Treatment of Sewage.

Raw Sewage

↓

Screening + Communitors/Macerators.

↓

Grit chamber (remove inorganic suspend particle)

↓

PST (Primary sedimentation tank)

↓

2° treatment (remove dissolve organic matter)

↓

SST (seconday sedimentation tank) → sludge digestion

↓

3° treatment (remove M.O.)

↓

Effluent

sludge digestion → Digested sludge on drying beds.

sludge digestion → Gases

Supernatant liquid → PST

① **Screening** (same as raw water)

→ Screening is the first operation carried out at sewage treatment plant and consist of passing the sewage through different types of screen so as to remove heavy solid from it. eg. floating matter, pieces of cloth, paper, wood cork, hair, fibre, kitchen refuse, fecal solids

→ Screening is done with the help of unit termed as screens, classified as:-

(i) Coarse screen/racks:
spacing between the bars is 50 mm or more.

(ii) Medium Screen
Here spacing between the bar is 6-40mm and angle of inclination is 30 to 60°.

(iii) Fine screen

spacing is 1.5 - 3mm

→ Head loss through screen.

$$H_L = \frac{K}{2g}(V^2 - u^2)$$

K = constant depend upon malerial of screen.

V = Velocily through screen.

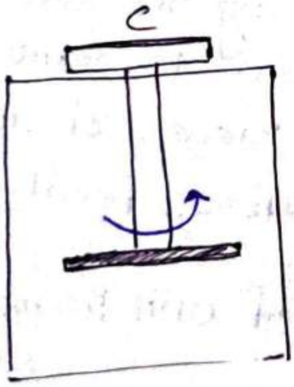
u = velocily of approach.

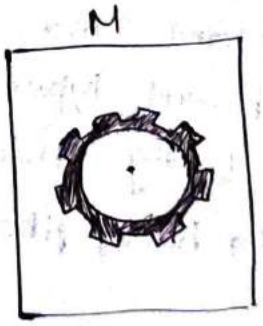
$$\frac{K}{2g} = 0.0729.$$

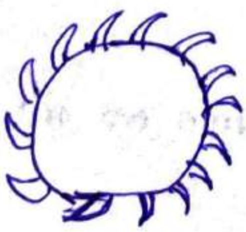
(iv) Comminutors/Macerators.

→ In real terms these are not the method of treatment of woste waler. they are only used to convert the heavier solid into the lighler ones, so as to carry out their effective disposal.

→ Here comminutor based upon cutting or tearing action whereas macerator based upon crushing and griding action.







Note:- Disposal of screening.

→ Material seprated by screen is called screenings.

→ It can be disposed by any of following method :-

(i) Buring in incinerators.

(ii) Disposal by burial (composting)

(iii) Land filling method.

(iv) Dumping into sea.

(v) Digestion along with sludge: Not successful.

(iii) Grit Chamber

→ Grit chambers are provided for the removal of inorganic suspended particles. of size greater than 0.15 to 0.2mm suspended particle in PST. and to pass forward the organic suspended particle in PST.

→ Grit quantity produced may vary between 0.004 - 0.037 m³/1000 m³ of sewage for seprate sewrage system and 0.004 - 0.180 m³/1000m³ for combined sewerage system.

→ It is not desirable to remove any organic matter in grit chamber because no further treatment to the removed grit is given.

→ They are further classified into:-

(i) Horizontal flow (non-aerated)

(ii) Aerated Grit chamber.

Horizontal flow type Grit chamber

→ These chamber are designed to give a horizontal constant velocity of flow or discharge which is achieved by providing velocity control devices such as proportional weir or sutro weir & partiall ~~flow~~ flume at the effluent end of the section.

→ If rectangular section is adopted proportional weir is provided and if parabolic section is adopled ~~[illegible]~~ parshall flume is provided.

→ surface over flow rate for 0.2mm sized particle is ~~[illegible]~~. 0.025 - 0.03m/s

→ The important point in the designed of the basin is that flow velocity should neither be too low as to cause settling of organic matter and nor should be so high so as not to cause settling of silt or grit. Moreover flow velocity should ~~not~~ also be enough to scour out settle organic matter.

$$V_H = 3 \text{ to } 4.5 \sqrt{gd(G-1)} \quad \text{[Shield's Equation]}$$

→ In order to prevent increase in velocity at peak flow and to avoid scouring of settled grit, these tanks are designed into the form of form 2-3 or 3 chambers for average discharged or dry wheather flow.

→ $t_d = 40 - 60 sec$.

→ $H = 1 - 1.8$ m

→ free Board = 0.3m

→ length of tank is increased by 25-30% over the design length to account for the torblence at inlet and outlet of tank.

→ These chamber are cleaned periodically at about 3 week interval so either manually or mechanically.

Note:- Here flow is transition

$$V_s = \sqrt{\frac{4}{3}\frac{(G-1)\, g\, .d}{C_D}}$$

$$C_D = \frac{24}{R_e} + \frac{3}{R_e^{1/2}} + 0.34 \quad \text{or} \quad C_D = \frac{18.5}{R_e^{0.6}}$$

$$\boxed{V_s = \left(\frac{g(G-1)\, d^{1.6}}{13.88 \cdot (\nu)^{0.6}}\right)^{0.714}}$$

$$V_s \propto d^{1.142}$$

Design

$$\underset{(L \times B)}{SA} = \frac{Q_0}{OFR}$$

$$\underset{(LBH)}{V} = Q_0 \cdot t_d$$

$$H = \frac{V}{SA}$$

$$L = V_f \cdot t_d$$

$$B = \frac{SA}{L}$$

(ii) Aerated Grit Chamber.

→ In this grit chamber the spiral or helical flow of waste water derives the grit into the hopper

→ The shearing action of air bubbles is supposed to strip the inert grit from organic particles that adheres to its surface.

→ The air diffusers are located about 0.45m to 0.6m above the normal bottom of the tank.

→ This is same as sedimentation tank except a grit hopper of 0.9m depth with steeply slopping side is located under the air diffuser.

→ The performance of this unit is the function of roll velocity & detention time.

→ If velocity of roll is high grit will be carried through chamber and if it is slow organic matter may also settle down.

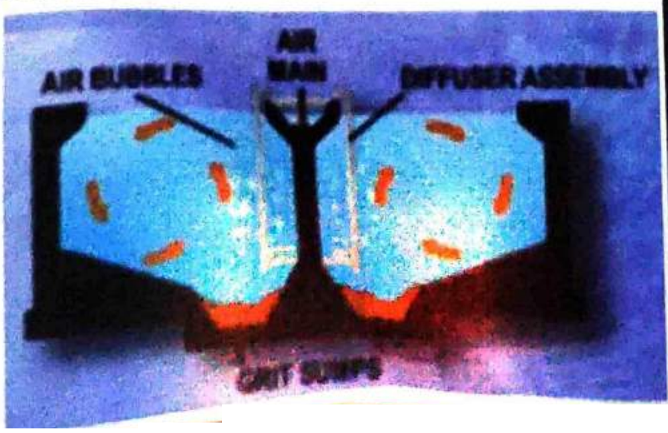
→ Air feed rate is kept in range of 0.15 to 0.45 m^3/min/m length of the tank. (0.3 m^3/min/m length)

→ Detention time = 3 minute. at maximum flow.

→ $\frac{L}{B} = 2.5 - 7$

⇒ $\frac{B}{H} = 1:1$ to $5:1$ $(2:1)$

⇒ Grit accumulation is approximately 90 m^3/M m^3 of sewage in combined sewerage system & 30 m^3/M m^3 in separate sewage system.



FLOW IN AN AERATED GRIT TANK

Q A rectangular grit chamber is designed to remove particle with diameter 0.2 mm., $G = 2.65$. A flow through velocity of 0.3 m/sec will be maintained by proportional weir. Determine the channel dimension for a maximum waste water flow of 12000 m^3/day

$Q_0 = 12000\ m^3/day.$

Assume V_s (SOF) $= 0.025\ m/sec$

$$SA = \frac{Q}{OFR} = \frac{12000}{86400 \times 0.025} = 5.55 m^2.$$

Assume, $t_d = 50 sec.$

$$V = Q_0 . t_d$$

$$= \frac{12000 \times 50}{60 \times 60 \times 24} = 6.99\ m^3$$

$$H = \frac{V}{SA} = \frac{6.99}{5.55} = 1.25 m$$

Provide free board of = 0.35 m

depth of sludge zone = 0.9 m

$$L = V_f . d$$

$$= 0.3 \times 50 = 15 m$$

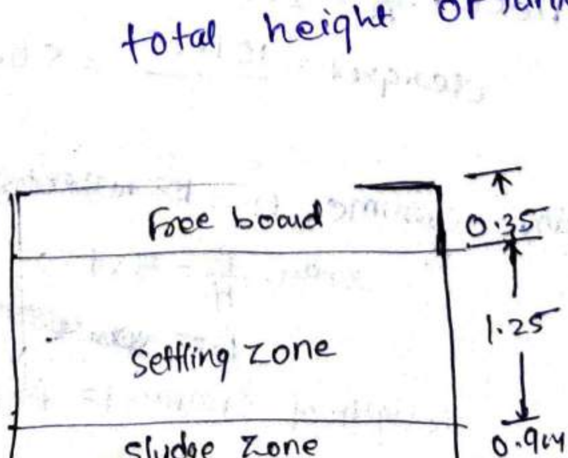
$$\text{width of tank} = \frac{\text{area}}{\text{Length}} = \frac{5.55}{15}$$

$$= 0.37 m$$

Increase the length by 30%,

$$L' = 19.5 m$$

total height of tank = 0.35 + 1.25 + 0.9
= 2.5 m



Q Design an aerated grit chamber for treating municipal waste water with average flow of $0.75\ m^3/s$ assume peak flow rate to be 3 times of average.

Design discharge, $Q_0 = 3\ Q_{avg} = 3 \times 0.75$
$= 2.25\ m^3/s$

Assume $t_d = 3$ min

Volume $= Q_0 \times t_d$
$= 2.25 \times 3 \times 60 = 405\ m^3$

In order to drain the channel periodically for routine maintenance, use two chamber

$$V_1 = V_2 = \frac{V}{2} = \frac{405}{2} = 202.5\ m^3$$

Assume, $H = 1m$

$$\frac{B}{H} = 2:1$$

$B = 2m = 2 \times 1$

$$\text{length of channel} = \frac{V}{H \times B} = \frac{202.5}{2 \times 1} = 101.25\ m$$

~~length of tank is increased by 25%.~~

~~$L' = 1.25 \times 101.25$~~
~~$= 126.56\ m$~~

Check, $\frac{L}{B} = 2.5$ to 7

$$\text{designed} = \frac{101.25}{2} = 50.6 > 7$$

Again, assume, $H = 1.2$

assume, $\frac{B}{H} = 5:1$

$B = 6m$

$$\text{length of channel} = \frac{V}{H \times B} = \frac{202.5}{1.2 \times 6}$$
$= 28.125$

$$\frac{L}{B} = \frac{28.125}{6} = 4.6875$$

length of tank is increased by 25%.

$L' = 1.25 \times 28.125 = 35.156 m$

Hence provide two chamber of size $= (\cancel{28.125}\ 35.156 \times 6 \times 1.2) m^3$

Air supply requirement $= 0.3 m^3/min/m$

Air supply requirement $= 0.3 \times 35.15$

$= 10.54 m^3/min$

Volume of grit produce @ $30 m^3/M m^3$ of sewage treated

$= 2.25 \times 86400 \times 30 \times 10^{-6}$

$= \cancel{97.76}\ 5.832 m^3/day$

IV Detritus tank

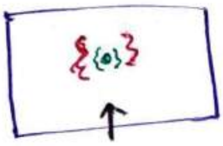
→ Detritus tank are grit chamber designed to flow with the smaller flow velocity of 0.09m/s and longer detention period of 3-4min.

→ They are designed to seprate out not only the larger grit, but very fine sand particles

→ Due to this large amount of organic matter will also settle out along with the inorganic particles.

→ All other details of this tank is same as grit chamber.

Note:- In order to remove organic particles from grit by the used of controlled aeration of flow through the tank in which air bubbles will then seprate the lighter organic matter from settling grit or by control of current in the tank through baffle wall.



V Skimming Tank

→ These tank are employed to remove oil and greese from waste water and are placed before PST

→ These oil and greece enters into the sewage from kitchen, resturdant, houses, motor garages, oil refineries etc.

→ If oil & grease is not removed from sewage they enter into the treatment plant and inhibits the growth of micro-organism required for biological treatment

→ It also generates odorous scum in sedimentation tank thereby effect the settling of particles.

→ Oil and grease from these tank is removed by passing air from the bottom of the tank, rising bubbles of which coagulate oil and grease & carries them to surface of the tank from where it can be easily skimmed off.

→ In order to increase the efficiency of these tanks Cl_2 gas may also be passed along with the compressed air which destroy the effect of enzymes that hold oil and grease in emulsified form.

→ These tanks are not used in India, as coagulation of oil and grease particles does not take place at high temperature.

→ $t_D = 3\text{-}5$ minutes.

→ compressed air required is 300-6000 m³/ML of sewage

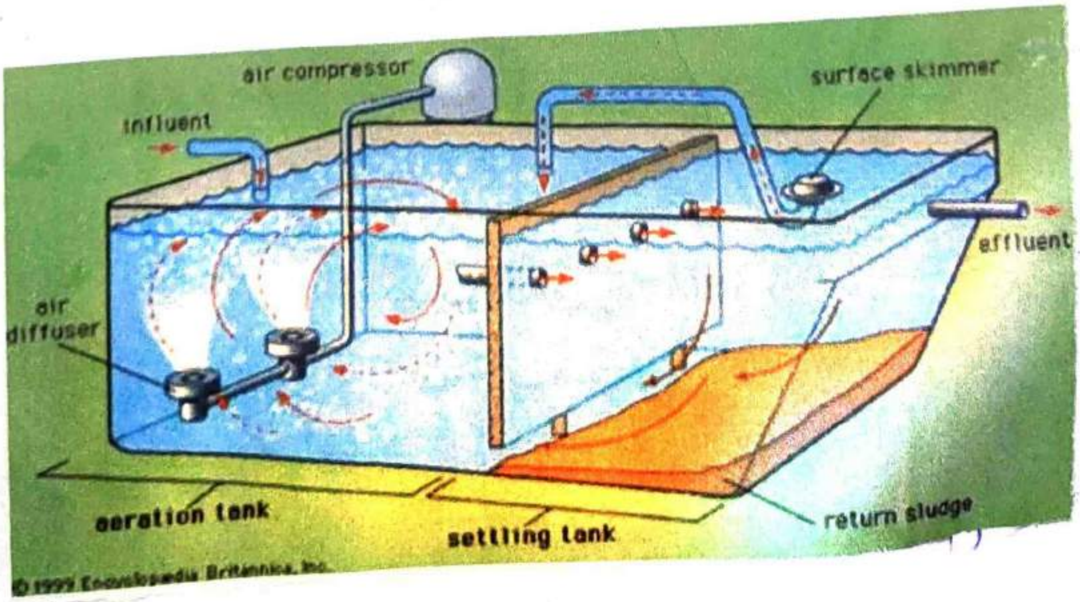
→ Area of tank is given by,

$$\boxed{A = 0.00622 \frac{q}{V_r}}$$

q = rate of flow of sewage in m³/day

V_r = minimum rising velocity of greasy material to be removed.

= 0.25 m/min.



Primary Sedimentation tank. (PST)

→ Depending upon concentration of particles and their tendency to interact with each other following types of settling may be observed during sedimentation:-

(i) TYPE-I : DISCRETE SETTLING

concentration of particles < 1000 mg/lit

→ It occurs when there is no change in shape, size and mass of particle during settling.

→ Settling velocity of discrete particles can be computed using stoke's law.

→ Settling is PST (raw water plant) or in grit chamber (WWP) is TYPE-I settling.

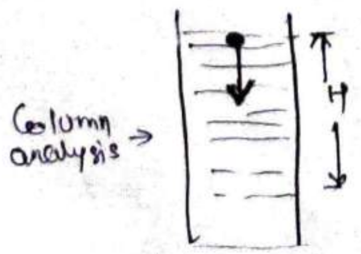
(ii) TYPE-II FLOCULANT SETTLING.

→ It is the type of settling in which shape, size and mass of particles changes during settlement due to formation of flocs.

→ It takes place when concentration of particle is comparatively low. 1000 mg/lit

→ There is no mathematical relationship to find settling velocity of particle in this case.

→ This settling is found in clariflloculator. (RWP) or in primary sedimentation tank. (PST) (WWP) or in upper portions of SST (secondary sedimentation tank).



$V_s = \frac{H}{t}$

Zone-III : ZONE/ HINDERED SETTLING.

→ When concentration of flocculated particles is in intermediate range (1500-1800)mg/lit, they are closed enough together so that their velocity field overlaps resulting in settling of particles in respective zone. and causing the hinderance to the individual settlement of the other particles.

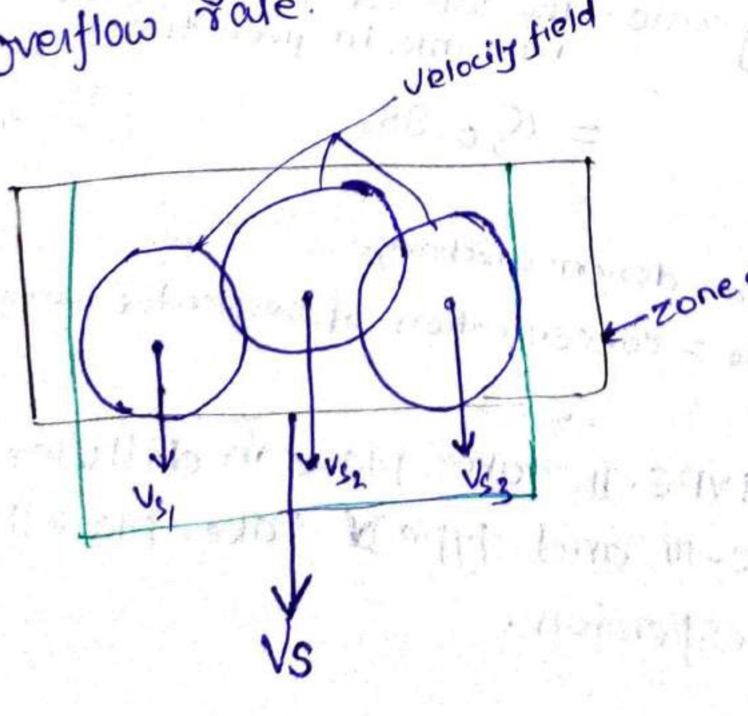
→ The settling of particles results in significant upward displacement of water.

