CE412 A

Water Supply & Wastewater Disposal Systems

Instructor :
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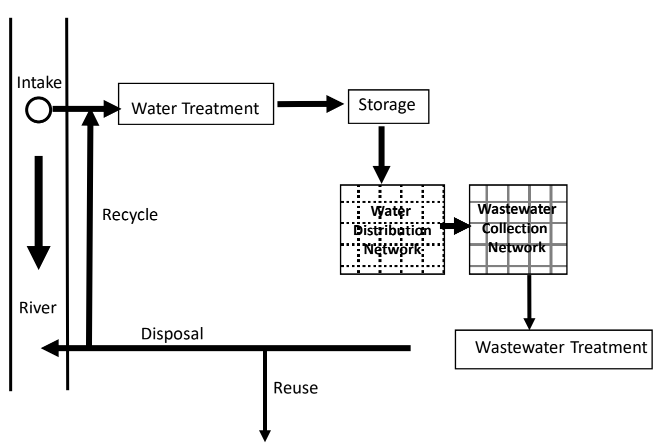
Urban Water Cycle



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 2018, Domestic Population = 9, 870; Temporary population = 1500
In 2038, Domestic Population = 12,300; Temporary population = 3000;

In 2018: 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 2038: 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

Wastewater Quality

Wastewater produced in domestic setting,

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

- 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

Fresh Sewage

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 2018:

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

Average Flow: 531 kg in 1..47 ML = 361 mg/L BOD_5

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

Av: BOD_5 = 361mg/L;
TKN = 36.1 mg/L (as N);
Total-P = 7.2 mg/L (as P)

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

In 2038:

BOD_5 added = 50(12300) + 25(3000) = 690 kg/d

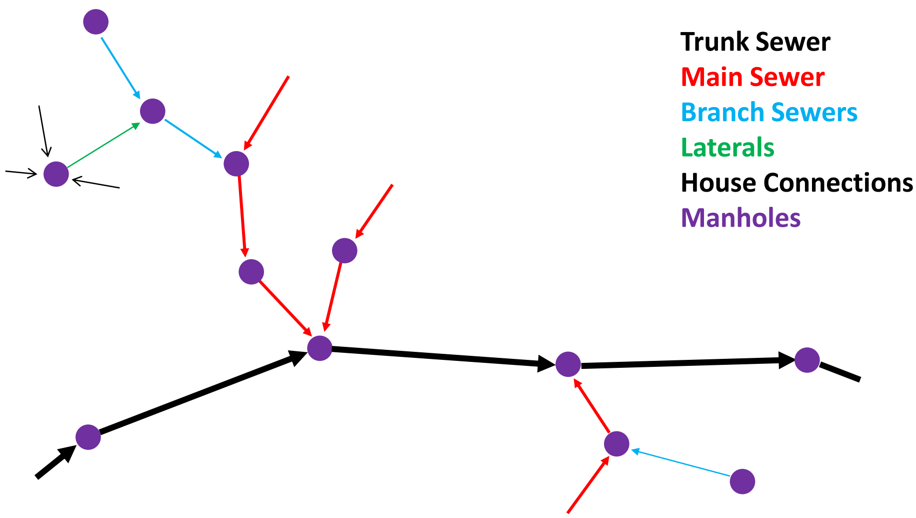
Average Flow: 690 kg in 2.46 ML = 281 mg/L BOD_5

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

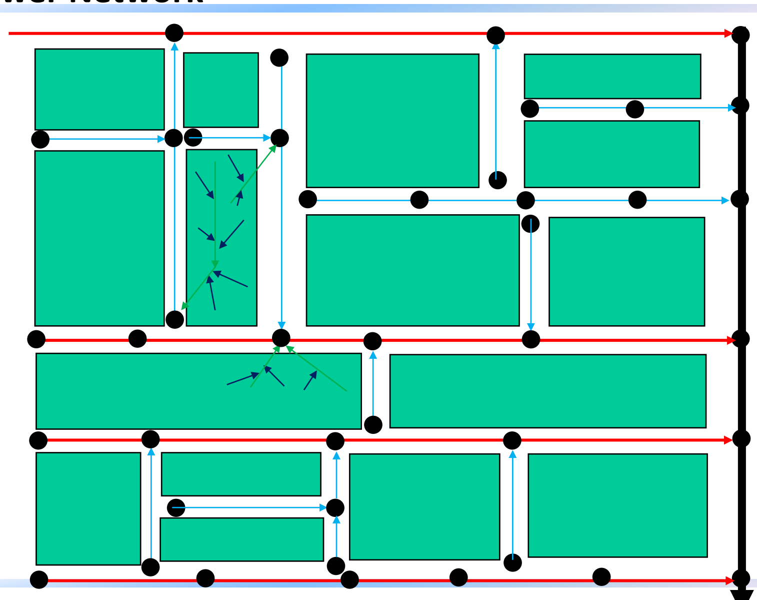
Av: BOD_5 = 281 mg/L;
TKN = 28.1 mg/L (as N);
Total-P = 5.6 mg/L (as P)

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

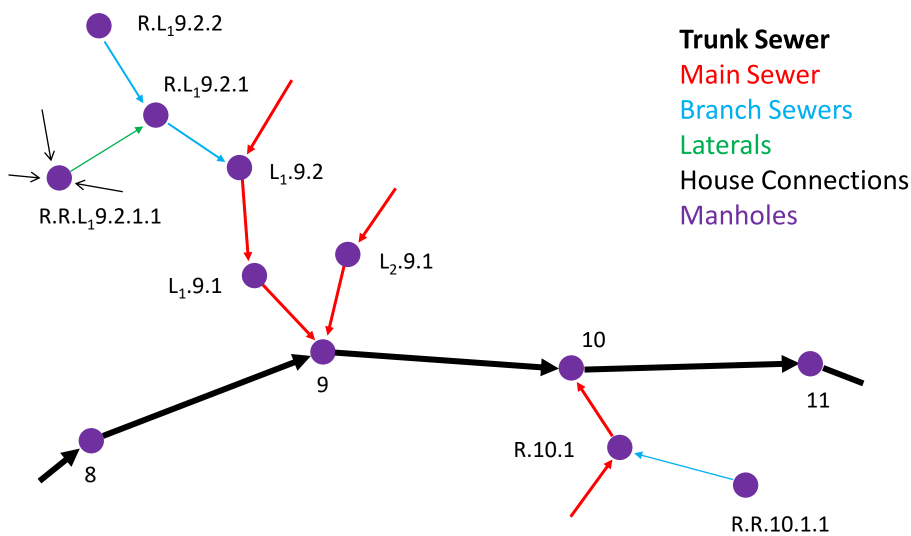
The Sewer System



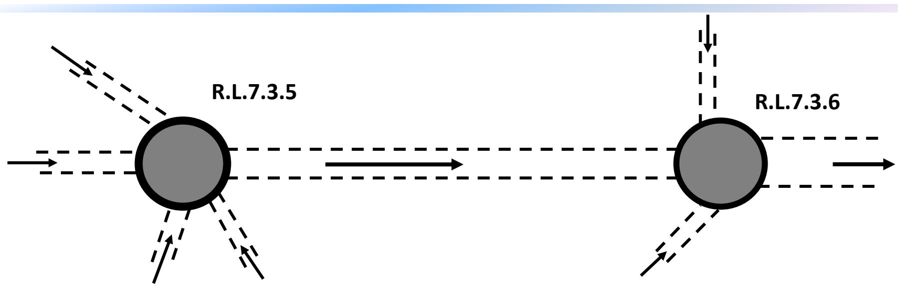
Sewer Network

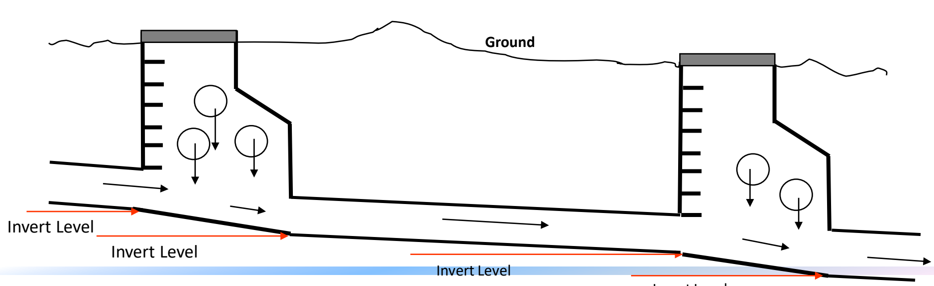


The Sewer System



Sewers and Manholes





Design of Sewers

- Manholes to be provided at the junction of sewers or at every change of alignment / gradient of the sewer.
- Even for straight sewers of uniform diameter and constant slope, manholes must be provided every 30 m for cleaning purposes.
- Branch sewers should connect to main sewers at angles ranging from 30 to 90 degrees
- 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

Design of Sewers

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

Groundwater Infiltration into Sewers

Ground water can infiltrate in sewers through, 1) leaky joints, 2) manholes. There are three ways of estimating infiltration as per CPHEEO manual,

Unit	Minimum	Maximum
Area (L/ha.d)	5000	50000
Length (L/km.d)	500	5000
Manhole (L per day)	250	500

During rainy season, there is a chance of storm water infiltration into sewers, either involuntarily or on purpose

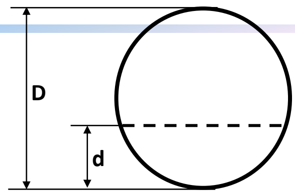
Gas Emission and Ventilation in Sewers

Anaerobic conditions develop in sewers due to emission of gases like methane, ammonia and hydrogen sulfide in sewers. There must be ample scope for ventilation of these gases. This is one of the reasons why sewers are designed to run only 0.8 full at the peak flow at the end of the design period. Hydrogen sulfide is a corrosive gas and may cause "**Crown Corrosion**" in sewers.

Solids Deposition in Sewers

Sewage contains suspended particles which may deposit in sewers if the horizontal velocity in sewers is less than the scouring velocity of these particles. Deposition of particles in sewers results in the reduction of sewer capacity and also in the increase of the value of manning's coefficient (n). Hence sewers are designed such that a horizontal velocity of 0.6 m/s is achieved in the sewer at least for some time every year starting from the beginning of sewer operation. If this is not possible, then sewers must be flushed manually one a year.

Design of a Sewer Section: Procedure



Peak hourly flow (2038) = 0.30 $m^3/s = q_1$
Peak hourly flow (2018) = 0.15 $m^3/s = q_2$

Design criteria: At $Q = q_1$, $d/D = 0.8$

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

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

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

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

Design of a Sewer Section: Procedure

d/D	v/V$_f$	q/Q$_f$
1.0	1.000	1.000
0.9	1.124	1.066
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0.4	0.902	0.337
0.3	0.776	0.196
0.2	0.615	0.088
0.1	0.401	0.021

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, $Q_f = 0.4142\ m^3/s$

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)

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

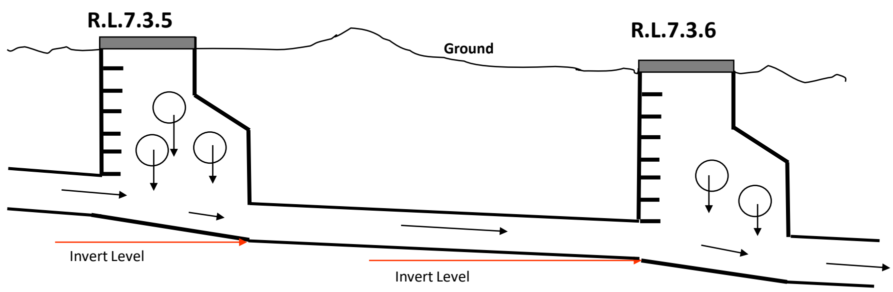
v (in 2012) = 0.902.(1.076) = 0.971 m/s (okay)

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

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

Tabular Calculations:

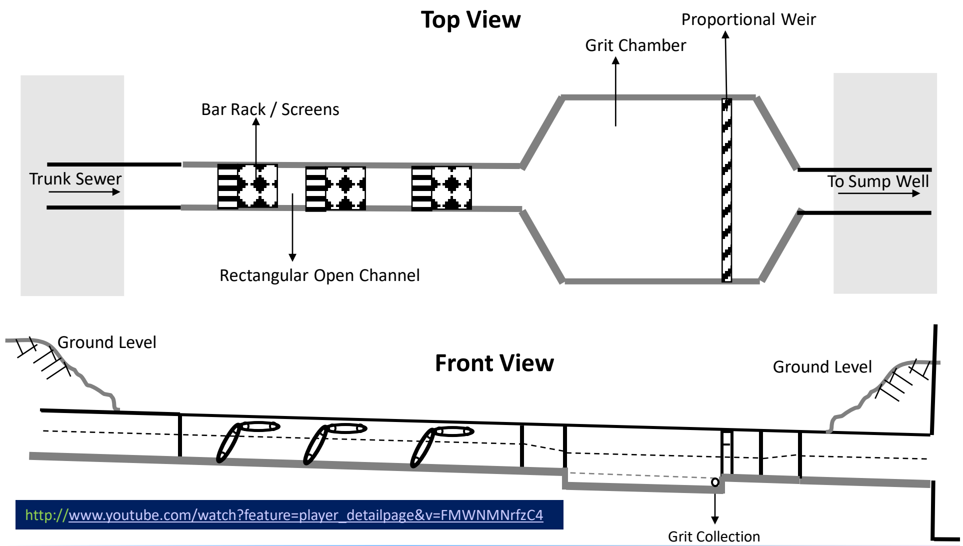
Manhole		Length (m)	Peak Flow (2032) m³/s	D (mm)	S	Discharge, m³/s (2032)		Velocity, m/s (2032)		Peak Flow (2012) m³/s	Self-Cleansing Velocity (m/s)	Total Fall, m	Invert Elevation (m)	
From	To					Full	Actual	Full	Actual				Upper	Lower
1	2	3	4	5	7	8	9	10	11	12	13	14	15	16
R.L.7.3.5	R.L.7.3.6	88	0.30	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

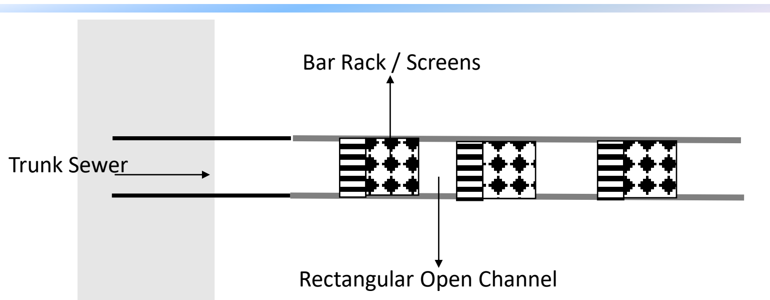
- The center-line of the branch (or main) sewer should be at least 0.4 m above the center-line of the main (or trunk) sewer leaving the manhole. This will ensure dissipation of excess energy of water in the smaller sewer.
- Manholes are connected over the center-line of the principal sewer. A minimum drop of 0.03m is provided between center-line of principal sewers entering and exiting manholes.
- The diameter of a sewer exiting a manhole must always be greater than or equal to diameter of the principal sewer entering the manhole.
- 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.

