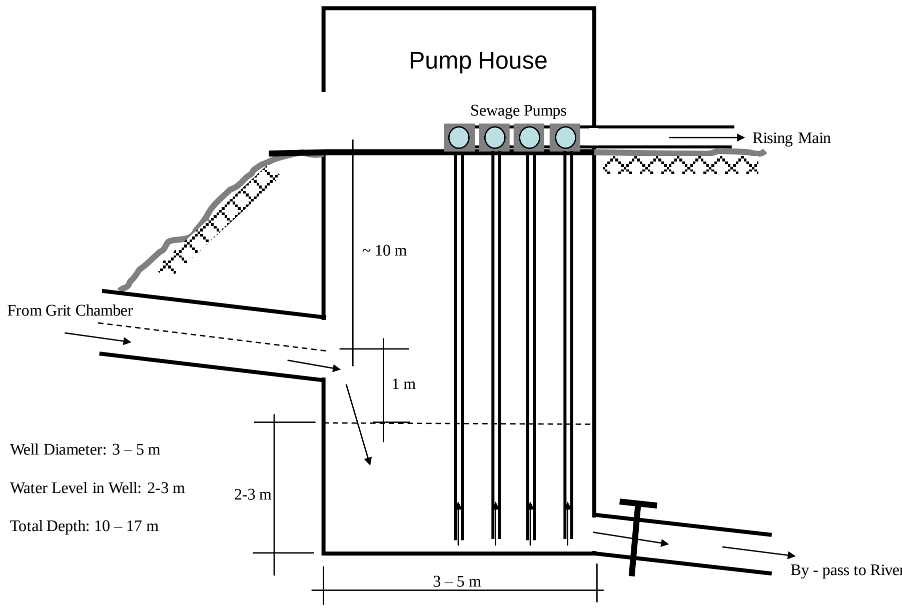
Thus, $[l.h^{\frac{1}{2}}]$ must be constant

If, Q and H are known,

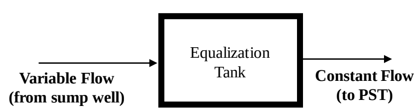
$$L = \frac{Q}{4.173.[H^{\frac{3}{2}}]};$$

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

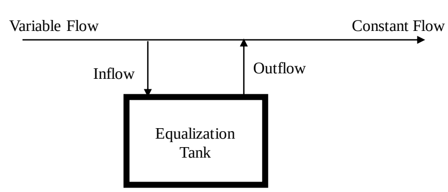
Sump Well



Equalization Tank (Flow Equalization)

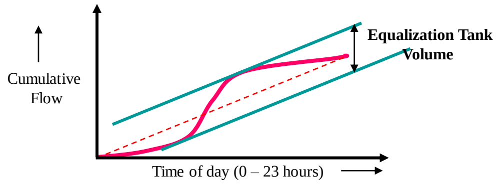


In- Line Equalization



Off - Line Equalization

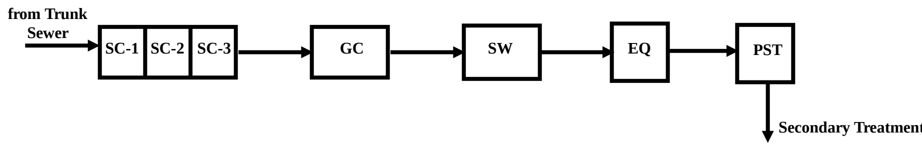
Determination of Equalization Tank Volume



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

Q = Maximum Flow (End of Design Period)

Preliminary and Primary Wastewater Treatment



Primary Sedimentation for Wastewater Treatment

Exactly same as primary sedimentation for water treatment in most respects

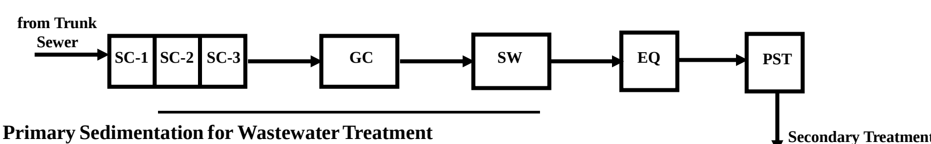
However,
1. Particles removed in this case are mostly organic particles of density 1200 kg/m^3

2. SOR = 32 m^3/m^2/d, organic particles: 60 μm, inorganic particles: 20 μm
3. SOR = 48 m^3/m^2/d, organic particles: 75 μm, inorganic particles: 25 μm

4. Type II settling, hence smaller particles are also removed

5. SS removal: 50 – 70 %
6. BOD_5 removal: 25 – 40 %

Preliminary and Primary Wastewater Treatment



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Sewage:
After Primary Sedimentation

Screenings

Grit

Fresh Sewage

Aged Sewage

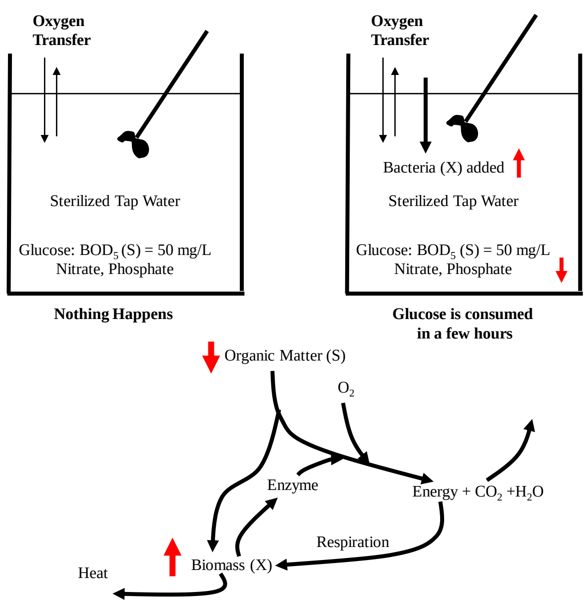
Sewage: After Primary Sedimentation

Sewage: Before and After Secondary Sedimentation

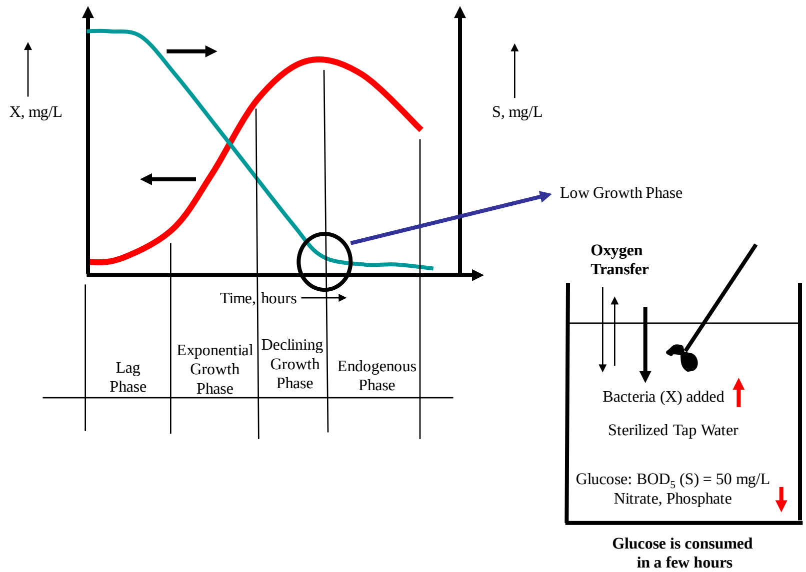
http://www.youtube.com/watch?v=GTLKoGGo0dM&feature=player_detailpage

Bacterial Consumption of Organic Carbon

$$C_6H_{12}O_6 + 6O_2 \rightarrow 6CO_2 + 6H_2O$$



Microbial Growth Curve



Microbial Growth: Definitions and Relations

X = Biomass Concentration, mg/L

S = Substrate Concentration, mg/L

$\frac{dS}{dt}$ = Substrate Utilization Rate, mg/L/d

$\left[\frac{dS}{dt}\right] / X = q$ = Specific Substrate Utilization Rate, /d

$\left[\frac{dX}{dt}\right]_T$ = Gross Biomass Growth Rate, mg/L/d

$\left[\frac{dX}{dt}\right]_G$ = Net Biomass Growth Rate, mg/L/d

$\left[\frac{dX}{dt}\right]_G / X = \mu$ = Specific Net Biomass Growth Rate, /d

$\left[\frac{dX}{dt}\right]_E$ = Biomass Decay Rate, mg/L/d

$\left[\frac{dX}{dt}\right]_E / X = k_d$ = Specific Biomass Decay Rate, /d

When S is large ($S >> K_s$), $q = q_{max}$ and hence, $\mu = \mu_{max}$

When S is small ($S << K_s$), $q = [q_{max}/K_s].S$

When $\mu = 0$, $q = k_d/Y_T$

When $q < k_d/Y_T$, $\mu < 0$

$$\left[\frac{dX}{dt}\right]_G = \left[\frac{dX}{dt}\right]_T - \left[\frac{dX}{dt}\right]_E$$

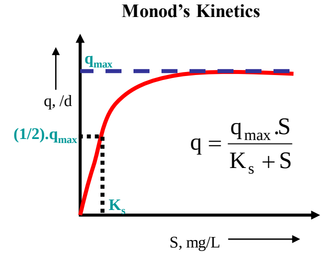
$$\left[\frac{dX}{dt}\right]_T = Y_T \left[\frac{dS}{dt}\right]$$

Y_T = Yield Coefficient, mg X produced (gross) / mg S consumed

$$\left[\frac{dX}{dt}\right]_G / X = Y_T \left[\frac{dS}{dt}\right] / X - \left[\frac{dX}{dt}\right]_E / X$$

$$\boxed{\mu = Y_T.q - k_d}$$

Monod's Kinetics



$$q = \frac{q_{max}.S}{K_s + S}$$

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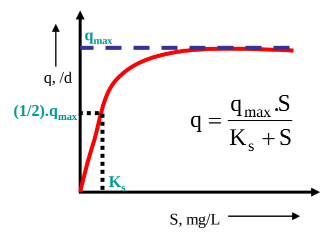
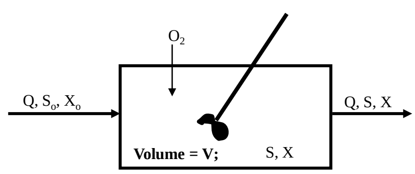


Microbial Kinetics in a Reactor



Completely Mixed Reactor

Microbial Kinetic Constants

q_{max} = 4 /d

K_s = 25 mg/L

K_d = 0.05 /d

Y_T = 0.50

Hydraulic Retention Time (θ) = V/Q

$$\boxed{\mu = Y_T.q - k_d}$$

$$\boxed{q = \frac{q_{max}.S}{K_s + S}}$$

Substrate Mass Balance:

$$Q.S_o = Q.S + \left[\frac{dS}{dt}\right].V$$

$$\boxed{Q.\frac{(S_o - S)}{V} = q.X}$$

Biomass Mass Balance:

$$Q.X_o + \left[\frac{dX}{dt}\right]_G .V = Q.X$$

$$\mu.X.V = Q.X$$

$$\boxed{\mu = Q/V = 1/\theta}$$

Example Problem

Q = 1 MLD; V = 1000 m^3; S_o = 300 mg/L

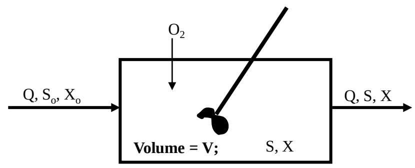
θ = V/Q = 1 day; $\mu = 1/\theta = 1$ / d

$$q = \frac{\mu + k_d}{Y_T} = \frac{1 + 0.05}{0.5} = 2.1\text{ /d}$$

$$S = \frac{q.K_s}{(q_{max} - q)} = \frac{(2.1).(25)}{(4 - 2.1)} = 27.63\text{ mg/L}$$

$$X = \frac{(S_o - S)}{\theta.q} = \frac{(300 - 27.63)}{1.(2.1)} = 129.7\text{ mg/L}$$

Microbial Kinetics in a Reactor



Completely Mixed Reactor

Microbial Kinetic Constants

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$K_s = 25$ mg/L

$K_d = 0.05$ /d

$Y_T = 0.50$

$\boxed{\text{Hydraulic Retention Time } (\theta) = V/Q}$

$$\boxed{\mu = Y_T.q - k_d}$$

$$\boxed{q = \frac{q_{max}.S}{K_s + S}}$$

Substrate Mass Balance:

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Biomass Mass Balance:

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Q = 1 MLD; V = 1000 m^3; $S_o = 300$ mg/L

$\theta = V/Q = 1$ day; $\mu = 1/\theta = 1$ / d

$$q = \frac{\mu + k_d}{Y_T} = \frac{1 + 0.05}{0.5} = 2.1\,/d$$

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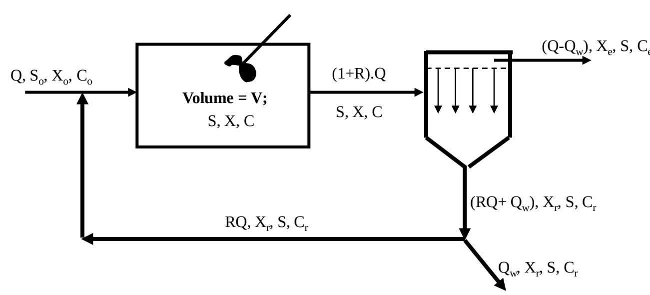
Microbial Kinetics in a Reactor with Recycle



