→ Aerobic Treatment of Waste water

1. Suspended Culture Systems

   Organisms chase food.(organic matter)

   Eg: Activated Sludge Process. (ASP).

2. Attached Culture Systems

   Food chase organisms.

   Eg: Trickling Filters.

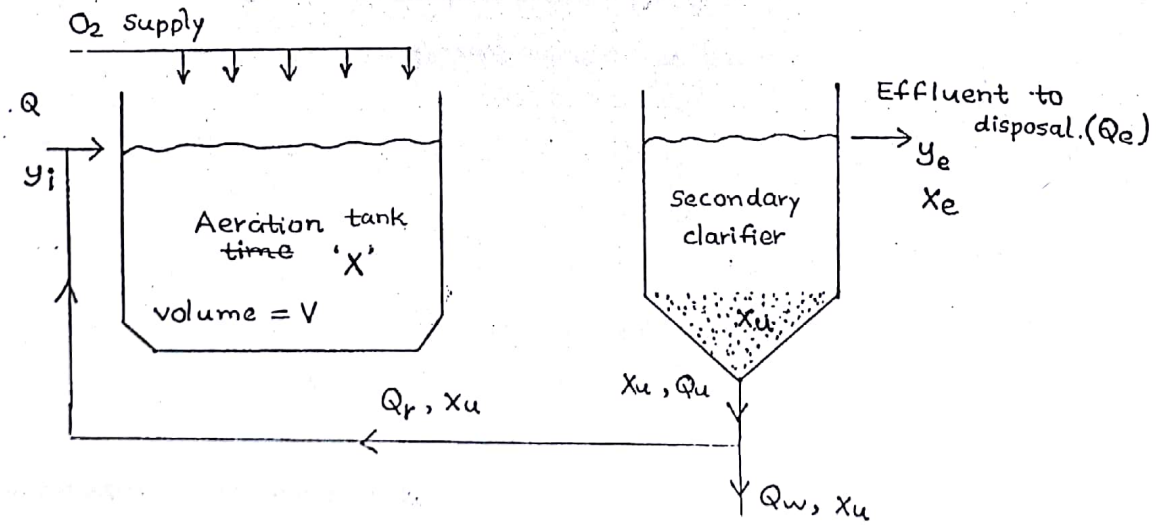
   Rotating biological contactors

   Biotowers. etc.

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Activated Sludge Process :

- ASP works on the principle of suspended culture system
- It employs two reactors
  1. Aeration tank : both food & O_2 supplied
  2. Secondary Settling tank : organisms seperated from waste water.



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$Q \rightarrow$ waste water flow rate.

$y_i \rightarrow$ influent BOD (S_i).

$V \rightarrow$ volume of aeration tank. ($L * B * H$).

$X \rightarrow$ MLSS (Mixed Liquor Suspended Solids)

(or)

MLVSS (Mixed Liquor Volatile Suspended Solids)

$Q_e \rightarrow$ flow rate effluent.

$X_e \rightarrow$ MLSS in effluent ($X_e = 0$ if not given)

$y_e \rightarrow$ Effluent BOD ($y_e = 0$ if not given)

$Q_u \rightarrow$ under flow rate (flow rate of sludge).

$X_u \rightarrow$ MLSS in under flow (MLSS in sludge).

$Q_w \rightarrow$ flow rate of waste sludge (taken to treatment).

$Q_r \rightarrow$ flow rate of recycling sludge.

* Design Parameters of ASP

1. $\frac{F}{M}$ ratio

$$\frac{F}{M} = \frac{\text{Mass of food applied/day}}{\text{Mass of micro organisms}}$$

$$\boxed{\frac{F}{M} = \frac{Q y_i}{VX}}$$

$\frac{F}{M}$: 0.2 day^{-1} to 0.4 day^{-1}

Sometimes, $\frac{F}{M}$ is considered as the ratio of mass of food consumed to mass of microorganisms.

$$\Rightarrow \frac{F}{M} = \frac{Q(y_i - y_e)}{VX}$$

(10% change may be there in value)

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2. Mean Cell residence time (MCRT), θ_c

- It is the average life of micro organisms (sludge age).
- It is defined as mass of cells (micro organisms or MLSS) in the reactor (aeration tank) to mass of cells wasted per day.

$$\theta_c = \frac{\text{Mass of MLSS in aeration tank.}}{\text{Mass of MLSS wasted/day}}$$

$$\boxed{\theta_c = \frac{VX}{Q_w X_u + Q_e X_e} = \frac{VX}{Q_w X_u + (Q - Q_w) X_e}}$$

$\{ Q = Q_e + Q_w \}$

θ_c : 4 to 15 days

3. BOD removal efficiency

$$\boxed{\eta = \left(\frac{y_i - y_e}{y_i}\right) * 100}$$

η : 85% - 95%

4. Volumetric loading rate (or) Organic loading rate (or) BOD loading rate. ($\forall_L$).

$$\forall_L = \frac{\text{Total BOD applied/day}}{\text{unit volume reactor}}$$

$$\boxed{\forall_L = \frac{Q y_i}{V}}$$

$\forall_L$: 0.3 to 0.6 kg of BOD/day/m^3

5. Aeration Period (Detention time).

$$\text{Aeration Period} = \frac{V}{Q} = \frac{L * B * H}{Q} \text{ ; 4 to 8 hr.}$$

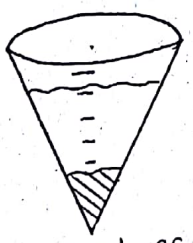
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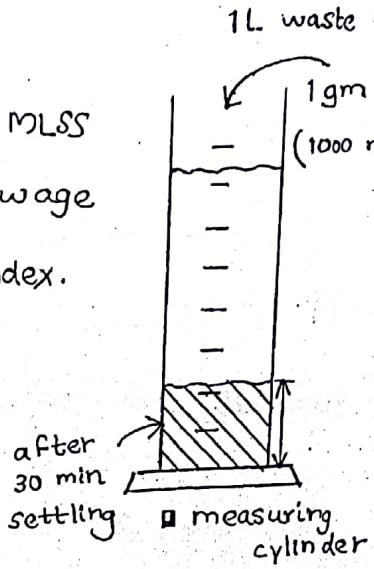
6. Sludge Volume Index, (SVI)

Space occupied in 'mL' by 1 gram MLSS after settling for 30 min from 1L sewage sample is known as Sludge Volume Index.

SVI : 50 to 150 mL/gm



imhoff cone.



7. Recycling Ratio

$$\frac{Q_r}{Q} = \frac{X}{\frac{10^6}{SVI} - X}$$

$$X_u \simeq \frac{10^6}{SVI}$$

$\frac{Q_r}{Q}$: 0.25 to 0.5

X_a : 1500 to 3000 mg/L

X_u : 9000 to 12000 mg/L

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4. $Q = 35000\ m^3/day$ $\quad X = 2500\ mg/L$

$V = 10,900\ m^3$ $\quad X_e = 30\ mg/L$

$y_i = 250\ mg/L$ $\quad X_u = 9700\ mg/L.$

$y_e = 20\ mg/L.$ $\quad Q_w = 220\ m^3/day.$

$$\text{Aeration period} = \frac{V}{Q} = \frac{10,900}{35000} = 0.311\ day = \underline{7.47\ hours}$$

5. $$\frac{F}{M} = \frac{Q y_i}{V X} = \frac{35000 \times 250}{10,900 \times 2500} = \underline{0.32}$$

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6. $\eta = \frac{y_i - y_e}{y_i} * 100 = \frac{250 - 20}{250} * 100 = 92\%$

7. Sludge age $= \frac{VX}{Q_w X_u + (Q - Q_w) X_e} = \frac{10,900 \times 2500}{9700 \times 220 + (35000 - 220) \times 30}$

$= 8.58$ days

2. $Q = 50 \times 10^6$ L/d $= 50000$ m³/d

$y_i = 180$ mg/L ; $\frac{F}{M} = 0.5$

$\frac{F}{M} = \frac{Qy_i}{VX}$

$0.5 = \frac{50000 \times 180}{V \times 1800}$

$V = 10000$ m³

301. $Q = 50000$ m³/day.

$y_i = 180$ mg/L.

$\forall_L = 0.55$ kg of BOD/day/m³.

$\forall_L = \frac{Qy_i}{V}$

$0.55 = \frac{50 \times 180}{V} \Rightarrow V = 16363$ m³

03. MLSS conc. $= 2000$ mg/L $= 2$ g/L

2 g/L MLSS occupies 176 mL.

∴ 1 g MLSS occupy $= \frac{176}{2} = 88$ ml/gm

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13. MLSS = 2800 mg/L = 2.8 g/L.

1 g occuppy = $\frac{200}{2.8}$ = 71.4

14. Q = 500 m^3/hour.

y_i = 150 mg/L

y_e = 10 mg/L.

θ Aeration period = $\frac{V}{Q}$ = 8 hours.