Preliminary Wastewater Treatment



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

Screen Channel



Screen Channel Design:

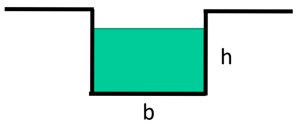
Velocity in screen channel should be > 0.6 m/s (self-cleansing velocity)

q_1 = 0.30 m^3/s;

Choose:

n = 0.013; S = 0.002;

b = 0.70 m (same a trunk sewer dia.)



Q = Design flow
b = Channel width
h = depth of flow
S = channel slope
A = flow cross section
V_H = flow velocity

Assuming rectangular channel, R = (b.h)/(b + 2.h); A = b.h;q_1 = A.V_H = (1/n).A.$(R)^{2/3}$.$(S)^{1/2}$;

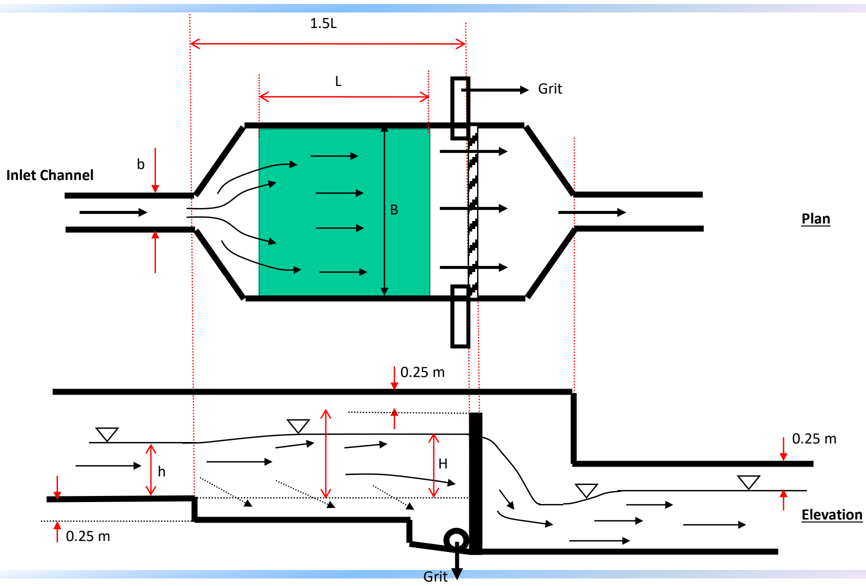
Choose h such that q_1 = 0.3 m^3/s; h = 0.504 m
V_H = q_1/(A) = 0.850 m/s; $V_H > V_{sc}$, hence okay

Checking for q_2: q_2 = A.V_H = (1/n).A.$(R)^{2/3}$.$(S)^{1/2}$;

Choose h such that q_1 = 0.15 m^3/s; h = 0.298 m
V_H = q_1/(A) = 0.720 m/s; $V_H > V_{sc}$, hence okay

<u>Grit Chamber:</u> To remove inorganic particles up to 0.25 mm in diameter

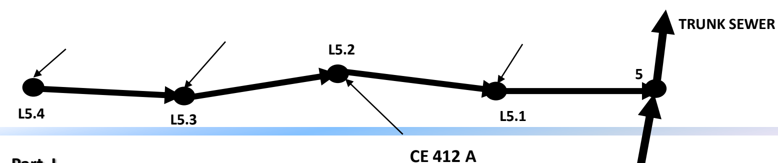
V_{sc} (0.25 mm particles) = 0.254 ms^{-1}; Thus, $V_H < V_{sc}$



Sewer Levels

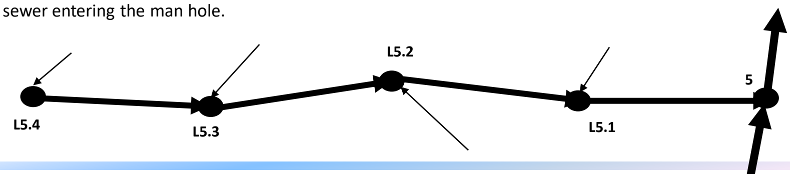
Consider the sewer system shown in the figure below. A main sewer with manholes, L5.4 (a starting manhole) to L5.1 discharges into manhole 5 of the trunk sewer. Branch sewers discharge into manhole L5.1 – L5.4 of the main sewer as shown in the figure. Invert levels of the branch sewers discharging into manhole M5.4 – M5.1 are 97.223, 98.775, 98.311, and 96.334 m respectively. Invert level of the trunk sewer leaving manhole 5 is 93.700 m. Fill up the last three columns of the design table for main sewer. Show all calculations. All branch sewers have 400 mm diameter. Diameter of the trunk sewer is 1500 mm.

Given:
1. Center-line of a branch sewer entering a manhole must be at least 0.40 m above the center-line of the sewer leaving the manhole
2. The top of a sewer leaving a manhole cannot be higher than the top of the principal sewer entering the manhole
3. Invert level of a sewer leaving a manhole must be at least 0.03 m below the invert level of the principal sewer entering the man hole.



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
L5.4	**L5.3**	**88**	**0.15**	500	0.002	0.169	**0.15**	0.86	0.972	**0.05**	0.688	0.176	96.773	96.597
L5.3	**L5.2**	**77**	**0.28**	700	0.002	0.414	**0.28**	1.076	1.152	**0.09**	0.829	0.154	96.397	96.243
L5.2	**L5.1**	**101**	**0.33**	700	0.002	0.414	**0.33**	1.076	1.184	**0.12**	0.861	0.202	96.213	96.011
L5.1	**5**	**122**	**0.45**	800	0.002	0.569	**0.45**	1.176	1.235	**0.15**	0.929	0.244	95.734	95.490

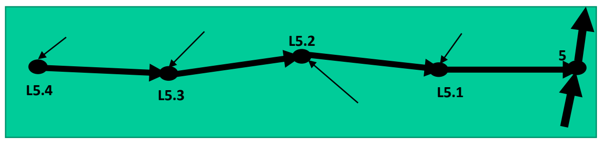
Given:
1. Center-line of a branch sewer entering a manhole must be at least 0.40 m above the center-line of the sewer leaving the manhole
2. The top of a sewer leaving a manhole cannot be higher than the top of the principal sewer entering the manhole
3. Invert level of a sewer leaving a manhole must be at least 0.03 m below the invert level of a principal sewer entering the man hole.



Calculation of Sewer Levels

Given:

- Center-line of a branch sewer entering a manhole must be at least 0.40 m above the center-line of the main sewer leaving the manhole
- The top of a main sewer leaving a manhole cannot be higher than the top of a main sewer entering the manhole
- Invert level of a main sewer leaving a manhole must be at least 0.03 m below the invert level of a main sewer entering the man hole.



Manhole		Invert Level (m)		Dia.
From	To	Upper	Lower	mm
L.5.4	L.5..3	96.773	96.597	500
L5.3	L.5.2	96.397	96.243	700
L.5.2	L.5.1			700
L5.1	5			800

Invert levels of the branch sewers discharging into manhole M5.4 – M5.1 are 97.223, 98.775, 98.311, and 96.334 m respectively. Invert level of the trunk sewer leaving manhole 5 is 93.700 m. All branch sewers have 400 mm diameter. Diameter of the trunk sewer is 1500 mm.

Invert level of branch sewer at L5.4 = 97.223 m
Diameter of the branch sewer at L5.4 = 0.400 m
R.L of the c/l of the branch sewer at L5.4 = 97.223 + 0.200 = 97.423 m
R.L of the c/l of sewer exiting L5.4 = 97.423 -0.400 = 97.023 m
Diameter of sewer exiting L5.4 = 0.500 m
Invert level of sewer exiting L.5.4 = 97.023 – 0.250 = 96.773 m

<u>Condition 1</u>

Fall from L5.4 – L5.3 = (88).(0.002) = 0.176 m
Invert level of sewer entering L5.3 = 96.773 – 0.176 = 96.597

Invert level of branch sewer at L5.3 = 98.775 m
Invert level of sewer leaving L5.3 = 96.597 – 0.2 = 96.397 m

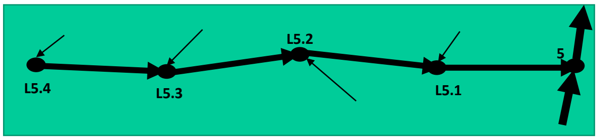
<u>Condition 2</u>

Check: R.L of c/l of sewer leaving L.5.3 = 96.397 + 0.350 = 96.747 m
R.L of c/l of branch sewer at L.5.3 = 98.775 + 0.200 = 98.975 m
c/l to c/l distance = 98.975 – 96.747 = 2.228 m (> 0.400 m, okay)

<u>Condition 1</u>

Fall from L5.3 – L 5.2 = (0.002).77 = 0.154 m
Invert level of sewer entering L5.2 = 96.397 – 0.154 = 96.243 m

Calculation of Sewer Levels (Continued)



Manhole		Invert Level (m)		Dia.
From	To	Upper	Lower	mm
L.5.4	L.5..3	96.773	96.597	500
L5.3	L.5.2	96.397	96.243	700
L.5.2	L.5.1	96.213	96.011	700
L5.1	5	95.734	95.490	800

Given:

- Center-line of a branch sewer entering a manhole must be at least 0.40 m above the center-line of the main sewer leaving the manhole
- The top of a main sewer leaving a manhole cannot be higher than the top of a main sewer entering the manhole
- Invert level of a main sewer leaving a manhole must be at least 0.03 m below the invert level of a main sewer entering the man hole.

Invert levels of the branch sewers discharging into manhole M5.4 – M5.1 are 97.223, 98.775, 98.311, and 96.334 m respectively. Invert level of the trunk sewer leaving manhole 5 is 93.700 m. All branch sewers have 400 mm diameter. Diameter of the trunk sewer is 1500 mm.

Invert level of sewer leaving L5.2 = 96.243 – 0.030 = 96.213 m

Check: R.L of c/l of sewer leaving L.5.2 = 96.213 + 0.350 = 96.563 m
R.L of c/l of branch sewer at L.5.2 = 98.311 + 0.200 = 98.511 m
c/l to c/l distance = 98.511 – 96.563 = 1.948 m (> 0.400 m, okay)

Condition 3

Fall from L5.2 – L 5.1 = 0.202 m
Invert level of sewer entering L5.2 = 96.213 – 0.202 = 96.011 m

Invert level of sewer leaving L5.1 = 96.011 – 0.100 = 95.911 m

Check: R.L of c/l of sewer leaving L.5.1 = 95.911 + 0.400 = 96.311 m
R.L of c/l of branch sewer at L.5.1 = 96.334 + 0.200 = 96.534 m
c/l to c/l distance = 96.534 – 96.311 = 0.223 m (not > 0.400 m, not okay)

Condition 2

True invert level of sewer leaving L5.1 is 96.534 - 0.400 – 0.400 = 95.734

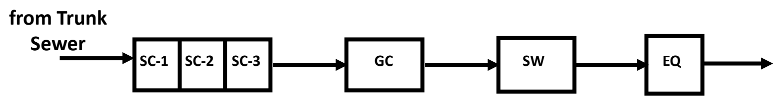
Fall from L5.1 – 5 = 0.244 m
Invert level of sewer entering 5 = 95.734 – 0.244 = 95.490 m

Invert level of sewer exiting 5 = 93.700 m

Condition 1

Check: R.L of c/l of trunk sewer leaving 5 = 93.700 + 0.75 = 94.450 m
R.L of c/l of main sewer at 5 = 95.490 + 0.400 = 95.890 m
c/l to c/l distance = 95.890 – 94.450 = 1.440 m (> 0.400 m, okay)

Preliminary Wastewater Treatment



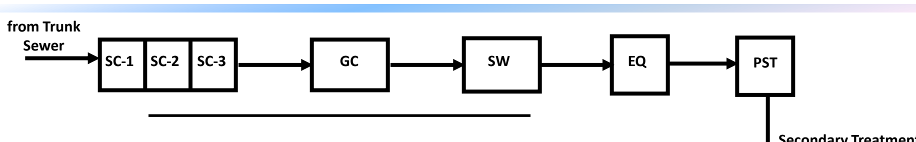
Screenings

Grit

Fresh Sewage

Aged Sewage

Preliminary and Primary Wastewater Treatment



Primary Sedimentation for Wastewater Treatment

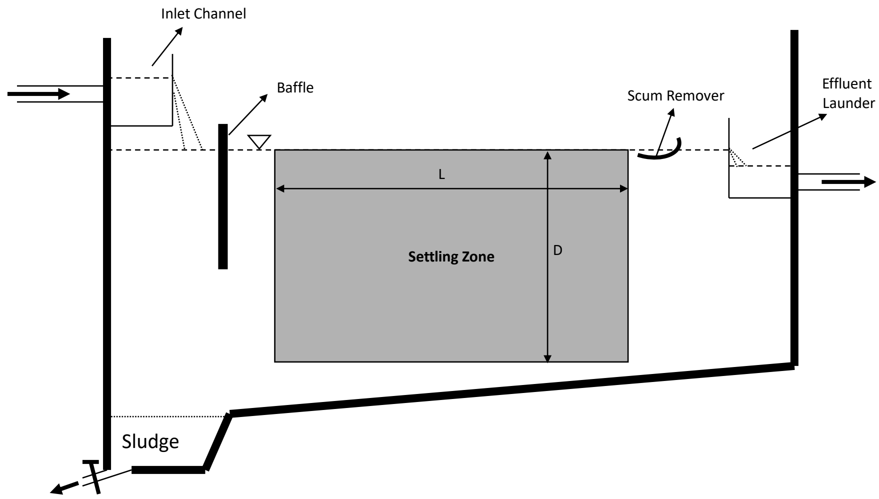
Exactly same as primary sedimentation for water treatment in most respects