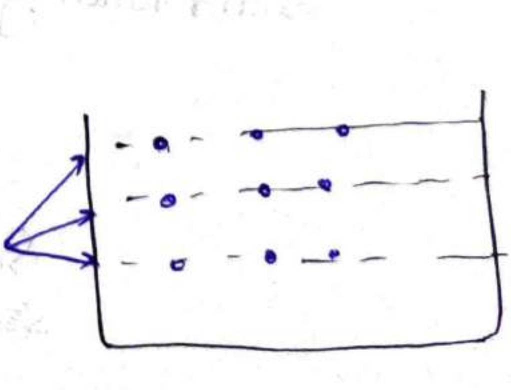
→ Particles maintained there relative position with respect to each other and the whole mass of the particles settles as one unit or zone.

→ This type of settling is found in SST after ASP.

→ In Hindered settling zone the concentration of particles increases from top to bottom leading to thickning of sludge.

→ Such secondary clarifier were zone settling occurs are designed on the basis of solid loading rate and overflow rate.

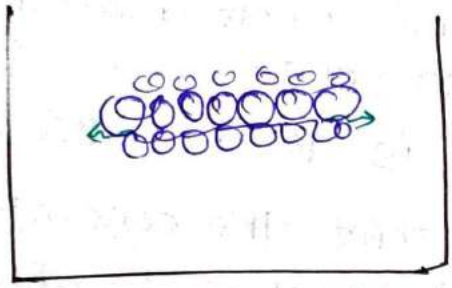




(iv) TYPE IV: Compression Settling.

→ In compression zone concentration of particles become so high that particles are in physical contact with each other and the lower layers supporting the weight of upper layer., consequently any further settling in this case, results due to compression of ~~solid~~ whole structure of particles and is accompanished by squeezing out of water from voids between the particles.

→ This settling phenomenon occurs at bottom of deep sludge mass as in bottom SST flowing trickling filter (TF)



→ Such secondary clarifier are also designed for over flow rate & solid loading rate.

Note:- (i) Overflow rate = V_s

Solid loading rate = The rate at which solid loaded in tank. in given time.

$$= Q_0 . SS_0 .$$ $\left[\frac{w}{V} \cdot \frac{V}{t}\right]$

Q_0 = design discharge

SS_0 = concentration of suspended particle.

(ii) TYPE-I and TYPE-II takes place in dillutes where as Type-III and type-IV takes place in concentrated suspension.

PST

→ It is provided for the removal of organic suspended particle from waste water.

→ It is designed for overflow rate ie,
Area of tank is computed using both average ~~value of both~~ and peak value of ~~area~~ OFR & maximum area computed is adopted.

PST	OFR ($m^3/m^2/day$)		Depth of tank (m)	detention time hr.
	Avg	Peak		
→ 1° settling only	25-30	50-60	2.5-3.5	2-2.5
→ 1° settling followed by TF.	35-50	80-120	2.5-3.5	2-2.5
→ 1° settling followed by ASP.	25-35	50-60	3.4-4.5	2-2.5

→ $V_f = 0.3\ m/min$

→ $B = 5-6m$

→ $\frac{L}{B} = 4-5$

$$SA = \frac{Q_{AVG} \text{ or } Q_0}{OFR_{avg}} \quad \& \quad SA = \frac{Q_{PEAK}}{OFR_{PEAK}} \Big\} \text{Maximum}$$

$$V = Q_0 \cdot t_d \quad , \quad L = V_f \cdot t_d$$

$$H = \frac{V}{SA_{max}} \quad , \quad B = \frac{SA}{L}$$

Coagulation aided Sedimentation.

→ Under normal treatment condition, coagulation aided sedimentation is avoided in PST. as:-

(i) The chemicals aided during coagulation inhibits the growth of micro-organism required for growth of bioligical treatment.

(ii) Cost of chemicals aided in this case comes out to be more (as it depend upon turbidity of water). which makes this process costly.

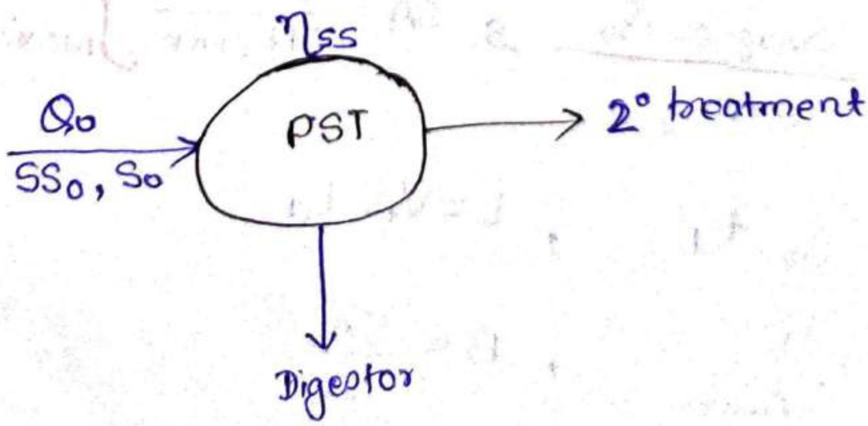
(iii) Large quantity of sludge is formed after coagulation, which makes the treatment of sludge costly

(iv) This process requires skilled supervision & handling of chemicals.

(v) For Biological treatment these days, method involves are complete in nature, hence donot require coagulation seprately

→ But in some extreme condition it can be used by addition of iron salts. ($FeCl_3$, $Fe_2(SO_4)_3$) as in case of :-

- when treatment is carried out in hilly areas where smaller plants are required due to limited availability of space.
- When disposal of sewage effluent is done in lakes & its eutrophication is avoided as coagulation ppt out nutrients of algae.



Sludge Digestion.

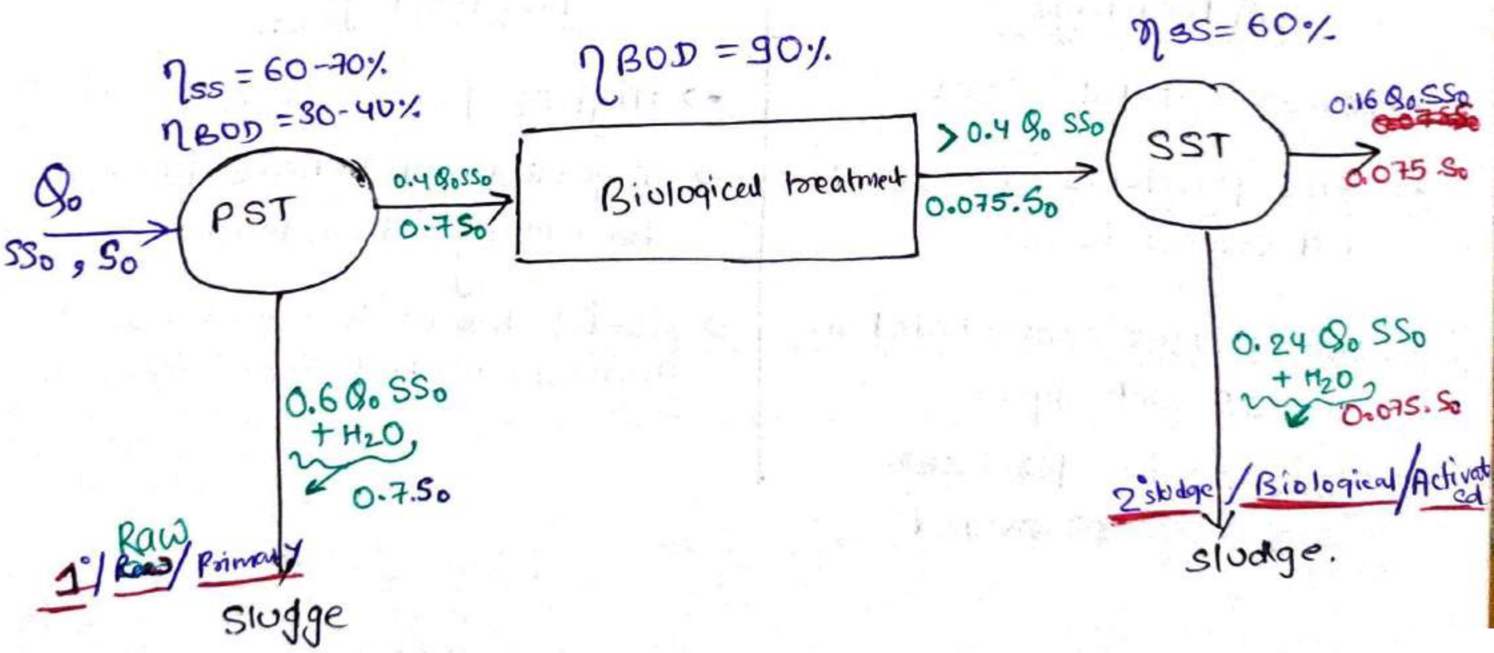
→ Sludge generated in sewage treatment plant is considerable hazard to the environment hence must be treated prior to its disposal.

→ Digestion of organic sludge can be done either in presence of oxygen or in absence of oxygen.

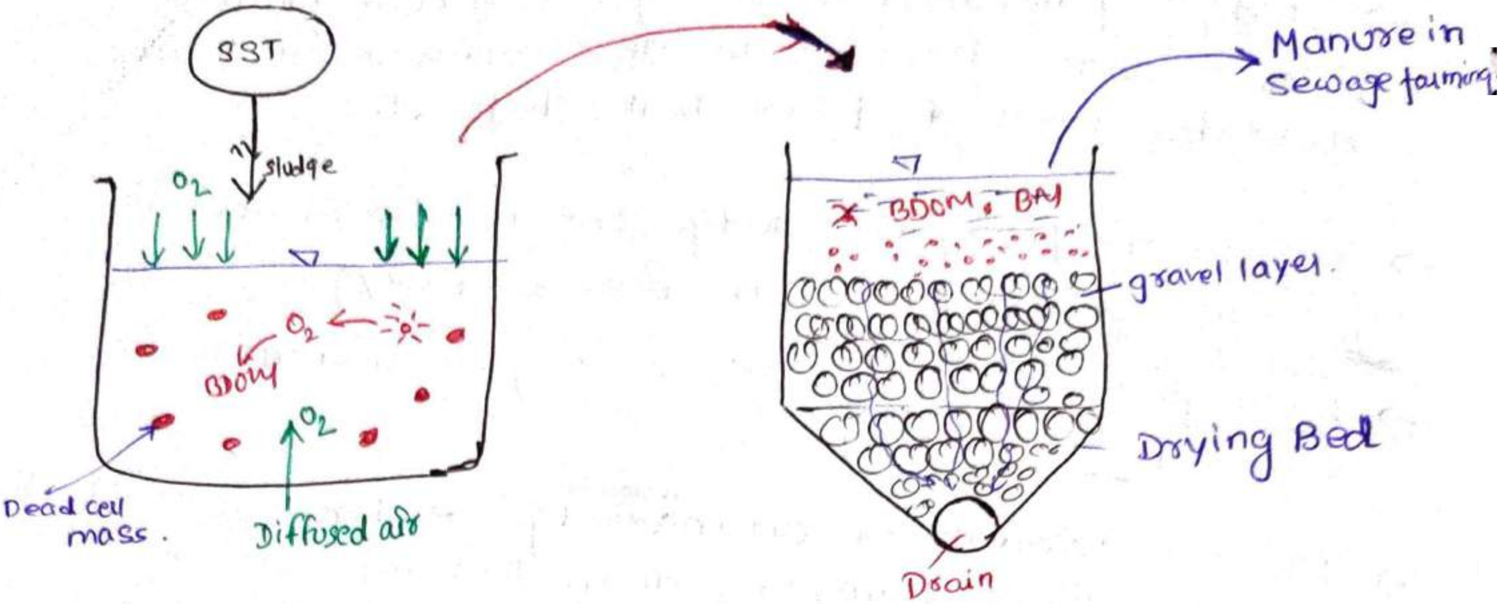
AEROBIC DIGESTION

→ Biological sludge (from SST) is lesser organic in nature is digested aerobically.

→ Due to scarcity of OM in Biological sludge endogenous respiration also takes place during digestion. that increase the concentration of dead cell mass in the sludge ~~which~~ that further makes the dewatering of sludge difficult.



Note :- It is just extension of extended aeration process.



Properties of aerobic digestion:-

Advantage	Disadvantage
→ lower capital cost.	→ Higher power cost.
→ End products are stable and odour less.	→ Digested solid have poorer dewatering characteristics.
→ Lower BOD concentrated in supernatant liquor.	→ Useful bio-product such as methane is not recoverd.
→ More basic fertilizer values are recovered.	

Anaerobic Digestion.

→ Raw sludge is digested anaerobically as if it is digested aerobically it would induce, uncontrolable growth of micro-organism in system.

→ The prime function of anerobic digestion is to convert most of the sludge into liquids and gases.

→ During anerobic digestion sludge is reduced into following consituents :→

(i) Digested Sludge: It is stable humus like solid matter, tarry black in colour with reduced moisture content and in therefore having reduced volume (about $\frac{1}{3}$ times of undigested sludge)

(ii) Supernatent liquid:- It includes liquified and finely divided solid matter and is having high BOD ≈ 3000 ppm.

(iii) Gases: Gases like methane (65-70%), CO_2 (30%) and traces of other inert gases like nitrogen, hydrogen sulphide etc are evolved. They may be collected [particularly (CH_4)] & used as a fuel.

Note:- The gas produced is approximately 14-18 l/c/d (17 l/c/d) after the digestion of sludge.

→ Anerobic digestion is being carried out by following set of micro-organism.

(i) Acid formers: These are anerobic or facultative MO, (- aerobic) which utilise organic matter from sludge and convert it into acid & alcohol of lower molecular masses.

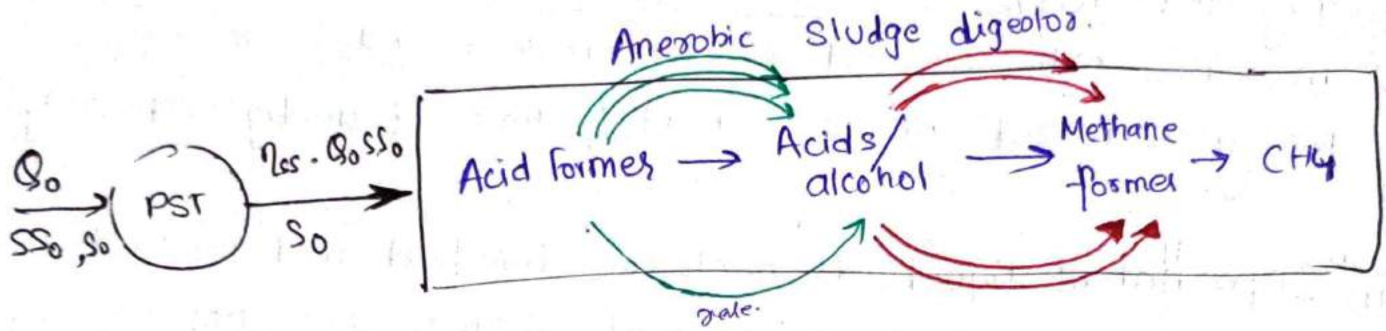
→ These are highly competitive M/O as they respond very quickly to the food given to them.

(ii) Methane formers:

These are strickly anerobic micro-organism and utilises acid & alcohol formed by acid former along with CO_2 and H_2O & convert it into methane.

→ They work in a narrow pH Range of 6.5-7.5

→ They are very delicale against shock loading as they do not respond very quickly to food given to them.



Stages in Sludge Digestion process:-

→ Digestion of sludge have been found to occur in following stages.

(i) Acid Fermentation / Acid Production stage.

→ In this first stage of sludge digestion the fresh sewage sludge begins to be acted upon by anerobic & facultative micro-organism called acid formes.

→ These MO solubilize the organic solid through "HYDROLYSIS"

→ The solube product are then fermented to volatile acid & organic alcohol of low molecular weight.

→ Intensive acid production makes the sludge highly acidic and lowers the pH value less than 6.

(ii) Acid Regression stage.

→ In this intermediate stage the volatile organic acid & nitrogenous compound of first stage are attacked by the M/O, so as to form acid carbonates and ammonia compounds.

→ The pH of the system in this stage rises upto 6.8

(iii) Alkaline Fermentation Stage.

→ In this stage of sludge digestion more resistant material like proteins and organic acid are attacked by anaerobic M/O called Methane formers and convert it into simple compound. like ammonia, organic acid and digested sludge.