Substrate Balance for the whole system:

$Q.S_o - [(Q - Q_w).S + Q_w.S] = [dS/dt].V$

$\boxed{Q.(S_o - S)/V = q.X}$ (A)

Biomass Balance for the whole System:

$[dX/dt]_g.V = Q_w.X_r$

$\boxed{\mu.X.V = Q_w.X_r}$ (B)

Biomass Balance for Aeration Tank:

$R.Q.X_r + [dX/dt]_g.V = (1 + R).Q.X$

$R.Q.X_r + \mu.X.V = (1 + R).Q.X$

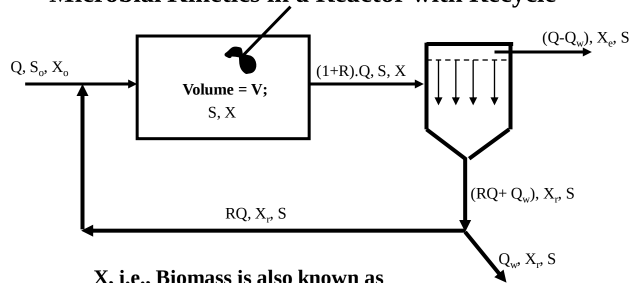
$\boxed{\mu.V = Q.[1 + R - R(X_r/X)]}$ (C)

Other Relationships:

$\boxed{\mu = Y_T.q - k_d}$ (D)

$\boxed{q = \frac{q_{max}.S}{K_s + S}}$ (E)

Microbial Kinetics in a Reactor with Recycle



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Sludge Wasted (ΔX) = $Q_w.X_r$

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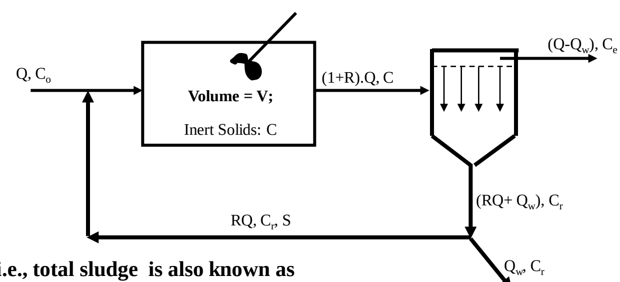
Please Note,

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

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

Inert Solids in a Reactor with Recycle



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

Total Sludge Wasted (ΔX + ΔC) = $Q_w.(X_r + C_r)$

Inert Solids Balance for the whole system:

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

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

Retention Time of Inert Solids:

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

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

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

Design of Activated Sludge Process: Part I

1. Given Q, assume θ (4-12 hrs) and calculate volume of aeration tank (V)
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Design of Activated Sludge Process: Part I

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

Aeration Tank Volume = Q.θ = 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**

$$q = \frac{q_{max}.S}{K_s + S} = \frac{(4).(4)}{(25+4)} = 0.55\ /d$$

$$\mu = Y_T.q - k_d = (0.5)(0.55) - 0.05 = 0.225\ /d$$

$\theta_c = 1/\mu = 4.43$ d;

For, S_o = 350 mg/L,

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

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

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

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

(C + X) = 3394 mg/L

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

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

$$X_r = 12500 - 3261 = 9239 \text{ mg/L}; \qquad Q_w = \frac{\Delta X}{X_r} = \frac{7082}{9239} = 0.766 \text{ MLD}$$

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

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

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

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

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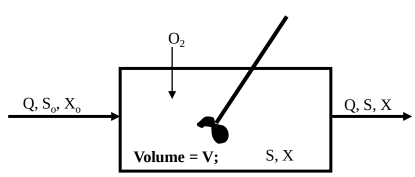
Microbial Kinetics in a Reactor



Completely Mixed Reactor

Microbial Kinetic Constants

$q_{max} = 4$ /d

$K_s = 25$ mg/L

$K_d = 0.05$ /d

$Y_T = 0.50$

Hydraulic Retention Time $(\theta) = V/Q$

$$\boxed{\mu = Y_T.q - k_d}$$

$$\boxed{q = \frac{q_{max}.S}{K_s + S}}$$

Substrate Mass Balance:

$$Q.S_o = Q.S + \left[\frac{dS}{dt}\right].V$$

$$\boxed{Q.\frac{(S_o - S)}{V} = q.X}$$

Biomass Mass Balance:

$$Q.X_o + \left[\frac{dX}{dt}\right]_G.V = Q.X$$

$$\mu.X.V = Q.X$$

$$\boxed{\mu = Q/V = 1/\theta}$$

Example Problem

$Q = 1$ MLD; $\quad V = 1000$ m^3; $\quad S_o = 300$ mg/L

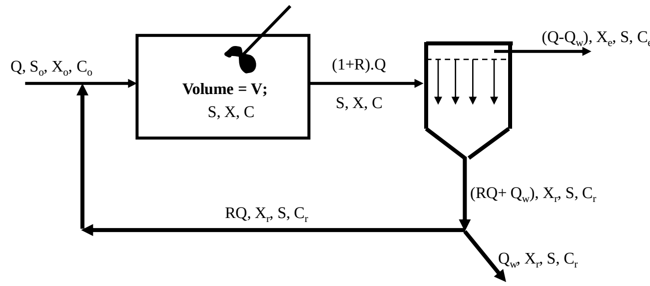
$\theta = V/Q = 1$ day; $\quad \mu = 1/\theta = 1$ / d

$$q = \frac{\mu + k_d}{Y_T} = \frac{1+0.05}{0.5} = 2.1\text{/d}$$

$$S = \frac{q.K_s}{(q_{max} - q)} = \frac{(2.1).(25)}{(4-2.1)} = 27.63 \text{ mg/L}$$

$$X = \frac{(S_o - S)}{\theta.q} = \frac{(300 - 27.63)}{1.(2.1)} = 129.7 \text{ mg/L}$$

Microbial Kinetics in a Reactor with Recycle



Substrate Balance for the whole system:

$Q.S_o - [(Q - Q_w).S + Q_w.S] = [dS/dt].V$

$\boxed{Q.(S_o - S)/V = q.X}$ (A)

Biomass Balance for the whole System:

$[dX/dt]_g.V = Q_w.X_r$

$\boxed{\mu.X.V = Q_w.X_r}$ (B)

Biomass Balance for Aeration Tank:

$R.Q.X_r + [dX/dt]_g.V = (1 + R).Q.X$

$R.Q.X_r + \mu.X.V = (1 + R).Q.X$

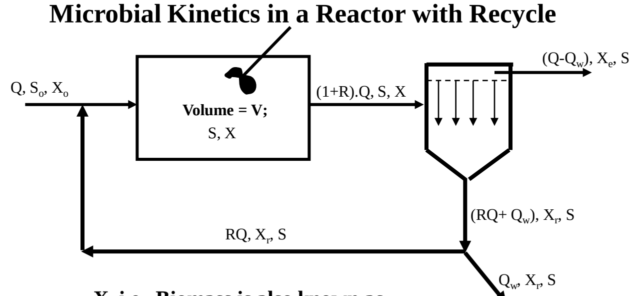
$\boxed{\mu.V = Q.[1 + R - R(X_r/X)]}$ (C)

Other Relationships:

$\boxed{\mu = Y_T.q - k_d}$ (D)

$\boxed{q = \frac{q_{max}.S}{K_s + S}}$ (E)

Microbial Kinetics in a Reactor with Recycle



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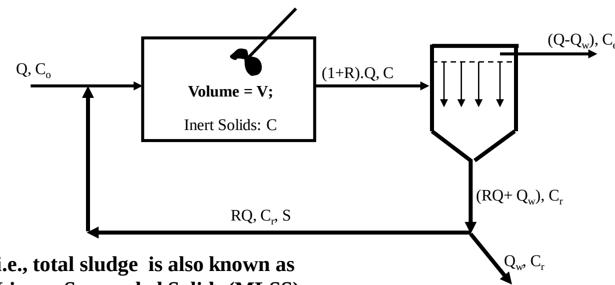

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Inert Solids Balance for the whole system:

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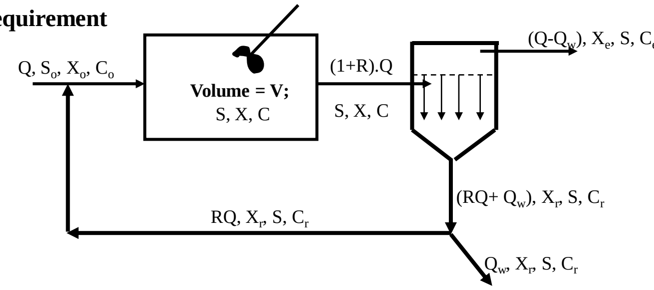
Oxygen Requirement



Organic carbon compounds and consumed by microorganisms and converted to biomass. Part of this biomass is oxidized by microorganisms to inorganic carbon (respiration). Energy produced through respiration is the driving force for sustenance of the microorganisms and production of new biomass.

Oxygen is required for respiration of aerobic microorganisms. Oxygen is the terminal electron acceptor.

Amount of Oxygen Required:
[Oxygen Required in Aeration Tank, kg/d] = [BOD_u Removed, in kg/d] – 1.42.[Sludge Wasted, kg/d]

Where, $BOD_u \sim 1.5.BOD_5$; So, O_2 Reqd, kg/d = 1.5.Q (S_o – S) – 1.42.[ΔX]

Biomass formula (dry basis): $C_5H_7O_2N$; Formula weight = 113

Respiration Equation: $C_5H_7O_2N + 5O_2 + H^+ \rightarrow 5CO_2 + 2H_2O + NH_4^+ + \text{energy}$

This, if 1 gm of biomass is produced, 1.42 g of oxygen requirement is saved.

Nutrient (N and P) Requirement:

In addition to organic carbon and oxygen, all microorganisms require N and P to grow.

Wastewater generally contains sufficient nutrients (in fact excess) from microorganism growth.

Nitrogen in wastewater is generally expressed as TKN (mg/L as N)

Phosphorus in wastewater is generally expressed as total – P (mg/L as P)

The BOD_5 (in mg/L) : TKN (in mg/L as N) : Total-P (in mg/L as P) ratio in domestic wastewater is ~ 100 : 5 : 1

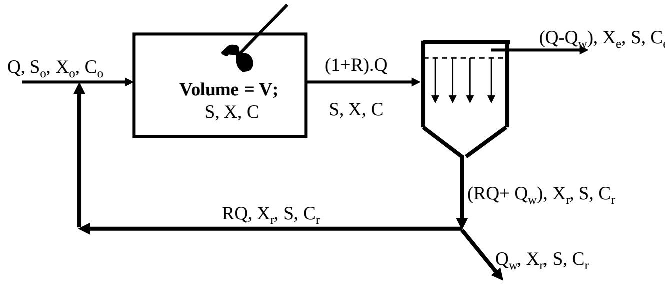
Biomass formula (dry basis): $C_{60}H_{87}O_{23}N_{12}P$; Formula Weight: 1382

For the growth of 1382 g biomass, 168 g N and 39 g P is required

Nutrient Requirement: 0.121 g N and 0.028 g P per g biomass produced (or wasted).

Hence the N and P concentration will decrease by this amount during ASP

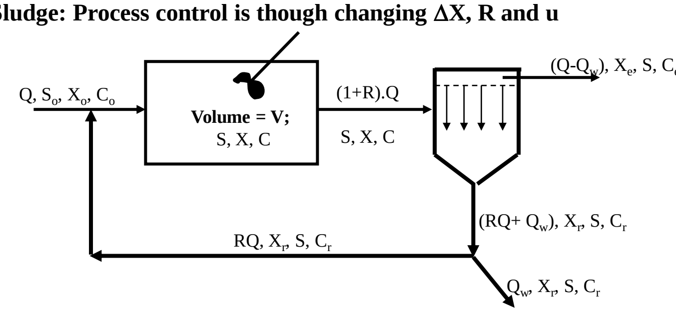
Activated Sludge: Process control is though changing ΔX, R and u



What happens if S_o (influent BOD_5) increases (or decreases) ??
Effluent quality will not decline provided enough oxygen is provided. However X concentration In aeration tank and sludge wastage Δ(C + X) will increase (or decrease).

What happens in ΔX (sludge wasting) is decreased/increased ??
Increase in Δ(C+X) will decrease X, C and θ_c. This will increase μ and q and ultimately S. Opposite will happen when Δ(C+X) is decreased.

Activated Sludge: Process control is though changing ΔX, R and u



What are the characteristics of a high-growth ASP system ??
In such a system, θ_c is low. Hence μ and q are high, S is relatively high. ΔX is high, hence O_2 requirement is low, but nutrient requirement is high. Chance of forming pin-point flocs with poor settling characteristics.