θ_c = 240 hours. = 10 days

V = 4000 m^3

MLSS = 2000 mg/L. = X

$$\frac{F}{M} = \frac{500 \times 150}{4000 \times 2000} = 0.225 \cdot day^{-1}$$

15. θ_c = $\frac{\text{mass of MLSS in aeration tank}}{\text{mass of MLSS wasted.}}$

$$\frac{240}{24} = \frac{4000 \times 2000 \cdot \times 10^3 \times 10^{-6}}{\text{Mass of MLSS wasted.}}$$

Mass of MLSS wasted = $\frac{4 \times 2000}{10}$ = 800 kg/day

16. MLSS = 4000 mg/L = 4 g/L

4 g/L of MLSS occupies 200 mL.

SVI = $\frac{200}{4}$ = 50 ml/g

12. V = L * B * H = 30 X 14 X 4.3 X = 1500 mg/L.

Q = 0.0796 m^3/s

y_i = 130 mg/L

X_u = 9100 mg/L

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- Aeration period $= \frac{V}{Q} = \frac{30 \times 14 \times 4.3}{0.0796} = 22688.4$ s.

$= 6.3$ hours

$- \frac{F}{M} = \frac{Q y_i}{V X_{MLVSS}} = \frac{0.0796 \times 130}{30 \times 14 \times 4.3 \times 1500} = 3.81 \times 10^{-6}\ s^{-1}$

$= 0.33\ day^{-1}$

$- SVI = \frac{230}{2.1} = 109.52.\ mL/g.$

$- \frac{Qr}{Q} = \frac{X}{\frac{10^6}{SVI} - X.}$

Return sludge rate, $Qr = \frac{0.0796 \times 21500}{\frac{10^6}{109.52} - 21500} = \frac{0.024}{0.0156}\ m^3/s$

$= 2054.21\ m^3/day$

08. SVI = 88 mL/g

$Xu = \frac{10^6}{SVI} = \frac{10^6}{88} = 11364$ mg/L

10. $V = 400\ m^3$

Q. Following data pertains to ASP.

Waste water flow rate, $Q = 10,000\ m^3/day$

Influent BOD, $y_i = 150$ mg/L

Effluent BOD, $y_e = 20$ mg/L

MLSS $= X = 3000$ mg/L

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MLSS in under flow, $X_u = 10000$ mg/L

$$\frac{F}{M} = 0.25 \text{ day}^{-1}$$

$$\theta_c = 10 \text{ days.}$$

Find the following:

a) Volume of aeration tank

b) aeration period.

c). BOD removal efficiency.

d) Volumetric loading rate.

e) Mass of solids wasted per day.

f). Volume of solids wasted per day

g) SVI.

h) Recycling ratio.

a) $\frac{F}{M} = \frac{Q y_i}{V X}$.

$$0.25 = \frac{10000 \times 150}{V \times 3000}$$

$$V = \underline{2000} \text{ m}^3$$

b) Aeration period, $= \frac{V}{Q} = \frac{2000}{10000} = 0.2 \text{ day} = \underline{4.8 \text{ hours}}$

c). BOD removal efficiency, $\eta = \frac{y_i - y_e}{y_i} \times 100 = \frac{150-20}{150} \times 100$

$= \underline{86.67\ \%}$

d). Volumetric loading rate, $V_L = \frac{Q y_i}{V} = \frac{F}{M} X. = 0.25 \times 3000$

$= \underline{0.75}$ kg of BOD /day/m^3

e). $\theta_c = \frac{\text{Mass of MLSS in aeration tank}}{\text{Mass of solids wasted per day.}}$

Mass of solids wasted per day $= \frac{2 \times 3000}{10} = \underline{600}$ kg /day.

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f) Volume of solids wasted per day = $\frac{\text{Mass of solids wasted}}{X_u}$

X_u = mg/L

$= \frac{600 \text{ kg/day}}{10000 \times 10^{-6}}$

OR

$= 60000 \text{ L} = 60 \text{ m}^3/\text{day}$

g) ~~SVI~~ $= Q_w X_u + (Q - Q_w) X_e = 600 \text{ kg/day}$

$\because X_e$ is not given. $\Rightarrow X_e = 0$.

$Q_w X_u = 600 \text{ kg/day}$.

Volume of solids wasted per day = Q_w.

$Q_w * X_u = 600 \text{ kg/day}$

(mLD). (mg/L)

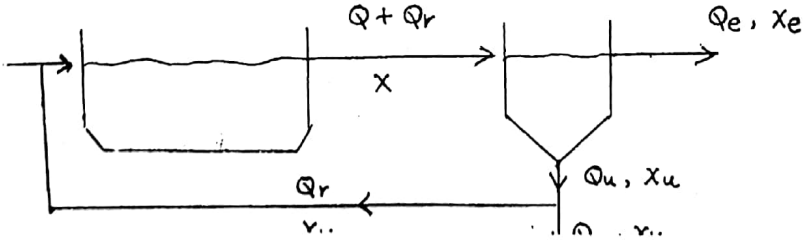
$\therefore Q_w = \frac{600}{10000} = 0.06 \text{ MLD} = 60 \text{ m}^3$

g) $X_u \simeq \frac{10^6}{SVI}$

$\therefore SVI = \frac{10^6}{X_u} = \frac{10^6}{10000} = 100$

h) $\frac{Q_r}{Q} = \frac{X}{\frac{10^6}{SVI} - X} = \frac{3000}{\frac{10^6}{100} - 3000} = 0.42$ (approximate)

* To find exact value of $\frac{Q_r}{Q}$, write mass balance equation around secondary clarifier.



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$\Rightarrow$ Mass in = mass out.

$$(Q + Q_r)\,X = Q_e X_e + Q_u X_u.$$

$$= (Q - Q_w)\,X_e + (Q_r + Q_w)\,X_u.$$

But $X_e = 0$ (not given).

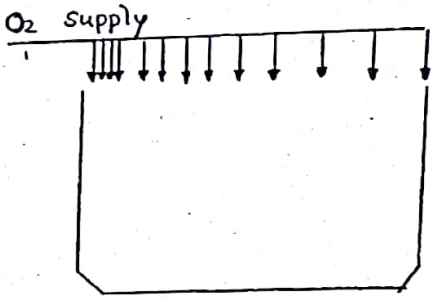
$$(10000 + Q_r)\,3000 = (Q_r + 60)\,10000$$

$$Q_r = 4277 \text{ m}^3/\text{day}$$

$$\frac{Q_r}{Q} = \frac{4277}{10000} = 0.43$$

→ Modifications made to conventional ASP:

1. Tappered Aeration.

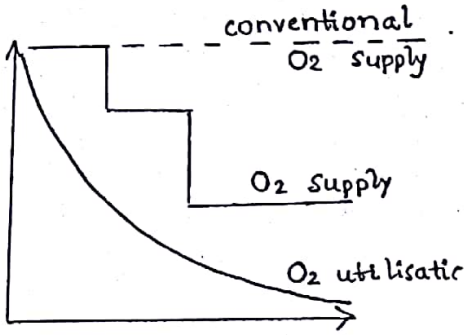


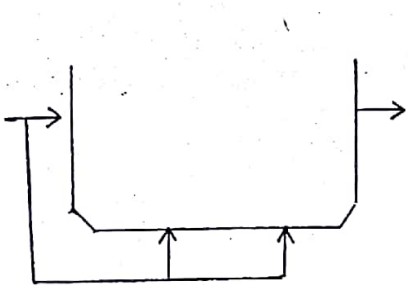
-Varying the O_2 supply from inlet to outlet.

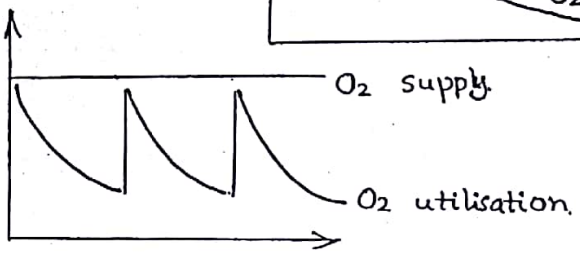
2. Stepped Aeration.

-Food supply @ multiple points.

-This will bring down gap blw O_2 supply and utilisation.







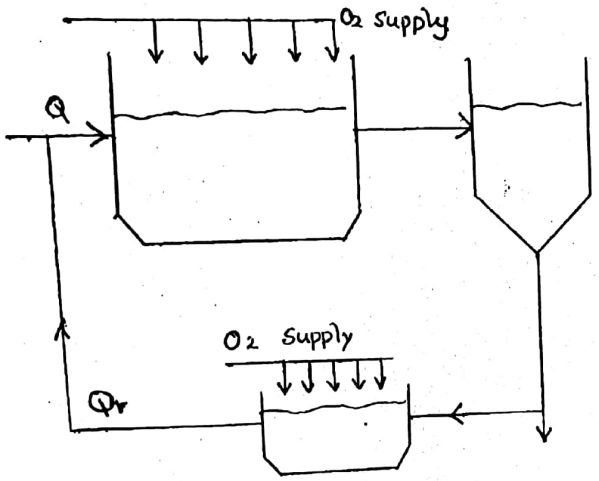
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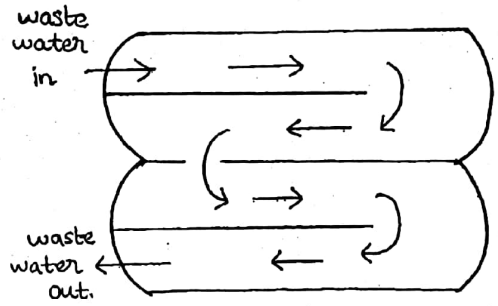
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3. Extended aeration

- No artificial supply of oxygen
- Waste water exposed to long periods to free atmosphere.
- Main features of extended aeration are:
  (i) Long detention time (18 to 24 hours)
  (ii) low $\frac{F}{M}$ ratio. (0.05 to 0.1 day^{-1}).
- An oxidation ditch (oval shaped channel) works on the principle of extended aeration.