→ This digested sludge collected is also called ripened sludge

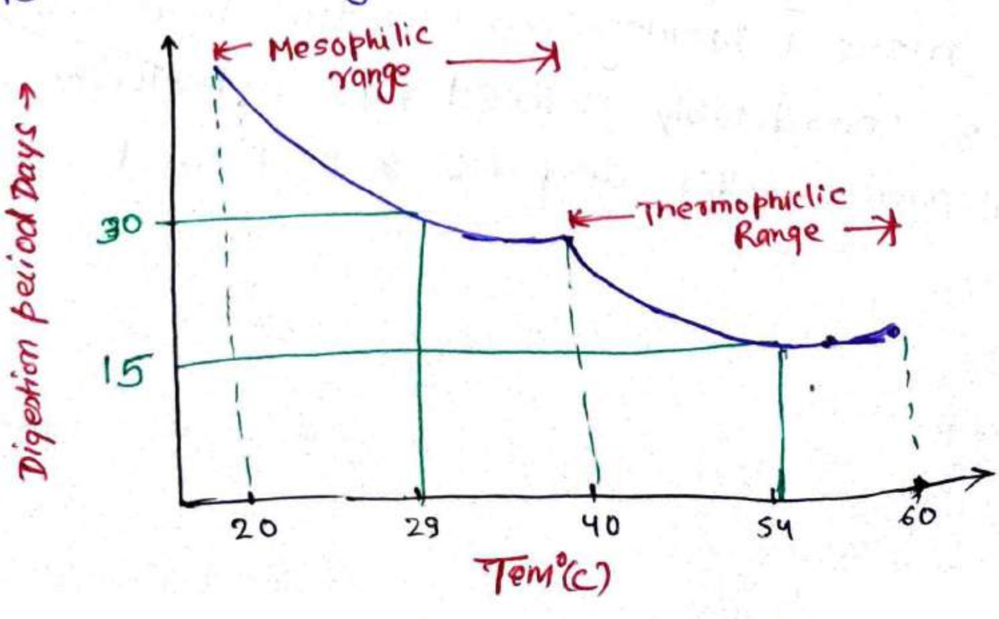
→ Digested sludge is alkaline, hence pH is slightly greater than 7.

→ Large volume of methane gas is also produced in this stage.

Factors affecting Anerobic Digestion.

(i) Temperature

The process of digestion is greatly influenced by temp., rate of digestion increases with increase in temperature due to the weaking of bond in organic matter.



(ii) pH

For optimum digestion pH must be in range of 6.5-7.5. for methane former to work effectively.

Note:- During digestion pH of the sludge varies more on acidic side due to → overdosing of raw sludge.

→ with overwithdrawal of digested sludge.

→ with sudden admission of industrial waste.

The remedy to this is to add hydrated lime in doses of 2.3-4.5 kg per 1000 person to the raw sludge.

(iii) Seeding with digested sludge.

→ When a sludge digestion tank is first put into service it is highly beneficial to seed it with the digested sludge from another tank.

→ without seeding, it may take a few month to get a tank into operation properly.

→ Proper seeding will help attain quick balance condition of reaction. in the digestor.

(iv) Mixing and stirring of the raw sludge with digested sludge.

→ if incoming fresh raw sludge is throughly mixed with the digested sludge by some effective method of agitation so as to make a homogenious mix. the time required for digestion is considerably reduced, as opportunity of contact between organic matter and Mo is increased.

(v)

(V) Nuisance Causing bacteria.

→ These are those bacteria which reduces

$$SO_4^{2-} \longrightarrow S^{2-} \text{(sulphide)}$$

where, S^{2-} is distrastrous for methane former.

Gases produced during Anerobic Digestion.

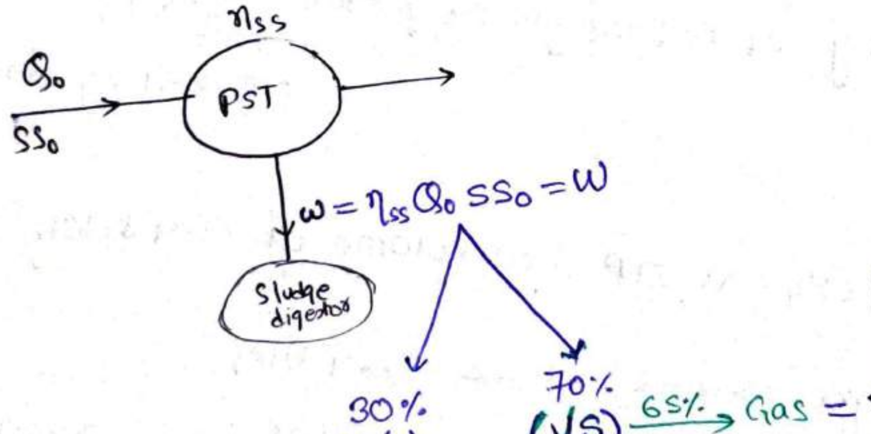
→ Of all the solids that comes into digestor from PST, (70%) are volatile, & 30% are fixed.

→ Out of total volatile solide 65% is reduced to gases and 35% form digested sludge.

→ Total volume of gases produced is $0.6\ m^3/kg$ of volatile solid present or $0.9\ m^3/kg$ of Volatile solid reduced to gases.

→ Out of total gases formed 65-70% is CH_4 and 30% CO_2 rest are remaining gases.

→ Calorific value of methane is $8600\ Kcal/m^3$



0.6 m^3/kg of VS present

$= 0.6 \times \underline{VS}$

$= 0.6 \times 0.7\,\omega$

$= 0.42\,\omega$

Or

0.9 m^3/kg VS reduced to gases.

$= 0.7\,\omega \times 0.65 \times 0.9$

$= 0.41\,\omega$

Note:- 1 kg of BOD of glucose produce 0.35 m³ of CH_4

$C_6H_{12}O_6 \rightarrow 3CH_4 + 3CO_2$

$BOD_{max} = COD = ThOD$

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

180gm   192gm

1 kg $= \frac{192}{180} \times 1 = 1.06$ kg of oxygen

$\boxed{\rightarrow \text{1 kg of glucose has 1.06 kg of BOD.}}$ — (i)

$C_6H_{12}O_6 \rightarrow 3CH_4 + 3CO_2$

1mole   3mole

180gm   48

1 kg $\frac{48}{180} \times 1\text{kg} = 0.267$ kg of methane

$\boxed{\rightarrow \text{1 kg of glucose produce 0.267kg of methane}}$ — (ii)

1.06 kg of BOD of glucose produces 0.267 kg of methane

1 kg of BOD of glucose produce $= \frac{0.267}{1.06} \times 1$

$= 0.251$ kg of methane

$\boxed{\text{1 mole of } CH_4 \text{ at STP has volume of 22.4 liter}}$

16 gm methane —— 22.4 liter

0.25 kg of methane —— $\frac{22.4}{16} \times 0.25 \times 10^3 \times 10^{-3}$

$= 0.35m^3$ of methane

1 kg of BOD of glucose produce 0.35 m³ of CH_4 (STP) (atm, 0°)

Q. A 10MLD waste water plant is to treat waste having 2000 mg/l COD, effluent can have 400 mg/l of COD in it. Raw sewage SS = 1300 mg/l. If BOD = 0.6 COD. find

(i) Fuel value of this waste

(ii) Effeciency of PST for removal of suspended solid.

Soln

(i) BOD of waste water $= (2000-400) \times 0.6 \times 10 \times 10^{6} \times 10^{-6}$

$= 9600$ kg/day

Volume of methane produced $= 9600 \times 0.35 = 3360\ m^3/day$ —(i)

fuel value of waste $= 3360 \times 8600$

$= 2.889 \times 10^{7}$ Kcal/day.

(ii) Volume of methane $= 0.6 m^3$ kg of volatile solid

(assuming it to be 70% of total gases produced) $= 0.6 \times [0.7 \times \eta_{ss} \times 10 \times 10^{6} \times 1300 \times 10^{-6}] \times 0.7$

$= 3822 \cdot \eta_{ss}$ —(ii)

from (i) and (ii)

$3822\, \eta_{ss} = 3360$

$\eta_{ss} = \frac{3360}{3822} = 87.91\%$

Types of Anerobic Digester.

→ Anerobic digestion can be carried out in any of the following units :-

(i) Standard Rate sludge Digester. (SRSD)
(ii) High Rate sludge Digestor (HRSD)

(i) SRSD

Raw sludge is directly fed into these digestor where they are acted upon by acid & methane formers in active zone leading to the digestion of sludge.

→ As no mixing is induced during digestion stratification of consistuents takes place in it.

→ Digested sludge being heaviest is collected from the bottom and disposed over drying beds.

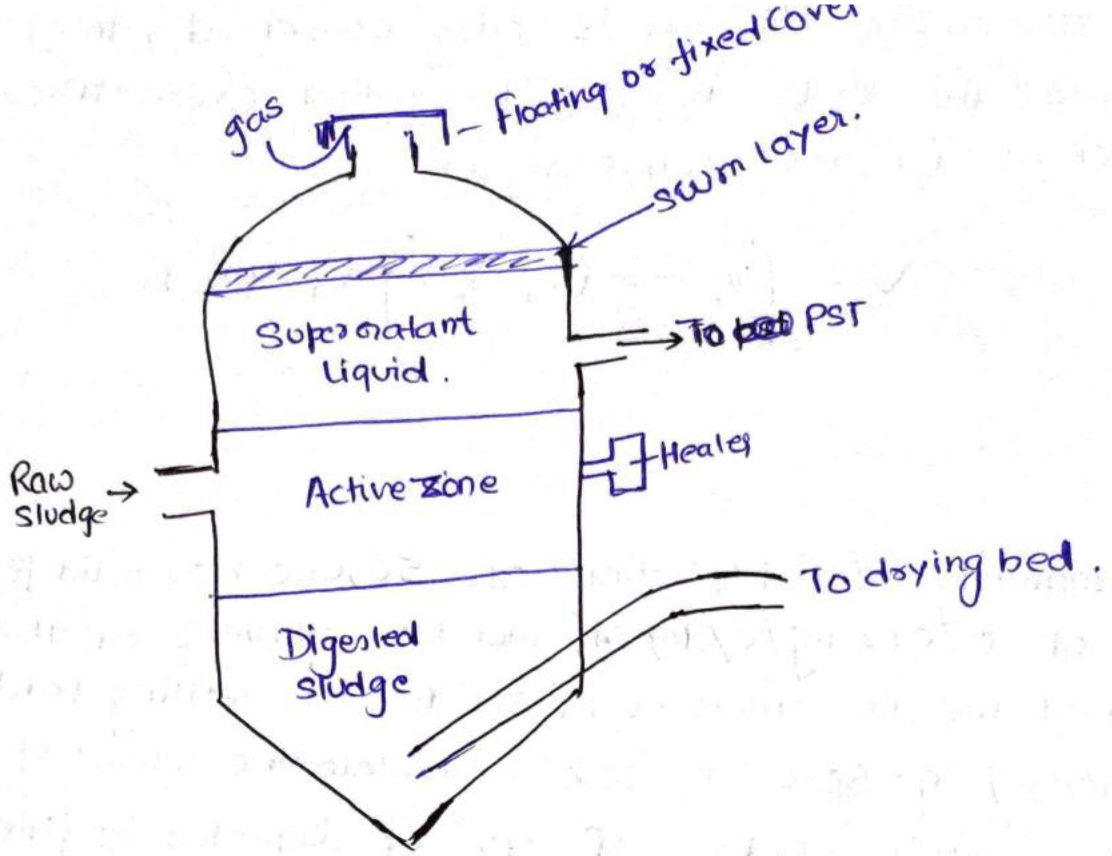
→ Gases being lightest rises to surface and is collected from the top.

→ Supernated liquid being highly organic (due to rising of organic suspended particles from active zone into it by the organic gases) it is send back to PST for removal of organic suspended solid from it.

→ Oil & grease if present in the sludge rises to top and leads to the formation of scum layer, which serve following purposes—

(i) It avoids the release odourous gases (H_2S), there by helps in maintaining sanitary condition around the digester.

(ii) They act as thermal insulator, thereby helps in increasing the rate of digestion and reducing the digestion period.



Design data of anerobic disaster:-

(i) Dia of tanks = 3-12m (~~18 m~~ may extend upto 18 m)

(ii) Depth of tank = 6m (may extend upto 12m)

(iii) Bottom hoppered floor of tank is given a ~~slope~~ slope of 1:1 to 1:3 (1H: 3V)

(iv) Degestion Period/Mean cell residence time = 30days.

(v) Volume of Digestor (v)

(a) If variation in quantity of sludge is assomed to be linear.

volume of tank = $V = \left(\frac{V_1 + V_2}{2}\right) \cdot t_d$

V_1 = Volume of raw sludge fed per day.

V_2 = volume of digested sluge withdrawn per day.

(b) If variation in quantity of sludge is assumed to be parabolic

$$V = \left[V_1 - \frac{2}{3}(V_1 - V_2)\right] \times t_d$$

— Our code use this ~~code~~ formula.

(vi) If monsoon storage is also considerd, then additional volume of V_2T is also provided over the design volume. (T = monsoon period)

$$V = \left[v_1 - \frac{2}{3}(v_1 - v_2)\right] t_d + V_2T$$

Q A community with population of 50,000 has solid production rate of 0.072 kg/c/day. If moisture content of the sludge formed due to settlement of solids in settling tank of efficiency $\eta = 60\%$ is 90%. calculate the volume of digester stablizing this sludge, if 4% of digestor is filled with fresh sludge daily. Assume ~~density of~~ G sludge = 1.03.

wt. of solids in sludge = $50000 \times 0.072 \times 0.6$
$= 2160$ kg/day.

$P = 90\%$.

10 kg of solid + 90 kg of water = 100 kg of sludge.

2160 kg of solid $= \frac{100}{10} \times 2160$
$= 21600$ kg of sludge.

Volume of raw/fresh sludge $= \frac{21600}{\rho} = \frac{21600}{1.03 \times 10^3}$
$= 20.97\ m^3/d$

$4\%\ V = V_1 \times 1$
$V = \frac{20.97}{0.04}$
$V = 524.25\ m^3/$

(ii) HRSD

→ In this case treatment is carried out in two stages where in the first stage complete ~~to~~ mixing of incoming raw sludge is done ~~mech~~ mechanically to increase the opportunity of contact betn organic matter and M/O. that in turn increase the rate of digestion.

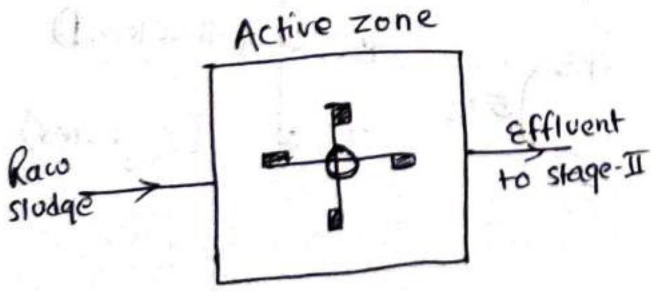
→ Here stage I can be referred as active zone

→ No change in the volume of sludge take place in stage I as dewatering is not permitted.

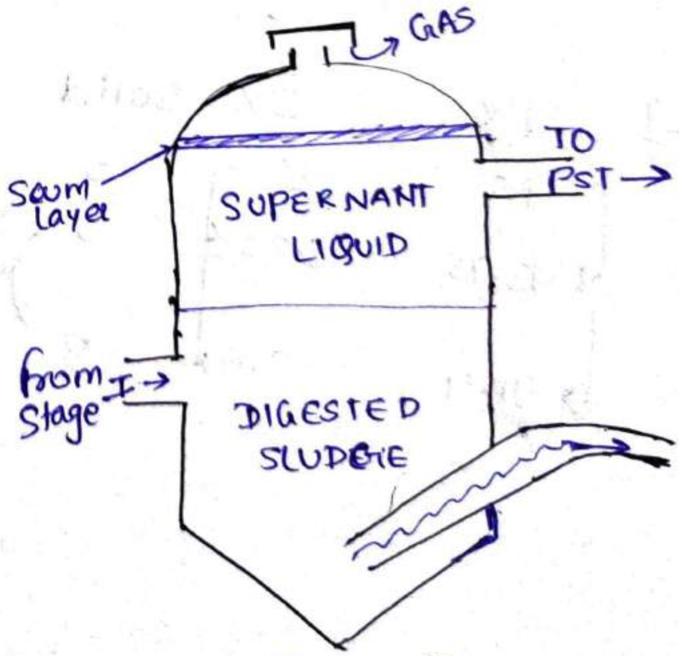
→ Operational stability of this process with respect to shock loading is comparatively more due to homogenious environment created by mixing.

$$V_I = V_1 \, t_{D_1} \quad (t_{d_1} = 10\text{-}15 \text{ days})$$

$$V_{II} = \left(V_1 - \frac{2}{3}(V_1 - V_2)\right) t_{D_2} + V_2 T \quad (t_{D_2} = 10\text{-}12 \text{ days}).$$



Stage-I



Stage-II

Sludge and its moisture content.

→ Moisture content of raw sludge is approximately 95%.

→ Moisture content of biological sludge after trickling filter is 96-98%.

→ Moisture content of biological sludge after ASP 98%.

moisture content $p\%$	S%	Solid (Kg)	water kg	Sludge (kg)	Solids (Kg) = Sludge (kg)
99	1	1	99	100	1 = 100
					$\Delta P = 4\%$
95	5	5	95	100	1 = 20
					$\Delta P = 9\%$
90	10	10	90	100	1 = 10

Note:- Volume of sludge is controlled by its moisture content. and density of sludge is controlled by its solid content.

eg

P%	S%	Solid	+ water	= sludge	
95	5	5	95	= 100	(raw)
93.75	6.25	5	75	= 80	(thickned)
		40%	57%		
91.4	8.6	3	32	= 35	(Digested)

Q Raw sludge (V_a, P_A) , Digested sludge [V_B, P_B)

$$\begin{bmatrix} P_A > P_B \\ V_A > V_B \\ \rho_A < \rho_B \end{bmatrix}$$

Solid + Water = Sludge

$100 - P_A$ $\quad$ P_A $\quad$ = 100kg of raw sludge

$= \left(\frac{100}{100-P_A} \times \omega\right)$ of raw sludge.