What are the characteristics of an extended growth ASP system??
In such a system, θ_c is high. Hence μ and q are low, S is relatively low. ΔX is low, hence O_2 requirement is high, but nutrient requirement is low. The inert matter concentration (C) in the aeration tank may be high. Chance of forming filamentous bulking sludge with poor settling characteristics.

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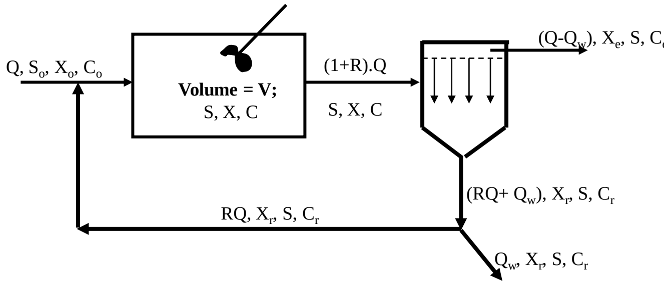

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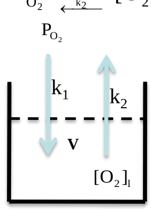
Activated Sludge Process: Aeration

Theory of Aeration:

$P_{O_2} \underset{k_2}{\overset{k_1}{\rightleftarrows}} [O_2]_l$ $[O_2]_l$ in moles/L P_{O_2} in atm.

Henry's Law: $P_{O_2} = H.[O_2]_L^S$

H is the Henry's Constant in atm - L/mole



Rate of oxygen input to liquid phase = $V.\dfrac{d[O_2]_l}{dt}$

= Rate of Oxygen Absorption – Rate of Oxygen Stripping = $k_1.P_{O_2} - k_2.[O_2]_l$

At equilibrium: $[O_2]_l = [O_2]_l^s$; $k_1.P_{O_2} = k_2.[O_2]_l^s$;

$P_{O_2} = \left(k_2/k_1\right).[O_2]_l^s$; $\boxed{k_2 = k_1.H}$

k_1 in moles/atm.-s

k_2 in L/s

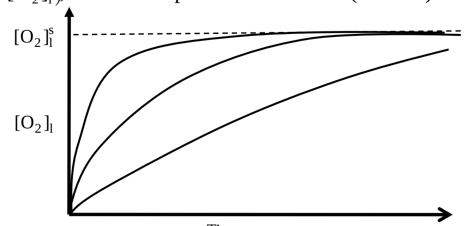
$$V.\frac{d[O_2]_l}{dt} = k_1.\left(H.[O_2]_l^s\right) - k_1.H.[O_2]_l = k_1.H.\left([O_2]_l^s - [O_2]_l\right); \quad k_1.H = \left(k_a.H\right).A = k_L.A$$

$$V.\frac{d[O_2]_l}{dt} = k_L.A.\left([O_2]_l^s - [O_2]_l\right); \quad k_L = \text{Mass transfer coefficient; } A = \text{contact area between liquid - gas phases}$$

$$\frac{d[O_2]_l}{dt} = k_L.\left(A/V\right).\left([O_2]_l^s - [O_2]_l\right) = k_L.a.\left([O_2]_l^s - [O_2]_l\right); \text{ where } a = \text{specific surface area } (m^2/m^3)$$

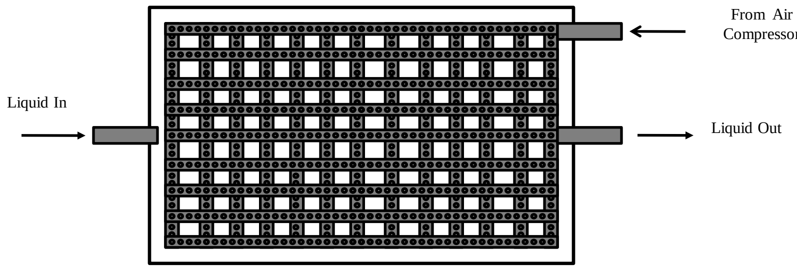
The oxygen transfer rate from the gas to the liquid phase can be increased by increasing the value of 'a'.

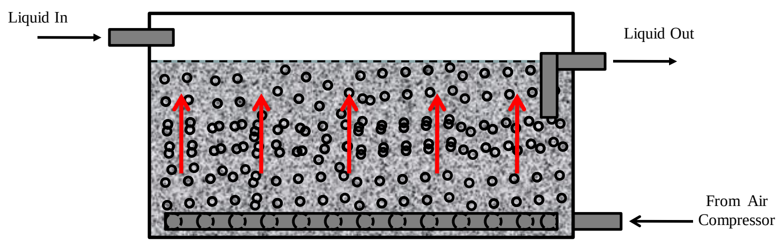
The value of 'a' may be increased by using mechanical aerators



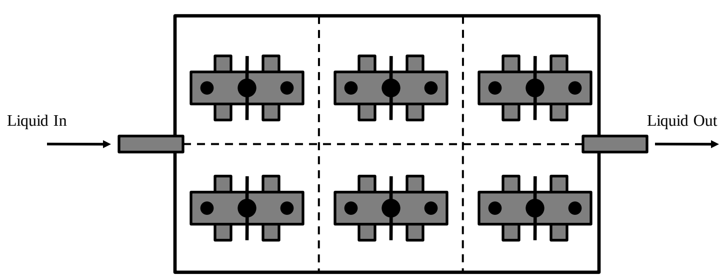
Two types of aerators are commonly used:

Bubble Aerators





Turbine Aerators



Fate of Nitrogen: Nitrification

In wastewater, nitrogen is present as TKN, i.e., a mixture of organic-N and ammonia-N

During BOD removal in ASP, a part of the TKN is incorporated into the cell biomass
Amount Incorporated = 0.124. (ΔX) kg/d

The remaining amount remains in wastewater as ammonia – N

Presence of high concentrations of ammonia-N in water gives it a disagreeable smell

$$NH_4^+ \rightleftarrows NH_3 + H^+ \qquad pK = 9.4$$

Presence of high concentrations of ammonia-N in water is also toxic to fish in surface water

It is thus desirable that the ammonia present in wastewater is converted to nitrate before release
The biological process for converting ammonia to nitrate is known as **Nitrification**.

Biological conversion of ammonia to nitrate is a two step process,
ammonia to nitrite conversion by *Nitrosomonas* species of bacteria
Nitrite to nitrate conversion by *Nitrobacter* species of bacteria

The first step of the process, i.e., conversion of ammonia to nitrite is rate limiting.

Both *Nitrosomonas* and *Nitrobacter* are aerobic chemo-autotrophic microorganisms.

Classification of Microorganisms

All microorganisms needs three things to survive, 1) Energy source,
2) Carbon / food source
3) Terminal electron acceptor

Aerobic microorganisms use ***oxygen*** as the terminal electron acceptor
Anaerobic microorganisms use chemicals ***other than oxygen*** as terminal electron acceptor

Heterotrophic microorganisms use *organic carbon* as energy source. Hence they need organic carbon to survive. They also use *organic carbon* as food source.

Chemo-autotrophic microorganisms use *chemical compounds other than organic carbon* as energy source. They do not need reduced carbon to survive. They use *inorganic carbon* as food source.

Photo-autotrophic microorganisms use light (photons) as energy source and *inorganic carbon* as food source.

Nitrosomonas / Nitrobactor: Energy source: Ammonia / Nitrite
Food Source: Inorganic Carbon
Electron acceptor: Oxygen

Aerobic, chemo-autotrophic microorganisms

Single Stage Nitrification

- Nitrification along with the ASP process itself
- Low θ_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

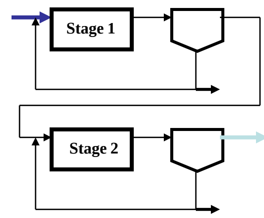
Two Stage Nitrification

Stage 1:

Low θ_c
Only BOD removal and no nitrification

Stage 2:

High θ_c
Removal of residual BOD
Nitrification



Essential Requirements for Efficient BOD and TKN Removal in an Aerobic Reactor

1. Availability of large concentration of Biomass
2. Availability of sufficient amount of oxygen

Any reactor where the above two conditions are satisfied is likely to show efficient removal of BOD and TKN

Suspended growth reactors: Biomass suspended in a tank, e.g., ASP
Attached growth reactors: Biomass attached to media kept in a tank

Concept of Attached Growth

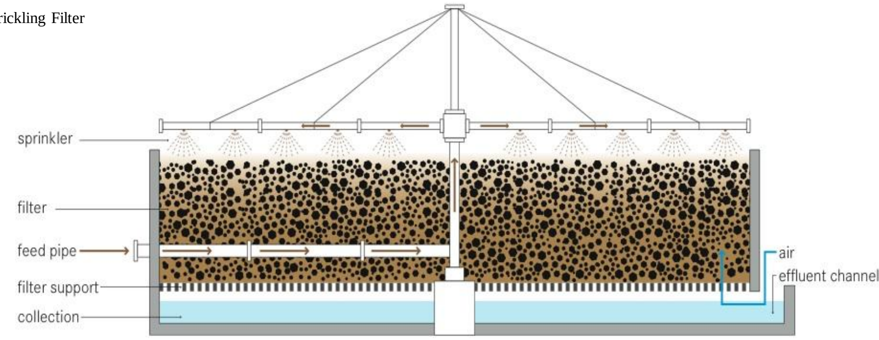
1. One of the essential requirements for efficient BOD/TKN removal in a bio-reactor is the maintenance of high biomass concentration in the reactor

2. In the suspended growth system, the biomass is allowed to escape from the reactor along with the treated effluent. However, the escaped biomass is captured in the SST and recycled back into the reactor. High biomass concentration is maintained in this way.

3. The attached growth concept is based on the observation that biomass prefers to attach itself to inert surfaces (if available).

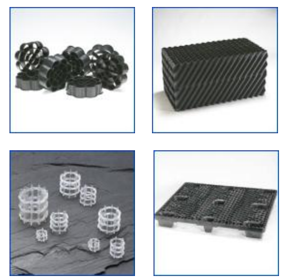
4. Hence if inert media is provided inside the reactor, biomass will grow attached to this media. Such biomass will not be able to escape from the reactor easily (since it is attached). Thus high biomass concentration can be maintained inside the reactor.

Trickling Filter

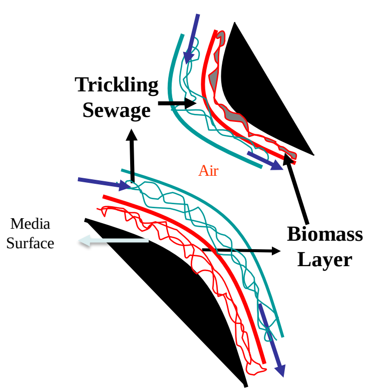


Actual TF Operation

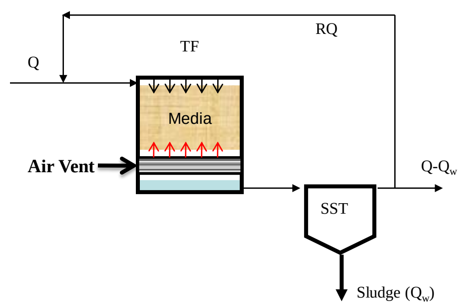
Plastic Media used in TF



Mode of Operation



Recirculation in TF



Media: 25 – 100 mm size, rock, gravel, plastic

The sewage must "trickle", i.e., air must be present inside the filter at all times.

Sludge Wastage by Sloughing:

Air supply through convective currents in the TF

Design Parameters:

Organic Loading Rate (OLR): $Q.S_o/V$
Units:
Kg BOD_5 applied / m^3 reactor volume/d

Hydraulic Loading Rate (HLR): $(Q + R.Q)/A_s$
Units:
m^3 sewage applied/ m^2 reactor surface area/d

Recirculation is from SST. Treated effluent (**not sludge**) is re-circulated.

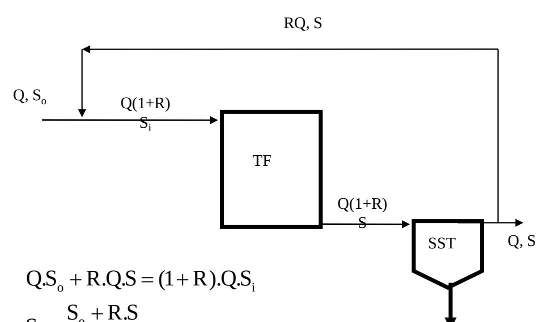
Recirculation enables the variation of HLR independent of OLR. This is important for high strength waste and also for maintaining the most of the media in wetted condition.

Depending on applied OLR, HLR and other factors, trickling filters can be divided into the following types,

Filter Type	Filter Medium	OLR, $kg/m^3/d$	HLR, $m^3/m^2/d$	% Removal	Depth (m)	R
Low Rate	Rock, Slag	0.1 - 0.3	1 - 4	80 - 85	1.8 - 3	0
Intermediate Rate	Rock, Slag	0.3 - 1.2	10 - 30	65 - 85	1 - 3	0.5 - 3
High Rate	Rock	1.2 - 3	40 - 90	65 - 85	2 - 5	1 - 4
Super High Rate	Plastic	3 - 4	60 - 120	65 - 80	4 - 12	1 - 4
Roughing	Plastic	4 - 6	60 - 180	40 - 65	4 - 12	1 - 4

Generally, higher values of OLR and HLR results in diminished filter performance. Increasing the reactor height improves filter performance.