4. Contact Stabilisation.



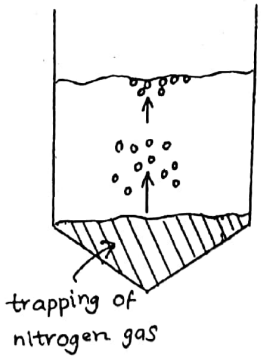


Plan of oxidation ditch.

→ Operational Problems associated with ASP

(i) Rising of Sludge.

Settled sludge contains large number of live cells. They oxidise the dead cells and release nitrogen. This nitrogen causes the sludge to rise and increases the Xe value.



Remedies:
- Increase Q_r — decrease θ_c
- decrease Q

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(ii) Bulking of Sludge.

– large number of dead cells in settled sludge and growth of filamentous organisms. (occurs when SVI $\simeq$ 150)

– Remedies:

(i) Bring down SVI ($\simeq$ 100).

(ii) increase pH.

(iii) Increase DO

(iv) Add nutrients.

(v) decrease $\frac{F}{M}$ ratio.

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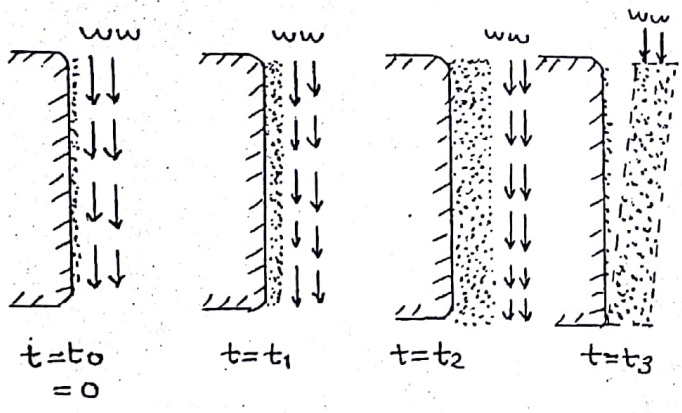
06. TRICKLING FILTERS

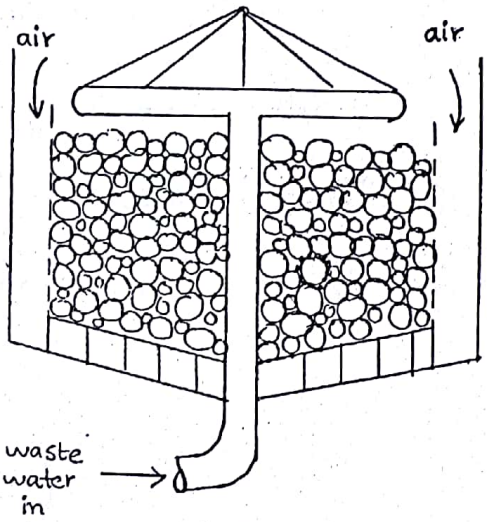
- Trickling Filters works on the principle of attached culture system.

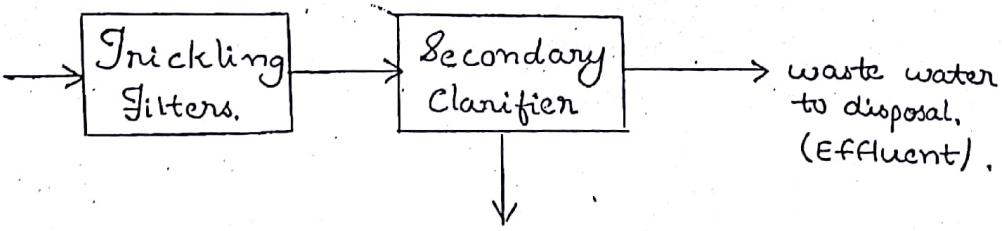
- Trickling Filters consists bed of highly permeable media to which micro organisms are attached through which waste water is passed.

- usually inert material is used as media. Size of media range from 25 mm to 100 mm size. (broken bricks or broken stones). Packed randomly in the fitters. To the surface of the media, microbial films are attached.

- Detachement of attached biofilm from the surface is called sloughing.







- Circular tank of diameter 3.6 to 60 m

Depth = 1.8 m

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→ Design of Trickling Filters.

- based on empirical formula by NRC (National Research Council)

* NRC Equation.

Efficiency of BOD removal, $\eta = \frac{y_i - y_e}{y_i} * 100$

$$\eta = \frac{100}{1 + 0.0044\sqrt{\frac{Qy_i}{VF}}}$$

where $\frac{Qy_i}{V}$ → organic loading rate (or) BOD loading rate.
(kg of BOD/day/ha.m)

V → volume of trickling filter (ha.m).

Qy_i → total BOD (kg/day)

F → recirculation factor.

$$F = \frac{1 + R/I}{\left(1 + 0.1\,R/I\right)^2}$$

where $\frac{R}{I}$: recirculation ratio

$$\frac{R}{I} = \frac{\text{amount of effluent recycled}}{\text{amount of influent applied}}$$

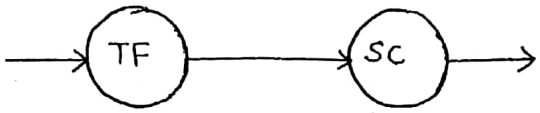
* Based on F, trickling filters are classified as:

1. Standard Rate Trickling Filters.

$R = 0 \Rightarrow \frac{R}{I} = 0$ & $F = 1$

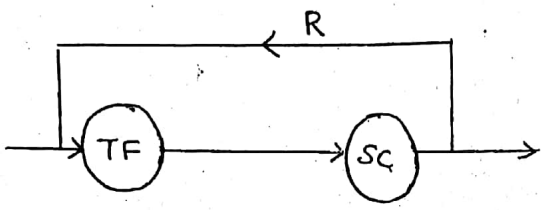
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2. High Rate Trickling Filters.

$R > 0 \Rightarrow \frac{R}{I} > 0$ & $F > 1$.



$$\eta = \frac{100}{1 + 0.0044\sqrt{\frac{Qy_i}{VF}}} \Rightarrow \downarrow V \propto Qy_i\downarrow$$

$$\downarrow V \propto \frac{1}{F\uparrow}$$

0-92

Q1. $y_i = 200$ mg/L, $y_e = 40$ mg/L

$$\eta = \frac{200 - 40}{200} * 100 = \underline{\underline{80\%}}$$

Q.2 $\frac{Qy_i}{V} = 0.175$ kg/m³/day $= 1750$ kg/ha.m/day.

$y_i = 150$ mg/L. ; $F = 1$ (if not given).

1 ha. m $= 10^4$ m³.

$1\text{m}^3 = \frac{1}{10^4}$ ha.m.

$$\eta = \frac{150 - y_e}{150} * 100 = \frac{100}{1 + 0.0044\sqrt{\frac{1750}{1}}}$$

$$\Rightarrow y_e = \underline{\underline{23.3}} \text{ mg/L}$$

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03. $Q = 6$ MLD.

$y_i = 150$ mg/L.

a). Organic loading rate $= 175\ g/m^3/day.$

$= 1750\ kg/ha.m/day.$

Surface loading rate $= 2500\ l/m^2/day.$

$= 2.5\ m^3/m^2/day$

$$0.175\ kg/m^3/day = \frac{6 \times 150}{V}$$

Volume of filter unit, $V = 5142.86\ m^3.$

Surface area $= \frac{6 \times 10^3}{2.5} = 2400\ m^2.$

Depth of filter unit $= \frac{5142.86}{2400} = \underline{\underline{2.143}}\ m$

b)

$$\eta = \frac{100}{1 + 0.004\sqrt{\frac{Qy_i}{VF}}} = \frac{100}{1 + 0.0044\sqrt{1750}}$$

$= \approx 84.45\%$

c). $\frac{y_i - y_e}{y_i} \times 100 = 84.45.$

$\Rightarrow y_e = \underline{\underline{23.32}}$ mg/L

04: $y_i = 120$ mg/L, $y_e = 20$ mg/L.