$\omega = \eta_{ss} Q_0 . SS_0$

$$\text{Volume of raw sludge} = \frac{100\omega}{100-P_A} \cdot \frac{1}{\rho_A} \quad \text{—— (i)}$$

Digested sludge

Solid + water = Sludge

$100 - P_B$ $\quad$ P_B $\quad$ = 100

$= \frac{100}{100-P_B} \times \omega'$ of digested sludge

$\omega' = NVS + 0.35VS$

$$\text{Volume of digested sludge} = \frac{100\,\omega'}{100-P_B} \times \frac{1}{\rho_B} \quad \text{—— (ii)}$$

Note:- Volume of sludge = $f(\omega, P/s, \rho)$

Assume, $\boxed{\omega = \omega' \quad (\omega > \omega')}$

$$\frac{V_A(100-P_A)}{\cancel{100}} \rho_A = \frac{V_B(100-P_B)}{\cancel{100}} \times \rho_B$$

Assume, $\rho_A = \rho_B$ ($\rho_B > \rho_A$)

$$\boxed{V_A(100-P_A) = V_B(100-P_B)}$$

Q If moisture content of digested sludge is 80%. what is moisture content of raw sludge.

$P_B = 80\%$

$P_A = ?$

$V_A(100 - P_A) = V_B(100 - P_A)$

Assume, $V_B = \frac{V_A}{3}$

$3\cancel{V_B}(100 - P_A) = \cancel{V_B}(100 - P_A)$

$3(100 - P_A) = (100 - 80)$

$P_A = 93.33\%$

Sludge Handling Process

Handling of sludge can be done by ~~any of the~~ following ~~method~~ sequence of operation.

(i) Primary operation.
(ii) Thickening
(iii) Stabilization/digestion.
(iv) Conditioning.
(v) Dewatering
(vi) Heat drying
(vii) Incineration.

(i) Primary Operation.

→ This process include:- ~~grinding~~
(a) Grinding - Particle size reduction.
(b) Screening - removal of fibrous material.
(c) Degriting - removal of grit.
(d) Blending - It is done for homogenization.
(e) Storage - flow equalization to avoid shock loading.

Note:- In some cases advanced primary treatment is also carried out by use of chemical termed as coagulant to remove not only suspended solid but also soluble organic matter. like ferric chloride.

(ii) THICKENING

→ It is the process of removing the water from sludge or to increase the solid content, so as to reduce the volume of sludge to be handled/digested by the plant, there by reducing cost of digestion

→ Volume reduction of approximately 30-80% can be reached by sludge thickening.

→ It can be done by any of following methods:-

(a) Gravity thickening.

→ It is done for primary sludge or primary sludge combined with ASP sludge.

→ It is never adopted for ASP sludge independently.

→ It is not effective, if in combined sludge the ASP sludge portion exceed 40% of total sludge.

→ It is designed for surface loading rate of approx 25-30 $kg/m^2/day$

→ In this the working is same as that of sedimentation tank. in which sludge is ~~about~~ allowed to stand to carryout settlement of suspended solid.

→ Depth of tank 3-4m.

→ t_d = 4 hours.

(b) Air floatation

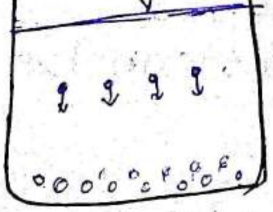
→ This unit employes floation of sludge by air under pressure or vaccume ~~so~~ is normally used for thickening of activated sludge.

→ The supernatant is pressurised at 3-5 kg/cm^2. and saturated with air in pressure tank.

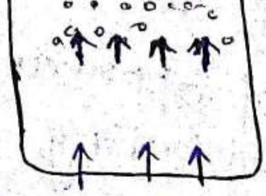
→ The excess dissolved air then rises up in the form of bubbles attaching themselves to the particle.

(c) Centrifugation.

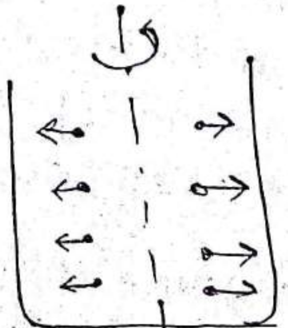
It is done where availability of space is limited.



Gravity thickner



Air floatation



Centrifugation.

III Stabilization/Digestion:

It is carried out for decomposition of organic suspended solid present in the sludge either aerobically or anerobically.

IV Conditioning

It is the process in which sludge solid are treated with chemicals or other means to prepare the sludge for dewatering.

→ It improves the drainability of digested sludge & sludge becomes more responsive/amenable to dewatering.

→ It can be achieved by various method such as:—

(a) Elutriation.
(b) Chemical conditioning.
(c) Heat treatment.
(d) Freezing.

Note:- Elutriation is a process of sludge conditioning whereby a sludge is washed either by fresh water or plant effluent to reduce the sludge alkalinity and fine particles thus decreasing the amount of required chemicals in further handling of sludge.

V Dewatering of sludge.

A physical unit operation used to reduce the moisture content of sludge is termed as dewatering

→ Dewatering can be done as follow:-
(a) Centrifugation.
(b) Bed filter press.
(c) Use of drying beds.

Unit operation - physical process
Unit process - chemical process

VI Heat Drying

It involve the ~~opera~~ application of heat to evaporate water & to reduce the moisture content of bio solids through conduction, convection or radiation, so as to reduce its transportation cost, improve storage, capability.

VII Incineration

It involves total conversion of organic solid to oxidise end product by burning it at high temp. in presence of oxygen.

VIII Sludge Disposal

It can be carried out by (a) land fill method
(b) sewage farming.
(c) Dumping into sea.

Q A 10MLD waste water plant treats 250mg/l of suspended solid in it, settling tank in the plant has efficiency of 65% then, specific gravity of organic & inorganic solid is 1.02 & 2.5 respectively. compute:

(i) Volume of raw sludg produce @ moisture content of 96%.

(ii) Volume of digested sludge if digestion reduced 70% of volatile solid into gases at moisture content of 80%.

(iii) Volume of thickned sludge if it reduces the moisture content to 90%.

(iv) ~~Digested~~ Volume of sludge after dewatering when it reduces the moisture content to 50%.

(v) Volume of heated sludge @ moisture content of 10%.

(vi) Volume of sludge after incineration.

(vii) Area of land required for disposal of incinerated sludge. If solid loading rate is 0.05 m³/hac/day.

Volume of raw sludge = Volume of solid + Vol. of water.

$$V_{RS} = (V_{VS} + V_{NVS}) + V_w$$

Total weight of solids in raw sludge

$$= \eta_{ss} \cdot Q_0 \cdot SS_0$$

$$= 0.65 \times 10 \times 10^6 \times 250 \times 10^{-6}$$

$$= 1625 \text{ kg/day.}$$

Assuming 70% of total solids to be volatile.

$$V_{VS} = \frac{0.7 \times 1625}{1.02 \times 10^3} = 1.115\ m^3$$

Assuming 30% of total solid to be NVS.

$$V_{NVS} = \frac{0.3 \times 1625}{2.5 \times 10^3} = 0.195\ m^3/day.$$

$P = 96\% \Rightarrow$ 4 kg of solid + 96 kg of water

Volume of water $V_w = \frac{96}{4} \times \frac{1625}{10^3} = 39\ m^3.$

$=$

$$V_{RS} = (V_{VS} + V_{NVS}) + V_w$$

$$= 1.115 + 0.195 + 39 = 40.31\ m^3/d.$$

(ii) $P = 90\% \Rightarrow$ 10 kg of solid + 90 kg of water

Volume of water in thickened sludge, $V_w = \frac{90}{10} \times \frac{1625}{10^3}$

$= 14.625\ m^3/day$

$$V_{TS} = (V_{VS} + V_{NS}) + V_w$$

$$= (1.115 + 0.195) + 14.625$$

$$= 15.935\ m^3/day.$$

(iii) $V_{VS} = \frac{0.7 \times 1625 \times 0.3}{1.02 \times 10^3} = 0.3345\ m^3/day.$

$V_{NVS} = 0.195\ m^3/day.$

828.75

$P = 80\% \Rightarrow$ 20 kg of solid + 80 kg of water

Volume of water in digested sludge, $V_w = \frac{\frac{80}{20} \times [(0.7 \times 0.3 + 0.3) \times 1625]}{10^3}$

$= 3.315\ m^3/day.$

$$V_{DS} = 0.334 + 0.195 + 3.315$$

$$V_{DS} = 3.844\ m^3/day.$$

(iv) $P = 50\%$

50 kg of solid + 50 kg of water

V_w in dewatered sludge $= \frac{50}{50} \times \frac{828.75}{10^3}$

$= 0.828 \; m^3/day$

Volume of dewater Slude

$V_{DWS} = (0.334 + 0.195) + 0.828$

$= 0.529 + 0.828$

$= 1.357 \; m^3/day$

(v) $P = 10\%$ ⇒ 90 kg of solid + 10 kg of water.

V_w of water in dried sludge $= \frac{10}{90} \times \frac{828.75}{10^3} = 0.092 \; m^3/day$

Volume of dried sludge $= V_w + V_s$

$= 0.092 + 0.529$

$= 0.62 \; m^3/day$

(vi) $V_{IS} = V_{NVS} = 0.195 \; m^3/day$

(vii) Area of land $= \frac{0.195 \; m^3/day}{0.05 \; m^3/hac/day}$

$= 3.9$ hact

Q Design a digestion tank for primary sludge with the help of following data

(i) Average flow = 20 MLD

(ii) Total suspended solid in raw sewage = 300 mg/l

(iii) Moisture content of digested sludge = 85%.

Assume any other data not given is required.

Let 60% of suspended solid settles in PST to form raw sludge having moisture content of 95%. & $\rho = 1000\ kg/m^3$

$$\text{Volume of raw sludge, } V_1 = \frac{100\, \omega}{(100-P)\rho} = \frac{100 \times \eta_{ss} \times Q_0 \times SS_0}{(100-P)\rho}$$

$$= \frac{100 \times \overbrace{\boxed{0.6 \times 20 \times 10^6 \times 300 \times 10}}^{3600\ kg/day}}{(100-95) \times 1000}$$

$$V_1 = 72\ m^3/day$$

$$V_2(100-P_2) = V_1(100-P_1)$$

$$V_2(100-85) = 72(100-95)$$

$$V_2 = 24\ m^3/day.$$

Assume, $t_d = 30$ days.

$$\text{Volume of digestor} = V = \left[V_1 - \frac{2}{3}(V_1 - V_2)\right] \times t_d$$

$$V = \left[72 - \frac{2}{3}(72 - 24)\right] \times 30$$

$$V = 1200\ m^3.$$

Assume, $H = 6\ m$.

$$SA = \frac{V}{Depth} = \frac{H}{Depth} = \frac{1200}{6} = 200\ m^2$$

$$\frac{\pi}{4} \times D^2 = 200$$

$$D = 16\ m > 12\ m$$

2°/ Biological treatment

→ Secondary or biological treatment is being carried out for the decomposition of dissolved organic matter present in waste water by the action of micro-organisms either in the presence or absence of oxygen.

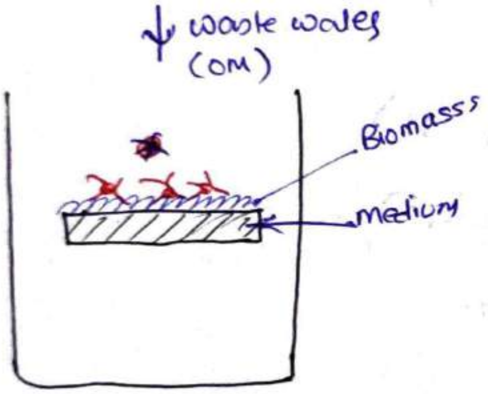
→ To establish the contact between organic matter and micro-organism following mechanism are being developed:

(i) Attached growth system.

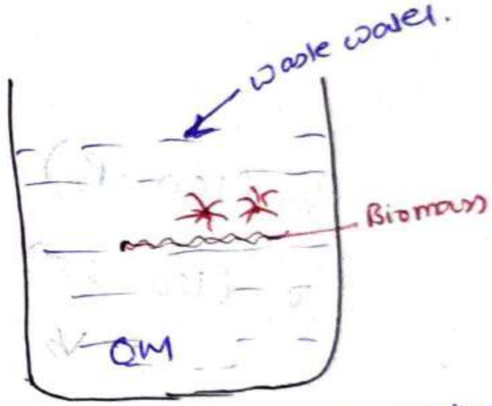
→ In this a medium is provided for attachment of biomass (m/o) and waste water is passed through it.

(ii) Suspended growth system.

→ In this bio-mass is suspended in waste water



Attached growth system



suspended growth system.

Types of Biological Treatment are as follows:-

Process	Contact Mechanism	Mode of decomposition
1. Trickling filter	Attached	Aerobic
2. Rotary Biological reactor	"	"
3. Activated sludge process	Suspended	"
4. Oxidation Pond	"	"
5. Septic tank	"	Anerobic
6. Inoff tank	"	Anerobic.

Upflow anerobic sludge balanced reactor	suspended	Anerobic

Trickling Filter [Aerobic Attached growth system]

→ As the waste water trickles through filter medium biomass get attached over the medium particles normally within 2-3 weeks, making the filter ready for operation.

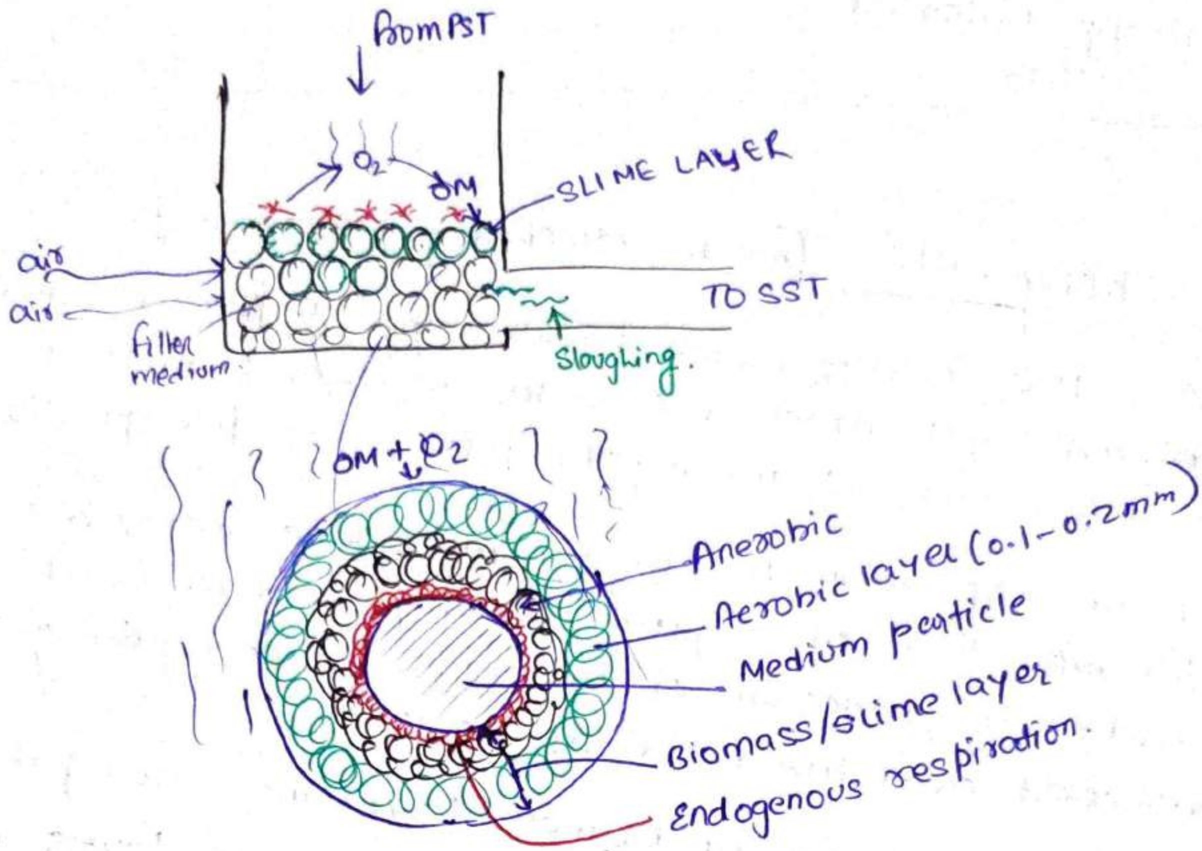
→ During operation MO in bio mass layer carry out the decomposition of organic matter present in waste water, leading to formation of bio mass. which again gets attached over the medium particle.

→ As the thickness of biomass layer increases penetration of OM and O_2 is limited only in the upper layers of the biomass, leading to the endogenous respiration at the interface that weakens the bond between bioman layer and medium particles.

→ At a particular stage during the operation: bio-mass layer gets removed of from the medium particle due to the turbulence ~~of~~ created by movement of waste water around it (This is termed as sloughing).

→ The sloughed off biomass is taken to SST for its settlement.

→ These filters are constructed above the ground with honey comb wall so as to ensure availability of oxygen througout volume of medium particles.



Application of waste water in trickling filter can be done by any of following method.

(i) Rotatory distribution system.

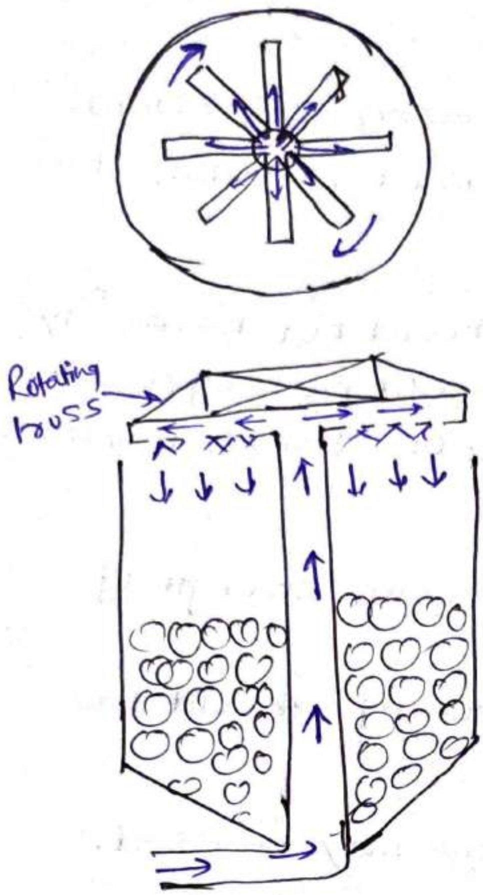
(ii) spray nozzle method.