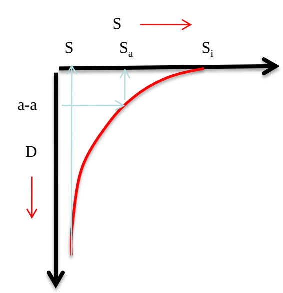
Plastic media is generally used if the filter height in more than 5 m.



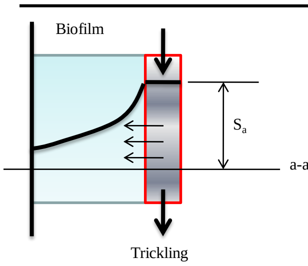
$$Q.S_o + R.Q.S = (1+R).Q.S_i$$

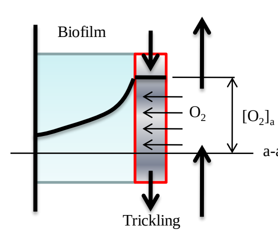
$$S_i = \frac{S_o + R.S}{(1+R)}$$

BOD_5 Profile in a Trickling Factor



A trickling filter is **NOT** a completely mixed reactor

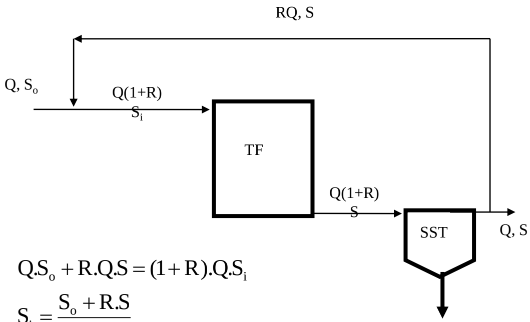




$$q = \left[\frac{q_m.S}{K_s + S}\right].\left[\frac{(q_m)_o.[O_2]}{(K_s)_o + [O_2]}\right]$$

When $[O_2]$ is large compared to $(K_s)_o$,

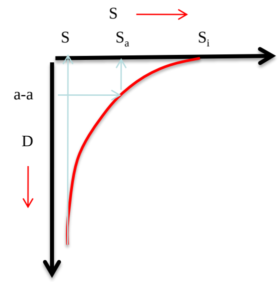
$$q = \left[\frac{q_m.S}{K_s + S}\right].(q_m)_o = \frac{q_{max}.S}{K_s + S}$$



$$Q.S_o + R.Q.S = (1+R).Q.S_i$$

$$S_i = \frac{S_o + R.S}{(1+R)}$$

BOD$_5$ Profile in a Trickling Factor



Filter Type	Filter Medium	OLR, kg/m^3/d	HLR, m^3/m^2/d	% Removal	Depth (m)	R
Low Rate	Rock, Slag	0.1 - 0.3	1 - 4	80 - 85	1.8 - 3	0
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High Rate	Rock	1.2 - 3	40 - 90	65 - 85	2 - 5	1 - 4
Super High Rate	Plastic	3 - 4	60 - 120	65 - 80	4 - 12	1 - 4
Roughing	Plastic	4 - 6	60 - 180	40 - 65	4 - 12	1 - 4

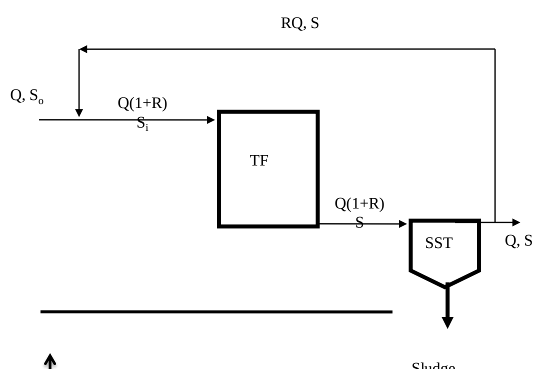
Design of a Trickling Filter

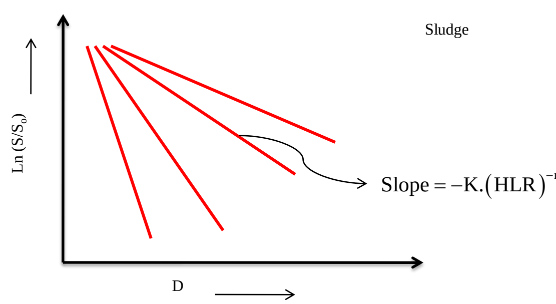
$$\frac{S}{S_i} = \mathrm{Exp}\left[-K.D.\left(\frac{Q.(1+R)}{A}\right)^{-n}\right]$$

K, n: Treatability Constants
D: Depth of Filter
A: Filter Cross-Sectional Area

To determine K and n:
Get data on percent BOD$_5$ Remaining [(S/S$_o$).(100)] through pilot tests (without recycle) conducted at various HLR values

Depth (m)	HLR (L/min/m²) 20	40	60	80
0.50	50	70	75	82
1.00	40	50	60	60
1.50	25	30	40	50
2.00	15	20	30	40

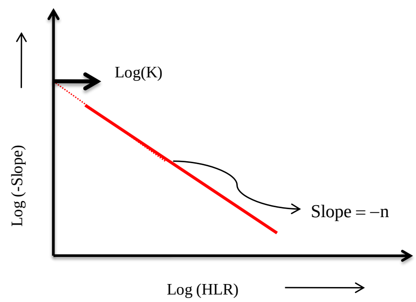




$$\mathrm{Ln}\left[\frac{S}{S_o}\right] = \left[-K.(HLR)^{-n}\right].D$$

$$Slope = -K.(HLR)^{-n}$$

$$Log(-Slope) = Log(K) - n.Log(HLR)$$



Without recycle, Hydraulic Loading Rate (HLR) =

$$\frac{Q}{A} = \frac{(0.6).10^3}{150} = 4\,m^3/m^2/d \qquad \text{(not okay)}$$

So, let R = 3; Hence, HLR =

$$\frac{(Q+RQ)}{A} = \frac{(0.6+1.8).10^3}{150} = 16\,m^3/m^2/d \qquad \text{(okay)}$$

Example Problem

A tricking filter with the following dimensions is available. Depth: 2 m, Surface area: 150 m^2. The media consists of stones of 7-10 cm diameter. This filter will be used to treat 0.6 MLD wastewater with BOD_5 = 300 mg/L. Based on this information, calculate the expected BOD_5 removal efficiency.
K = 1.36; n = 0.5

Volume of trickling filter = D.A = (2).150 = 300 m^3

Organic Loading Rate (OLR) =

$$\frac{Q.S_o}{V} = \frac{(0.6.10^6).(300)}{300.10^6} = 0.6\,Kg/m^3/d$$

(okay for intermediate rate)

BOD Removal Efficiency:

$$\frac{S}{S_i} = \exp[-k..D.(HLR)^{-n}] \qquad S_i = \frac{S_o + R.S}{(1+R)}$$

$$\frac{S}{S_i} = \exp[(-1.36).(2).(16)^{-0.5}] = 0.507 = \frac{S.(1+R)}{S_o + R.S}$$

$$0.507.S_o + 1.520.S = 4.S \qquad \frac{S}{S_o} = 0.204$$

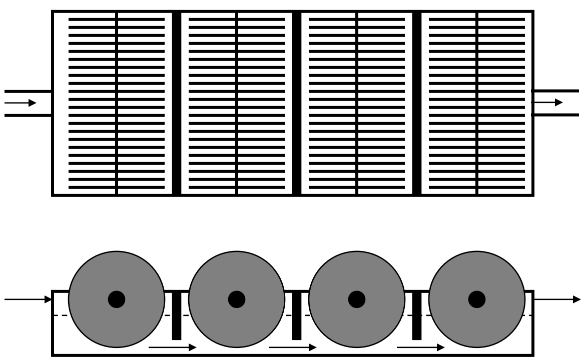
$$\left(1 - \frac{S}{S_o}\right) = 1 - 0.204 = 0.796$$

79.6% Removal

Commonly used aerobic bio-reactors (other than ASP) are,

1. Trickling Filter (TF): attached growth, does not require mechanical aeration
2. Rotating Biological contactor (RBC): attached growth, does not require mechanical aeration
3. Sequential Batch Reactor (SBR): suspended growth, does not require separate SST
4. Mixed Bed Biofilm Reactor (MBBR): Hybrid suspended-attached growth
5. Membrane Bio-Reactor (MBR) : suspended growth, membrane instead of SST

Rotating Biological Contactor



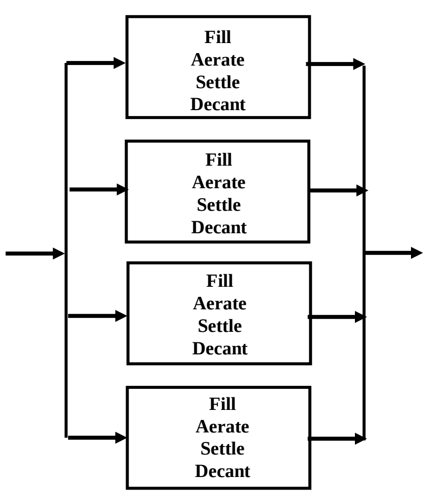
- Attached growth of biomass on the rotating disks.
- The attached biomass absorbs substrate when the disks are submerged in water
- The attached biomass absorbs oxygen when exposed to air
- The attached biomass sloughs off when the layer on the disk becomes too thick
- Hence there is some suspended biomass in the tank which is separated in the SST provided after the RBC.
- Supernatant from the SST is the treated effluent

Main Characteristics

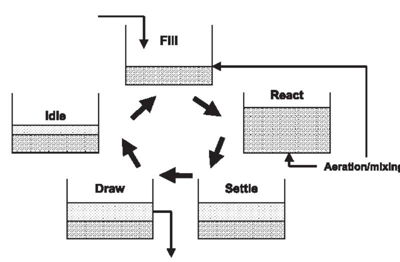
1. Attached growth system
2. High biomass concentration maintained in the reactor
3. High level of nitrification in multiple stage systems
4. Hydraulic retention times are generally higher than ASP, hence comparatively larger tank volumes are required.
5. Used mainly for treating small flows.

Actual Installation

Sequential Batch Reactor (SBR)



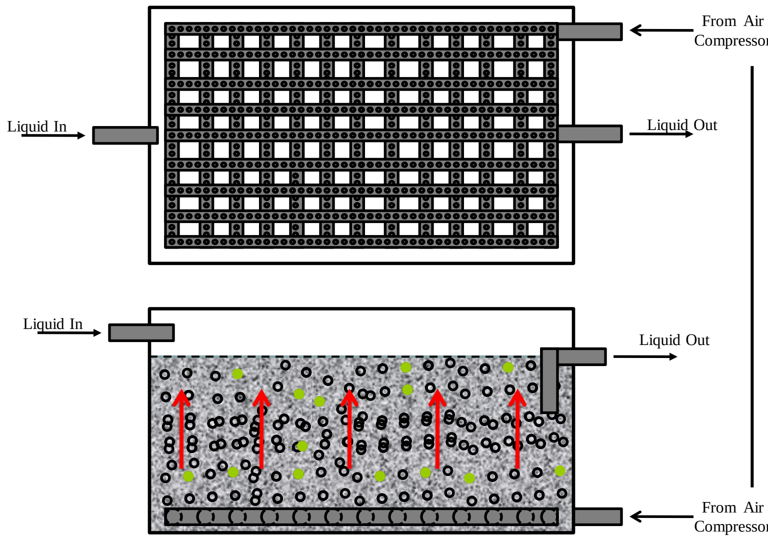
Operating Cycle of SBR



Main Characteristics

1. Suspended growth sequential batch process
2. Fine bubble aeration
3. High quality effluent
4. Moving system of weirs/decanters required
5. Complex operation requiring electronic controls

Mixed Bed Biofilm Reactor (MBBR)

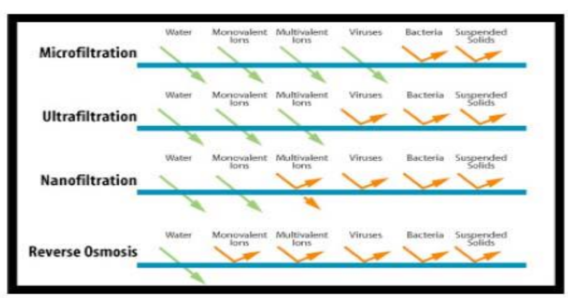


Addition of carriers can increase biomass (MLVSS) in the aeration tank without increasing the concentration of inert substances `

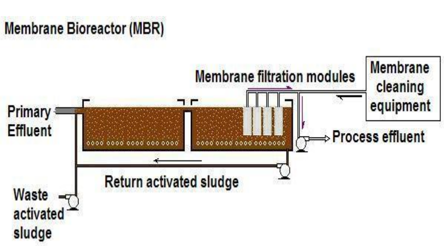
Membrane Bioreactors

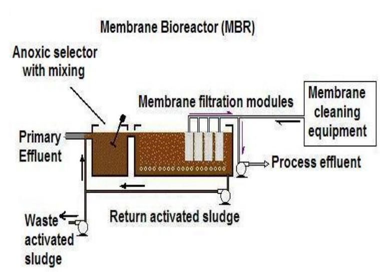
Similar to the conventional activated sludge system, except that the SST has been replaced by a membrane (micro-filtration) module.