$Q = 2200\ m^3/day.$

$$\eta = \frac{y_i - y_e}{y_i} * 100 = \frac{120 - 20}{120} * 100$$

$= 83.33\%$

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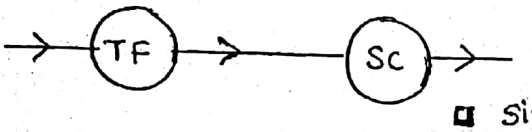
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$$\frac{R}{I} = \frac{4000}{2200} = 1.81$$

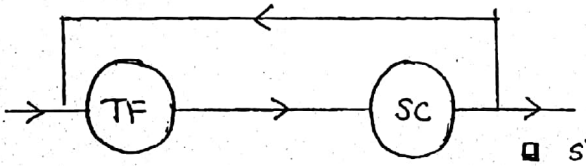
$$F = \frac{1 + R/I}{\left(1 + 0.1\,\frac{R}{I}\right)^2} = \frac{1 + 1.8}{(1 + 0.1 \times 1.8)^2} = 2.02.$$

$$0.83 = \frac{1}{1 + 0.0044\sqrt{\frac{2.2 \times 120}{V \times 2.02}}}$$

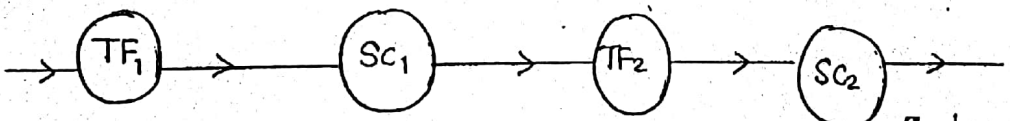
$$\Rightarrow V = 633\ m^3$$



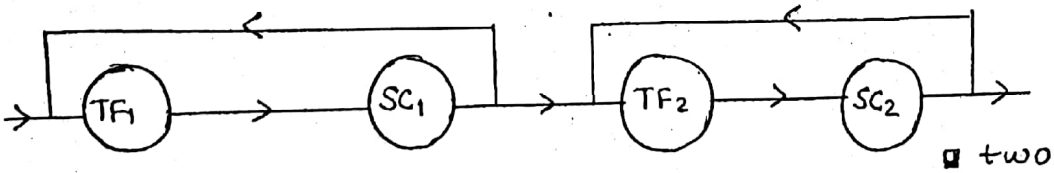
- Single stage standard TF



- single stage high rate TF



- two stage std. rate TF



- two stage high rate TF

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→ Operational Problems associated with TF

1. Psychoda Problems – fly nuisance.

2. Ponding Problems – odour nuissance.

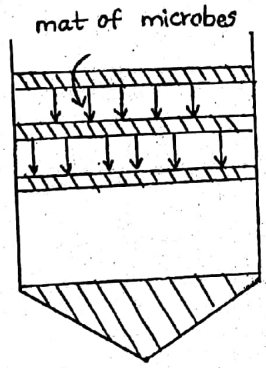
These problems are not present in ASP (relatively modern)

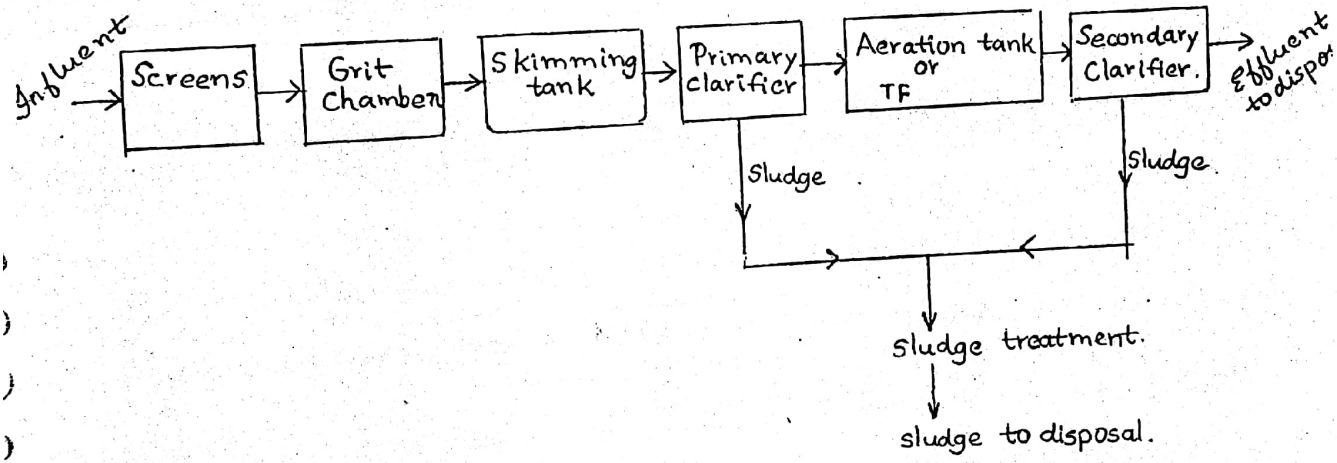
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SECONDARY CLARIFIER

- Secondary settling tank
- Secondary clarifiers follow TYPE IV settling
- Type IV settling is known as 'Compression settling or Zone settling'
- Design of secondary clarifier is based on 'Solid's Flux Rate'.





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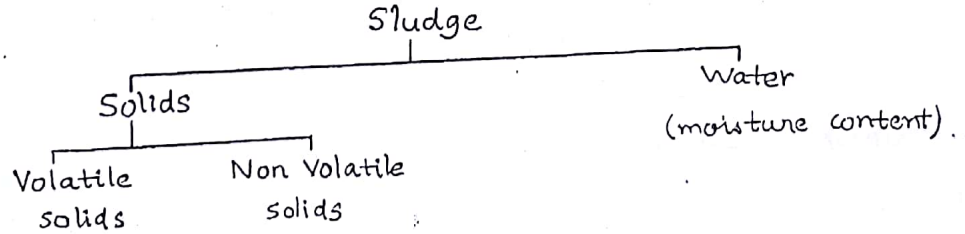
07. SLUDGE TREATMENT

- Sludge treatment is aimed to achieve:
  1. Volume reduction
  2. Strength reduction.
- Volume and strength reduction of sludges are achieved by:
  1. Sludge thickening.
  2. Sludge digestion
  3. Sludge conditioning.
  4. Sludge dewatering and drying.

- Sludge thickening is achieved by concentrating the sludges by expulsion of water.
- Sludge digestion is achieved by anaerobic decomposition to reduce strength.
- Sludge conditioning is done to improve drainability of sludges.
- Sludge dewatering and drying removes remaining moisture from sludge.

→ Composition of Sludge.

- Sludge is a mixture of water and solids.
- It contains 0.25% to 12% solids and.
  88% to 99.75% water



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→ Mass Volume Relation of Sludge.

$$\rho_{sludge} = \frac{\text{Mass of Sludge}}{\text{Volume of Sludge.}}$$

$$S_{sludge} = \frac{\rho_{sludge}}{\rho_{water}}$$

A + B + C ⟶ D

%A	%B	%C.	100% D.
(S_A)	(S_B)	(S_C)	$S_D = ?$

$$\frac{100}{S_D} = \frac{\%\,A}{S_A} + \frac{\%\,B}{S_B} + \frac{\%\,C}{S_C}$$

$$\frac{100}{S_{solids}} = \frac{\%\text{ volatiles}}{S_{vol}} \qquad \frac{\%\text{ non-volatiles}}{S_{non-vol}}$$

$$\frac{100}{S_{sludge}} = \frac{\%\text{ solids}}{S_{solids}} + \frac{\%\text{ water}}{S_{water}}$$

$$\frac{100}{\rho_{sludge}} = \frac{\%\text{ solids}}{\rho_{solids}} + \frac{\%\text{ water}}{\rho_{water}}$$

Q. Sludge contains 98% water and remaining is solid. 70% solids are volatile whose sp.gr. is 1.7 and remaining are non volatile with sp.gr. 2.7. Find sp.gr of sludge and ρ_{sludge}.

$$\frac{100}{S_{solids}} = \frac{70}{1.7} + \frac{30}{2.7} \Rightarrow S_{solids} = 1.91$$

$$\frac{100}{S_{sludge}} = \frac{98}{S_{water}} + \frac{2}{S_{solids}}$$

$$= \frac{98}{1} + \frac{2}{1.91} \Rightarrow S_{sludge} = 1.0096$$

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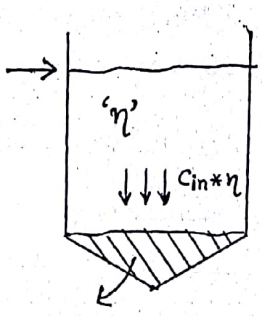
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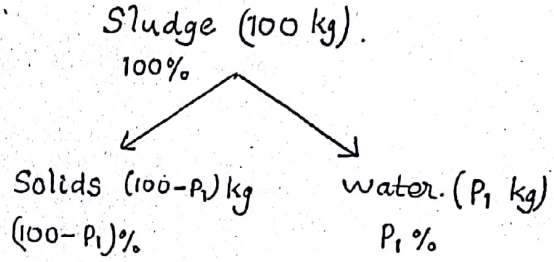
$$\rho_{sludge} = 1.0096 \times 1000 = \underline{1009.6}\ kg/m^3$$

→ Volume Reduction of Sludge.

Let P_1 be the initial percent moisture content of sludge and V_1 be the initial volume of sludge.

If % moisture content of sludge is reduced to P_2 and volume decreased to V_2.





conc. of solids in sludge = conc. of solids removed = $C_{in} * \eta$.

Concentration of solids (dry) in sludge = conc. of solids applied * efficiency of removal.

$$C_{sludge} = C_{in} * \eta \quad (mg/L).$$

Total mass of dry solids in sludge, 'M' (kg/day) = Q (MLD) * C_{sludge} (mg/L).