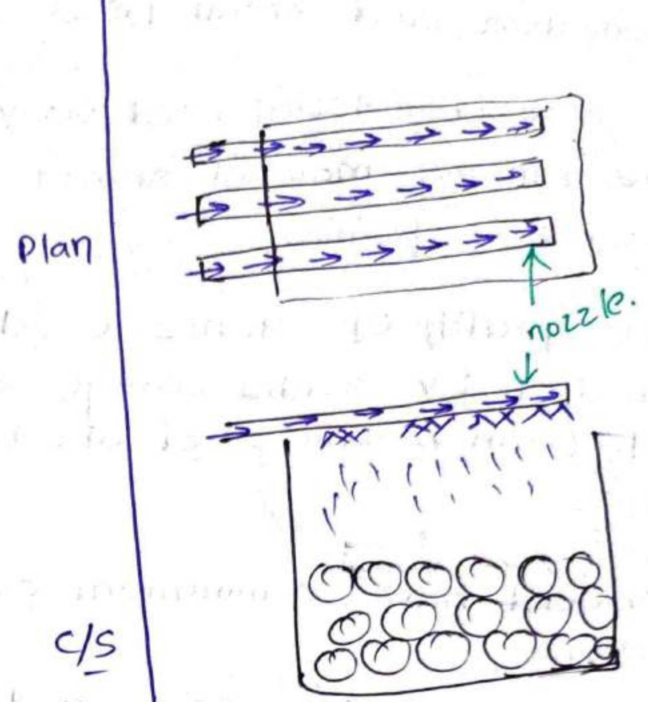
→ Application of waste water is uniform throughout the medium in rotatory method moreover application ~~is also~~ ~~continuous~~ of waste water is also continuous.

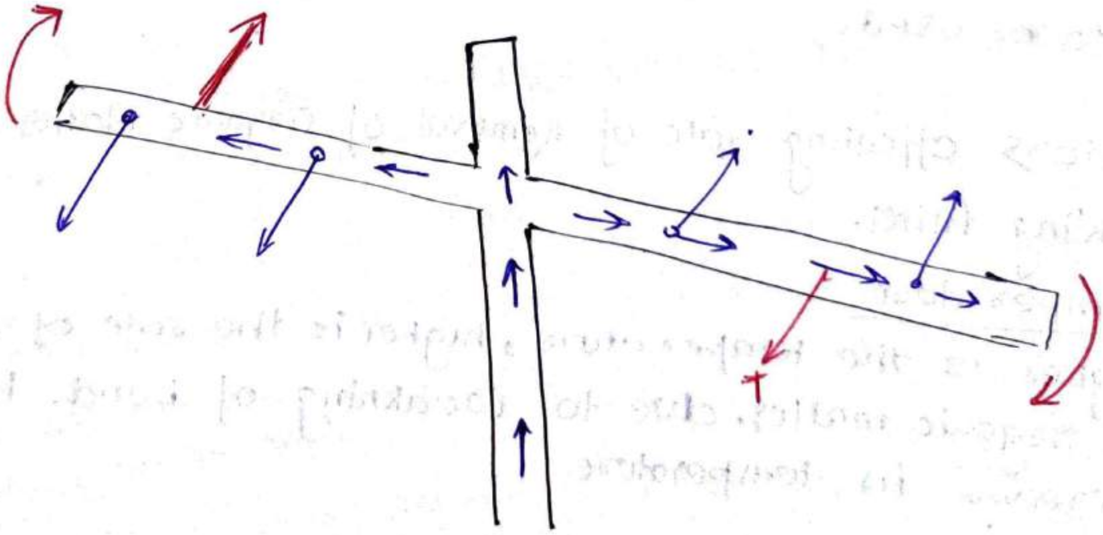
→ Distribution of waste water is intermittent in spray nozzle method due to which waste water do not remain fresh at all the time hence causes odour problem.

(i) Rotatory Distribution System.

(ii) spray nozzle method







→ The filtering media in this case consist of coarser material like cubically broken stones or slag free from dust and small pieces.

→ The size of material used vary between 25-75mm.

→ The filtering material should be washed before it is placed in position.

→ The quality of stones used should not be easily effected by acidic sewage & should be sufficiently hard. (with hardness of about 12 on Brinels Hardness Scale).

→ It should have a minimum compressive strength of $100 N/mm^2$.

→ Its resistance to freezing and throwing should be comparitavely more.

→ Eg. Rocks like granite or lime stone may be used.

→ In some cases plastic medium instead of stones can also be used.

Factors affecting rate of Removal of Organic Matter in Trickling Filter.

(i) Temperature.

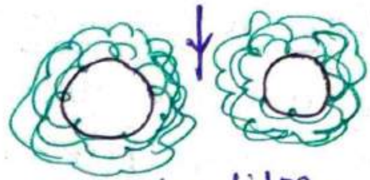
→ Higher is the temperature, higher is the rate of removal of organic matter. due to weakning of bonds by increase in temperature.

(ii) Hydraulic Loading rate.

Higher is the hydraulic loading rate, higher is the rate of sloughing. which increases the rate of removal of organic matter.

(iii) Organic Loading Rate. (OLR) = $Q_0 \cdot S_0$.

Higher is the organic loading rate lesser is the rate of removal.



(iv) Rate of Diffusibility of food and O_2 in bio-film.

→ Higher is this rate, higher the rate of removal of organic matter.

Properties of trickling filler.

Adv

→ Rate of filter loading is high, hence area required is comparitively less.

→ Effluent obtain from trickling filler is sufficiently nitrified and has BOD removal 75 to 80 %.

→ Working of trickling filler is simple and doesnot require any supervision.

→ They are flexible in operation and can treat different sewage having different concentration of BOD.

→ They are self cleansing.

→ Mechanical weir and tear is small.

→ Moisture content of sludge obtained from trickling filler is in range of 96-98%.

→ They operate more efficiently in warm weather

→ The head loss through these fillers make automatic dosing of filler necessary.

→ Cost of construction is more.

→ They cannot treat raw sewage and primary treatment require prior to it.

→ These filter pose no. of operational problem.

(i) Fly nuiscane: Due to the presence of nutrient of the insect ~~they grow over the~~ in waste water they grow over filter medium. and lay their eggs (larve) which act as a energy foo flies. (Psychoda)

→ Over the period of time the entire filter is flooded with flies making the operation of filter difficult & reducing the rate of removal of organic matter through it.

→ In order to avoid this flooding of waste water over the medium is done. or insecticides like DDT, Chlorance, benzene hexachloride is applied over the medium.

(ii) Odour Nuisance ← presence of H_2S : This is found only if spray nozzle method of distribution of waste water is adopted as in this case supply is intermittent and waste water does not remain fresh at all the times

→ In order to avoid this waste water is chlorinated before feeding it in trickling filter or by increasing the hydraulic loading rate.

(iii) Pounding Problem:
Due to the growth of algae and fungi in voids of filter medium chocking of medium takes place which leads to ponding of waste water over the medium surface.

→ To avoid this, Cl_2, $Ca(OH)_2$, $CaSO_4$ is added in waste water prior to trickling filter.

Types of trickling Filter.

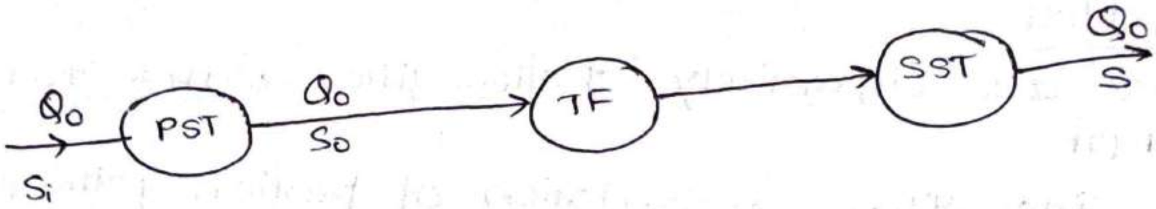
⇒ Trickling filters are generally of two types:-

(i) standard Rate trickling filter. (SRTF)

(ii) High rate trickling filter. (HRTF)

(i) SRTF

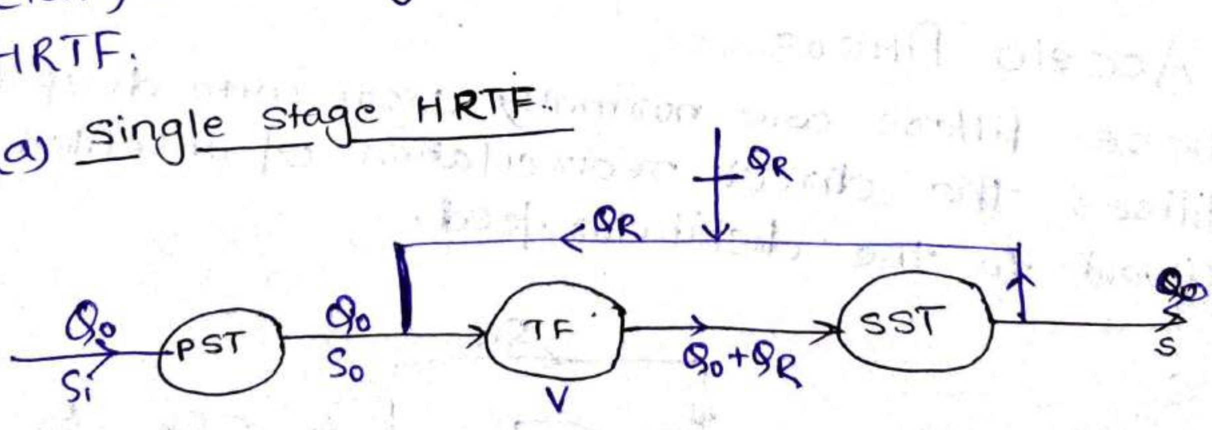
It is type of trickling filter in which rate of removal of organic matter is comparatively less, due to its lower hydraulic loading rate. as there is no provision of recirculation in it.



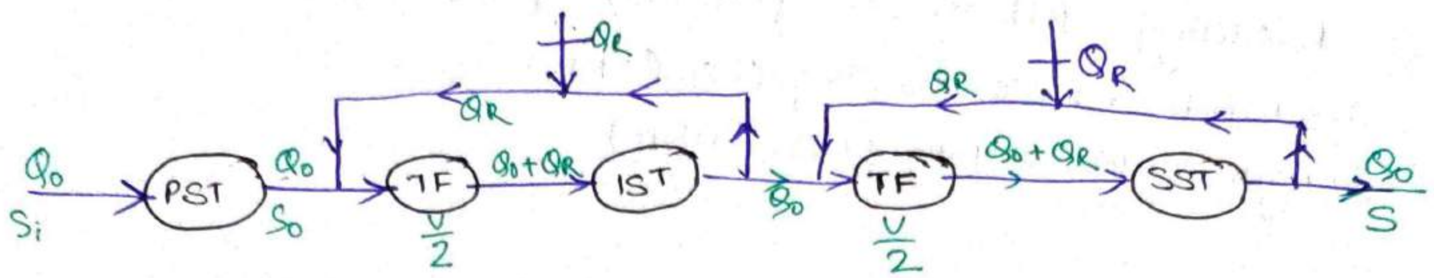
(ii) HRTF

These are type of trickling filter which offers higher rate of removal of organic matter due to its higher HLR, which is increased by recirculating a part of sewage effluent back into filter medium either in single stage or in multiple stages. According to which these are classified as single stage HRTF or Multiple stage HRTF.

(a) Single stage HRTF.



(b) Two Stage HRTF



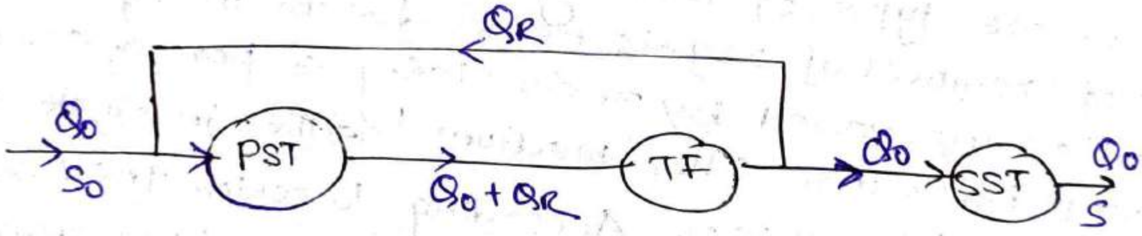
Types of High Rate trickling filter.

High rate trickling filter are further classified as :-

(i) Bio-filter

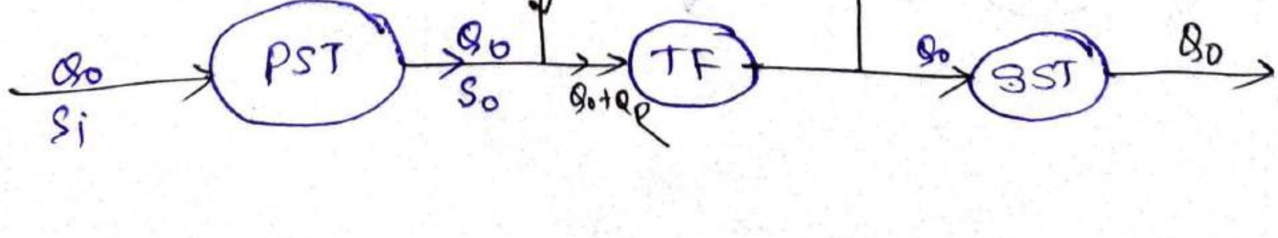
→ These are comparatively shallow filters with 1.2 to 1.85 m in depth.

→ These filter utilises recirculation of portion of the filter effluent to the primary settling tank. for a second passage through the filter.



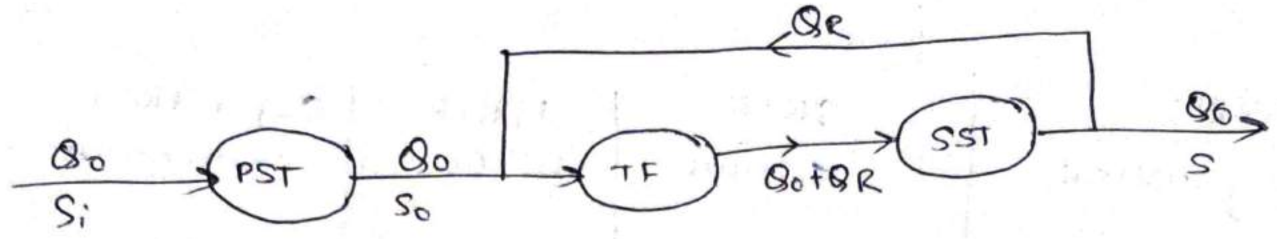
(ii) Accelo Filters

→ These filters are normally 1.8 to 2.4 m deep and utilises the direct recirculation of unsettled filter effluent to the distributor feed.



(iii) Aero filter

→ These filters are 1.8m deep and utilises the recirculation from SST



Design data for trickling filter:-

Parameters	SRTF	HRTF	SUPER HRTF
1. Hydraulic Loading Rate. ($m^3/m^2/d$)	1-4	10-40 (Including recirculation)	40-200
2. Organic loading Rate ($Kg/m^3/day$)	0.11 - 0.37	0.37-1.85 (excluding recirculation).	1-6
3. Depth (m)	1.5-3	1-2	4-12
4. Recirculation ratio	0	1-4	1-4.

SRTC

① $PA = \frac{Q_o}{HLR}$

② $\frac{\pi D^2}{4} = PA \Rightarrow D = ?$

③ $V = \frac{Q_o \cdot S_o}{OLR}$

④ $H = \frac{V}{PA}$

HRTF

① $PA = \frac{Q_o}{HLR_{ER}}$ or $PA = \frac{Q_o + Q_R}{HLR_{IR}}$

② $\frac{\pi}{4} D^2 = PA \Rightarrow D?$

③ $V = \frac{Q_o S_o}{OLR_{ER}}$ or $V = \frac{Q_o S_o + Q_R S}{OLR_{IR}}$

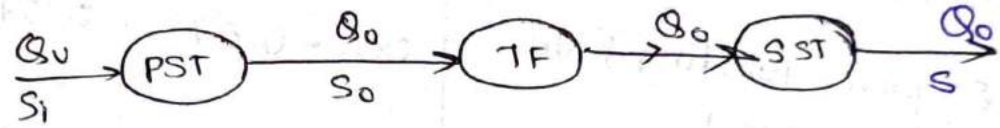
④ $H = \frac{V}{PA}$

Note:- TF are designed for average discharge but under-drainage system & distribution system is designed for <u>peak discharge</u>.