This ensures better solid-liquid separation and hence higher quality effluent fit for recycling is guaranteed on a regular basis.

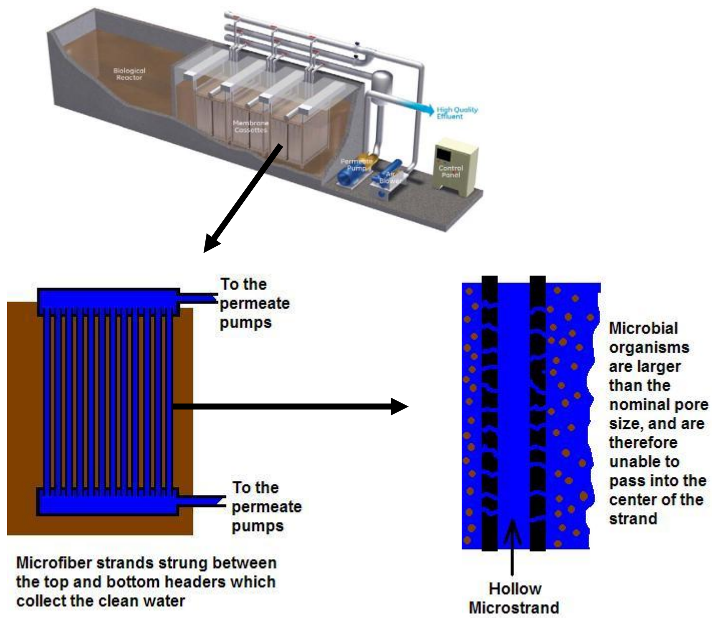


Membrane Process Characteristics

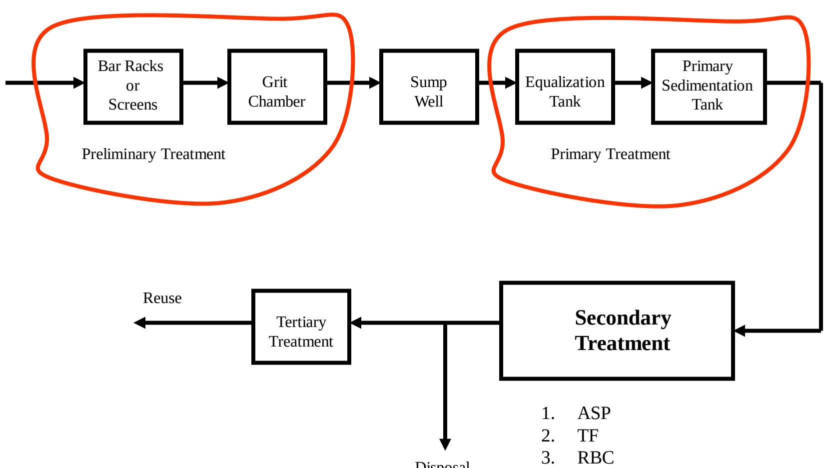




Microbial Bioreactor (Continued)

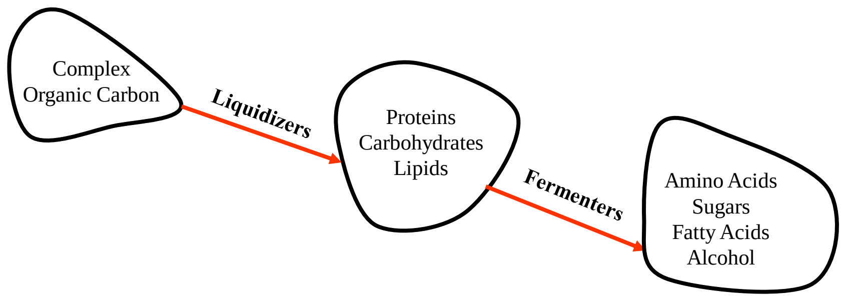


Wastewater Treatment Process Schematic



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

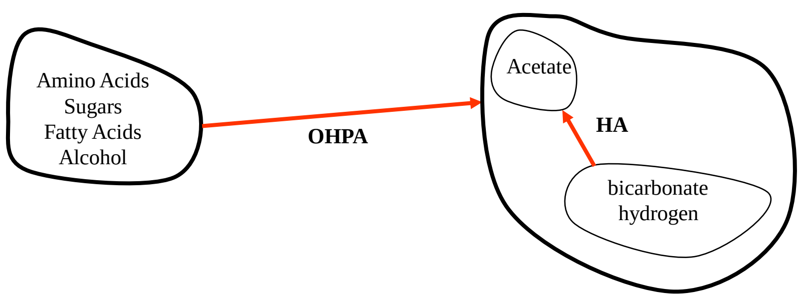
Biochemistry of Anaerobic Biodegradation….Fermentation



Liquidizers: Anaerobic, Heterotrophic
Energy Source: Complex Organic Carbon
Food Source: Organic Carbon
Electron Acceptor: Organic Carbon
By products: Proteins, Carbohydrates, Lipids

Fermentors: Anaerobic Heterotrophic
Energy Source: Organic Carbon (Proteins, Carbohydrates, Lipids)
Food Source: Organic Carbon
Electron Acceptor: Organic Carbon
By-products: Amino Acids, Sugars, Fatty Acids, Alcohol

Biochemistry of Anaerobic Biodegradation….Acid Formation

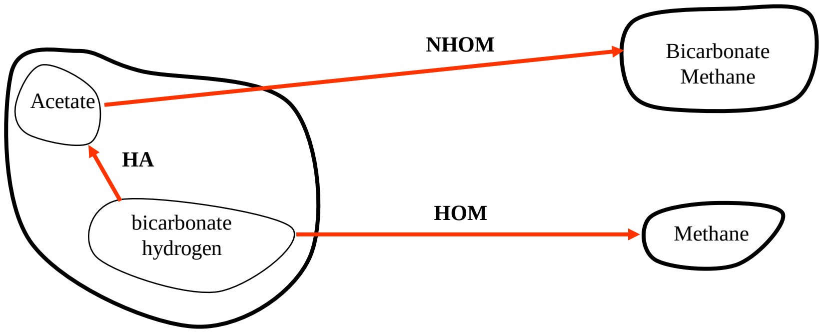


Obligate Hydrogen Producing Acetogens (OHPA): Anaerobic Heterotrophic
Energy Source: Organic Carbon
Food Source: Organic Carbon
Electron Acceptor: H_2O
By-products: Acetate, inorganic carbon, hydrogen

Homoacetogens (HA): Anaerobic, Autotrophic
Energy Source: H_2
Food Source: Inorganic Carbon
Electron Acceptor: Inorganic Carbon
By-product: Acetate
Equation:

$$2HCO_3^- + 4H_2 + H^+ \rightarrow CH_3COO^- + 4H_2O$$

Biochemistry of Anaerobic Biodegradation….Methane Formation



Acetoclastic or Non-Hydrogen Oxidizing Methanogens (NHOM): Anaerobic, Heterotrophic

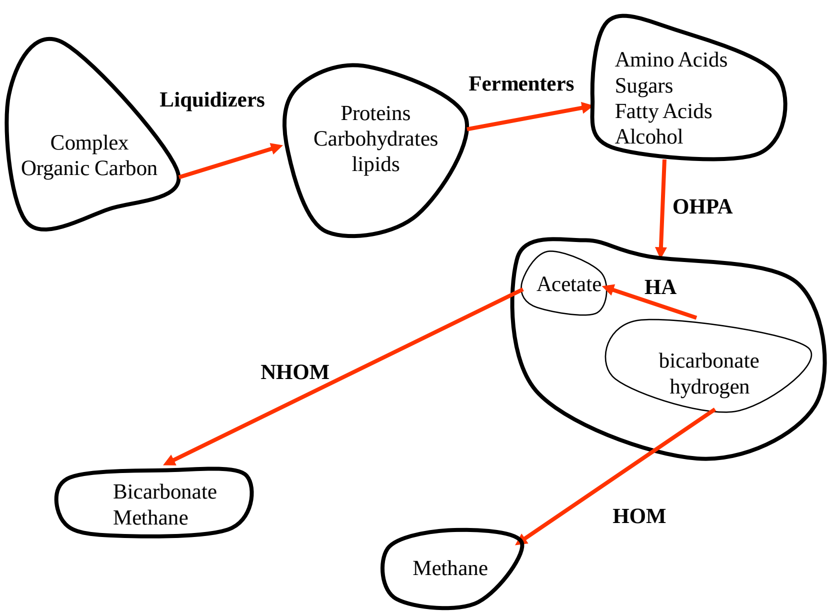
Energy Source:	Acetate
Food Source:	Acetate
Electron Acceptor:	Acetate
By-products:	Methane and Bicarbonate
Equation:	$CH_3COO^- + H_2O \rightarrow HCO_3^- + CH_4$

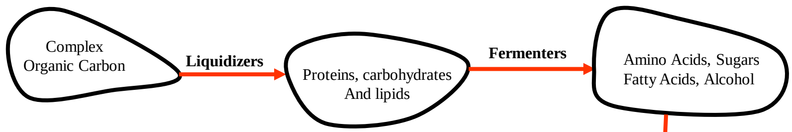
Hydrogen Oxidizing Methanogens (HOM): Autotrophic Anaerobic

Energy Source:	H_2
Food Source:	Inorganic Carbon
Electron Acceptor:	Inorganic Carbon
By-product:	Methane
Equation:	$HCO_3^- + 4H_2 + H^+ \rightarrow CH_4 + 3H_2O$

Complex Organic Matter is Converted to Methane and Inorganic Carbon

Biochemistry of Anaerobic Biodegradation…….Overall





Importance of Hydrogen

Increase in H_2 partial pressure has a negative impact on OHPA activity

This leads to the accumulation of higher aliphatic acids and reduces the production of acetate

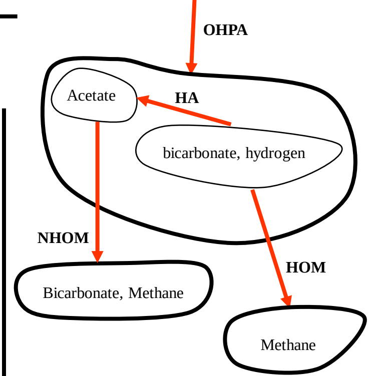
Lower acetate means lower methanogenic activity.

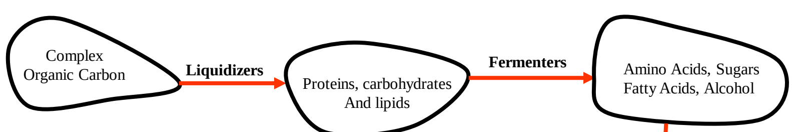
Propionate to acetate → below 10^{-4} atm. H_2 partial pressure

Butryate to acetate → below 10^{-4} atm. H_2 partial pressure

Ethanol to acetate → below 1 atm. H_2 partial pressure

Lactate to acetate → below 1 atm. H_2 partial pressure





Importance of Reactor pH
Methanogens are active in the pH range of 7.5 – 8.5

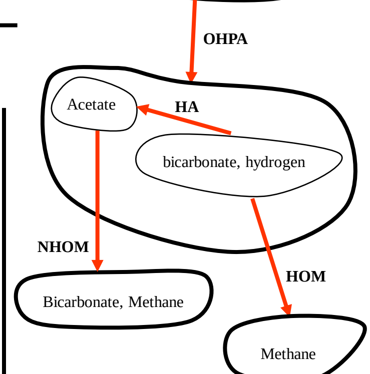
Lowering of pH due to accumulation of acids disrupts methane production

What Happens to the Oxygen Demand of Wastewater ??

Oxygen demand is removed from the wastewater and transferred primarily to the methane gas.

Thus oxygen demand is not satisfied, merely transferred to the gaseous phase (methane).