$(100 - P_1)$ kg of dry solids produce = 100 kg of wet sludge.

1 kg dry solids produce = $\frac{100}{100 - P_1}$ kg of wet sludge.

M kg/day of dry solid produce = $\frac{100}{100 - P_1} * M$ kg/day of wet sludge.

Mass of sludge produced at m.c P_1 = $\frac{100}{(100 - P_1)} * M$ kg/day

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→ Strength Reduction

- Strength reduction of sludge; ie, BOD removal is carried out by anaerobic decomposition known as Sludge digestion.
- Sludge digestion occur in two stages
  1. Acid Fermentation stage.
  2. Methane Fermentation.
- Acid Fermentation.

Sludges ⟶ Acid Formers. (anaerobic acid forming bacteria) ⟶ Acids { Acetic acid, Butyric acid, Propionic acid. }

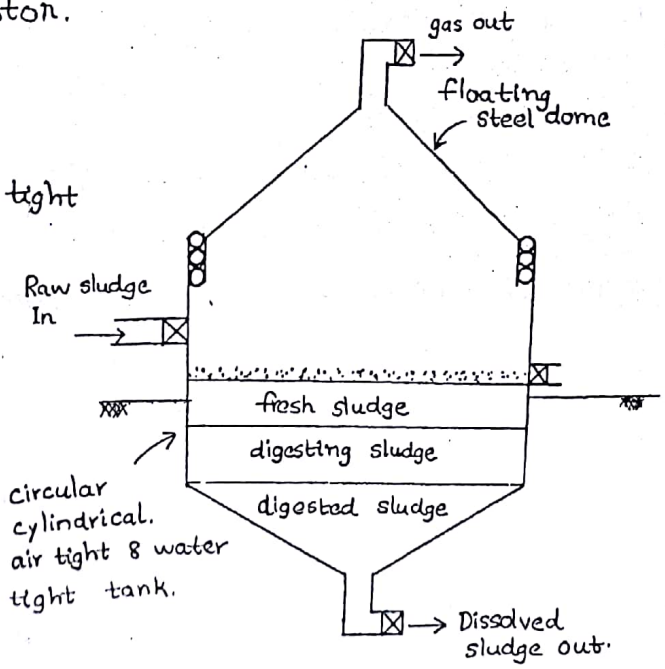
- Methane Fermentation

Acids ⟶ Methane formers. (anaerobic methane forming bacteria) ⟶ Gases. { CH_4, CO_2, H_2S }

- Sludges are anaerobically digested by using a reactor known as 'Sludge Digestor'.

→ Sludge Digestor

- air tight and water tight circular cylindrical tank
- diameter 3m to 60 m
- height 6m to 12 m
- provided with floating steel dome to collect gases.



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- Capacity of sludge digester. $= \left(\frac{V_f + V_d}{2}\right) * \text{digestion period}$
  (Volume of digester)

- Digestion period depends on type of microorganisms employed
  (i) Mesophilic organisms → 30 to 60 days (temp 20°c - 40°c)
  (ii) Thermophilic organism → 15 to 25 days (temp 45°c - 60°c).

- Rate of gas production:

  0.9 m^3/kg volatiles destroyed.

  70% → CH_4

  20-30% → CO_2

  Traces → remaining gases.

- Calorific value (Energy content) = 5200 to 5800 kcal/m^3 of CH_4.

V_f → volume of fresh sludge/day.

$$V_f = V_1 = \frac{100}{(100 - P_1)} * \frac{M}{(\rho_{slu})_1}$$

V_d → volume of digested sludge/day.

$$V_d = V_2 = \frac{100}{(100 - P_2)} * \frac{M}{(\rho_{sludge})_2}$$

P-96

08. Concentration of volatile suspended solids destroyed.

$$= 200 * \frac{90}{100} * \frac{70}{100} * \frac{65}{100}$$

$$= 81.9 \text{ mg/L}.$$

Total volatiles destroyed = 1 ML * 81.9 mg/L.

$$= 81.9 \times 10^6 \text{ mg} = \underline{\underline{81.9 \text{ kg}}}$$

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Volume of gas produced = rate of gas production * total volatiles destroyed.

$= 0.9\ m^3/kg * 81.9 = 73.71\ m^3$

CH_4 content of gas $= 73.71 * 0.65 = 47.91\ m^3$

Total fuel value (total energy) $= 5200\ kcal/m^3 * 47.91$

$= 249139.8\ kcal$

02. Mass of dry solids produced per day $= Q * C_{in} * \eta$.

$= 20 * 300 * 0.65$

$= 3900\ kg/day.$

$S_{raw\ sludge} = 1.02 \qquad P_{raw\ sludge} = 95\%$

$$\frac{100}{1.02} = \frac{5}{S_{solids}} + \frac{95}{1.}$$

$S_{solids} = 1.645$

$S_{digested\ sludge} = ?$

$$\frac{100}{S_{d.s}} = \frac{15}{1.645} + \frac{85}{1}$$

$S_{ds} = 1.0625$

$$V_f = V_1 = \frac{100}{100-95} * \frac{3900}{1.02 \times 1000} = 76.47\ m^3$$

$$V_d = V_2 = \frac{100}{100-85} * \frac{3900}{1.062 \times 1000} = 24.47\ m^3$$

Volume of digester $= \left(\frac{V_f + V_D}{2}\right) \times 30 = 1514.11\ m^3$

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⊙ If the height of digester is 10m, find diameter.

$$\text{c/s area} = \frac{\text{Volume}}{\text{Height}} = \frac{1514.25}{10} = 151.425\ m^2.$$

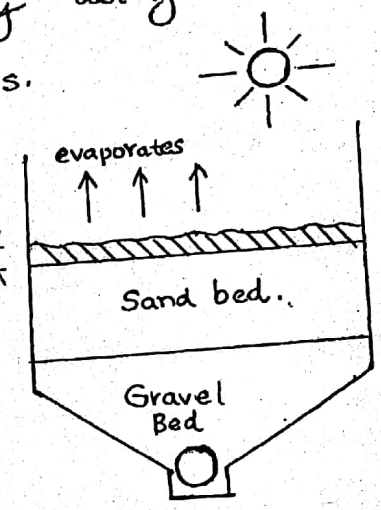
$$\frac{\pi}{4} d^2 = 151.425$$

$$\Rightarrow \text{diameter, } d = 13.88\ m$$

Digested sludges are dewatered and dried by using solar drying beds known as sludge drying beds.

- Dry sludge cake is used as manure (or) disposed by incineration (burning) or burying or filling low lying areas.

- 'Elutriation': washing of sludge before dewatering by mechanical driers.



◘ sludge drying bed.

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Classification of Micro organisms

1. *Nutritional Requirements*: On the basis of chemical form of carbon required, microorganisms are classified as
    a. Autotrophic: organisms that use CO_2 or HCO_3^- as their sole source of carbon.
    b. Heterotrophic: organisms that use carbon from organic compounds.

Energy Requirements: On the basis of energy source required, microorganisms are classified as

. Phototrophs: organisms that use light as their energy source.
    a. Chemotrophs: organisms that employ oxidation-reduction reactions to provide energy. They are further classified on the basis of chemical compounds oxidized (i.e., electron donor)
        i. Chemoorganotrophs: Organisms that use complex organic molecules as their electron donor.
        ii. Chemoautotrophs: Organisms that use simple inorganic molecules such as hydrogen sulfide or ammonia as their electron donor.

Temperature Range: On the basis of temperature range within which they can proliferate, microorganisms are classified as

. Psychrophilic: organisms whose growth is optimum within 15 to 30°C.
    a. Mesophilic: organisms whose growth is optimum within 30 to 45°C.
    b. Thermophilic: organisms whose growth is optimum within 45 to 70°C.

Oxygen Requirements: On the basis of oxygen requirement microorganisms are classified as

. Aerobes: organisms that use molecular oxygen as electron acceptor.
    a. Anaerobes: organisms that use some molecule other than molecular oxygen as electron acceptor.
    b. Facultative organisms : organisms that can use either molecular oxygen or some other chemical compound as electron acceptor.

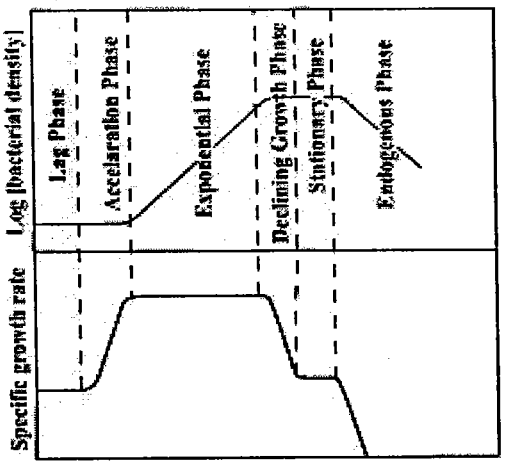
Growth Pattern of Micro organisms

When a small number of viable bacterial cells are placed in a close vessel containing excessive food supply in a suitable environment, conditions are established in which unrestricted growth

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takes place. However, growth of an organism do not go on indefinitely, and after a characteristic size is reached, the cell divides due to hereditary and internal limitations. The growth rate may follow a pattern similar to as shown in figure:

Characteristic Growth Curves of Cultures of Microorganisms



The curve shown may be divided into six well defined phases:

1. *Lag Phase*:adaptation to new environment, long generation time and null growth rate.
2. *Accelaration phase*: decreasing generation time and increasing growth rate.
3. *Exponential phase*: minimal and constant generation time, maximal and constant specific growth rate and maximum rate of substrate conversion.
4. *Declining growth phase*: increasing generation time and decreasing specific growth rate due to gradual decrease in substrate concentration and increased accumulation of toxic metabolites.
5. *Stationary phase*: exaustion of nutrients, high concentration of toxic metabolites, and cells in a state of suspended animation.
6. *Endogenous phase*: endogenous metabolism, high death rate and cell lysis.