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 mm, inorganic particles: 20 mm
3. SOR = 48 m^3/m^2/d, organic particles: 75 mm, inorganic particles: 25 mm
4. Type II settling, hence smaller particles are also removed
5. SS removal: 50 – 70 %; BOD_5 removal: 25 – 40 %

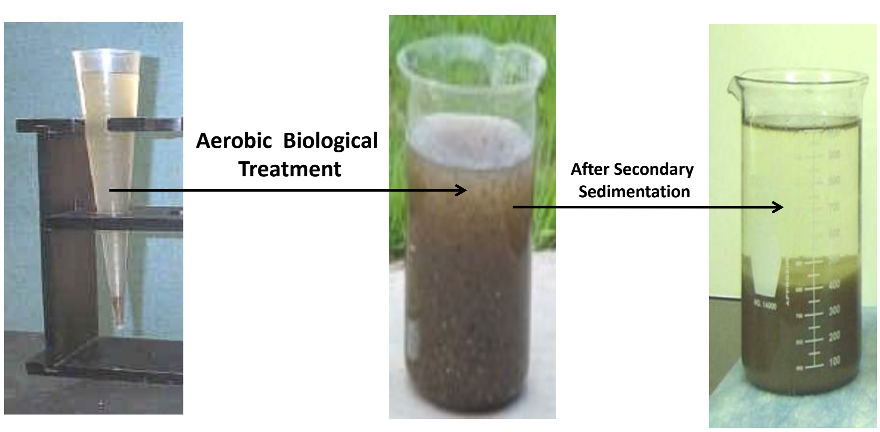
Sewage:
After Primary Sedimentation

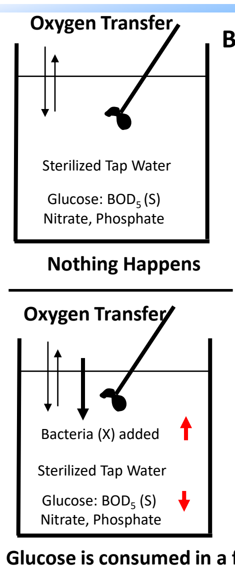
Primary Sedimentation Tank (To remove organic particles up to 60 mm size completely)



Secondary Treatment: Aerobic Biological Treatment

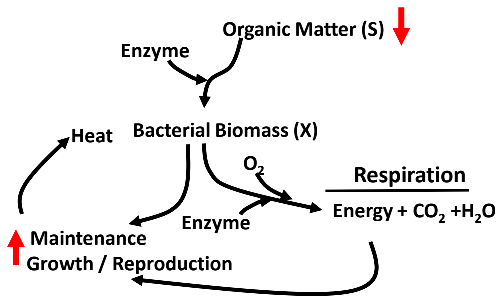
Sewage: After Primary Sedimentation



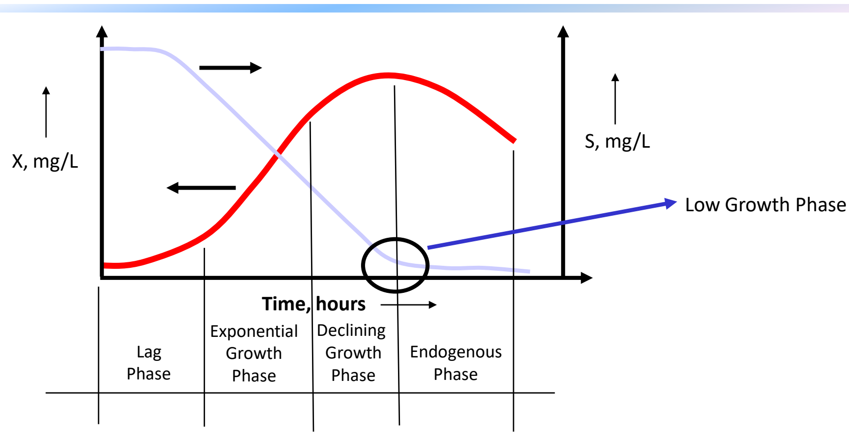


Bacterial Consumption of Biodegradable Organic Carbon

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



Microbial Growth Curve



Objective of aerobic wastewater treatment is to have low S output in a continuous basis. This is possible in a system where the biomass (X) is maintained in a low-growth phase, i.e., under conditions where X is high with respect to S, i.e., conditions where food to microorganism ratio is low

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

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

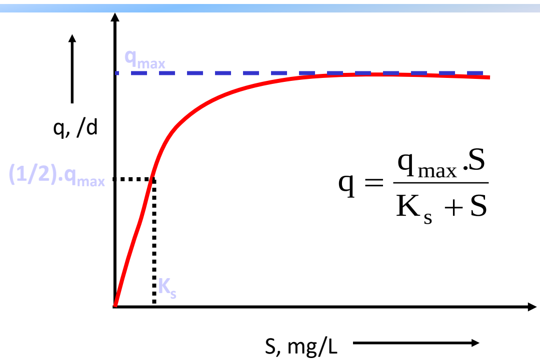
$$\left[\frac{dX}{dt}\right]_T = Y_T.\left[\frac{dS}{dt}\right]$$

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



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

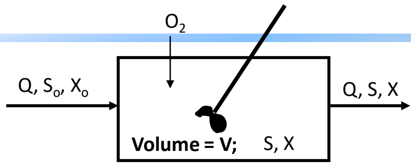
When S is large (S >> K_s), q = q_{max} and hence, m = m_{max}

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

When m = 0, q = k_d/Y_T

When q < k_d/Y_T, m < 0

Microbial Kinetics in a Reactor



Completely Mixed Reactor

Hydraulic Retention Time (q) = V/Q

$$\mu = Y_T.q - k_d$$

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

Substrate Mass Balance:

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

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

Biomass Mass Balance:

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

$$\mu.X.V = Q.X; \quad \mu = Q/V = 1/\theta$$

Microbial Kinetic Constants

q_{max} = 4 /d

K_s = 25 mg/L

K_d = 0.05 /d

Y_T = 0.50

Example Problem

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

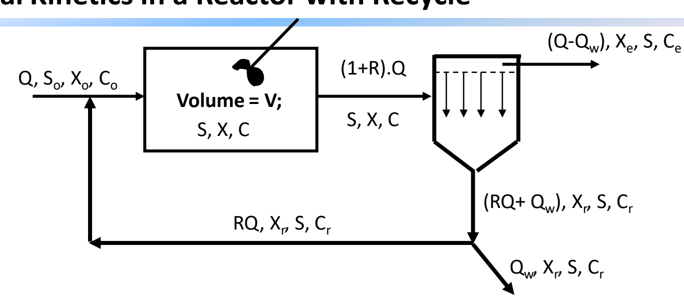
q = V/Q = 1 day; m = 1/q = 1 / d

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

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

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

Microbial Kinetics in a Reactor with Recycle



Substrate Balance for the whole system:

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

$Q.(S_o – S)/V = q.X$ (A)

Biomass Balance for the whole System:

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

$m.X.V = Q_w.X_r$ (B)

Biomass Balance for Aeration Tank:

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

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

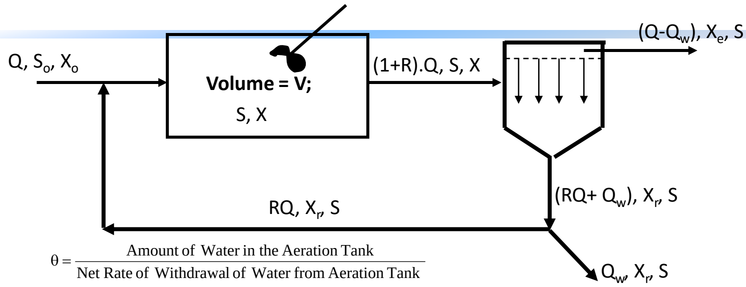
$m.V = Q.[1+ R – R(X_r/X)]$ (C)

Other Relationships:

$$\mu = Y_T.q - k_d \quad (D)$$

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

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) = $Q_w.X_r$

Hydraulic Retention Time (q):

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

Biological Solids Retention Time (q_c):

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

Please Note,

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

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

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

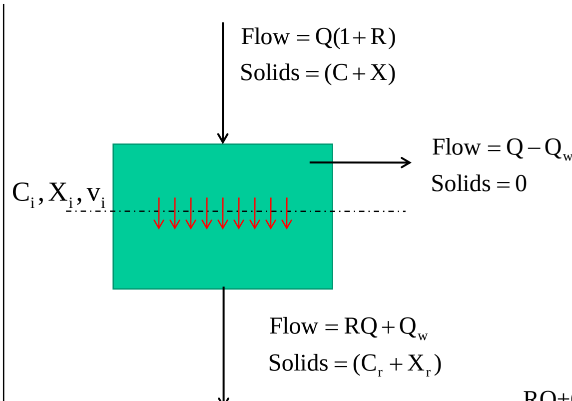
Underflow Flux (SF_u)= ($C_i + X_i$).u

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



Underflow Velocity (u) = $\frac{RQ+Q_w}{A_s}$

Design of Secondary Sedimentation Tank

Consider a clarifier with solids conc. (C + X) on top calculated as 3394 mg/L and the solids concentration at the bottom ($C_r + X_r$) being chosen as 12500 mg/L

It is also given that Q = 50 MLD, while the values, R = 0.352 and Q_w = 0.766 MLD were established through design

Assuming no solids escape with the treated effluent, solids input rate to the clarifier must be equal to the sludge output rate from the clarifier, i.e., Q.(1 + R).(C + X) = ($RQ + Q_w$).($C_r + X_r$) = 9557 kg/hr

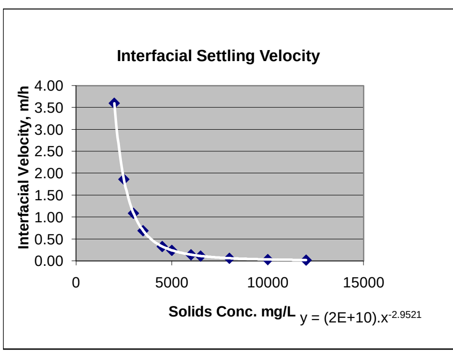
If cross sectional area is A_s, then solid flux (kg/m²/hr) through the tank,

$$SF = \frac{Q(1+R).(C+X)}{A_s} = \frac{(RQ+Q_w).(C_r+X_r)}{A_s}$$

It is also obvious that the SF through the tank cross-section is the same both at the top and bottom of the tank and also at any intermediate height in the tank.

The question is then, what should be the value of SF.

As soon as we decide on the value of SF, the tank surface area can be calculated.



Solids	V	Gravity Flux	u, m/hr	Underflow Flux	Total Flux
mg/L	m/hr	kg/m2/hr		Kg/m2/hr	kg/m2/hr
2000	*3.60*	7.196	0.100	0.200	7.396
2500	*1.86*	4.655	0.100	0.250	4.905
3000	*1.09*	3.261	0.100	0.300	3.561
3500	*0.69*	2.414	0.100	0.350	2.764
4500	*0.33*	1.478	0.100	0.450	1.928
5000	*0.24*	1.203	0.100	0.500	1.703
6000	*0.14*	0.843	0.100	0.600	1.443
6500	*0.11*	0.721	0.100	0.650	1.371
8000	*0.06*	0.481	0.100	0.800	1.281
10000	*0.03*	0.311	0.100	1.000	1.311
12000	*0.02*	0.218	0.100	1.200	1.418

Underflow Velocity (u) = $\frac{(Q_w + RQ)}{A_{prov}} = 0.1$ m/hr

A_{prov} = ($RQ + Q_w$)/u = 18.35.(1000/24)/0.1 = 7640 m²

SF_L = 1.28 kg/m²/hr

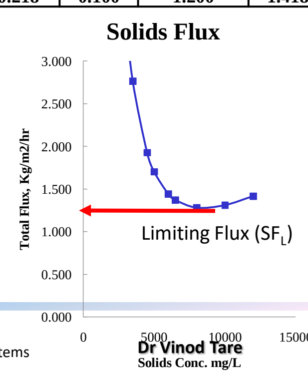
A_{req} = (1 + R).Q.(C+X) /SF_L = 9557/1.28 = 7461 m²

$A_{prov} > A_{req}$ hence, okay;

Actual Solid Flux (SF) = 9557/7640 = 1.25 kg/m²/hr

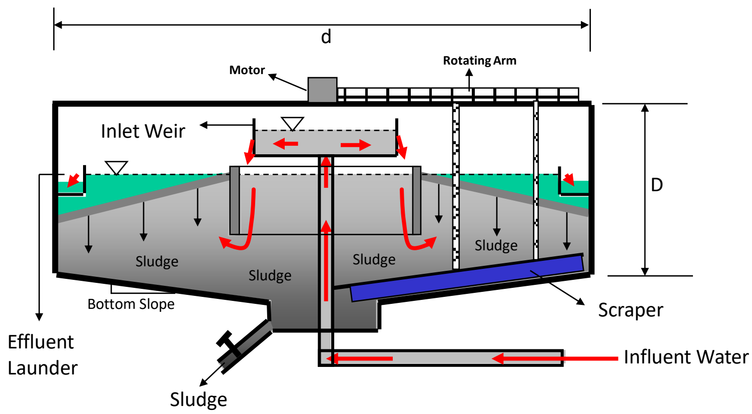
Provide 5 tanks, each of 44.13 m diameter and 2 m depth

Hydraulic Retention time = 5.30 hrs

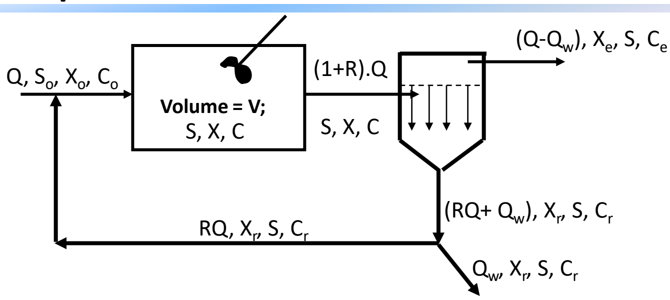


Activated Sludge Process: Secondary Sedimentation

Circular Sedimentation Tank



Oxygen Requirement



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

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 ~ 1.5.BOD_5 ; So, O_2 Reqd, kg/d = 1.5.Q (S_o – S) – 1.42.[DX]

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

Respiration Equation:

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 31 g P is required
- Nutrient Requirement: 0.121 g N and 0.022 g P per g biomass produced (or wasted).