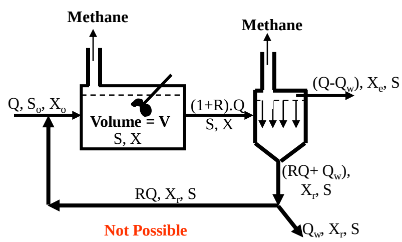
Thus, COD of methane gas produced ~ COD reduction of wastewater



Bio-kinetic Constant Comparison: Aerobic vs Anaerobic Growth

Aerobic (BOD basis)	Anaerobic (COD basis)
$Y_T = 0.50$	$Y_T = 0.04$
$K_d = 0.05$ / d	$K_d = 0.015$ / d
$K_s = 40$ mg/L	$K_s = 2224.[10^{0.046(35-T)}]$
	$(K_s)_{20} = 10892$ mg/L
	$(K_s)_{35} = 2224$ mg/L
	$(K_s)_{45} = 771$ mg/L
$q_m = 4$ /d	$q_m = 6.67.[10^{-0.015(35-T)}]$
	$(q_m)_{20} = 3.97$ /d
	$(q_m)_{35} = 6.67$ /d
	$(q_m)_{45} = 9.42$ /d

Suspended Growth Anaerobic Reactor: Similar to ASP



<u>Maximum Removal</u>

Assuming $\mu = 0$; $q = k_d/Y_T$

$S = q.K_s/(q_m - q)$

q(aerobic) = 0.05/0.5 = 0.1 / d

S = (0.1).(40)/(4 – 0.1) = 1.02 mg/L

q(anaerobic) = 0.015/0.04 = 0.375 / d

S (at 45°C) = 0.375.(771)/(9.42 - 0.375) = 32 mg/L

S (at 35°C) = 0.375.(2224)/(6.67 – 0.375) = 132 mg/L

S (at 20°C) = 0.375.(10892)/(3.97 – 0.375) = 1136 mg/L

Additionally,

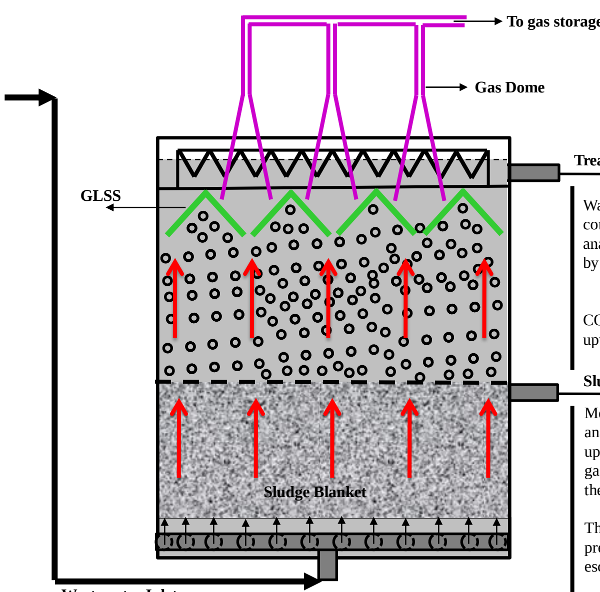
Anaerobic sludge has very poor settling characteristics.

Due to unfavorable biodegradation and poor settling characteristics of anaerobic sludge means that suspended growth anaerobic reactors are not feasible for domestic wastewater treatment due to poor effluent quality.

However, such reactors are used extensively for biological sludge digestion and industrial wastewater treatment.

Up-flow Anaerobic Sludge Blanket (UASB) Reactor

This is neither a suspended growth, nor an attached growth system. It is a "retained biomass" reactor.



Wastewater influent to the reactor from bottom.

Primary sedimentation of the influent wastewater is optional

Treated Water Outlet

Wastewater passes through a 'sludge blanket' containing mainly inert material with attached anaerobic bacteria maintained in a fluidized state by the up-flowing sewage.

COD reduction of sewage taken place during the upward flow through the sludge blanket.

Sludge Outlet

Methane and carbon dioxide is formed due to anaerobic biodegradation. The gas bubbles travel upwards through the reactor and are collected by the gas domes. COD reduction of wastewater ~ COD of the methane produced.

The gas-liquid-solid separator (GLSS) helps to prevent sludge particles attached to gas bubbles from escaping from the reactor effluent.

Sludge is wasted once a day or even more infrequently.

Advantages of UASB Reactor

- No aeration required. Hence energy requirements are low.
- Operation of the reactor is straight forward. Skilled manpower not required.
- Low sludge production.

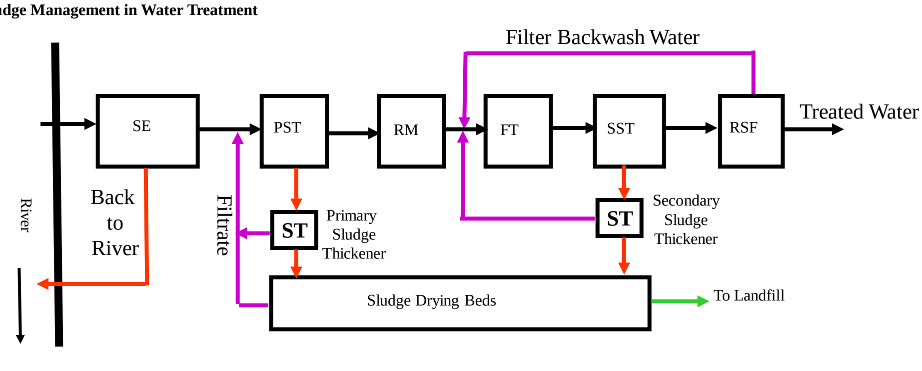
Disadvantages of UASB Reactor

- Poor effluent quality. Polishing treatment required.
- Unreliable performance due to periodic sludge escape with the treated effluent.
- Methane recovery is low for low strength domestic wastewater and hence unviable.
- Ineffective at low temperatures. External heating needs to be provided.
- No nitrification of the effluent.
- Poor removal of pathogens
- sulfate present in wastewater is partially converted to sulfide through the action of sulfur reducing bacteria present in UASB reactor. Metal sulfides are precipitated, which often gives the effluent a black color. Production an escape of H_2S gas produced odor problems. H_2S gas is highly corrosive. Production of sulfide reduces methane formation, hence COD removal by methane formation is reduced. The sulfide remains in water and is exerted instantaneously (through consumption of DO) when released in natural water bodies.

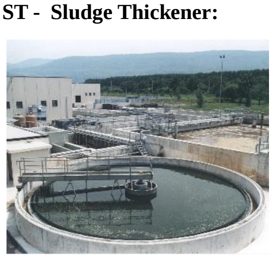
On the whole, UASB treatment of domestic wastewater is infeasible unless accompanied by comprehensive aerobic polishing treatment.

UASB treatment is not recommended for high sulfate wastewater due to the associated problems with sulfide production

Sludge Management in Water Treatment



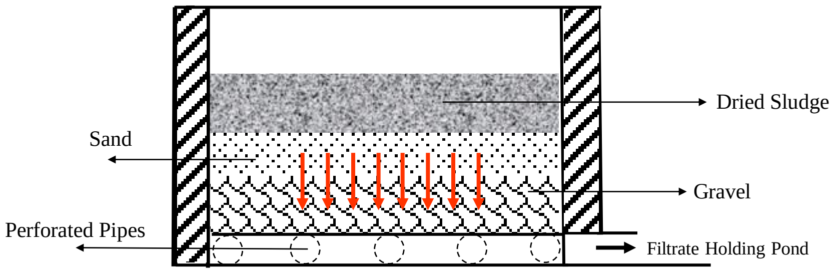
ST - Sludge Thickener:



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.

Sludge Drying Beds



The solids content of the sludge influent to sludge drying bed = ~ 4 percent

Solids loading to sludge drying bed = 1.5 kg solids (dry basis) /m^2 / cycle

The solids content of the dried sludge is ~ 30 - 40 percent.

Drying time ~ 2 weeks

Dried primary sludge can be used for land application

Dried secondary sludge must be disposed in a land fill

Alternatives to Sludge Drying Beds: Centrifuge

Sludge Dewatering Centrifuge

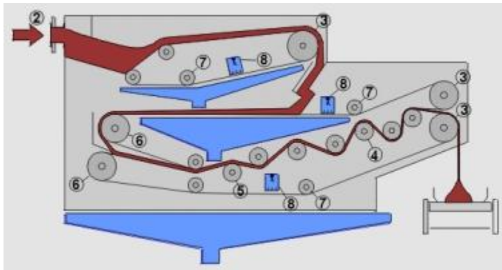
Video Link

http://www.youtube.com/watch?v=FhS5vN4r5LA&feature=player_detailpage

https://www.youtube.com/watch?v=_IHmvh3pe9o

https://www.youtube.com/watch?v=ao_62cLsVBs

Alternatives to Sludge Drying Beds: Belt Filter Press



Schematic

Actual Machine

The sludge is put between two fabric filters and passed through rollers. Water in the sludge is squeezed out and the dried sludge is collected.

Dewatered Sludge
(~40 percent solids)

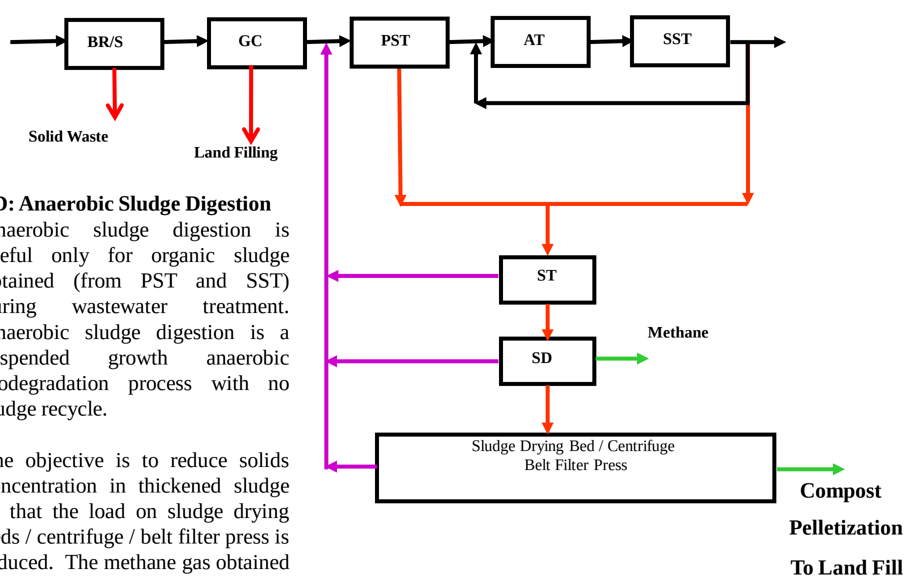
Video Link

https://www.youtube.com/watch?v=6voXE1HxYsY

https://www.youtube.com/watch?v=DSS6F0UlGlQ

https://www.youtube.com/watch?v=KAUB4QQvZqI

Sludge Management in Wastewater Treatment

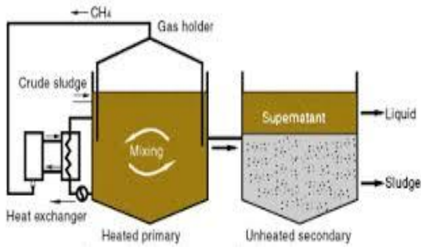


SD: Anaerobic Sludge Digestion

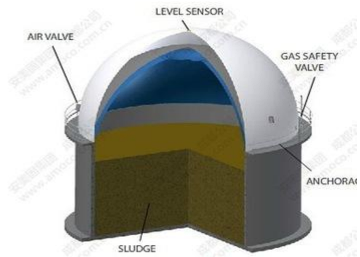
Anaerobic sludge digestion is useful only for organic sludge obtained (from PST and SST) during wastewater treatment. Anaerobic sludge digestion is a suspended growth anaerobic biodegradation process with no sludge recycle.

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

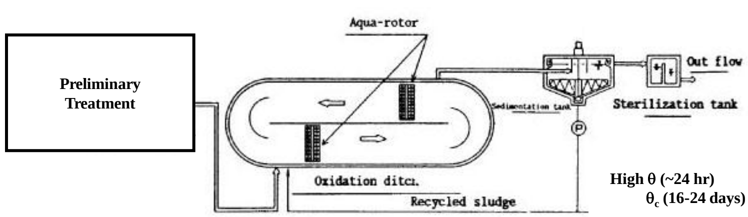
Anaerobic Sludge Digester

Gas Storage

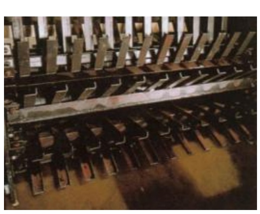
Natural Methods of Wastewater Treatment: Applicable in Rural Areas

These treatment technologies try to mimic BOD and nutrient removal processes in nature. When properly designed and operated, such treatment methods can produce treated effluent similar to secondary treated effluent in other engineered systems. In some cases the effluent quality is even better, i.e., closer to that produced by tertiary treatment in engineered systems. However such treatment processes require large land area and hence are only feasible in places where relatively cheap land is available, i.e., in rural areas.

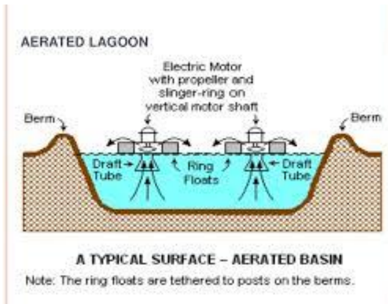
Wastewater Treatment in Oxidation Ditch



High θ (~24 hr)
θ_c (16-24 days)



Wastewater Treatment in Aerobic Lagoon: ASP without Recycle



Aerated lagoon is similar to ASP without recycle

Pond Systems for Wastewater Treatment

Sewage after preliminary treatment is passed through a series of ponds for treatment. Various types of ponds are possible,

1. Anaerobic Ponds
2. Facultative Ponds
3. Oxidation Ponds
4. Polishing Ponds
5. Fish Pond

Pond Systems for Wastewater Treatment

Preliminary Treatment → **Facultative Pond OR Aerobic Lagoon** → **Oxidation Pond**

Algae Harvesting

Oxidation Pond → **Polishing Pond OR Fish Pond**

Fish Harvesting

Effluent (better than secondary treated Effluent)

Facultative Pond: Deep Pond (3-4 m). Solids settle to the bottom and are degraded anaerobically. Top part of the pond is aerobic due to algal photosynthesis. Some aerobic biodegradation of BOD takes place on the top part. Hydraulic retention time is ~ 2 days.