PROPERTES	SRTF	HRTF	Super HRTF
1. Dosing Interval	< 5 min	15 - 60 sec	Continuous
2. Sloughing.	Intermediate	Constinuous	Coustinuous
3. Effluent Quality.	Fully nitrified	Partially nitrified	Nitrified

Efficiency of Trickling filter.

i) SRTF

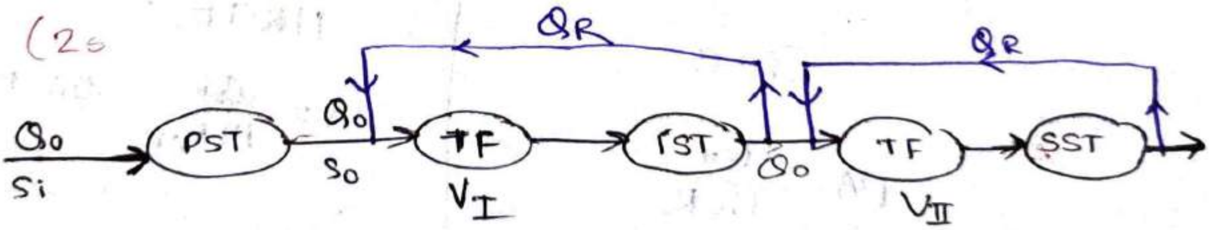


$$\eta = \frac{100}{1 + 0.0044\sqrt{U}}$$

$U = OLR$ (Kg/hac-m/day).

$$\boxed{1\,kg/day/m^3 = 10^4\,kg/day/ha\text{-}m}$$

(ii) HRTF (2



I^{st} state

$$\boxed{\eta_I = \frac{100}{1 + 0.0044\sqrt{\frac{W_1}{V_1 F_1}}}}$$

$W_1 = Q_0 S_0$ (kg/day)

V_1 = Volume of TF in stage I - (hac-m)

F_1 = recirculation factor

$$\boxed{F_1 = \frac{1+R}{(1+0.1R)^2}}, \quad R = \frac{Q_R}{Q_0}$$

II^{nd} stage

$$\eta_{II} = \frac{100}{1+\frac{0.0044}{1-\eta_I}\sqrt{\frac{W_2}{V_2 F_2}}}$$

$W_2 = (1-\eta_I)\,W_I$ (kg/d)

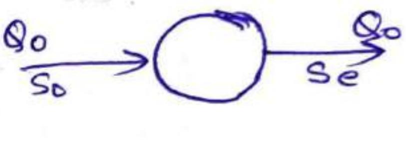
V_2 = Volume$_{II}$ (hac-m)

$F_2 = \dfrac{1+R}{(1+0.1R)^2}$, $R = \dfrac{Q_R}{Q_0}$

$$\boxed{\text{Overall efficiency, } \eta = \eta_I + (1-\eta_I)\,\eta_{II}}$$

Note:- Efficiency of trickling filter can also be computed using "ecken felder equation".

(i) SRTF

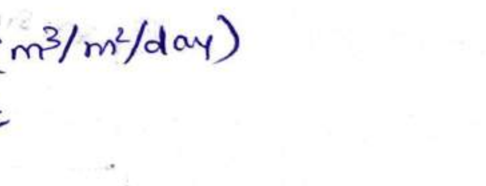


$$\frac{Se}{S_0} = e^{-\left(\frac{KD}{Q_L^n}\right)}$$

Se = effluent BOD in (mg/l)
S_0 = influent BOD (mg/l)
K = Rate constant
D = depth of filter (m)
Q_L = hydraulic loading rate (m^3/m^2/day)
n = filter media constant

$$\eta(\%) = \frac{Q_0 S_0 - Q_0 S}{Q_0 S_0} \times 100$$

(ii) HRTF



$$\frac{Se}{S_0} = \frac{e^{-\left(\frac{KD}{Q_L^n}\right)}}{(1+R) - R e^{-\left(\frac{KD}{Q_L^n}\right)}}$$

$$Sa = \frac{Q_0 S_0 + Q_R Se}{Q_0 + Q_R} = \frac{S_0 + R\,Se}{1+R}$$

$$\boxed{R = \frac{Q_R}{Q_0}}$$

Q] A town having a population of 30,000 person is producing following sewage

(i) Domestic sewage @ 120 l/c/day. having BOD = 200 mg/l

(ii) Industrial sewage @ 3×10^5 l/day having BOD = 800 mg/l

Design high rate trickling filter of single stage for treating above waste water.

Assume that PST removes 35% of BOD allow an organic loading rate of 10000 kg/hectare m/day excluding recirculation. The recirculation ratio is 1. and surface loading rate should not exceed 170 ML/hac/day including recirculation

→ Also find the efficiency (n) of filter and effluent BOD.

$$Q_0 = 120\times30000\times10^{-3} + 3\times10^5\times10^{-3}$$

$$= 3600 + 300$$

$$= 3900\ m^3/day.$$

$$BOD_i = \frac{3600\times200 + 300\times800}{3900}$$

$$= 246.15\ mg/l$$

Since PST remove 35% BOD $= 246.15 - 0.35\times246.15$

$= 159.99$

$= 160\ mg/l.$

$$\eta_I = \frac{100}{1+0.0044\sqrt{\frac{w_1}{V_1F_1}}}$$

$$= \frac{100}{1+0.0044\times\sqrt{\quad}}$$

$$F_1 = \frac{1+R}{(1+0.1R)^2}$$

$$= \frac{2}{(1+0.1\times1)^2}$$

$$= 1.652$$

$$SA = \frac{Q_0+Q_R}{HLR} = \frac{2\times3.9\times10^6}{170\times10^6} = 0.0458\ hectare = 458.8\ m^2$$

$$\frac{\pi D^2}{4} = 458.8$$

$$D = 24.14\ m.$$

$$V = \frac{Q_0 S_0}{OLR_{EN}} = \frac{3.9 \times 10^{6} \times 160 \times 10^{-6}}{10^{4}} \times 10^{4} = 624\,m^2$$

$$H = \frac{V}{SA} = \frac{624}{458.8} = 1.36\,m.$$

$R = 1,\; F = \frac{1+R}{(1+0.1\times R)^2} = 1.65$

Efficiency

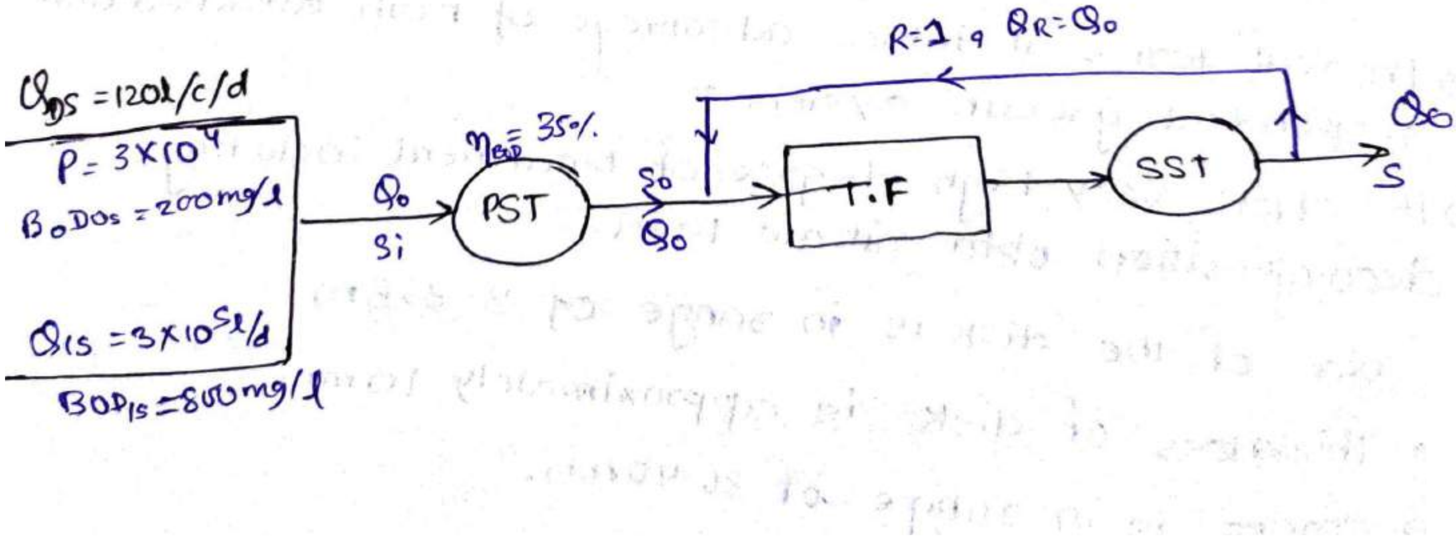
$$\eta = \frac{100}{1 + 0.0044\sqrt{\frac{\omega}{VF}}} = \frac{100}{1 + 0.0044\sqrt{\frac{10^4}{1.65}}}$$

$$= 74.48.$$

$$\eta = \frac{Q_0 S_0 - Q_0 S}{Q_0 S_0} \times 100$$

$$74.48 = \frac{160 - S}{160} \times 100$$

$$S = 40.8\,mg/l$$



ROTATORY BIOLOGICAL REACTOR/ CONTRACTOR (RBR/RBC)
(Aerobic attached growth system).

→ In this case closely spaced rotating disk is used as a medium for the growth of bio-mass layer (slime layer)

→ These disk are kept emerged in waste water upto 40% of their size. and are exposed alternatively to the atmosphere and the waste water due to which micro-organism in the bio-mass layer comes in contact with organic matter when disk is imersed in waste water and utilises the oxygen from the atmosphere for its decomposition. leading to the formation of biomass which again gets attached over the disk.

→ As the thickness of biomass layer increases penetration of OM and O_2 is limited only in upper layers leading to endogenous respiration at the interface, which weaknen the bond between the two.

→ This biomass layer gets removed off from the disk by the turblence created by the disk through the waste water.

→ In real terms it takes advantage of both attached and suspended growth system.

→ It offers very high degree of treatment including decomposition upto nitrate level.

→ Dia of the disk is in range of 3-3.5m

→ Thickness of disk is approximately 10mm.

→ spacing is in range of 30-40mm.

→ These disk are mounted on common shaft

~~otter~~

→ It is same as that of trickling filter with only difference ie in this case it is biomass layer which passes through waste water.

→ RPM = 1-2

Q A preliminary design for rotating disc type installation is done to serve 1000 person assuming organic loading of 20gm BOD/m²/day and 3m diameter disc spaced 5cm apart on center @ 54gm, use BOD at 54gm/c/d and sewage flow of 200 l/c/d. Assume 0.2m total clearence for the disk and depth of the tank is 2m.
Permissible BOD in effluent = 49mg/l. Determine:-

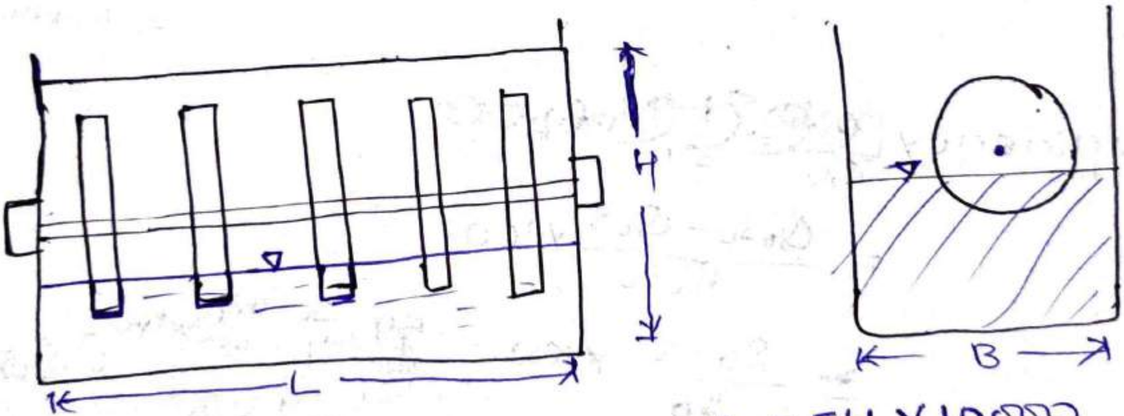
(i) No. of disk require and tank volume

(ii) Hydraulic loading rate.

(iii) Surface loading rate.

(iv) Efficiency

(v) Excess sludge at the rate of 0.6kg/kg of BOD removed.



(i) Total BOD to be removed = 54 X 10000
= 5.4×10^4 gm/day.

Total area required (A) for removal of BOD = $\frac{5.4 \times 10^4}{20} = 2700 \, m^2$.

Area of one disk available for removal of BOD.

$A_1 = \frac{\pi d^2}{4} \times 2 = \frac{\pi \times 3^2}{4} \times 2 = 14.137 m^2$

No. of disk required $N = \frac{A}{A_1} = \frac{2700}{14.136} = 191$

(ii) Height of tank $= 2m$

width $= d + 0.2 = 3.2m$.

length = no. of disk × spacing

$= 191 \times 5 \times 10^{-2} = 9.55m$.

Volume $= L \times B \times H$

$= 2 \times 3.2 \times 9.55 = 61.12 m^3$.

(iii) Hydraulic loading rate $= \frac{Q_0}{N \times \frac{\pi d^2}{4} \times 2} = \frac{2000 \times 1000}{191 \times \pi \times \frac{3^2}{4} \times 2}$

$= 740.1\ l/m^2/day$.

(iv) Surface loading rate $= \frac{Q_0}{X\text{-area}} = \frac{200 \times 1000}{3.2 \times 2 \times 0.4} \times 10^{-3}$

$= 78.1\ m^3/m^2/d$.

(v) Efficiency $=$

$= \frac{Q_0 S_0 - Q_0 S}{Q_0 S_0} \times 100$

$= \frac{S_0 - S}{S_0} \times 100 =$

$= \frac{\frac{54}{200} \times 10^3 - 49}{\frac{54}{200} \times 10^3} \times 100 = 81.85\%$

(vi) BOD removed,

$= Q_0 (S_0 - S)$

$= 200 \times 1000 \left(\frac{54}{200} \times 10^3 - 49 \right) \times 10^{-6}$

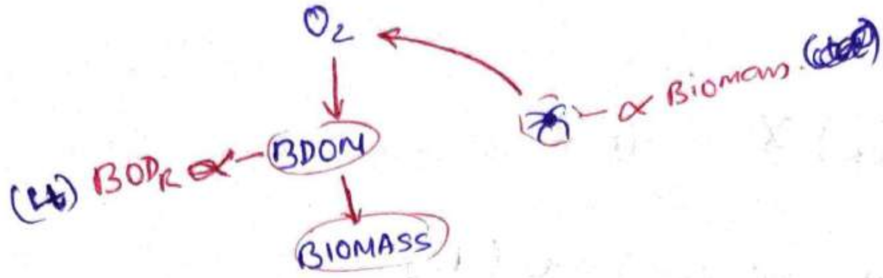
$= 44.2\ kg/day$.

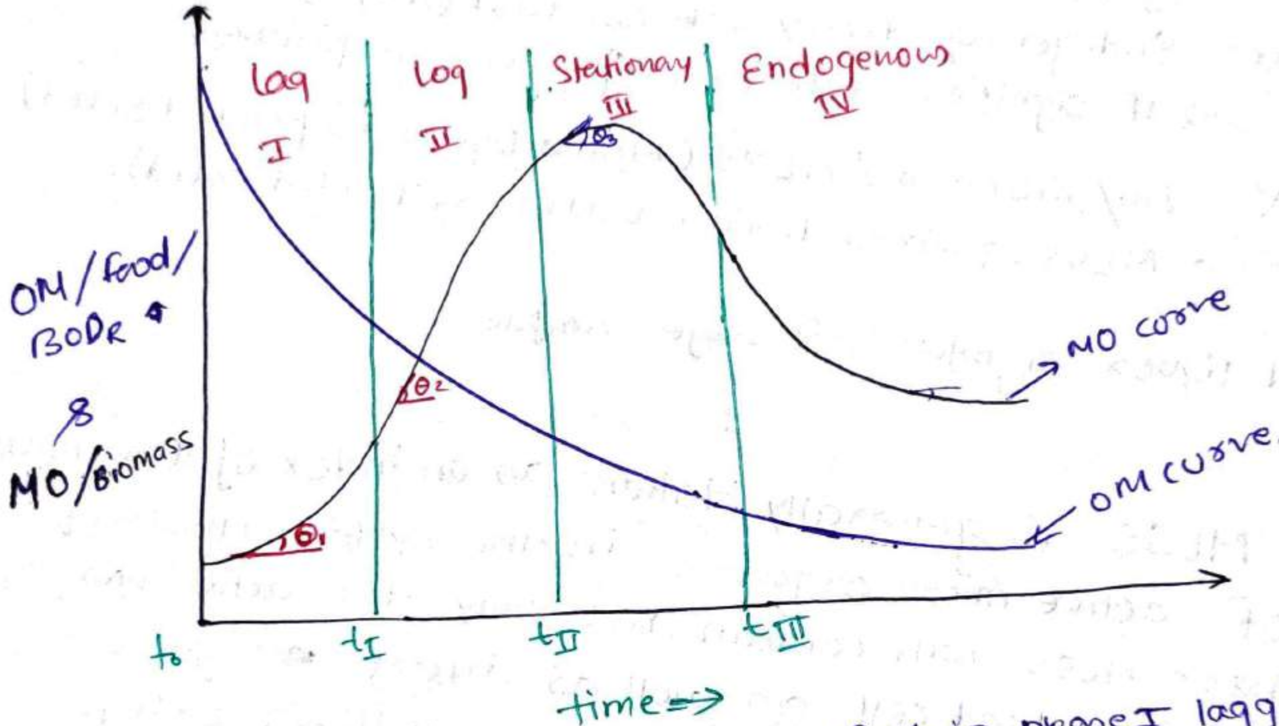
Excess solid produced $= 0.6 \times 44.2$

$= 26.52\ kg/day$.

III Activated Sludge Process (ASP)

(Aerobic suspended growth system)





$\theta_1 << \theta_2$ → Rate of growth of M/o in phase I lagging.

(i) OM/ Food/ BOD_R

$$\frac{ds}{dt} \propto -s$$

$$\frac{ds}{dt} = -K_D s \quad \text{—(A)}$$

K_D = deoxygenation constant (d^{-1})

S = Remaining BOD (mg/l)

(i) MO/ Biomass.

$$\cancel{\frac{dx}{dt} \propto x}$$

$$\cancel{\frac{dx}{dt} = Kx - K_{ER} X}$$

$$\frac{dx}{dt} \propto X$$

$$\frac{dx}{dt} = (k - K_{ER})\, X \quad —(B)$$

K = overall growth rate constant (day^{-1})
= It signifies the rate of growth of biomass.

K_{er} = Endogenous decay rate constant (day^{-1})
→ It signifies rate of endogenous respiration.

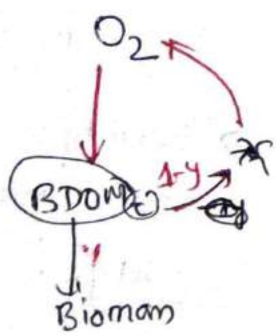
X = MO/ Biomass / MLSS (Mixed liquor suspended solid)
= MLVSS (Mixed liquour volatile suspended solid).