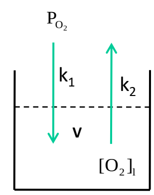
Hence the N and P concentration will decrease by this amount during ASP

Theory of Aeration:

$P_{O_2} \underset{k_2}{\overset{k_1}{\rightleftarrows}} [O_2]_l$ $\quad [O_2]_l$ in moles/L $\quad P_{O_2}$ 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: $[O_2]_l = [O_2]_l^s$; $\quad k_1.P_{O_2} = k_2.[O_2]_l^s$;

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

k_1 in moles/atm.-s

k_2 in L/s

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

$$k_1.H = (k_a.H).A = k_L.A$$

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

k_L = Mass transfer coefficient; A = contact area between liquid - gas phases

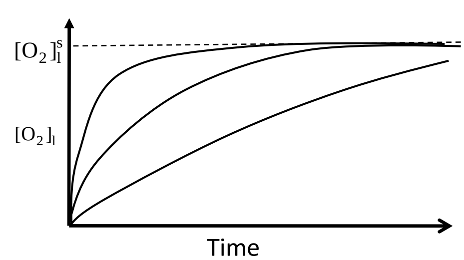
Theory of Aeration (Continued)

$$\frac{d[O_2]_l}{dt} = k_L.\left(\frac{A}{V}\right).\left([O_2]_l^s - [O_2]_l\right) = k_L.a.\left([O_2]_l^s - [O_2]_l\right)$$

a = Specific Surface Area (m^2/m^3)

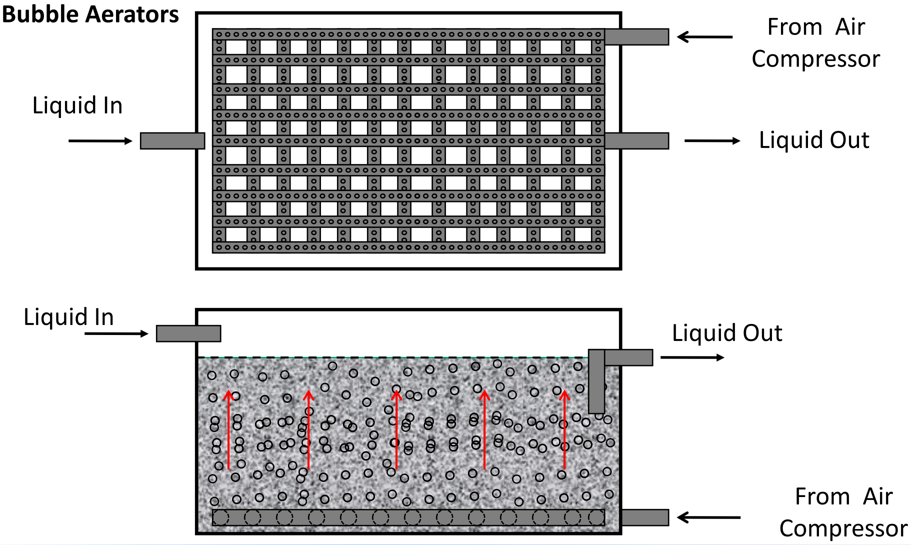
The oxygen transfer rate from the gas to the liquid phase can be increased by increasing the value of 'a'.

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

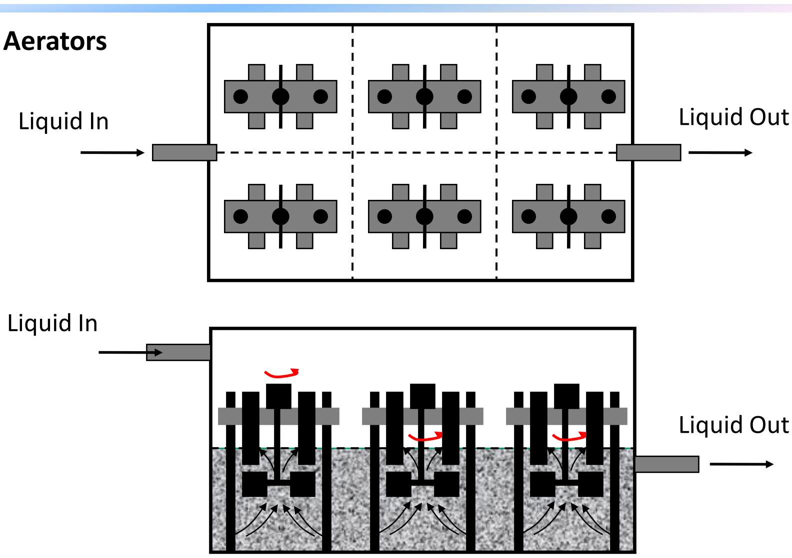


Two types of aerators are commonly used:

(a) Bubble Aerators



(b) Turbine Aerators



Turbine Aerator: Design Procedure

Aerator Rating: 1, 2, 5 KW ; Area of Influence: 5m x 5m x 3m (depth)

Aerator Rating: 10, 25, 50 KW; Area of Influence: 6m x 6m x 4m (depth)

Standard O_2 transfer efficiency (SOT) is given by the manufacturer as, kg O_2 transferred per hour per KW under <u>standard conditions</u>

<u>Standard Conditions:</u> T = 20°C; $[O_2]_l = 0.0$; In tap water

SOT calculation: (Generally specified by the manufacturer)

Fill up a 5x5x3 m or 6x6x4 m tank (depending on aerator size) with tap water

Aerate overnight. Measure DO in the morning. This is $[O_2]_l^s$. Measure temp.

Add sodium sulfite to de-aerate water, i.e., $[O_2]_l = 0.0$

Aerate from t = 0 to t = t. Measure $[O_2]_l$ at various times during aeration.

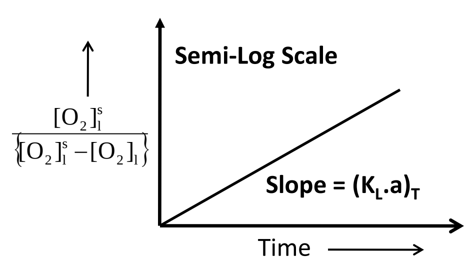
Turbine Aerator: Design Procedure (Continued)

$$\frac{d[O_2]_l}{dt} = K_L.a\left\{[O_2]_l^s - [O_2]_l\right\}$$

Integrating,

$$\int_{[O_2]_l}^{0} \frac{d[O_2]_l}{\left\{[O_2]_l^s - [O_2]_l\right\}} = \int_{t}^{o} K_L.a.dt$$

$$\ln\left\{\frac{[O_2]_l^s}{[O_2]_l^s - [O_2]_l}\right\} = K_L.a.t$$



$$\frac{d[O_2]_l}{dt} = k_L.a.\left([O_2]_l^s - [O_2]_l\right)$$

$$V.\frac{d[O_2]_l}{dt} = k_L.a.\left([O_2]_l^s - [O_2]_l\right)V$$

$$\boxed{SOT = (K_L.a)_{20}.[O_2]_l^{s_{20}}.(V)}$$

$$(K_L.a)_{20} = \frac{(K_L.a)_T}{(1.02)^{T-20}}$$

T in celcius

Given SOT: To Find the Actual Oxygen Transfer Efficiency (AOT)

both $K_L.a$ and $[O_2]_l^s$ depend on temperature

$$\alpha = \frac{(K_L.a)_T^W}{(K_L.a)_T^F} = 0.8;\ \beta = \frac{[O_2^W]_l^s}{[O_2^F]_l^s} = 0.9$$ Impact of wastewater

$$(K_L.a)_{20} = \frac{(K_L.a)_T}{(1.02)^{T-20}};\ \text{T in celcius}$$

$[O_2]_l^s$ decreases with tempearture, obtained from tables

$[O_2]_l = 1\ \text{mg/L or } 3\ \text{mg/L (in case of nitrification)}$

$$SOT = (K_L.a)_{20}^F.[O_2^F]_l^{s_{20}}.(V)$$ Generally 1.5 – 2.0 kg O_2 / h/KW

$$AOT = (K_L.a)_T^W.\{[O_2^W]_l^{s_T} - [O_2]_l\}.(V)$$

$$AOT = (K_L.a)_{20}^F.(\alpha.[1.02]^{T-20})\{\beta.[O_2^F]_l^{s_T} - [O_2]_l\}.(V)$$

Energy requirements for maintaining completely mixed conditions in the aeration tank

For an aeration tank of volume V, the P/V ratio for maintaining completely mixed conditions is 15 – 20 W/m^3

Hence in actual design power requirement in an aeration tank must be calculated both from O_2 requirement and mixing perspectives, and the larger value adopted.

In many cases, the mixing requirement becomes the controlling factor for provision of power to the aeration tank

Example Problem: Aerator Design

Given the oxygen requirement of 8400 kg/d, design the aeration system for an ASP using turbine aerators. Assume that two aeration tanks will be provided in parallel and depth of the tanks will be 4 m. Total volume of aeration tanks is 4320 m^3. No nitrification occurs in the aeration tank. Turbine Aerators are available with power rating 10, 25, 50 KW with area of influence being 36 m^2. Manufacturers specify that the oxygen transfer capacity of these aerators is 2.0 Kg O_2/kW-h under standard conditions. Based on this information, design an adequate aerator arrangement for each aeration tank. The operating temperature of the aeration tank is expected to be 30^{o}C. Saturation concentration of oxygen in water at 20^{o}C is 9.1 mg/L and at 30^{o}C is 7.5 mg/L.

$$\alpha = \frac{(K_L.a)_T^W}{(K_L.a)_T^F} = 0.8 \quad \beta = \frac{C_s\ (\text{wastewater})}{C_s\ (\text{tapwater})} = 0.9 \quad (K_L.a)_T^F = (K_L.a)_{20}^F.\gamma, \quad \gamma = 1.02^{(T-20)}$$

Solution:

c/s area of each tank = 4320/4/2 = 540 m^2

Area of influence of each aerator = 36 m^2

Let length of each tank be 30 m and breadth be 18 m. Hence aerators are provided in a 5x3 grid

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

$$[O_2^F]_l^{s20} = 9.1\,\text{mg/L} \qquad [O_2^F]_l^{s30} = 7.5\,\text{mg/L} \qquad [O_2]_l = 1.0\,\text{mg/L}$$

$$SOT = (K_L.a)_{20}^F.[O_2^F]_l^{s20}.(V) \qquad 2 = (K_L.a)_{20}^F.(V).9.1$$

$$AOT = (K_L.a)_{20}^F.\left([0.8].[1.02]^{30-20}\right)\{[0.9].[7.5]-1\}.(V)$$

$$AOT = (K_L.a)_{20}^F.(V).(5.607) = \frac{2}{9.1}.(5.607) = 1.232\ \text{kg}\ O_2/\text{h}/\text{KW}$$

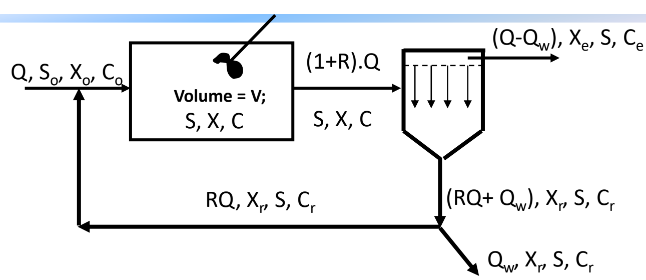
Oxygen transfer per aerator = 4200/15/24 = 11.67 kg/hr;

Hence, Power Reqd. = 11.67/1.232 = 9.56 KW; Provide 15 no. aerators@10KW/tank

Power Reqd. for mixing/tank = 540.(4).(20)/1000 = 43.2 KW

Power provided for aeration/tank = 15.(10) = 150 KW. Hence, completely mixed conditions will prevail.

Activated Sludge: Process control



Q and S_o input to the aeration tank is variable. Daily variations are damped by the equalization tank. However sewage is generally less concentrated in summer months (when wastewater production is high) as compared to winter months. Further, Q is low at the beginning of the design period, but increases gradually. S_o also gradually decreases over the design period. Hence, though the ASP is designed for a particular flow and S_o, actual plant operation is under for a variety of conditions that may be encountered during its operation over the design period.

The ASP is generally designed in modules, with the number of modules in operation increasing with increase in flow over the design period. Individual modules can further be controlled by varying the sludge wastage rate [D(X + C) = Q_w.(X_r+ C_r) and the Recycle Ratio (R).