Aerobic Lagoon: Natural aeration in a facultative pond may be enhanced through the provision of surface aerators. Such a system is known as an Aerobic Lagoon. Hydraulic retention time is ~ 12-24 hours.

Oxidation Pond: Shallow Pond (< 1 m deep). Completely aerobic. Oxygen provided for aerobic biodegradation of BOD through algal photosynthesis. Both BOD and nutrient removal occur. Hydraulic retention time is ~ 4-5 days.

Polishing Pond: Shallow Pond. Completely aerobic. Fish cultivation with algae and bacteria as fish food. Fish harvesting is possible. Hydraulic retention time is ~ 1-2 days.

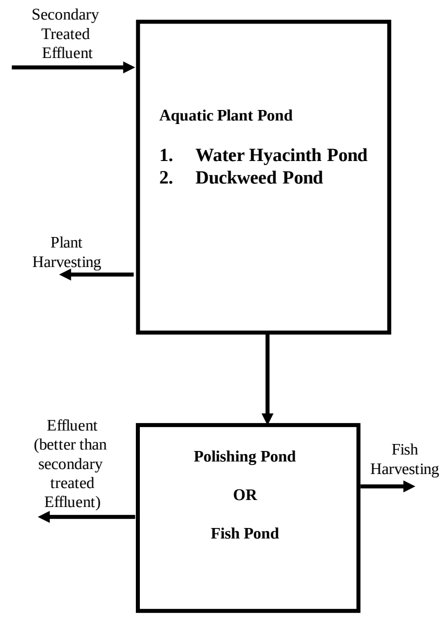
Treated Effluent: Treated effluent is low in both BOD and nutrients, but may contain some suspended particulate matter.

Aquatic Plant Ponds for Nutrient Removal

Aquatic Plant Pond:
Aquatic plants take nutrient from the water and carbon dioxide from air. They release oxygen directly into air. Hence DO levels are generally low in aquatic plant ponds. Such ponds are however, good for nutrient removal from water. Depth is generally 3-4 m. HRT is 1-2 days. Regular harvesting of plants is essential.

Polishing Pond:
Algal and bacterial action in this pond, along with the presence of fish result in removal of residual BOD, nutrients and solids for the effluent.

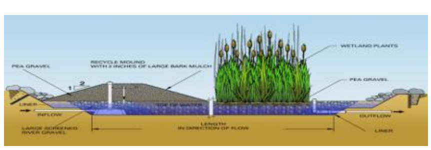
Effluent is almost equivalent to tertiary treated effluent from an engineered process. The effluent may still have some solids.



Constructed Wetlands
Sewage after preliminary treatment is allowed to flow through a constructed wetland. Various natural processes in the wetland, i.e., sedimentation, filtration, aerobic and anaerobic biodegradation, nitrification, denitrification, algal growth, etc. combine to treatment the influent sewage to almost tertiary level.

Constructed wetlands can be used as stand-alone treatment systems or in conjunction with pond systems (as a replacement for polishing ponds). Also, constructed wetlands may be used for tertiary treatment of secondary treated effluent, mainly for nutrients removal and filtration.

As with other natural treatment systems, the HRT (~ 5-10 days) is high requiring large land area.



Subsurface Flow

Subsurface + Overland Flow

Actual picture of a constructed Wetland actually in Operation

Topic 1: Design of an Oxidation Pond:

1 MLD of wastewater to be treated in an oxidation pond. The influent BOD_5 (S_o) in an oxidation pond is 80 mg/L. Desired effluent BOD_5 (S) is 5 mg/L.

For Oxidation Pond:

K = 0.1 L/mg/d, where K is the first order microbial substrate utilization rate ($K = q_m/K_s$), (i.e.., assuming $K_s >> S$)

Y_T = 0.5 mg/mg; K_d = 0.05 /d (based on BOD_5)

Formula for microbial biomass: $C_{60}H_{87}O_{23}N_{12}P$

Average intensity of solar radiation: 150 calories/cm²/d

Solar energy utilization efficiency for algae: 6 percent

Energy requirement of algal bio-mass: 6000 calories/g algae

Equation for algal photosynthesis:

$$106CO_2 + 16NO_3^- + HPO_4^{--} + 122H_2O + 18H^+ \rightarrow C_{106}H_{263}O_{110}N_{16}P + 138O_2$$

Solution:

$q = K.S = 0.1.(5) = 0.5$ /d; $\mu = 0.20$ /d; $\theta = \theta_c = 1/\mu = 5$ days

$X = (S_o - S)/(\theta.q) = (80\text{-}5)/(0.5)(5) = 30$ mg/L; Sludge Production (ΔX) = Q.X = 30 kg/d

Oxygen Requirement = $1.5.Q.(S_o - S) - 1.42.(\Delta X) = 69.9$ kg/d; $V = \theta.Q = 5000\ m^3$

Assuming depth = 0.5 m; Surface Area (A) = 10000 m²

Algae Production = $(150).(10^4)(0.06)/6000 = 15\ g/m^2/d$; Total algae production = 150 kg/d

from, photosynthesis equation, 1.3 kg oxygen production / kg algae production

Oxygen Production = 1.3.(150) = 195 kg/d

Assuming 50 percent of the algal oxygen produced is available for microbial respiration,

Oxygen available = 97.5 kg/d (>> oxygen required)

Topic 2: Advantages/Disadvantages of Anaerobic Treatment

Advantages of Anaerobic Treatment

- No aeration required. Hence energy requirements are low.
- Operation of the reactor is straight forward. Skilled manpower not required.
- Low sludge production.

Disadvantages of Anaerobic Treatment

- Poor effluent quality. Polishing treatment required.
- Unreliable performance due to periodic sludge escape with the treated effluent.
- Methane recovery is low for low strength domestic wastewater and hence unviable.
- Ineffective at low temperatures. External heating needs to be provided.
- No nitrification of the effluent.
- Poor removal of pathogens
- sulfate present in wastewater is partially converted to sulfide through the action of sulfur reducing bacteria present in UASB reactor. Metal sulfides are precipitated, which often gives the effluent a black color. Production an escape of H_2S gas produced odor problems. H_2S gas is highly corrosive. Production of sulfide reduces methane formation, hence COD removal by methane formation is reduced. The sulfide remains in water and is exerted instantaneously (through consumption of DO) when released in natural water bodies.

On the whole, anaerobic treatment of domestic wastewater is infeasible unless accompanied by comprehensive aerobic polishing treatment.

Anaerobic treatment is not recommended for high sulfate wastewater due to the associated problems with sulfide production

Topic 3: 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.

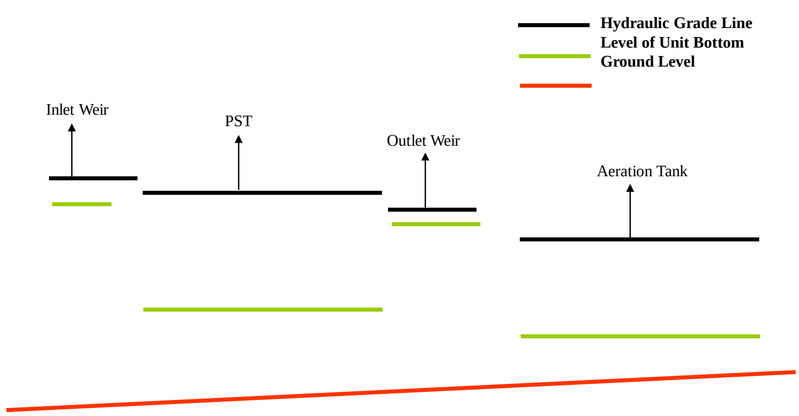
A treatment plant is generally designed to keep pumping to a minimum. Pumping is generally done at the beginning of the treatment train. Water is then expected to flow through various treatment units by gravity.

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.

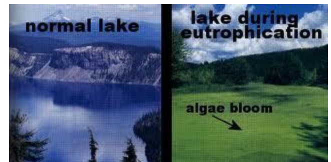
Topic 4: **Consequences of Pollutant Loading to Natural Water Bodies**

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.

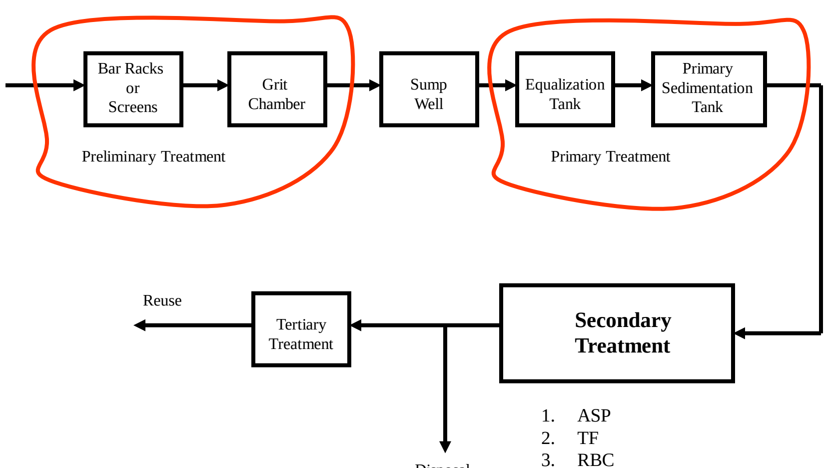
Consequences of Nutrient Addition in Lakes



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

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

1. Nutrient Removal
    a. Nitrogen Removal: Denitrification
    b. Phosphorus Removal: Precipitation as Calcium Phosphate

2. Suspended Solids Removal
    a. Rapid Sand Filtration/Pressure Filtration
    b. Membrane Filtration: Microfiltration

3. Micro-pollutants Removal
    a. Activated Carbon Adsorption
    b. Ozonation/other Advanced Oxidation processes (AOPs)

4. Dissolved Inorganic Solids Removal
    a. Ion Exchange
    b. Reverse Osmosis

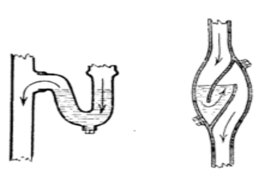
5. Disinfection
    a. Chlorination/other disinfectants
    b. UV disinfection

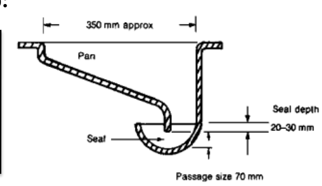
Sanitation

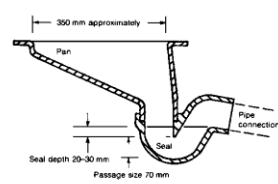
A WHO Study Group in 1986 formally defined ‘sanitation’ as "the means of collecting and disposing of excreta and community liquid wastes in a hygienic way so as not to endanger the health of individuals and the community as a whole".

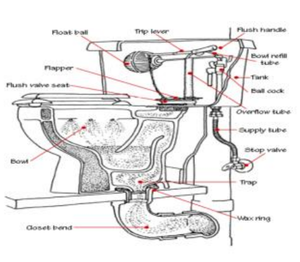
Based on the above definition, some types of sanitation are called “improved” while others are “unimproved”. Wet latrines are an important component of “improved” sanitation. Wet latrines are of two types, 1) flush latrines, 2) pour flush latrines.

Wet Latrines must have a P-Trap:









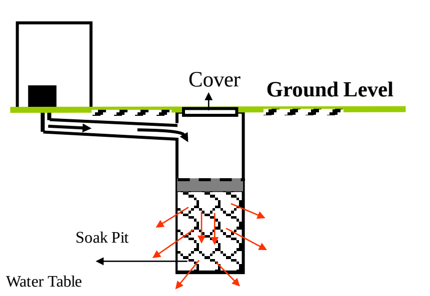
A P-Trap fitted to a toilet provides a water seal, which helps in odor control. Toilets with P-traps can easily be integrated with a flushing system.

Flush Latrines: Back-End
A flush toilet with a P-trap and connected to the sewer system is the most desired alternative. However, such systems may always not be possible in many areas due to the absence of a sewer system.

Sewage from flush latrines flow into natural depressions / ponds in the locality and evaporate or infiltrate into the ground. This creates an unhealthy and aesthetically depressing scenario.

Sewage from flush latrines flow in surface drains and then into natural rivulets ('nalas') and finally into rivers. Surface drains often do not have the slope to provide the required self-cleansing velocity to the flowing sewage leading to solids deposition and choking / overflowing of the drain. Such drains have to be cleaned regularly.