Mixed liquor signifies sewage sample.

Note: MLSS is generally taken as an index of the mass of active micro-organism in the system, However these MLSS will contain not only the active micro-organism but also dead cell as well as inert organic and inorganic matter derived from influent sewage.

→ For this reason, MLVSS is also sometime is used and may be prefered than MLSS as it eliminates the effect of of inorganic matter.

Note:- During bio-logical activity only Certain fraction of organic matter undergoing decomposition will be converted into energy (ie, consumed by Microorganism) and rest converted into Biomass.



→ If during metabolism ie (catabolism) y fraction of food is converted into biomass (waste), (1-y) fraction would be consumed by micro-organism for deriving the energy.

$$-y\frac{ds}{dt} = \frac{dX}{dt} \quad , \quad \frac{dx}{dt} = KX$$

eg

t	OM	BIO MASS.
3 pm	2 kg	200 gm
3:30pm.	1.5 kg	400 gm.

$dt = 3{:}30 - 3 = 30\,min$

$ds = 1.5 - 2 = -0.5\,kg = -500\,gm$

$dx = 400 - 200 = 200\,gm.$

$y = \frac{200}{500} = 40\%$

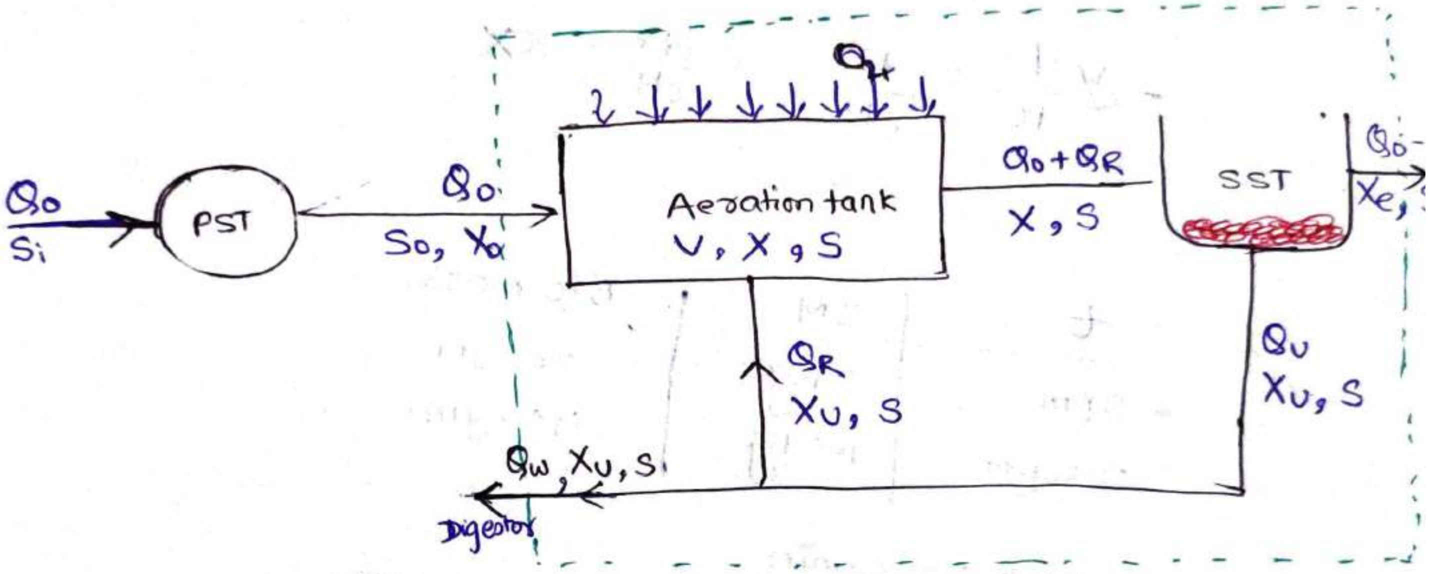
$$-0.4\frac{(-500)}{30}\frac{gm}{min} = \frac{200\,gm}{30\,min.}$$

$$-y\frac{ds}{dt} = \frac{dx}{dt}$$

Activated sludge process.

→ Sludge generated in SST most consist of living or active micro organism, thereby it is termed as "activated sludge."

→ In this process the sludge is recirculated back into the system in order to carry out the decomposition of organic matter in presence of oxygen in suspension, hence this process is termed as activated sludge process.



$S_0 > S$

X, X_0

$Q_0 > Q_U$

$X_U > X$

$X_0 > X_e$

(i) Biomass.

Biomass ~~which~~ IN + BIOMASS GROWTH = BIOMASS OUT

$$\cancel{Q_0 X_0}^{0} + \left(\frac{dx}{dt}\right) V = \cancel{(Q_0 - Q_\omega) X_e}^{0} + Q_\omega X_u$$

$$(K - K_{ER}) X V = Q_\omega X_u$$

$$K = \frac{Q_\omega X_u}{XV} + K_{ER} \quad \text{—(i)}$$

(ii) Food

FOOD IN − FOOD CONSUMED = FOOD OUT

$$Q_0 S_0 - \left(\frac{ds}{dt}\right) V = (Q_0 - Q_\omega) S + Q_\omega S$$

$$\left[-Y\frac{ds}{dt} = \frac{dx}{dt} \quad , \frac{dx}{dt} = KX\right]$$

- $$Q_0 S_0 - \frac{KXV}{Y} = Q_0 S$$

$$K = \frac{Q_0 (S_0 - S) Y}{VX} \quad \text{—(ii)}$$

From (i) and (ii)

$$\boxed{\frac{Q_\omega X_u}{XV} + K_{ER} = \frac{Q_0 (S_0 - S) Y}{VX}}$$

Assumption made in above analysis:-

(i) Influent (X_0) and effluent (X_e), bioman concentration is neglected.

(ii) BOD removal is not consider in SST.

(iii) Mixture is considerd to homogenious inside the aeration tank (It is achieved by mixing inducing mixing in aeration tank.).

→ In such a case variation of organic matter is given by:-

$$Q_0 S_0 - \left(-\frac{ds}{dt}\right) V = (Q_0 - Q_\omega) S + Q_\omega S$$

$$Q_0 S_0 - K_D S V = Q_0 S$$

$$Q_0 S_0 - K_d S Q_0 t = Q_0 S$$

$$S (1 + K_d t) = S_0$$

$$\boxed{S = \frac{S_0}{1 + K_d t}} \quad \text{or,} \quad L_t = \frac{L_0}{1 + K_d t}$$

→ If several reactors are arranged in series, the effluent of one reactor becomes the influent of the next.

→ A substrate balance written across a series of n reactor results in following equation.

$$S_n = \frac{S_0}{\left(1 + K_d \frac{t}{n}\right)^n}$$

S_0 = initial influent
S_n = final influent

Design Parameter of ASP.

(i) Hydraulic Retention time (HRT)

→ It is defined as the ratio of volume of aeration tank to the rate of flow of waste water into it excluding recirculation.

$$HRT\ (t) = \frac{V}{Q_0}\ (hrs)$$

(ii) Organic Loading Rate (OLR)

It is defined as amount of organic matter entering into system to the volume of aeration tank.

$$OLR = \frac{Q_0 S_0}{V}\ (Kg/m^3/day)$$

(iii) Specific food/substrate/OM/BODR utilisation rate.

→ It is defined as amount of organic matter removed in the system, to the mass of biomass in the aeration tank.

$$U = \frac{Q_0 S_0 - [(Q_0 - Q_w)S + Q_w S]}{VX}$$

$$U = \frac{Q_0 (S_0 - S)}{VX}\ (Kg/Kg/day)$$

(iv) Sludge Age (θ_c)

→ It signifies the time for which bio-mass remains in the system

→ It is defined as mass of biomass in the aeration tank to the mass of bio-mass leaving the system per day.

$$\theta_c = \frac{VX}{(Q_0 - Q_w)X_e + Q_w \times X_u}$$

If X_e is neglected,

$$\theta_c = \frac{VX}{Q_\omega \cdot X_u} \quad (\text{kg-d/kg})$$

Note:-

$$\frac{Q_\omega X_u}{VX} + K_{ER} = \frac{Q_0 (S_0 - S) Y}{VX}$$

$$\boxed{\frac{1}{\theta_c} + K_{ER} = UY}$$

(iv) $\frac{F}{M}$ ratio =

It is defined as food entering into the system to the mass of bio-mass present in the system.

$$\frac{F}{M} = \frac{Q_0 S_0}{VX}$$

(i) $\frac{F}{M} \uparrow$ $(F\uparrow, M\downarrow) \Rightarrow \eta \downarrow$

(ii) $\frac{F}{M} \downarrow$ $(F\downarrow, M\uparrow) \Rightarrow \eta \uparrow$

unit = (kg/kg/d)
= day^{-1}

$$\Rightarrow \boxed{\frac{F}{M} \propto \frac{1}{\eta}}$$

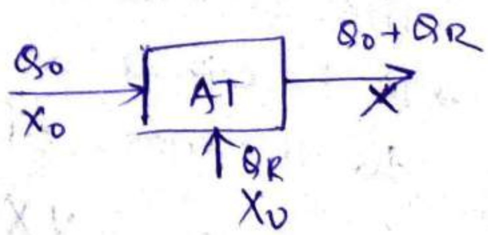
(vi) Recirculation Ratio $\left(R = \frac{Q_R}{Q_0}\right)$

It is defined as ratio of recirculated sludge discharge to the original discharge.

$$\cancelto{0}{Q_0 X_0} + Q_R \times X_u = (Q_0 + Q_R) X$$

$$Q_R (X_u - X) = Q_0 X$$

$$\boxed{\frac{Q_R}{Q_0} = \frac{X}{X_u - X}}$$



(vii) Sludge volume Index (SVI)

→ It signifies the concentration of bio-mass in the sludge that decides the rate of return of sludge, corresponding to desired MLSS concentration, for a particular $\frac{F}{M}$ ratio. to achieve required degree of treatment.

→ It is defined as volume occupied in "ML" by 1 gm of solid in mixed liquor when allow to settle for 30 minute.

→ It is determined by collecting 1l of mixed liquor from discharged end of aeration tank into a calibrating container and allow to stand for 30 minute to know the vol. of solid settle in it. "V_s (ml/l)".

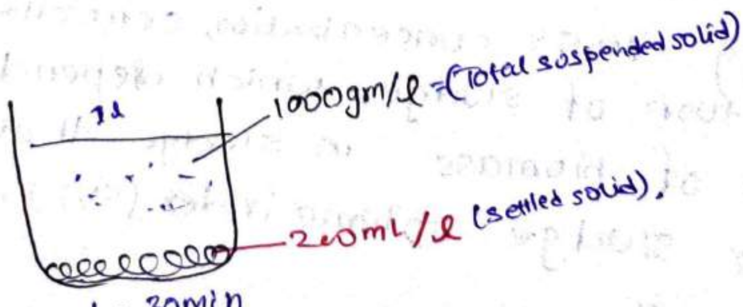
→ The sample after being remixed is further tested for concentration of MLSS in it by performing the standard test. "X_s (mg/lit)".

→ for municipal sewage sludge volume Index SVI

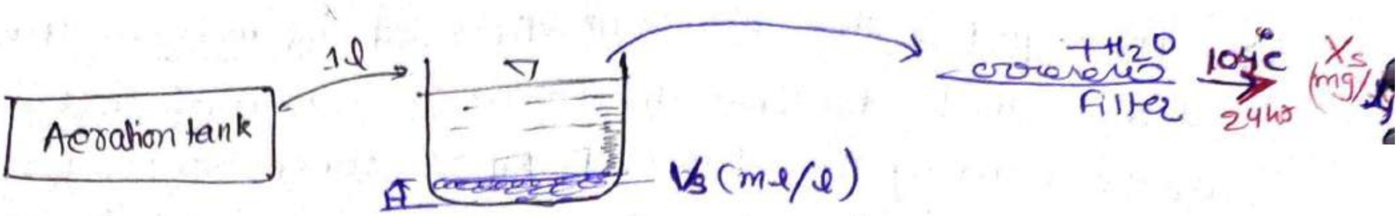
$$\boxed{SVI = 50\text{-}150 \text{ ml/gm}}$$

$$SVI \longrightarrow X_U \longrightarrow Q_R \longrightarrow X \longrightarrow \frac{F}{V} \longrightarrow \eta(\%)$$

$$\left[Q_R = \frac{X Q_0}{X_U - X}\right], \left[Q_R \propto X\right], \left[\frac{F}{M} = \frac{Q_0 S_0}{VX}\right], \left[\frac{F}{M} \propto \frac{1}{\eta}\right]$$



$$SVI = \frac{200}{1000} = 0.2 \text{ ml/gm}.$$



$$SVI = \frac{V_s}{X_s} \times 10^3 \ (ml/gm)$$

→ If it is assumed that the sedimentation of suspended solid in the laboratory is similar to that in sedimentation tank then

X_u (mg/l)

SVI (ml/gm)

$$X_u = \frac{1}{SVI} \ (gm/ml)$$

$$X_u = \frac{1}{SVI}\left(\frac{gm}{ml}\right) \frac{\times 10^3 (mg/gm)}{10^{-3} (l/ml)}$$

$$X_u = \frac{10^6}{SVI} \ (mg/l)$$

→ The most important point in operation of ASP is to maintain proper $\frac{F}{M}$ ratio, corresponding to influent BOD, which can achieved by increasing or decreasing MLSS concentration, controlled by rate of return of sludge, which depend upon concentration of biomass in sludge, that is governed by sludge volume index (SVI) which is found experiments.

$\frac{10^3}{10^{-3}}$

→ The excess sludge generated is wasted out i.e. being sent into the digestor for digestion.

$$(S_0, S) \longrightarrow \eta(\%) \rightarrow \frac{F}{M} \rightarrow X \rightarrow Q_R \rightarrow X_U \longrightarrow SVI$$

$$(Q_w = Q_U - Q_R)$$

→ In order to indicate the settleability of sludge in a secondary clarifier or effluent that is related to calculation of SVI a parameter termed as SDI (sludge density Index) is used

→ Sludge Density Index is defined as weight in gram of solid of 100 ml of sludge after settling for 30 minutes.

$$SDI = \frac{100}{SVI} \left(\frac{gm}{mL}\right)$$

$SDI \propto \frac{1}{S}$

[Note $SVI \propto \frac{1}{X_U}$, $SDI \propto X_U$]

Q An activated sludge process is operating at equilibrium with the flowing information waste water related data:

- waste water related data.

$Q_0 = 500\ m^3/hr$

$BOD_i = 150\ mg/l$

$BOD_e = 10\ mg/l$

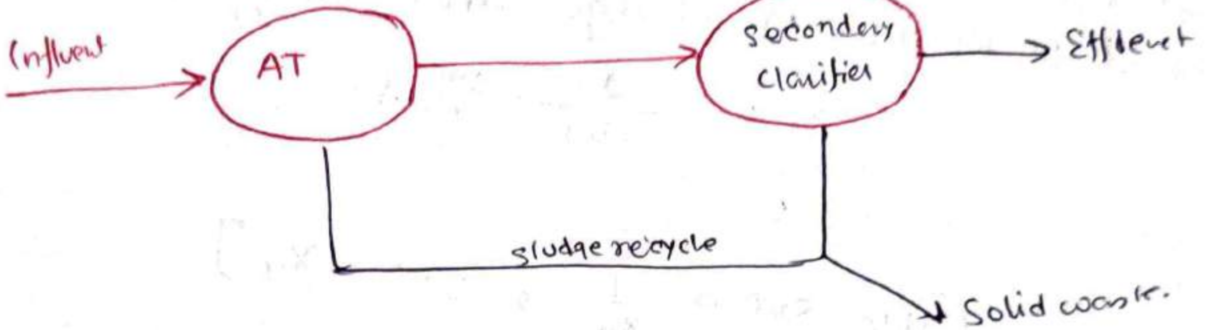
- Aeration tank related datas -

HRT = 8 hr

Mean cell resistance time or slude age / solid retention time $\theta_c = 240\ hr$

Volume of aeration tank = $4000\ m^3$.

MLSS = 2000 mg/l



Compute (i) $\frac{F}{M}$ ratio in (kg/kg/day)

(ii) Mass of solid wasted (kg/day)

(i) $FM = \dfrac{Q_0 S_0}{V X} = \dfrac{500 \times 150 \times m^3/hr \times mg/l}{4000 \times 2000\ mg/l}$

$= 9.375 \times 10^{-3}$ (kg/kg/hr)

$= 9.375 \times 10^{-3} \times 24$

$= 0.225$ (kg/kg/day)

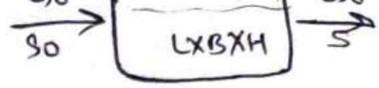
(ii) Mass of solid wasted = $Q_w \times X_u$

$$\theta_c = \frac{V X}{(Q_0 - Q_w) X_e + Q_w X X_u}$$

$$240 = \frac{4000 \times 2000}{Q_w \times X_u} \Rightarrow Q_w X_u = \frac{4000 \times 10^3 \times 2000 \times 10^{-6}}{10}$$

$= 800$ kg/day.

Q A waste water having an organic concentration of 54 mg/l is flowing at a steady state 0.8 m³/day through a detention tank of dimension 2m x 4m x 2m. If the contents of the tank are well mixed and decay constant is 0.1 d^{-1} the outlet concentration in mg/l is ______.