Typical Variation in Q and S_o of a STP Over Design Period

		Q MLD	BOD_5 mg/L
2038	Max	45.00	111.11
	Av	25.00	200.00
	Min	20.00	250.00
2033	Max	36.00	122.22
	Av	20.00	220.00
	Min	16.00	275.00
2028	Max	27.00	133.33
	Av	15.00	240.00
	Min	12.00	300.00
2023	Max	18.00	144.44
	Av	10.00	260.00
	Min	8.00	325.00
2018	Max	9.00	155.56
	Av	5.00	280.00
	Min	4.00	350.00

ASP is designed in modules.

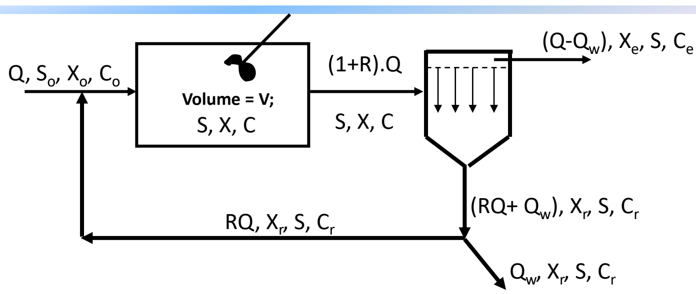
Each module consists of an aeration tank, followed by one or more secondary sedimentation tanks (SSTs).

Performance of the ASP can be controlled by changing the sludge wasting rate [$(D(X+C) = Q_w.(X_r+ C_r)$]

What happens in sludge wasting rate is decreased ??
Decrease in D(C+X) will increase X and C in the aeration tank. This will decrease m (increase q_c) and q and ultimately S. Opposite will happen when D(C+X) is increased.

What happens if S_o and or Q increases??
Effluent quality (S) will not decline provided enough oxygen is provided and q_c is maintained by adjusting sludge wastage D(C + X) and recirculation ratio (R).

Activated Sludge: High Growth, Standard and Extended Aeration process



What are the characteristics of a high-growth ASP system ??
In such a system, q_c is low. Hence m and q are high, S is relatively high. DX 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, q_c is high. Hence m and q are low, S is relatively low. DX 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.

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. (DX) 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

$$NH_4^+ \rightleftarrows NH_3 + H^+$$

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 :

- Energy source
- Carbon / food source
- 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

- 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

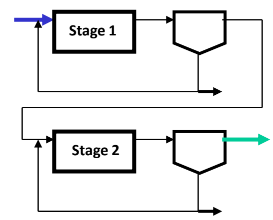
Two Stage Nitrification

Stage 1:

Low q_c
Only BOD removal and no nitrification

Stage 2:

High q_c
Removal of residual BOD
Nitrification



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

- Availability of large concentration of Biomass
- Availability of sufficient amount of oxygen

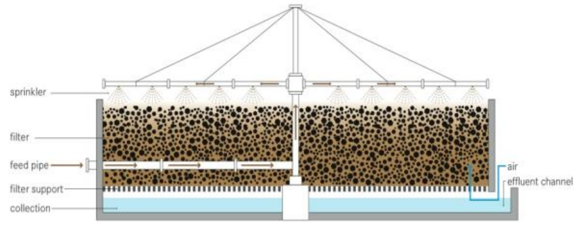
Any reactor where the above two conditions are satisfied is likely to show efficient removal of BOD and TKN

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

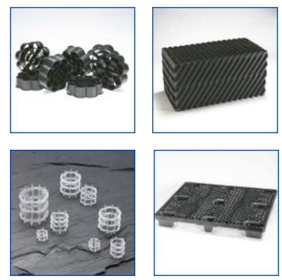
- One of the essential requirements for efficient BOD/TKN removal in a bio-reactor is the maintenance of high biomass concentration in the reactor
- In the suspended growth system, the biomass is allowed to escape from the reactor along with the treated effluent. However, the escaped biomass is captured in the SST and recycled back into the reactor. High biomass concentration is maintained in this way.
- The attached growth concept is based on the observation that biomass prefers to attach itself to inert surfaces (if available).
- 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

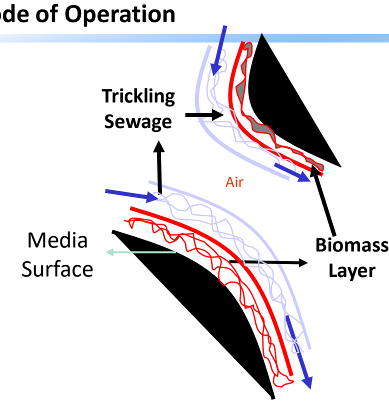


Actual TF Operation

Plastic Media used in TF



Mode of Operation



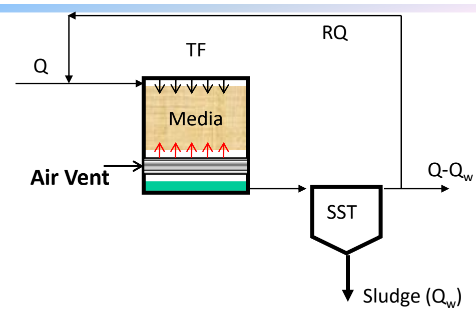
Media: 25 – 100 mm size, rock, gravel, plastic

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

Recirculation in 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.

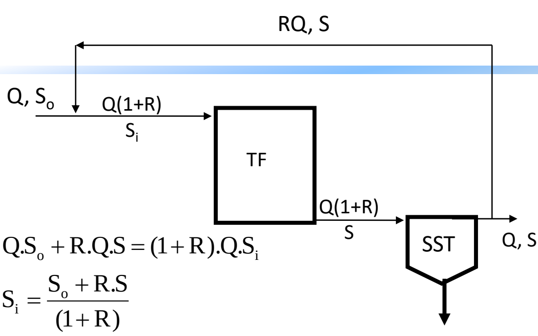
Recirculation enables the variation of HLR independent of OLR. This is important for high strength waste and also for maintaining the most of the media in wetted condition.

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.

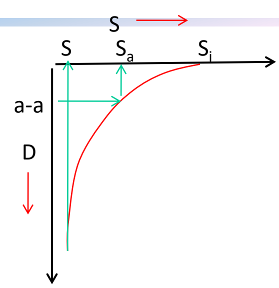
Plastic media is generally used if the filter height in more than 5 m.



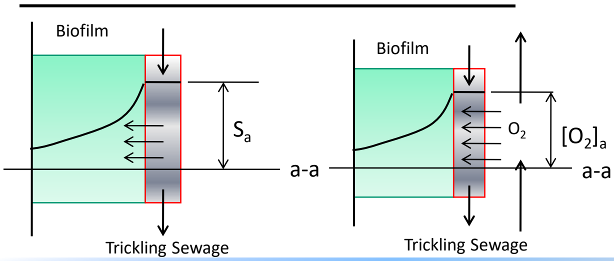
$$Q.S_o + R.Q.S = (1+R).Q.S_i$$

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

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 $(K_s)_o$,

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

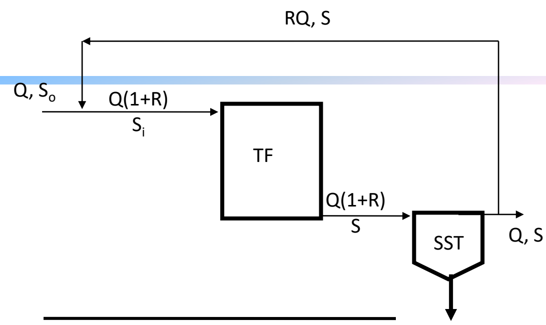
Design of a Trickling Filter

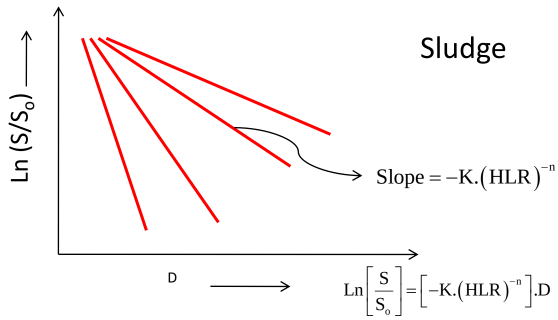
$$\frac{S}{S_i} = \text{Exp}\left[-K.D.\left(\frac{Q.(1+R)}{A}\right)^{-n}\right]$$

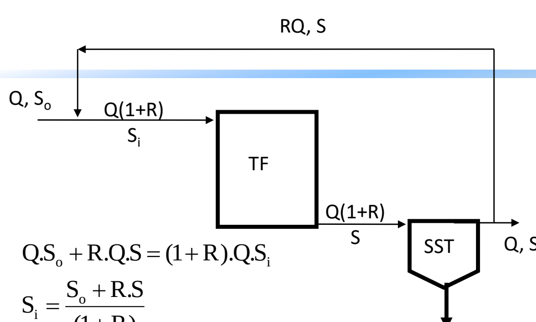
K, n: Treatability Constants
D: Depth of Filter
A: Filter Cross-Sectional Area

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



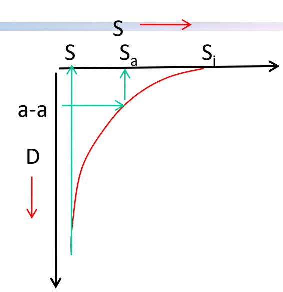




$$Q.S_o + R.Q.S = (1+R).Q.S_i$$

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

BOD_5 Profile in a Trickling Factor



Filter Type	Filter Medium	OLR, kg/m³/d	HLR, m³/m²/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
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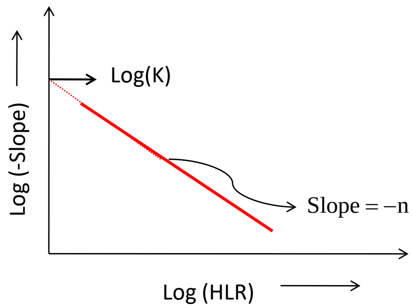
Super High Rate	Plastic	3 - 4	60 - 120	65 - 80	4 - 12	1 - 4
Roughing	Plastic	4 - 6	60 - 180	40 - 65	4 - 12	1 - 4

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



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)

Without recycle, Hydraulic Loading Rate (HLR) = $\frac{Q}{A} = \frac{(0.6).10^3}{150} = 4\ m^3/m^2/d$ (not okay)

So, let R = 3; Hence,

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

BOD Removal Efficiency:

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

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

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

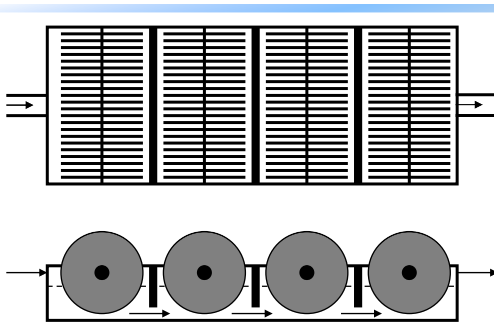
$$\left(1 - \frac{S}{S_o}\right) = 1 - 0.204 = 0.796$$

79.6% Removal

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

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

Rotating Biological Contactor



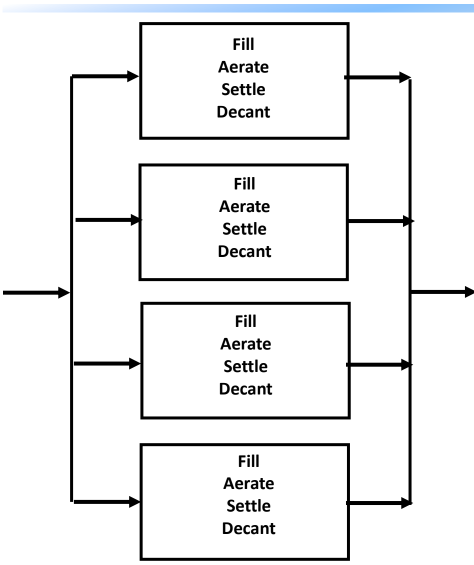
- Attached growth of biomass on the rotating disks.
- The attached biomass absorbs substrate when the disks are submerged in water
- The attached biomass absorbs oxygen when exposed to air
- The attached biomass sloughs off when the layer on the disk becomes too thick
- Hence there is some suspended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

Actual Installation

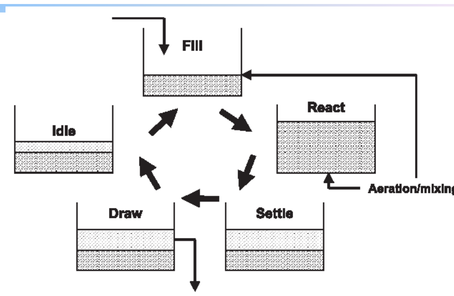
Main Characteristics

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

Sequential Batch Reactor (SBR)



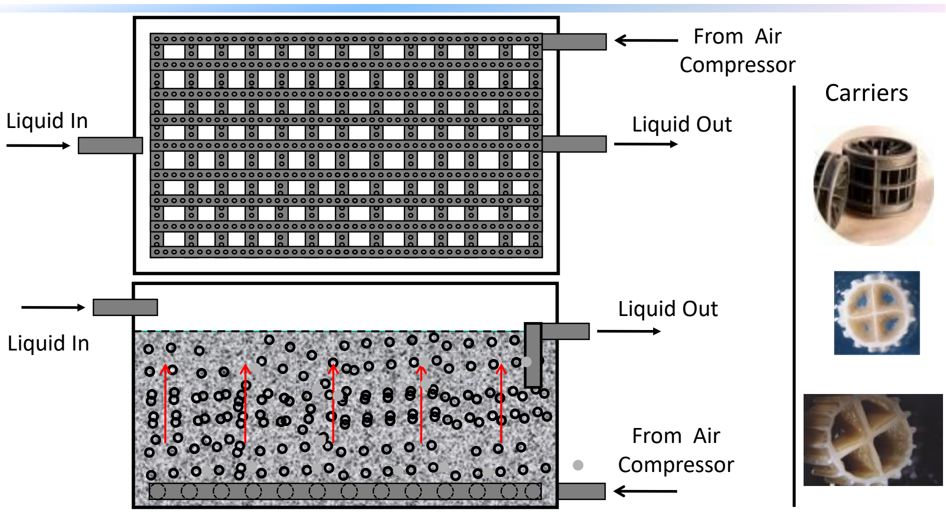
Operating Cycle of SBR



Main Characteristics

- Suspended growth sequential batch process
- Fine bubble aeration
- High quality effluent
- Moving system of weirs/decanters required
- Complex operation requiring electronic controls

Mixed Bed Biofilm Reactor (MBBR)

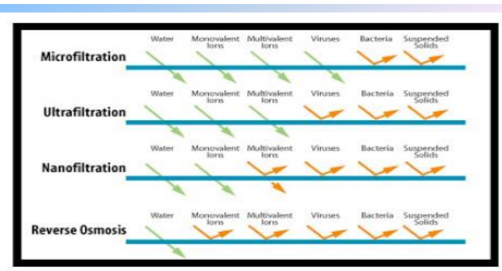


Addition of carriers can increase biomass (MLVSS) in the aeration tank without increasing the concentration of inert substances

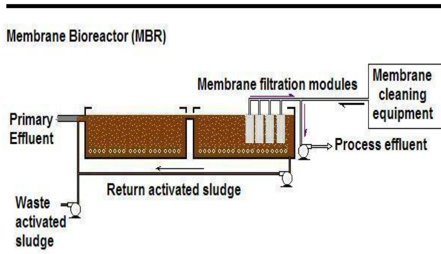
Membrane Bioreactors

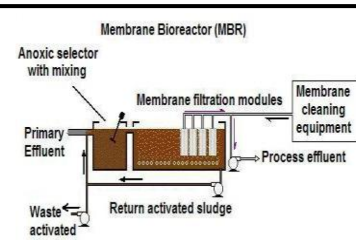
Similar to the conventional activated sludge system, except that the SST has been replaced by a membrane (micro-filtration) module.