Biomass Growth Rate

The most widely used expression for the growth rate of micro organisms is given by Monod:

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Totalrate of microbial growth, $\frac{dx}{t} = \frac{\mu_m XS}{K_s + S}$

where,

μ_m=Θmaximum specific growth rate

X = micro organism concentration

S = substrate concentration

K_s= substrate concentration at one half the maximum growth rate

Similarly, rate of substrate utilization,

$$\frac{dS}{dt} = \frac{kX\,S}{K_s + S}$$

where,

k = maximum specific substrate utilization rate

Maintenance as Endogenous Respiration

Net growth rate of micro organisms is computed by subtracting from the total growth rate, the rate of micro organisms endogenously decayed to satisfy maintenance energy requirement. Therefore,

Net rate of microbial growth $= \frac{\mu_m X\,S}{K_s + S} - k_d X$

where,

k_d = endogenous decay coefficient

Growth Yield

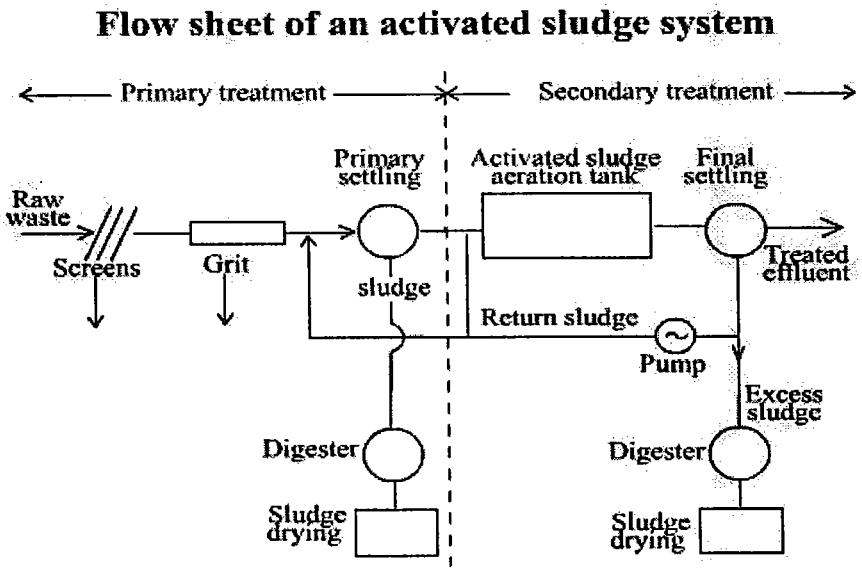
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Growth yield is defined as the incremental increase in biomass which results from the utilization of the incremental amount of substrate. The maximum specific growth rate is given by: $\mu_m = Y.k$

where, Y is the maximum yield coefficient and is defined as the ratio of maximum mass of cells formed to the mass of substrate utilized. The coefficients Y, k_d, k and K_s are designated as kinetic coefficients. The values of kinetic coefficients depend upon the nature of wastewater and operational and environmental conditions in biological reactor. The biological reactors can be completely mixed flow or plug flow reactor with or without recycle.

Activated Sludge Process

The most common suspended growth process used for municipal wastewater treatment is the activated sludge process as shown in figure:



Activated sludge plant involves:

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1. wastewater aeration in the presence of a microbial suspension,
2. solid-liquid separation following aeration,
3. discharge of clarified effluent,
4. wasting of excess biomass, and
5. return of remaining biomass to the aeration tank.

In activated sludge process wastewater containing organic matter is aerated in an aeration basin in which micro-organisms metabolize the suspended and soluble organic matter. Part of organic matter is synthesized into new cells and part is oxidized to CO_2 and water to derive energy. In activated sludge systems the new cells formed in the reaction are removed from the liquid stream in the form of a flocculent sludge in settling tanks. A part of this settled biomass, described as activated sludge is returned to the aeration tank and the remaining forms waste or excess sludge.

Activated Sludge Process Variables

The main variables of activated sludge process are the mixing regime, loading rate, and the flow scheme.

Mixing Regime

Generally two types of mixing regimes are of major interest in activated sludge process: *plug flow* and *complete mixing*. In the first one, the regime is characterized by orderly flow of mixed liquor through the aeration tank with no element of mixed liquor overtaking or mixing with any other element. There may be lateral mixing of mixed liquor but there must be no mixing along the path of flow.

In complete mixing, the contents of aeration tank are well stirred and uniform throughout. Thus, at steady state, the effluent from the aeration tank has the same composition as the aeration tank contents.

The type of mixing regime is very important as it affects (1) oxygen transfer requirements in the aeration tank, (2) susceptibility of biomass to shock loads, (3) local environmental conditions in the aeration tank, and (4) the kinetics governing the treatment process.

Loading Rate

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A loading parameter that has been developed over the years is the *hydraulic retention time* (HRT), Θ, d

$$\Theta = \frac{V}{Q}$$

V= volume of aeration tank, m^3, and Q= sewage inflow, m^3/d

Another empirical loading parameter is *volumetric organic loading* which is defined as the BOD applied per unit volume of aeration tank, per day.

A rational loading parameter which has found wider acceptance and is preferred is *specific substrate utilization rate*, q, per day.

$$q = \frac{Q(S_O - S_e)}{V X}$$

A similar loading parameter is *mean cell residence time* or *sludge retention time* (SRT), Θ_c, d

$$\Theta_c = \frac{V X}{Q_w X_r + (Q - Q_w X_e)}$$

where S_O and S_e are influent and effluent organic matter concentration respectively, measured as BOD_5 (g/m^3), X, X_e and X_r are MLSS concentration in aeration tank, effluent and return sludge respectively, and Q_w= waste activated sludge rate.

Under steady state operation the mass of waste activated sludge is given by

$$Q_w X_r = YQ(S_O - S_e) - k_d XV$$

where Y= maximum yield coefficient (microbial mass synthesized / mass of substrate utilized) and k_d = endogenous decay rate (d^{-1}) .

From the above equation it is seen that $1/\Theta_c = Yq - k_d$

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If the value of S_e is small as compared S_O, q may also be expressed as *Food to Microorganism ratio*, F/M

$F/M = Q(S_O - S_e) / XV = QS_O / XV$

The Θ_c value adopted for design controls the effluent quality, and settleability and drainability of biomass, oxygen requirement and quantity of waste activated sludge.

Flow Scheme

The flow scheme involves:

- the pattern of sewage addition
- the pattern of sludge return to the aeration tank and
- the pattern of aeration.

Sewage addition may be at a single point at the inlet end or it may be at several points along the aeration tank. The sludge return may be directly from the settling tank to the aeration tank or through a sludge reaeration tank. Aeration may be at a uniform rate or it may be varied from the head of the aeration tank to its end.

Conventional System and its Modifications

The conventional system maintains a plug flow hydraulic regime. Over the years, several modifications to the conventional system have been developed to meet specific treatment objectives. In *step aeration* settled sewage is introduced at several points along the tank length which produces more uniform oxygen demand throughout. *Tapered aeration* attempts to supply air to match oxygen demand along the length of the tank. *Contact stabilization* provides for reaeration of return activated sludge from from the final clarifier, which allows a smaller aeration or contact tank. *Completely mixed* process aims at instantaneous mixing of the influent waste and return sludge with the entire contents of the aeration tank. Extended aeration process operates at a low organic load producing lesser quantity of well stabilized sludge.

Design Consideration

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The items for consideration in the design of activated sludge plant are aeration tank capacity and dimensions, aeration facilities, secondary sludge settling and recycle and excess sludge wasting.

Aeration Tank

The volume of aeration tank is calculated for the selected value of Θ_c by assuming a suitable value of MLSS concentration, X.

$$VX = \frac{YQ\Theta_c(S_O - S)}{1+\Theta k_d \Theta_c}$$

Alternately, the tank capacity may be designed from

$$F/M = QS_O / XV$$

Hence, the first step in designing is to choose a suitable value of Θ_c *(or F/M)* which depends on the expected winter temperature of mixed liquor, the type of reactor, expected settling characteristics of the sludge and the nitrification required. The choice generally lies between 5 days in warmer climates to 10 days in temperate ones where nitrification is desired alongwith good BOD removal, and complete mixing systems are employed.