The Soak-Pit

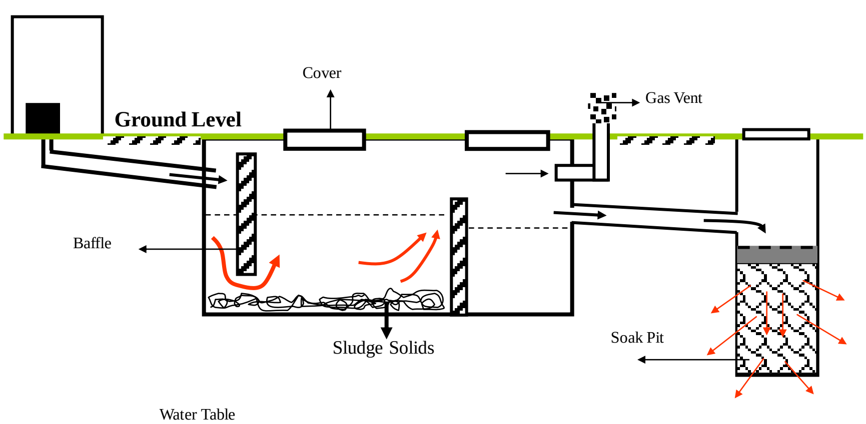


Inadequate "back-end" solution.

The soak pit chokes after some time.

Soaking sewage may cause groundwater pollution.

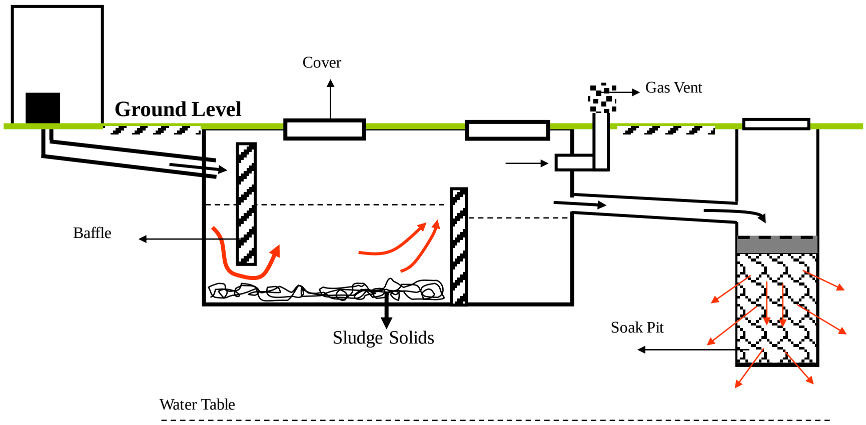
Flush Toilets with Septic Tank and Soak Pits



Inadequate "back-end" solution.
Septic Tank Cleaning issues
The soak pit chokes after some time.
Soaking sewage may cause groundwater pollution.

The effluent from septic tank is called septage. When soak pits are choked, the septage often flows over ground. Sometimes the septage outlet is connected to surface drains. Septage management is a huge problem

Flush Toilets with Septic Tank and Soak Pits



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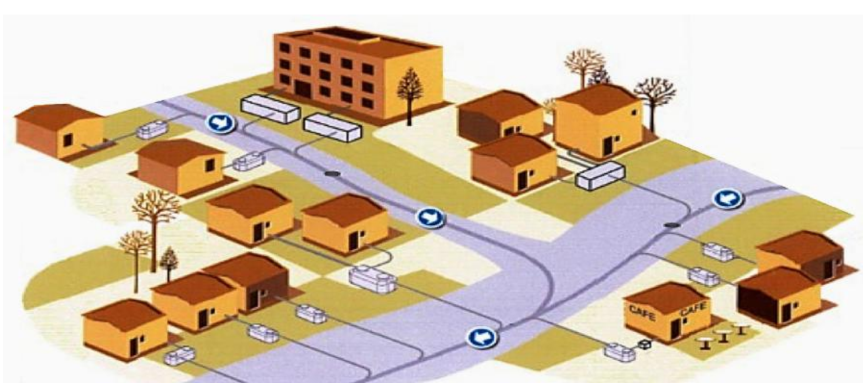
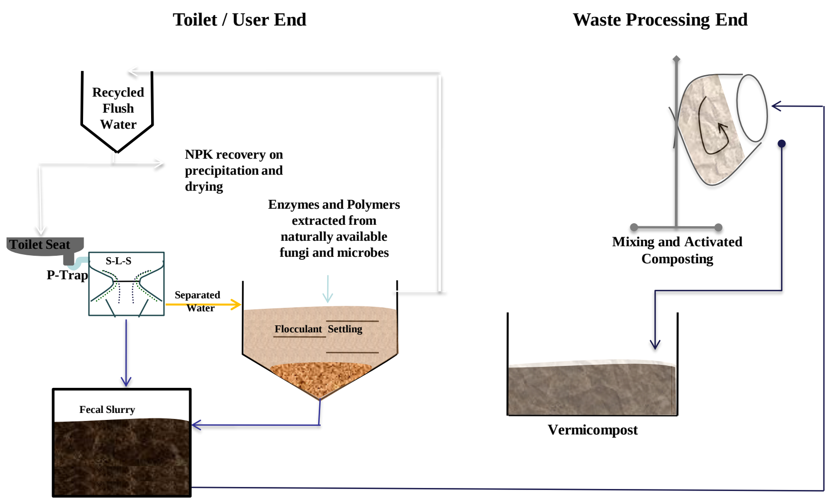
Small-Bore Sewer System:
Small bore sewer system, also known as solids free sewer, divides the sewage into two components at the source itself using an interceptor (similar to a septic tank). One is the decanted liquid fraction (supernatant of the sewage) and the other is settled sewage solids (sludge). The solids which accumulate in the interceptor tanks should be removed periodically for safe disposal. Sewer lines are designed to receive only the liquid portion of household wastewater for off-site treatment and disposal.

SBS system requires small diameter piping because it conveys only liquid, hence it is economical. Because of the lower costs of construction and maintenance and the ability to function with little water, small bore sewers can be used where conventional sewerage would be inappropriate.



Flush Toilets in Slums / Rural Areas

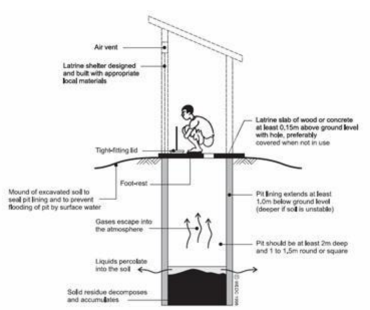
Zero-Discharge Toilet System (ZDTS)



Adequate back-end solution

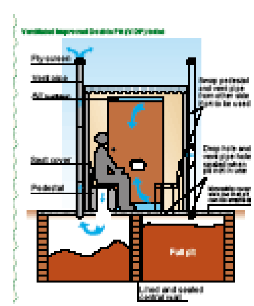
Dry Sanitation

Single Pit Dry Latrine



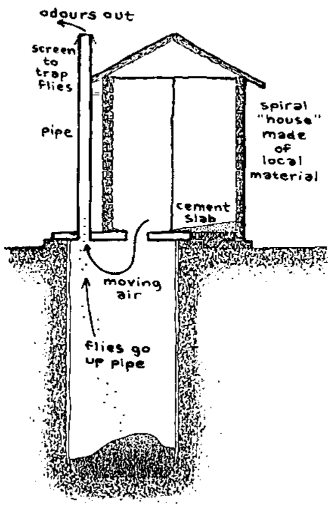
Such toilets should be used for a maximum of six months, then closed for six months, after which time the toilet contents should be removed and the toilet put back into operation. The toilet contents may be composted after mixing with garden waste, agricultural wastes or organic solid wastes.

Double Pit Dry Latrine

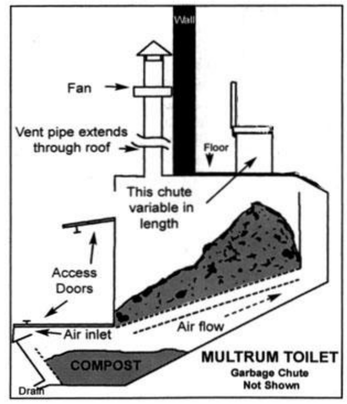


Such toilets contain two pits. One pit is used for six month and then closed for six months. During this time the second pit is used. The first pit is put back into operation after six months and the second pit is closed.

Ventilated Improved Pit (VIP) Latrine



Composting Toilet



Urine Diversion Dehydration Toilets (UDDT)
This is a type of dry toilet which can be adapted to accommodate anal cleaning water use.

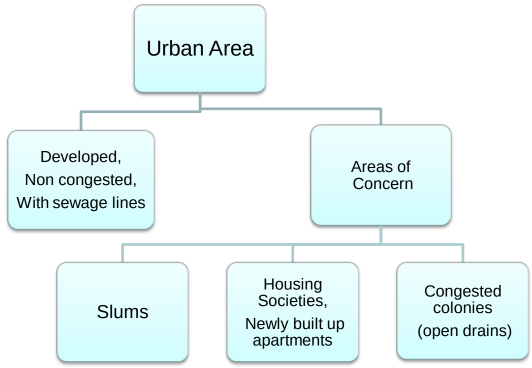
UDDTs divert all liquids i.e. urine and anal cleansing water, from the feces to keep the processing chamber contents dry. UDDTs make use of desiccation (dehydration) processes for the hygienically safe on-site treatment of human excreta. Adding wood ash, lime, sawdust, dry earth etc. after defecation helps in lowering the moisture content and raising the pH. The system thus creates conditions of dryness, raised pH and pathogen die-off.

If wet anal cleansing habits prevail in a community, anal cleansing water must be diverted (by providing a separate washbowl) from the feces.

The main disadvantage is that some discipline is required during defecation. This has made adaptation of such toilets difficult.

Adequate back-end solution

Different Types of Human Settlements in Urban Areas



Sanitation solutions are different for each of these areas.

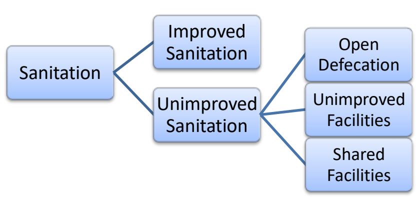
Different Types of Human Settlements in Rural Areas

- Congested settlements ("Mohallas")
  - Relatively well-off
  - Relatively poor
- Dispersed settlements
  - Relatively well off
  - Relatively poor

Sanitation solutions are different for each of these areas.

Unimproved Sanitation

According to World Health Organization, Sanitation can be classified as 'improved' and 'unimproved', as shown below.



Unimproved Sanitation
Systems which are unhygienic and/or lack proper technological inputs to facilitate a minimum comfort level are termed as unimproved sanitation. Following systems fall into this category:

Open Defecation
When human feces is disposed of in fields, forests, bushes, open bodies of water, beaches or on railway tracks or other open spaces or disposed of with solid waste.

Unimproved Facilities
These facilities do not ensure hygienic separation of human excreta from human contact. Unimproved facilities include pit latrines without a slab or platform, hanging latrines and bucket latrines.

Shared Sanitation Facilities
Sanitation facilities of an otherwise acceptable type shared between two or more households. Only facilities that are not shared or not public are considered improved.

Indian Sanitation Scenario

		Year	1990	2000	2011
		Population (x1000)	873785	1053898	1241492
		Percentage of Urban Population	26	28	31
Urban		Improved	50	54	60
	Unimproved	Shared	17	18	20
		Unimproved	5	6	7
		Open Defecation	28	22	13
Rural		Improved	7	14	24
	Unimproved	Shared	1	3	4
		Unimproved	2	4	6
		Open Defecation	90	79	66
National		Improved	18	25	35
	Unimproved	Shared	5	6	7
		Unimproved	3	5	6
		Open Defecation	74	63	50

A critical assessment of traditional sanitation practices and present sanitation conditions in India leads to following observations,

Open defecation cannot be recommended under any circumstances. This practice does not allow defecation with dignity and privacy and may be unhygienic if done improperly.

Toilets that need daily manual cleaning are not recommended under any circumstances since they offend basic human dignity and contravene the Manual Scavenging Act.

Hanging toilets, i.e., toilet constructed directly over water bodies or cesspools cannot be recommended under any circumstances. Such toilets create extremely unhygienic conditions.

Indian practice of using anal cleansing water renders the use of pit latrines difficult. The pits cannot be maintained dry and this leads to odor and fly problems. Defecation under such conditions becomes unhygienic and uncomfortable, and people soon abandon pit latrines and revert to open defecation.

Use of Urine Diversion and Dehydration Toilets (UDDT) is difficult, since the present models require following a certain discipline during defecation. An improved version of UDDT, specially attuned to Indian conditions is required.

Flush and pour-flush latrines connected to open drains are problematic. Since the open drains follow the contours of the ground, in flat areas slopes cannot be maintained for flow of sewage at self-cleansing velocities. This leads to the deposition of sewage solids in the drain and subsequent choking and overflowing of the drains, creating unhygienic conditions.

Flush and pour-flush latrines connected directly to soak pits or connected to septic tanks followed by soak pits is problematic in congested areas, especially when water table is high. The chances of groundwater pollution are very high under such conditions.

Flush and pour-flush latrines connected to small bore sewer system may be a viable option in Indian context.

Shared or communal toilet facilities must be given due importance. Such facilities, which are conceptually different from public toilets, may be the only workable solutions under certain conditions.

The cost of adequate sanitation in India, i.e., when both ‘front-end’ and “back-end” is taken care of, is Rs. 5/person/d. This is irrespective of what sanitation solution is adopted.