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

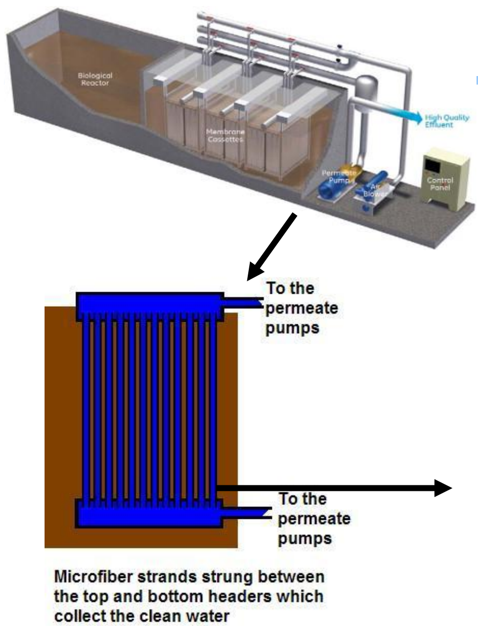


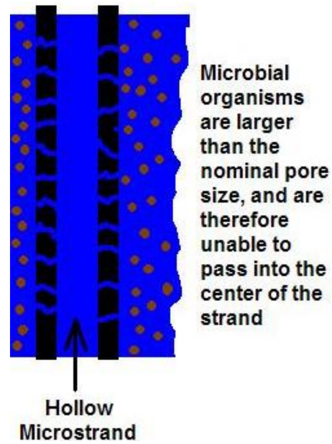
Membrane Process Characteristics



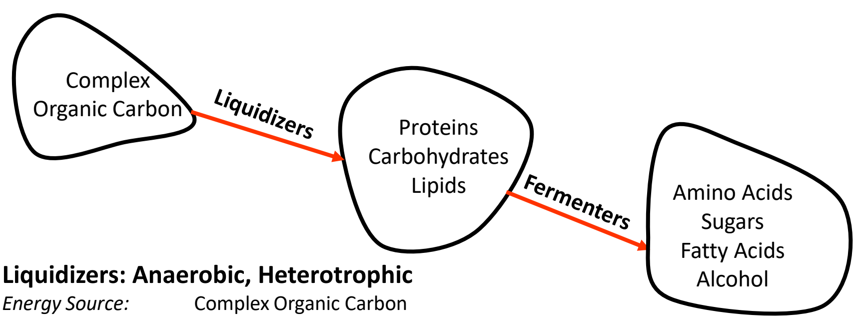


Microbial Bioreactor (Continued)





Biochemistry of Anaerobic Biodegradation….Fermentation



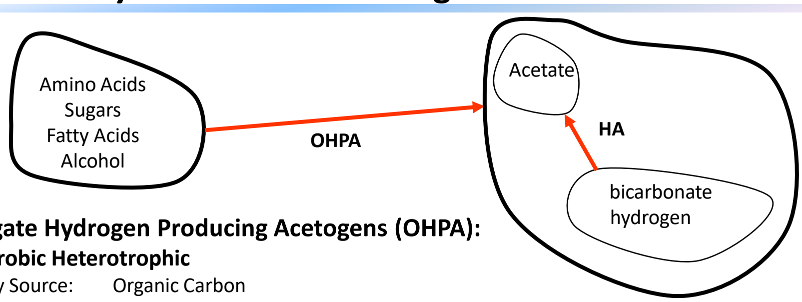
Liquidizers: Anaerobic, Heterotrophic

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

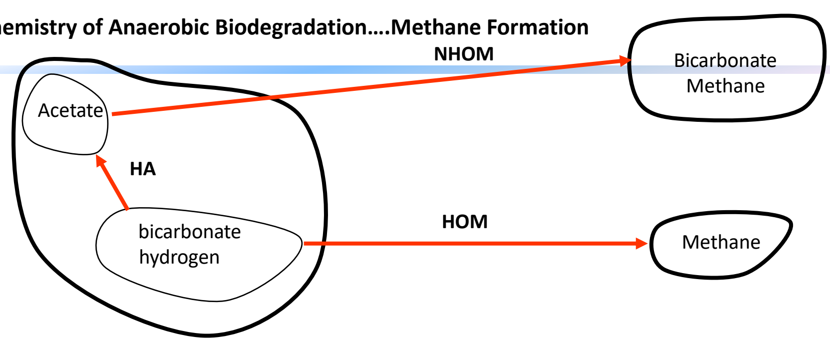
Biochemistry of Anaerobic Biodegradation….Acid Formation



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

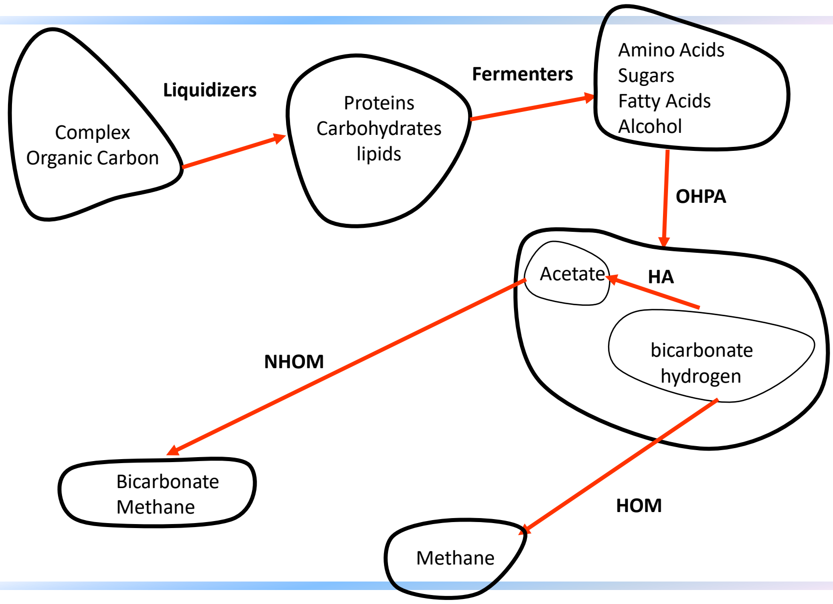


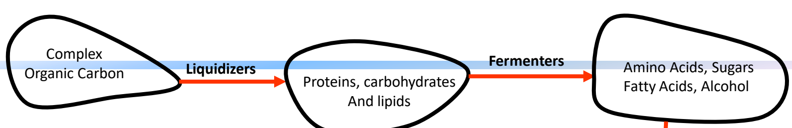
Acetoclastic or Non-Hydrogen Oxidizing Methanogens (NHOM): Anaerobic, Heterotrophic
Energy Source: Acetate
Food Source: Acetate
Electron Acceptor: Acetate
By-products: Methane and Bicarbonate
Equation: $CH_3COO^- + H_2O \rightarrow HCO_3^- + CH_4$

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

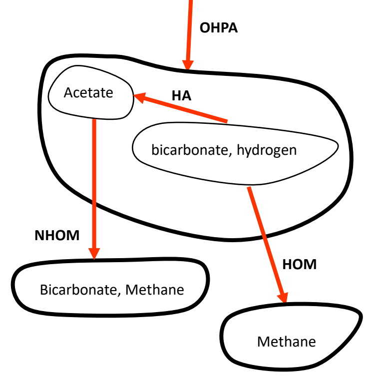
Lower acetate means lower methanogenic activity.

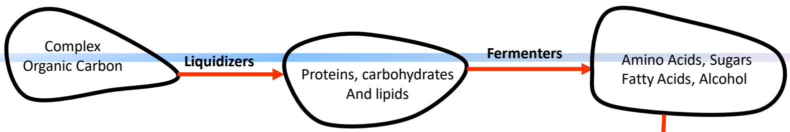
Propionate to acetate → below 10^{-4} atm. H_2 partial pressure

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

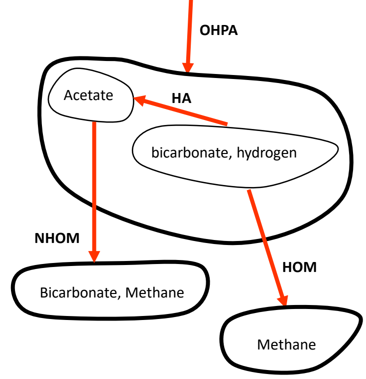
Lowering of pH due to accumulation of acids disrupts methane production

What Happens to the Oxygen Demand of Wastewater ??

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

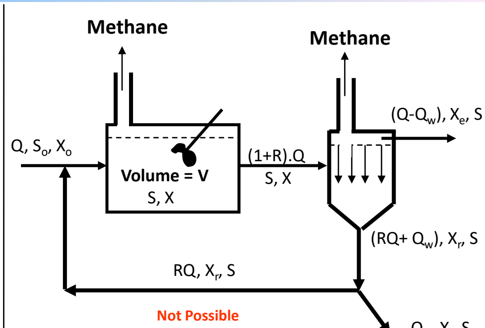
Thus oxygen demand is not satisfied, merely transferred to the gaseous phase (methane).

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

Maximum Removal

Assuming m = 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,

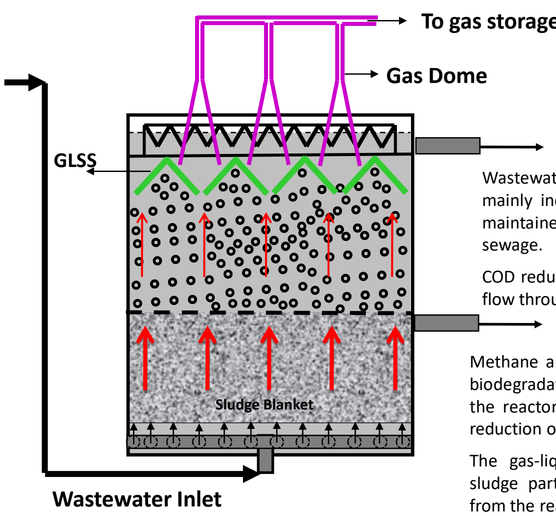
Anaerobic sludge has very poor settling characteristics.

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

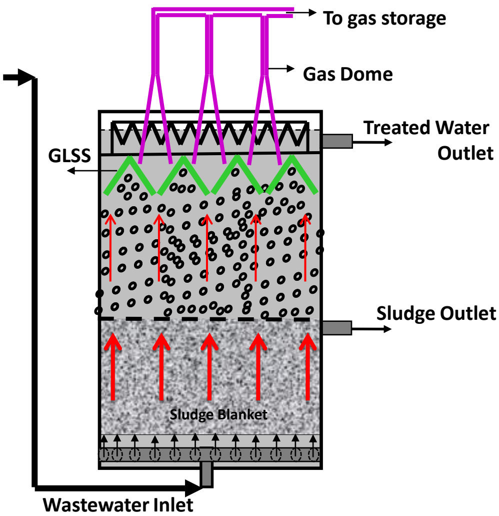
Methane and carbon dioxide is formed due to anaerobic biodegradation. The gas bubbles travel upwards through the reactor and are collected by the gas domes. COD reduction of wastewater ~ COD of the methane produced.

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.