The second step is to select two interrelated parameters *HRT, t and MLSS concentration*. It is seen that economy in reactor volume can be achieved by assuming a large value of X. However, it is seldom taken to be more than 5000 g/m^3. For typical domestic sewage, the MLSS value of 2000-3000 mg/l if conventional plug flow type aeration system is provided, or 3000-5000 mg/l for completely mixed types. Considerations which govern the upper limit are: initial and running cost of sludge recirculation system to maintain a high value of MLSS, limitations of oxygen transfer equipment to supply oxygen at required rate in small reactor volume, increased solids loading on secondary clarifier which may necessitate a larger surface area, design criteria for the tank and minimum HRT for the aeration tank.

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The length of the tank depends upon the type of activated sludge plant. Except in the case of extended aeration plants and completely mixed plants, the aeration tanks are designed as long narrow channels. The width and depth of the aeration tank depends on the type of aeration equipment employed. The depth control the aeration efficiency and usually ranges from 3 to 4.5 m. The width controls the mixing and is usually kept between 5 to 10 m. Width-depth ratio should be adjusted to be between 1.2 to 2.2. The length should not be less than 30 or not ordinarily longer than 100 m.

Oxygen Requirements

Oxygen is reqiured in the activated sludge process for the oxidation of a part of the influent organic matter and also for the endogenous respiration of the micro-organisms in the system. The total oxygen requirement of the process may be formulated as follows:

$$O_2 \text{ required (g/d)} = \frac{Q(S_O - S)}{f} - 1.42\, Q_w X_r$$

where, f = ratio of BOD_5 to ultimate BOD and 1.42 = oxygen demand of biomass (g/g)

The formula does not allow for nitrification but allows only for carbonaceous BOD removal.

Aeration Facilities

The aeration facilities of the activated sludge plant are designed to provide the calculated oxygen demand of the wastewater against a specific level of dissolved oxygen in the wastewater.

Secondary Settling

Secondary settling tanks, which receive the biologically treated flow undergo zone or compression settling. *Zone settling* occurs beyond a certain concentration when the particles are close enough together that interparticulate forces may hold the particles fixed relative to one another so that the whole mass tends to settle as a single layer or

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"blanket" of sludge. The rate at which a sludge blanket settles can be determined by timing its position in a settling column test whose results can be plotted as shown in figure.

Compression settling may occur at the bottom of a tank if particles are in such a concentration as to be in physical contact with one another. The weight of particles is partly supported by the lower layers of particles, leading to progressively greater compression with depth and thickening of sludge. From the settling column test, the limiting solids flux required to reach any desired underflow concentration can be estimated, from which the rquired tank area can be computed.

The solids load on the clarifier is estimated in terms of (Q+R)X, while the overflow rate or surface loading is estimated in terms of flow Q only (not Q+R) since the quantity R is withdrawn from the bottom and does not contribute to the overflow from the tank. The secondary settling tank is particularly sensitive to fluctuations in flow rate and on this account it is recommended that the units be designed not only for average overflow rate but also for peak overflow rates. Beyond an MLSS concentration of 2000 mg/l the clarifier design is often controlled by the solids loading rate rather than the overflow rate. Recommended design values for treating domestic sewage in final clarifiers and mechanical thickeners (which also fall in this category of compression settling).

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Sludge Recycle

The MLSS concentration in the aeration tank is controlled by the sludge recirculation rate and the sludge settleability and thickening in the secondary sedimentation tank.

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

where Q_r = Sludge recirculation rate, m^3/d

The sludge settleability is determined by sludge volume index (SVI) defined as volume occupied in mL by one gram of solids in the mixed liquor after settling for 30 min. If it is assumed that sedimentation of suspended solids in the laboratory is similar to that in sedimentation tank, then $X_r = 10^6/SVI$. Values of SVI between 100 and 150 ml/g indicate good settling of suspended solids. The X_r value may not be taken more than 10,000 g/m^3 unless separate thickeners are provided to concentrate the settled solids or secondary sedimentation tank is designed to yield a higher value.

Excess Sludge Wasting

The sludge in the aeration tank has to be wasted to maintain a steady level of MLSS in the system. The excess sludge quantity will increase with increasing F/M and decrease with increasing temperature. Excess sludge may be wasted either from the sludge return line or directly from the aeration tank as mixed liquor. The latter is preferred as the sludge concentration is fairly steady in that case. The excess sludge generated under steady state operation may be estimated by

$$q_c = \frac{VX}{Q_w X_r}$$

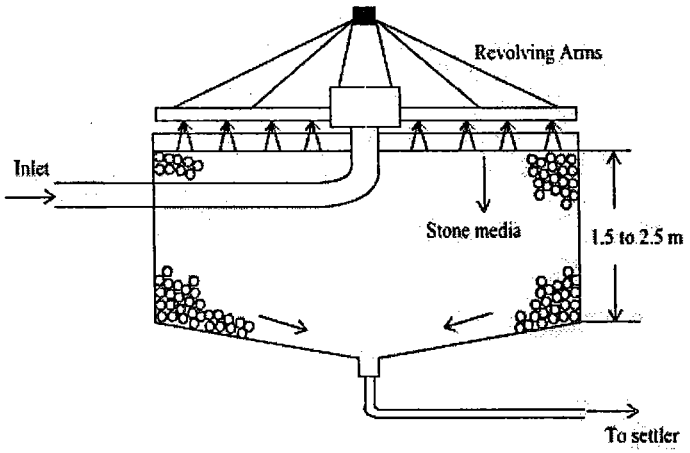
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or $Q_w X_r = YQ (S_O - S) - k_d XV$

Trickling Filters

Trickling filter is an *attached growth process* i.e. process in which microorganisms responsible for treatment are attached to an inert packing material. Packing material used in attached growth processes include rock, gravel, slag, sand, redwood, and a wide range of plastic and other synthetic materials.

High Rate Trickling Filter



Process Description

- The wastewater in trickling filter is distributed over the top area of a vessel containing non-submerged packing material.
- Air circulation in the void space, by either natural draft or blowers, provides oxygen for the microorganisms growing as an attached biofilm.
- During operation, the organic material present in the wastewater is metabolised by the biomass attached to the medium. The biological slime grows in thickness as the organic matter abstracted from the flowing wastewater is synthesized into new cellular material.
- The thickness of the aerobic layer is limited by the depth of penetration of oxygen into the microbial layer.

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- The micro-organisms near the medium face enter the endogenous phase as the substrate is metabolised before it can reach the micro-organisms near the medium face as a result of increased thickness of the slime layer and loose their ability to cling to the media surface. The liquid then washes the slime off the medium and a new slime layer starts to grow. This phenomenon of losing the slime layer is called *sloughing*.
- The sloughed off film and treated wastewater are collected by an underdrainage which also allows circulation of air through filter. The collected liquid is passed to a settling tank used for solid- liquid separation.

Types of Filters

Trickling filters are classified as high rate or low rate, based on the organic and hydraulic loading applied to the unit.

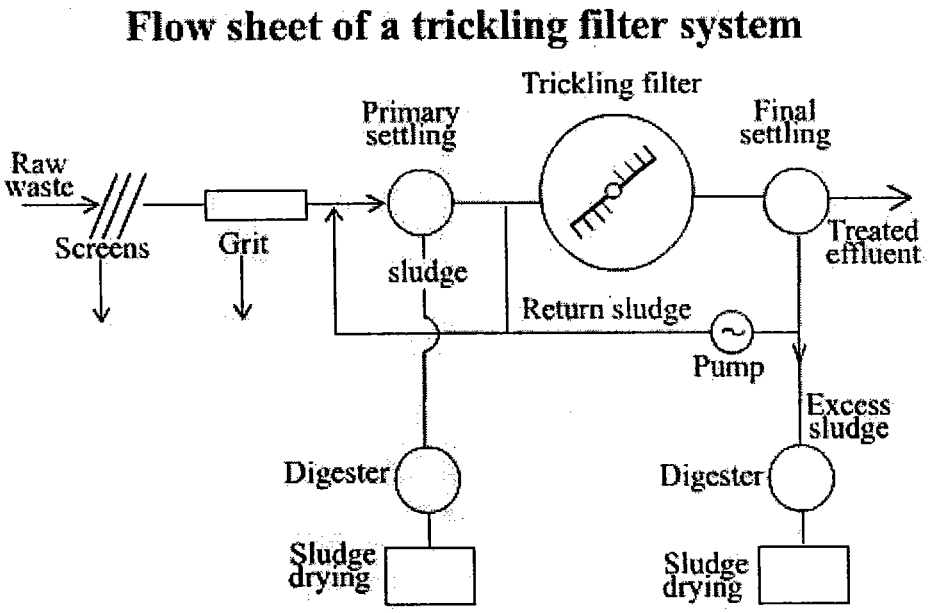
S.No.	Design Feature	Low Rate Filter	High Rate Filter
1.	Hydraulic loading, $m^3/m^2.d$	1 - 4	10 - 40
2.	Organic loading,kg BOD / $m^3.d$	0.08 - 0.32	0.32 - 1.0
3.	Depth, m	1.8 - 3.0	0.9 - 2.5
4.	Recirculation ratio	0	0.5 - 3.0 (domestic wastewater) upto 8 for strong industrial wastewater.