Up-flow Anaerobic Sludge Blanket (UASB) Reactor

Design of an UASB Reactor



$T = 40^oC$; $Y_T = 0.04$;
$k_d = 0.015$ /d;
$q_m = 6.67.[10^{-0.015(35-T)}]$

$q_m = 7.93$ / d;
$K_s = 2224.[10^{0.046(35-T)}]$

$K_s = 1310$ mg/L
S_o (COD)= 600 mg/L;
$q_c = 120$ d; m = 0.008 /d;
q = 0.583 / d; S = 104 mg/L
COD Removal = 83 percent

Height of water in the Reactor (H)
= 6 m; (3-6 m range)

Height of Sludge Blanket (h)
= 3 m; (2-3 m range)

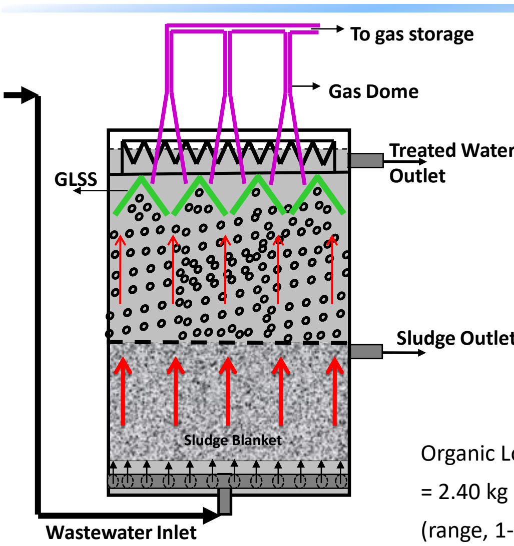
Flow (Q) = 10 MLD;

Detention time (q) in sludge blanket
= 6 hrs;

Total detention time (q_T) = 12 hrs

Up-flow Anaerobic Sludge Blanket (UASB) Reactor

Design of an UASB Reactor



$X = (S_o-S)/(q.q) = 3401$ mg/L;

Influent solids (C_o) = 150 mg/L

Solids (non-biomass) in sludge blanket (C) = $C_o.(q_c/q)$ = 72000 mg/L;

Total Solids in sludge blanket = (C+X) = 75401 mg/L;

Up-flow velocity (u) = 0.5 m/hr

Volume of sludge blanket (V) = (Q.q) = (10).1000.(6/24) = 2500 m^3;

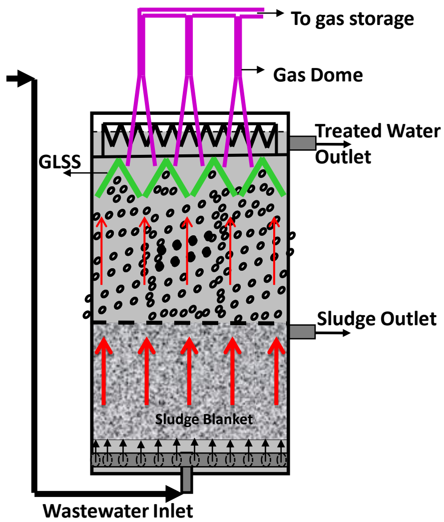
c/s Area of sludge blanket (A) = (V/h) = 833 m^2

Organic Loading Rate (OLR) = $(Q.S_o)/V$ = (10).600/2500

= 2.40 kg COD/m^3 sludge blanket volume

(range, 1-3 kg COD/m^3 sludge blanket volume)

Up-flow Anaerobic Sludge Blanket (UASB) Reactor



Design of an UASB Reactor

Kg COD / kg biomass = $Q.S_o/(X.V)$ = (10.600).1000/(3401.2500) = 0.71 (Range: 0.3 – 1.0)

Biomass wasted (DX) = m.X.V = (0.008.3401.2500/1000) = 71 kg/d; Volume Sludge wasted (Q_w) = DX/X = 20.83 m^3/d

Other solids (non-biomass) wasted (DC) = $Q_w.C$ = 20.83.(72000/1000) = 1500 kg/d;

DC + DX = 1571 kg/d

COD Removed = 4960 kg/d;

Stoichiometry:

COD of methane = 4 kg/kg; Methane produced (Theoretical) = 4960/4 = 1240 kg/d

$$CH_4 + 2O_2 \rightarrow CO_2 + 2H_2O$$

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.

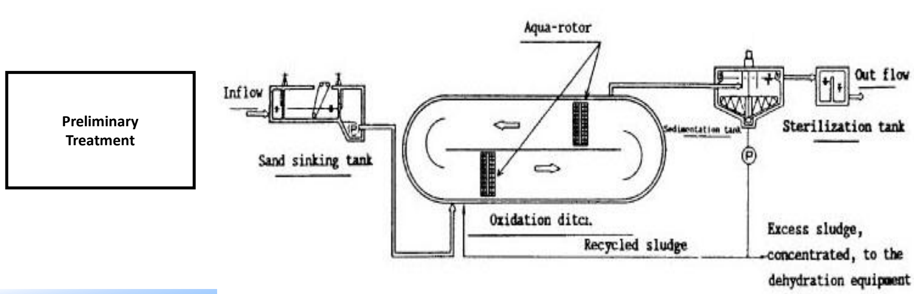
On the whole, anaerobic treatment of domestic wastewater is infeasible unless accompanied by comprehensive aerobic polishing treatment.

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

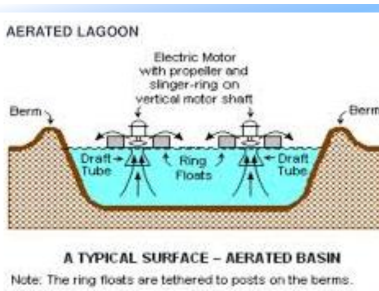
Natural Methods of Wastewater Treatment: Applicable in Rural Areas

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



Wastewater Treatment in Aerobic Lagoon: ASP without Recycle



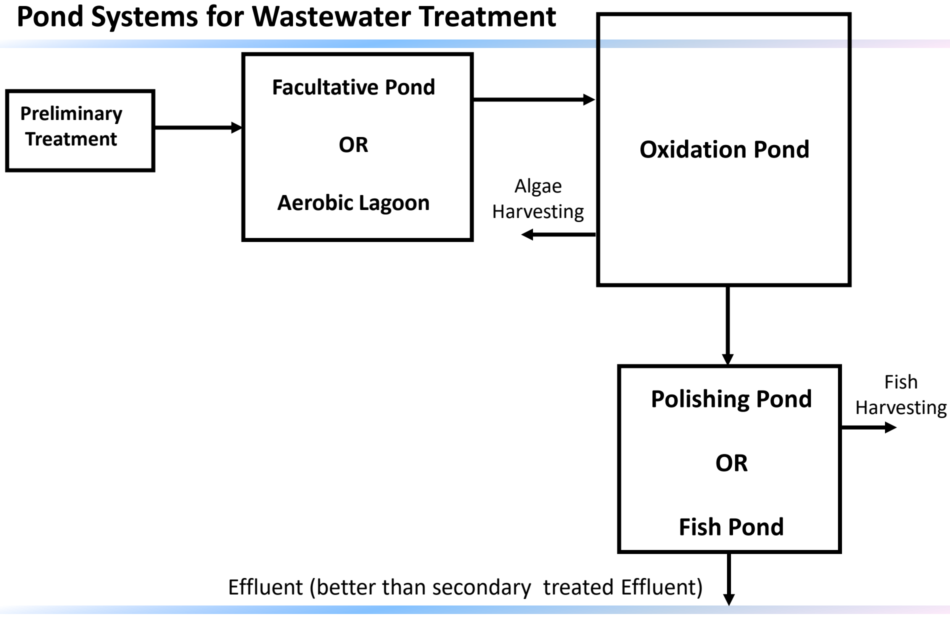
Aerated lagoon is similar to ASP without recycle

Pond Systems for Wastewater Treatment:

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

- Anaerobic Ponds
- Facultative Ponds
- Oxidation Ponds
- Polishing Ponds
- Fish Pond

Pond Systems for Wastewater Treatment



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.

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 (
), (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: $C_{60}H_{87}O_{23}N_{12}P$

$$K = {}^{q_m}\!/\!{}_{K_s}$$

Average intensity of solar radiation:	150 calories/cm^2/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; m = 0.20 /d; q = q_c = 1/m = 5 days
X = (S_o – S)/(q.q) = (80-5)/(0.5)(5) = 30 mg/L; Sludge Production (DX) = Q.X = 30 kg/d

Oxygen Requirement = 1.5.Q.(S_o –S) - 1.42.(DX) = 69.9 kg/d; V = q.Q = 5000 m^3

Assuming depth = 0.5 m; Surface Area (A) = 10000 m^2

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,

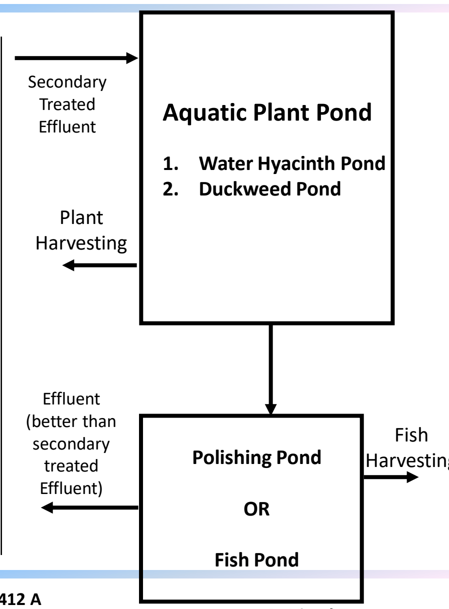
Oxygen available = 97.5 kg/d (>> oxygen required)

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.

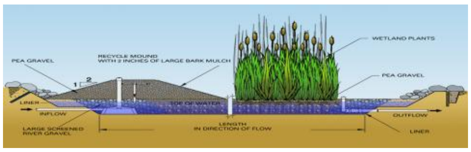


Constructed Wetlands

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.

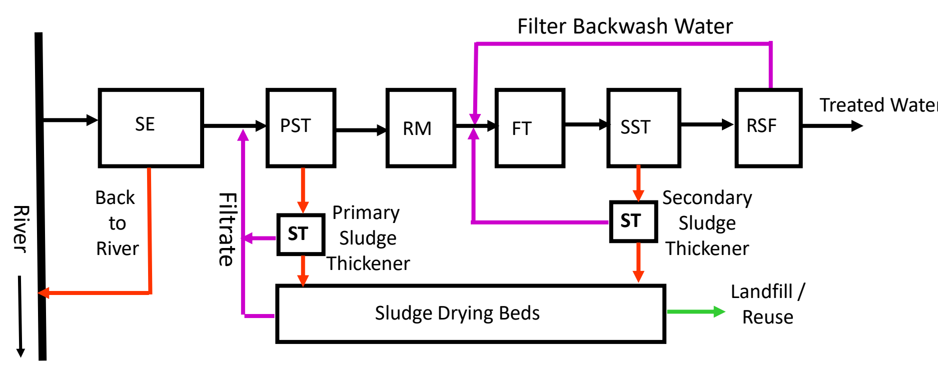


Subsurface Flow

Subsurface + Overland Flow

Actual picture of a constructed Wetland actually in Operation

Sludge Management in Water Treatment

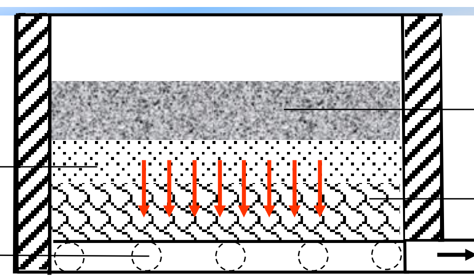


ST - Sludge Thickener:

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

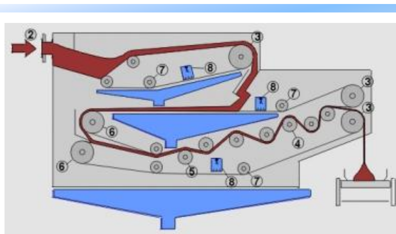
Sludge Dewatering Centrifuge **Sludge Inlet**

Feed and Polymer Inlet

Video Links

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

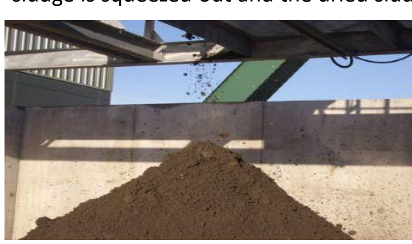
Alternatives to Sludge Drying Beds: Belt Filter Press



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.

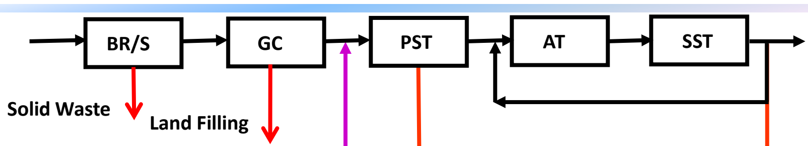


Dewatered Sludge (~40 percent solids)

Video Link

https://www.youtube.com/watch?v=6voXE1HxYsY
https://www.youtube.com/watch?v=KAUB4QQvZqI
https://www.youtube.com/watch?v=DSS6F0UlGlQ

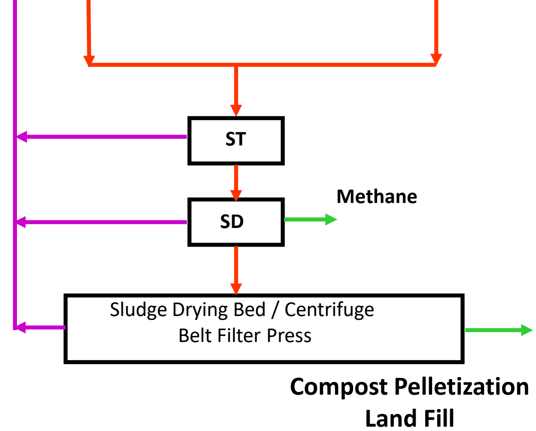
Sludge Management in Wastewater Treatment



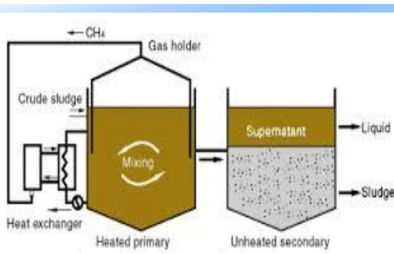
SD: Anaerobic Sludge Digestion

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.

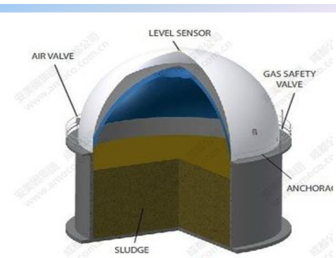
The objective is to reduce solids concentration in thickened sludge so that the load on sludge drying beds / centrifuge / belt filter press is reduced. The methane gas obtained as a product from the sludge digestion process may have economic value.



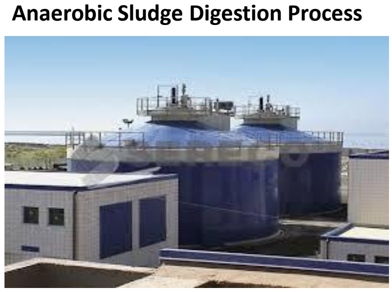
Anaerobic Sludge Digestion



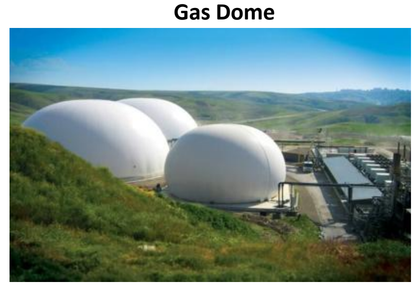
Anaerobic Sludge Digestion Process



Gas Dome



Anaerobic Sludge Digester



Gas Storage

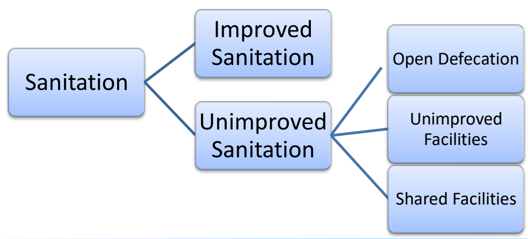
Sanitation

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".

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:

Unimproved Sanitation

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.

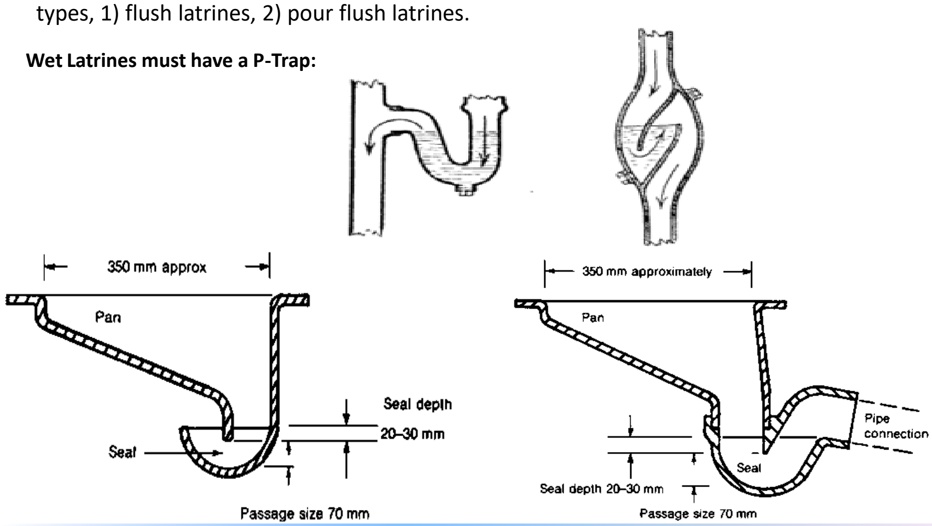
Shared Sanitation Facilities

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.

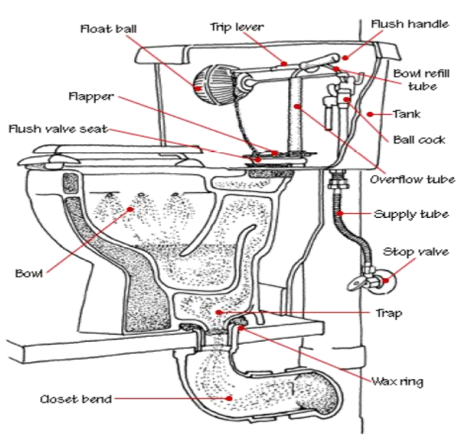
Improved Sanitation

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.



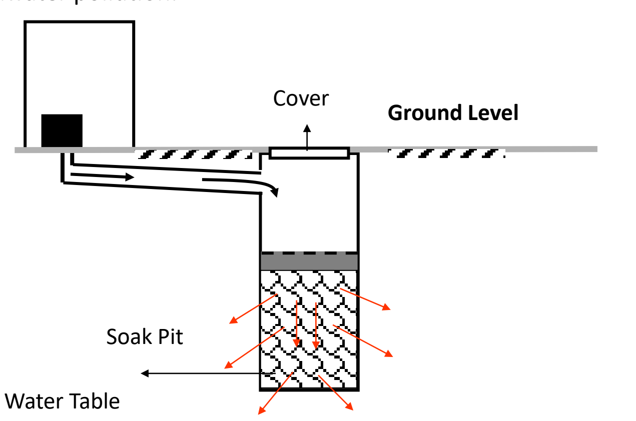
Flush Latrines: Back-End

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. Other alternatives are:

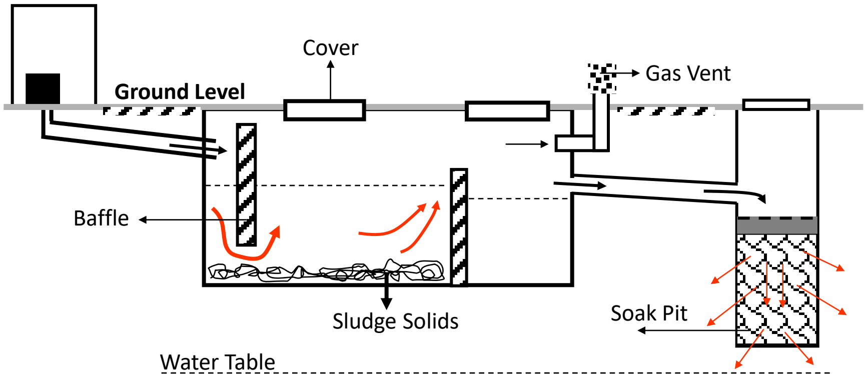
- Sewage from flush latrines flow into natural depressions / ponds in the locality and evaporate or infiltrate into the ground. This creates an unhealthy and aesthetically depressing scenario.
- Sewage from flush latrines flow in surface drains and then into natural rivulets ('nalas') and finally into 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:**

The soak pit chokes after some time. Soaking sewage may cause groundwater pollution.



Flush Toilets with Septic Tank and Soak Pits



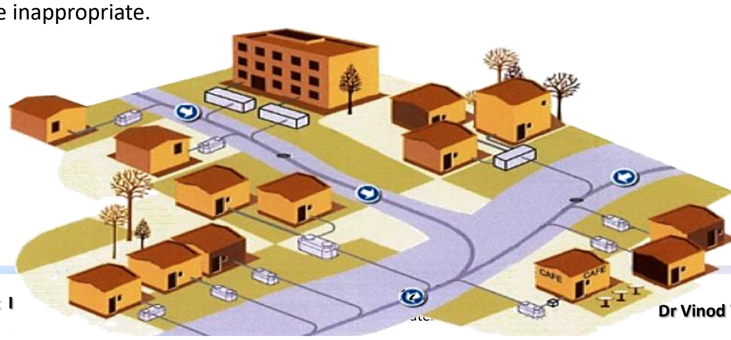
Septic Tank Cleaning issues
The soak pit chokes after some time.
Soaking sewage may cause groundwater pollution.

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.

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.

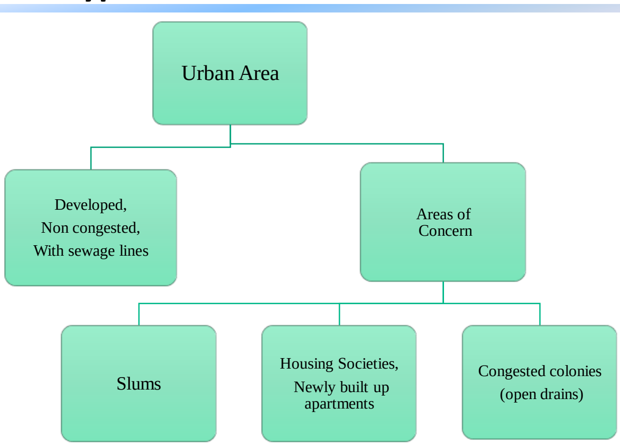
SBS system requires small diameter piping because it conveys only liquid, hence it is economical. Because of the lower costs of construction and maintenance and the ability to function with little water, small bore sewers can be used where conventional sewerage would be inappropriate.



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

Different Types of Human Settlements in Urban Areas



Sanitation solutions are different for each of these areas.

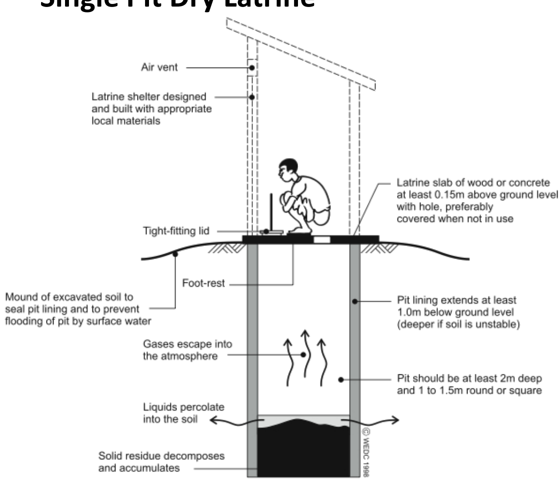
Different Types of Human Settlements in Rural Areas

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

Dispersed settlements
- ➢ Relatively well off
- ➢ Relatively poor

Dry Sanitation

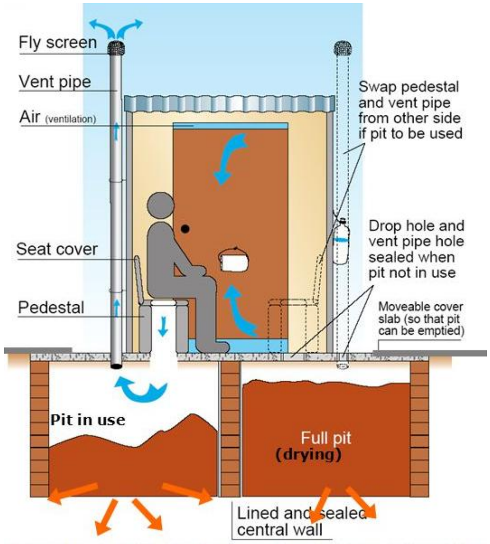
Single Pit Dry Latrine



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.

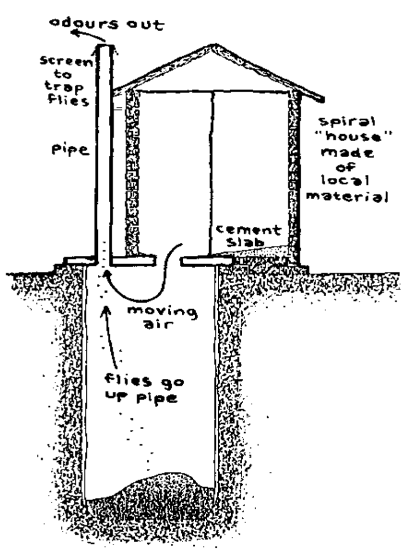
Part I
CE 412 A
Water Supply & Wastewater Disposal Systems
Dr Vinod Tare

Double Pit Dry Latrine

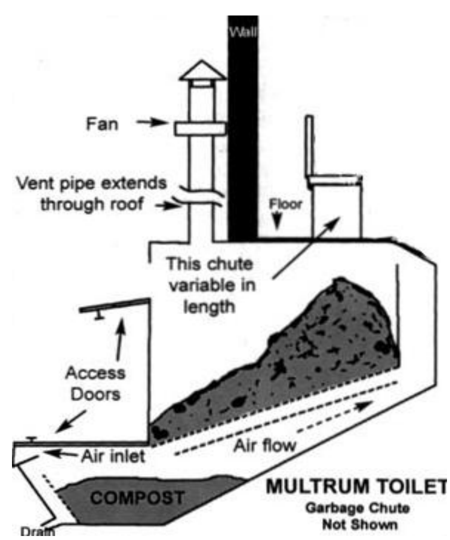


Such toilets contain two pits. One pit is used for six month and then closed for six months. During this time the second pit is used. The first pit is put back into operation after six months and the second pit is closed.

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.

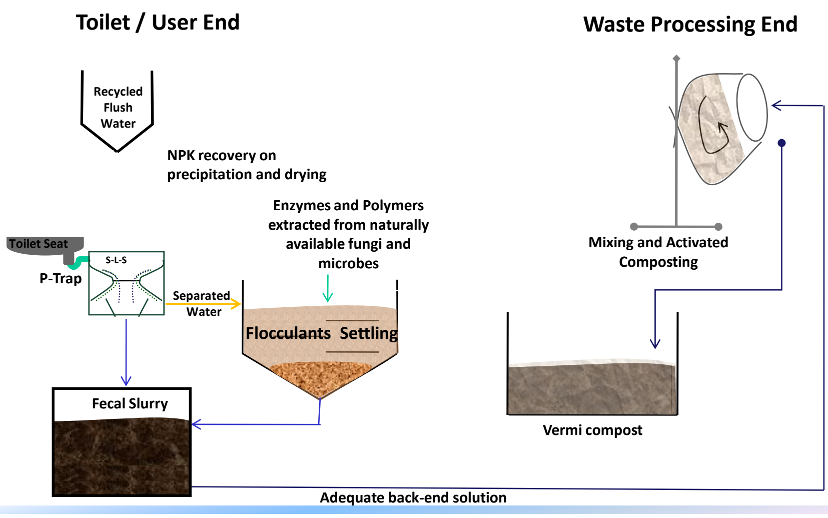
UDDTs divert all liquids i.e. urine and anal cleansing water, from the feces to keep the processing chamber contents dry. UDDTs make use of desiccation (dehydration) processes for the hygienically safe on-site treatment of human excreta. Adding wood ash, lime, sawdust, dry earth etc. after defecation 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.

Flush Toilets in Slums / Rural Areas

Zero-Discharge Toilet System (ZDTS)



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 (dry sanitation) 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.