- The hydraulic loading rate is the total flow including recirculation appied on unit area of the filter in a day, while the organic loading rate is the 5 day 20°C BOD, excluding the BOD of the recirculant, applied per unit volume in a day.
- Recirculation is generally not adopted in low rate filters.
- A well operated low rate trickling filter in combination with secondary settling tank may remove 75 to 90% BOD and produce highly nitrified effluent. It is suitable for treatment of low to medium strength domestic wastewaters.
- The high rate trickling filter, single stage or two stage are recommended for medium to relatively high strength domestic and industrial wastewater. The BOD removal efficiency is around 75 to 90% but the effluent is only partially nitrified.

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- Single stage unit consists of a primary settling tank, filter, secondary settling tank and facilities for recirculation of the effluent. Two stage filters consist of two filters in series with a primary settling tank, an intermediate settling tank which may be omitted in certain cases and a final settling tank.

Process Design



Generally trickling filter design is based on empirical relationships to find the required filter volume for a designed degree of wastewater treatment. Types of equations:

1. NRC equations (National Research Council of USA)

   NRC and Rankin's equations are commonly used. NRC equations give satisfactory values when there is no re-circulation, the seasonal variations in temperature are not large and fluctuations with high organic loading. Rankin's equation is used for high rate filters.

NRC equations: These equations are applicable to both low rate and high rate filters. The efficiency of single stage or first stage of two stage filters, E_2 is given by

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$$E_2 = \frac{100}{1+0.44(F_{1.BOD}/V_1.Rf_1)^{1/2}}$$

For the second stage filter, the efficiency E_3 is given by

$$E_3 = \frac{100}{[(1+0.44)/(1-E_2)](F_{2.BOD}/V_2.Rf_2)^{1/2}}$$

where E_2= % efficiency in BOD removal of single stage or first stage of two-stage filter, E_3=% efficiency of second stage filter, $F_{1.BOD}$= BOD loading of settled raw sewage in single stage of the two-stage filter in kg/d, $F_{2.BOD}= F_{1.BOD}(1- E_2)$= BOD loading on second-stage filter in kg/d, V_1= volume of first stage filter, m^3; V_2= volume of second stage filter, m^3; Rf_1= Recirculation factor for first stage, R_1= Recirculation ratio for first stage filter, Rf_2= Recirculation factor for second stage, R_2= Recirculation ratio for second stage filter.

Rankins equation: This equation also known as Tentative Method of Ten States USA has been successfully used over wide range of temperature. It requires following conditions to be observed for single stage filters:

1. Raw settled domestic sewage BOD applied to filters should not exceed 1.2 kg BOD_5/day/ m^3 filter volume.
2. Hydraulic load (including recirculation) should not exceed 30 m^3/m^2 filter surface-day.
3. Recirculation ratio (R/Q) should be such that BOD entering filter (including recirculation) is not more than three times the BOD expected in effluent. This implies that as long as the above conditions are satisfied efficiency is only a function of recirculation and is given by:

$$E = \frac{(R/Q)+1}{(R/Q)+1.5}$$

Stabilization Ponds

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- The *stabilization ponds* are open flow through basins specifically designed and constructed to treat sewage and biodegradable industrial wastes. They provide long detention periods extending from a few to several days.
- Pond systems, in which oxygen is provided through mechanical aeration rather than algal photosynthesis are called *aerated lagoons*.
- Lightly loaded ponds used as tertiary step in waste treatment for polishing of secondary effluents and removal of bacteria are called *maturation ponds*.

lassification of Stabilization Ponds

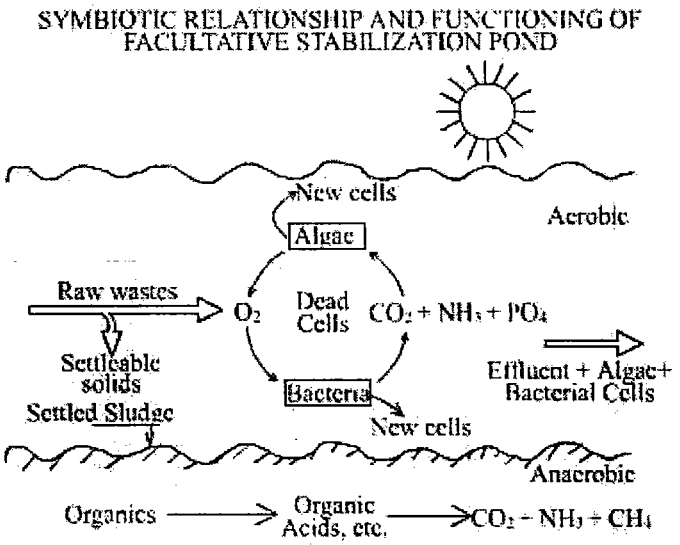
Stabilization ponds may be aerobic, anaerobic or facultative.

- *Aerobic ponds* are shallow ponds with depth less than 0.5 m and BOD loading of 40-120 kg/ha.d so as to maximize penetration of light throughout the liquid depth. Such ponds develop intense algal growth.
- *Anaerobic ponds* are used as pretreatment of high strength wastes with BOD load of 400-3000 kg/ha.d Such ponds are constructed with a depth of 2.5-5m as light penetration is unimportant.
- *Facultative pond* functions aerobically at the surface while anaerobic conditions prevail at the bottom.They are often about 1 to 2 m in depth. The aerobic layer acts as a good check against odour evolution from the pond

Mechanism of Purification

- The functioning of a facultative stabilization pond and symbiotic relationship in the pond are shown below. Sewage organics are stabilized by both aerobic and anaerobic reactions. In the top aerobic layer, where oxygen is supplied through algal photosynthesis, the non-settleable and dissolved organic matter is oxidized to CO_2 and water. In addition, some of the end products of partial anaerobic decomposition such as volatile acids and alcohols, which may permeate to upper layers are also oxidized periodically. The settled sludge mass originating from raw waste and microbial synthesis in the aerobic layer and dissolved and suspended organics in the lower layers undergo stabilization through conversion to methane which escapes the pond in form of bubbles.

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Factors Affecting Pond Reactions

Various factors affect pond design:

- wastewater characteristics and fluctuations.
- environmental factors (solar radiation, light, temperature)
- algal growth patterns and their diurnal and seasonal variation)
- bacterial growth patterns and decay rates.
- solids settlement, gasification, upward diffusion, sludge accumulation.

The depth of aerobic layer in a facultative pond is a function of solar radiation, waste characteristics, loading and temperature. As the organic loading is increased, oxygen production by algae falls short of the oxygen requirement and the depth of aerobic layer decreases. Further, there is a decrease in the photosynthetic activity of algae because of greater turbidity and inhibitory effect of higher concentration of organic matter.

Gasification of organic matter to methane is carried out in distinct steps of acid production by acid forming bacteria and acid utilization by methane bacteria. If the second step does not proceed satisfactorily, there is an accumulation of organic acids resulting in decrease of pH which would result in complete inhibition of methane bacteria. Two possible reasons for imbalance between activities of methane bacteria are: (1) the waste may contain inhibitory

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substances which would retard the activity of methane bacteria and not affect the activity of acid producers to the same extent. (2) The activity of methane bacteria decreases much more rapidly with fall in temperature as compared to the acid formers.

Thus, year round warm temperature and sunshine provide an ideal environment for operation of facultative ponds.

Algal Growth and Oxygen Production

Algal growth converts solar energy to chemical energy in the organic form. Empirical studies have shown that generally about 6% of visible light energy can be converted to algal energy.

The chemical energy contained in an algal cell averages 6000 calories per gram of algae.

Depending on the sky clearance factor for an area, the average visible radiation received can be estimated as follows:

Avg. radiation= Min. radiation + [(Max. radiation Min.radiation) X sky clearance factor]

Oxygen production occurs concurrently with algal production in accordance with following equation:

$$106C0_2 + 16NO_3 + HPO_4 + 122H_2O + 18H^+ \longrightarrow C_{106}H_{263}O_{110}N_{16}P_1 + 138O_{2s}$$

On weight basis, the oxygen production is 1.3 times the algal production.

Areal Organic Loading

The permissible areal organic loading for the pond expressed as kg BOD/ha.d will depend on the minimum incidence of sunlight that can be expected at a location and also on the percentage of influent BOD that would have to be satisfied aerobically. The Bureau of Indian Standards has related the permissible loading to the latitude of the pond location to aerobically stabilize the organic matter and keep the pond odour free. The values are applicable to towns at sea levels and where sky is clear for nearly 75% of the days in a year. The values may be modified for

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elevations above sea level by dividing by a factor (1 + 0.003 EL) where EL is the elevation of the pond site above MSL in hundred meters.

Detention Time

The flow of sewage can approximate either plug flow or complete mixing or dispersed flow. If BOD exertion is described by first order reaction, the pond efficiency is given by:

for plug flow: $L_e/L_i = e^{-k1t}$

for complete mixing: $L_e/L_i = \frac{1}{1+k_1t}$

For dispersed flow the efficiency of treatment for different dgrees of intermixing is characterized by dispersion numbers.Choice of a larger value for dispersion number or assumption of complete mixing would give a conservative design and is recommended.

Depth

Having determined the surface area and detention capacity, it becomes necessary to consider the depth of the pond only in regard to its limiting value. The optimum range of depth for facultative ponds is 1.0 - 1.5 m.

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