S

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

$$= \frac{2 \times 4 \times 2}{0.8} = \frac{16}{0.8} = 20$$

$$S = \frac{S_o}{1 + K_d \cdot t} = \frac{54}{1 + 0.1 \times 20} = 18\ mg/l$$

Q Consider the reactor shown in figure. The flow rate through the reactor is Q m³/hr. The concentration is in mg/lit of a compound in the effluent and effluent are C_o and C respectively. The compound is degraded in the reactor having following first order reaction. The mixing condition of the reactor can be varied such that the reactor becomes either a completely mixed flow reactor or plug flow reactor (PFR) the length of the reactor can be adjusted in these two mixing conditions to L_{CMFR} & L_{PFR} while keeping the cross-section of the reactor constant. Assuming steady state & for $C/C_o = 0.8$ the value of L_{CMFR}/L_{PFR} is ______.
plug flow reactor

$$\frac{C}{C_o} = 0.8$$

$$C = 0.8\,C_o$$

$$\frac{C_o - 0.8C_o}{C_o} = \frac{0.2}{1}$$

$$\eta = 20\%$$

c/c_o

$c = 0.8c_o$

Tor complele CMFR $\Rightarrow$ $C = \frac{C_0}{1+Kt}$

$$\frac{C}{C_0} = \frac{1}{1+Kt}$$

$$0.8 = \frac{1}{1+Kt}$$

$$0.8 + 0.8\,Kt = 1$$

$$t = \frac{0.25}{K}$$

FOR PFR $\Rightarrow$ $C = C_0 e^{-Kt}$

$$\frac{C}{C_0} = 0.8 = e^{-Kt}$$

$$t_{PFR} = \frac{0.233}{K}$$

for sleady stale condition.

$$V = \text{Const}$$

$$L = V \times t$$

$$\frac{L_{CMFR}}{L_{PFR}} = \frac{V \times t_{CMFR}}{V\, t_{PFR}} = \frac{0.25\ K}{K\ (0.223}$$

$$= 1.221$$

Types of ASP

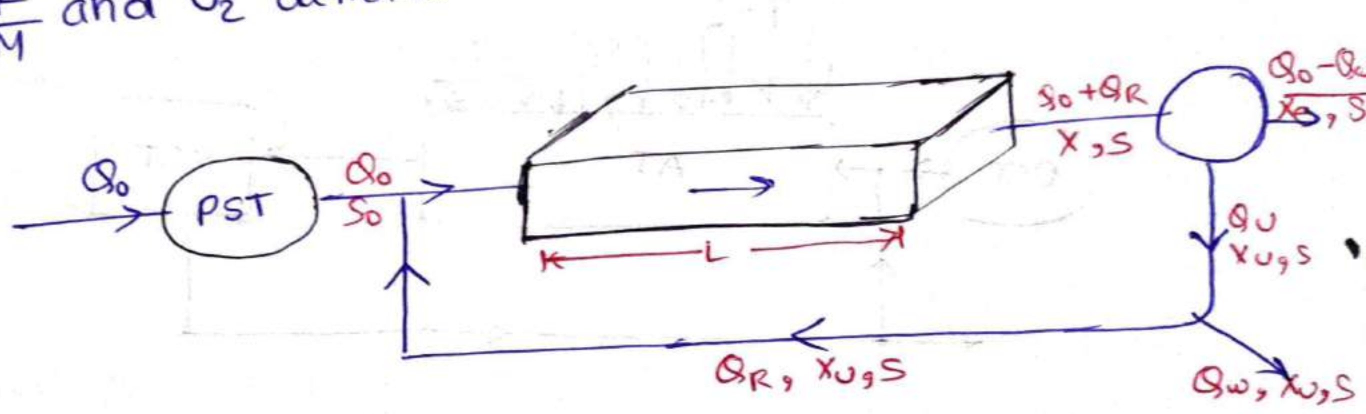
(i) Plug flow/conventional ASP

→ ~~It is used for~~ Plug flow implies that waste water and activated sludge moves progressively along the length of the aeration tank, remaining essentially unmixed throughout the volume of tank

→ It is suitable for plant having large capacity (even greater than 300MLD

→ long aeration channel are adopted in this case that results in higher efficency but no mixing is induced hence rate of removal is comparatively less.

$\frac{F}{M}$ and O_2 demand reduces along the length of the tank.



→ The main limitation of plug flow process are:- the

(i) aeration tank volume requirement is very high

(ii) The lack of operational stability at times of excessive variation in rate of flow or its BOD concentration.

(iii) Uniform distribution of O_2 in this case leads to deficiency of O_2 in initial section of tank & wastage of O_2 in later section of tank.

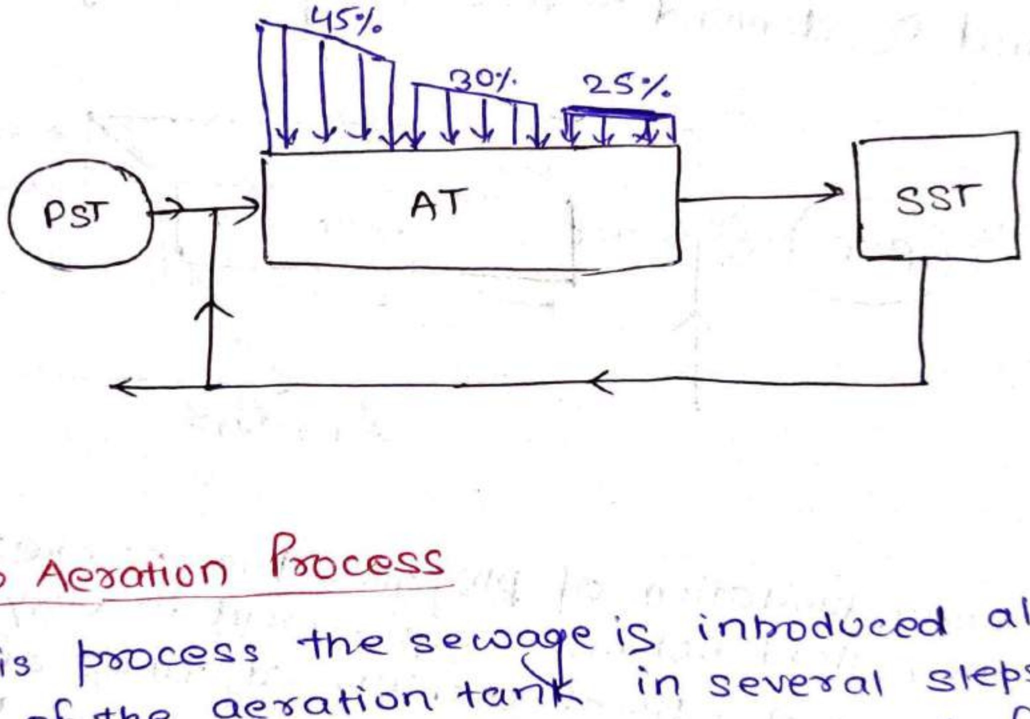
→ Limitation of conventional system are overcome over a period of time, leading to the development modified ASP as follows:-

(ii) Tapered Aeration Process.

→ This process involves a very little modification of the conventional process and ensures high air supply at the inlet and the initial length of the tank as compared to the downstream length.

→ It offers comparatively betler utilisation of oxygen along the length of the tank.

→ ~~Such a process~~ Ordinerily 45% of air is supplied to first 1/3 length of tank followed by 30% and 25% for remaining two, $\frac{1}{3}$ length of tank respectively.



(iii) Step Aeration Process

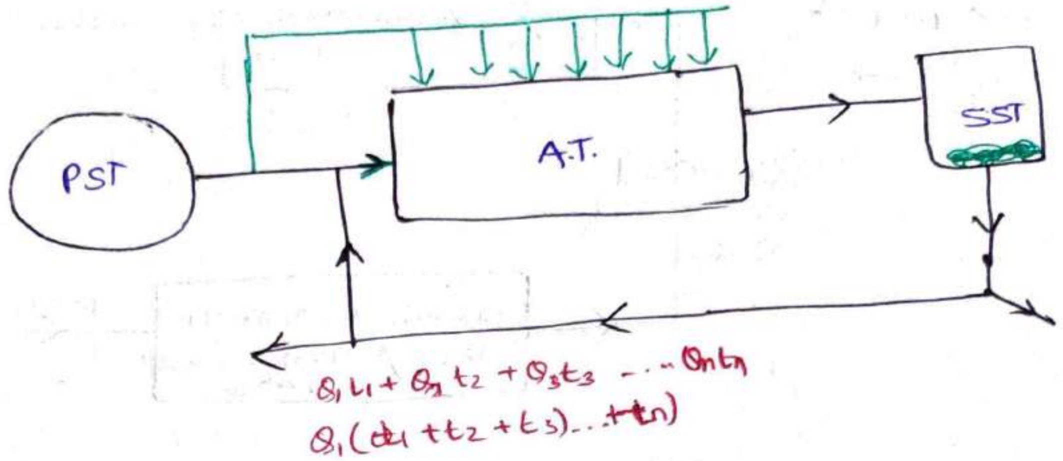
→ In this process the sewage is introduced along the length of the aeration tank in several steps while the return sludge is introdue at the head of aeration tank.

→ Such an arrangement results in uniform air requirement along the entire length of tank, hence the uniform air supply of the conventional plants can be efficiently used.

→ This process enables an appreacible reduction in aeration tank volume, without lowering BOD removal efficiency.

→ This method has considerable capacily to absorb shock organic loading.

→ It can be used for the plant of capacily upto 1000 MLD



(iv) Contact Stabilisation process (BIO SORPTION PROCESS)

→ This process has been designed for treating colloidal waste water.

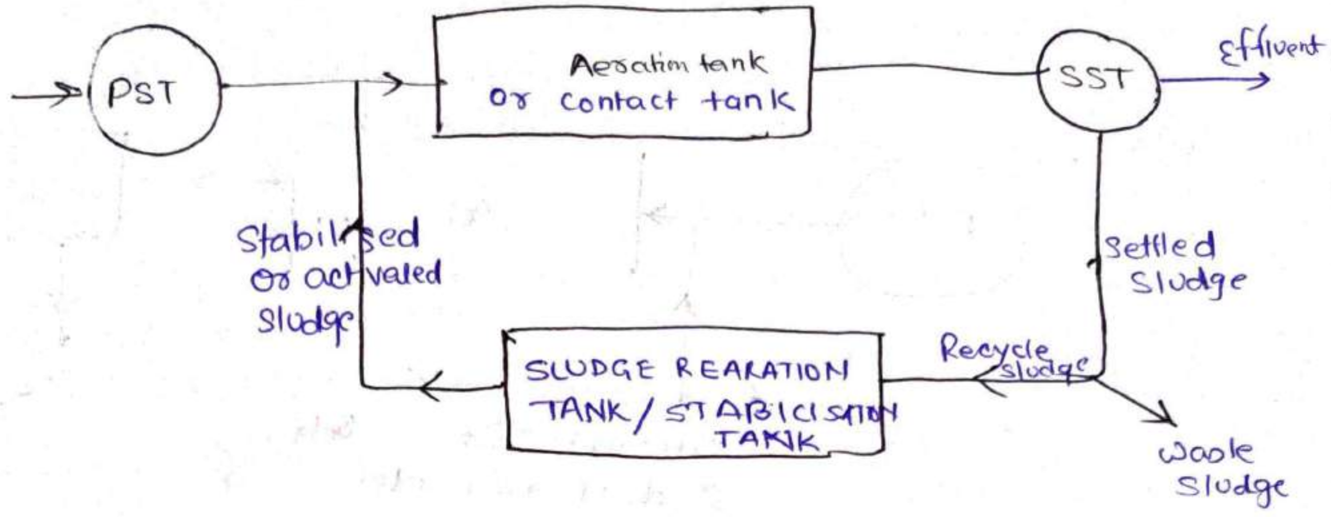
→ In this process, the sewage is recycled or returned sludge are mixed and aerated for a compouatively shorter period of 0.5-1.5hr in a special mixing tank called as contact tank.

→ The mixing will allow the suspended particles and dissolve organic matter to be sorbed to the activated sludge floc.

→ The sorbed organic and flocks are retained in SST where the effluent from the contact tank enters.

→ The settled sobred organics & flocs are then transfered to a sludge reaeration tank termed as stabilisation tank Aerodigester where the organics are decomposed over a period of about 3 to 5 hours. (6hr) before it is fed back into the contact aeration tank.

→ The stabilised sludge is then mixed with the infivent waste water again



→ Compare to conventional system, this process has greater capacity to handle shock loading because biological buffering capacity of the sludge reaeration tank.

→ This process also prevents greater resistance to toxic substance in sewage as biological mass is exposed to main stream of sewage

→ The air requirement of this plant is same as that for conventional process. and is divided equally between contact aeration tank & stabilisation tank.

→ It is used for small/medium sized plant having capacity of 40MLD.

V Complete Mix Process.

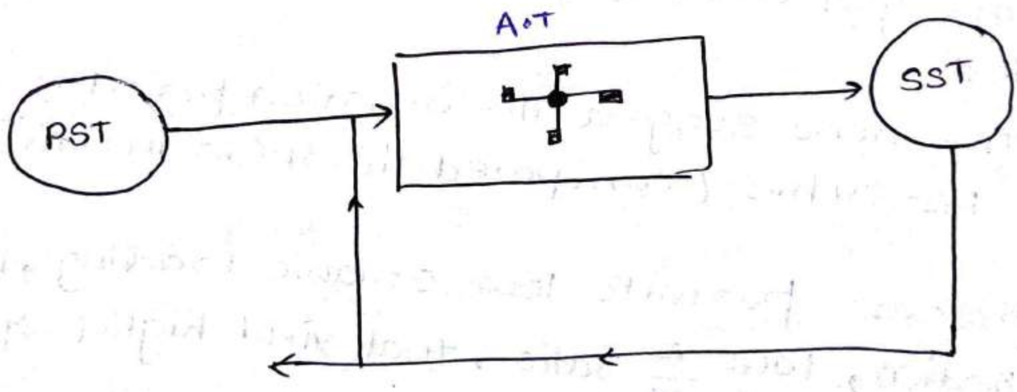
→ It is provided for smaller plants having capacity less than 20 MLD, particullary for ~~time~~ town where munincipal & industrial waste water flow together.

→ complete mix signifies that complete mixing of incoming activated sludge is done with waste water, which increase the opportunity of contact between organic matter and micro-organism, thereby increases the rate of removal of organic matter.

→ Square or circular tank are provided in this case with mechanical aerators

→ $\frac{F}{M}$ & O_2 demand remains constant through out the tank due to homogenious environment created by mixing.

→ Operational stability of this process ~~is com~~ with respect to shock organic loading is comparitively more. than conventional process.



VI Modified Aeration Process.

→ When an intermediate quality of sewage containing higher BOD is permissible for disposal such as at place where effluent is to be used for sewage farming. this modified aeration plant may be adopted as it leads to to substantial saving in construction and aeration cost.

→ Such a process does not need any PST as it is required in conventional plant.

→ The process insures short aeration period, high volumetric loading, high $\frac{F}{M}$ ration and low percentage of sludge return, low concentration of MLSS.

→ This process is suitable for large plant of capacity greater than 200MLD.

VII Extended Aeration Process.

→ The flow pattern in this process is essentially complete mix.

→ PST is frequently avoided in this process, but grit removal, grit chamber and commonitors are often provided.

→ As the name suggest the aeration period is comparatively more. 12-24 hrs. (compared to 4-6hr in conventional plant)

→ This process permits low organic loading, high MLSS concentration, low $\frac{F}{M}$ ratio, that yield higher efficiency of 95-98%.

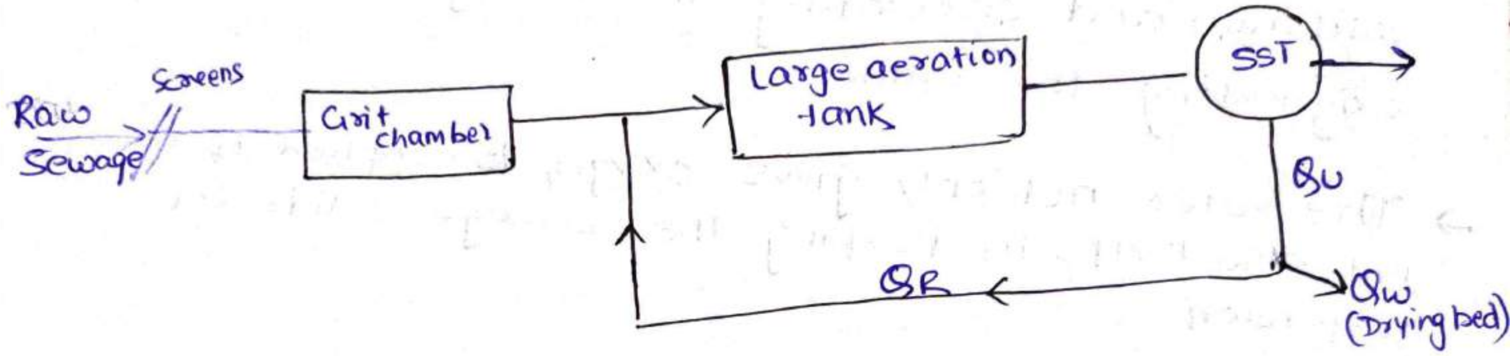
→ The air or oxygen requirement is quite high, that increases the running cost of the plant.

→ This also offers of no seprate requirement of sludge digester, because due to higher efficiency endogenous respiration takes place in it.

which increases the concentration of dead cell mass in the sludge produced.

→ This process is suitable for small communities having flow less than 4 MLD.

→ The typical plant based upon this principle is known as "oxidation ditch."



Note:-

Oxidation Ditch.

→ It is also termed as (Pasveer type Plant) or extenled Aeration lagoon.

→ This plant operates on the principle of extended aeration only.

→ Oxidation ditch plant is economical upto the population of 1.5 lakhs only, compare to conventional ASP

→ Such a plant also proves better than simple oxidation pond in terms of area requirement (It requires 20% area of oxidation pond)

→ The construction of an oxidation ditch generally envolves no. of ditches channel placed together side by side having depth of about 1-1.5 m

→ The width is limited to 1-5 m

→ The length of the ditches is not too important but may be kept such that perimeter of wall reduced to minimum and is in range of 150-~~1000~~ 1000m.

→ These ditches can be constructed either in earthwork or stone masonry.

→ These ditches are equipped with special type of horizontal axis rotor, which serves the purpose of agitation and circulating the sewage and there by oxygenating the same.

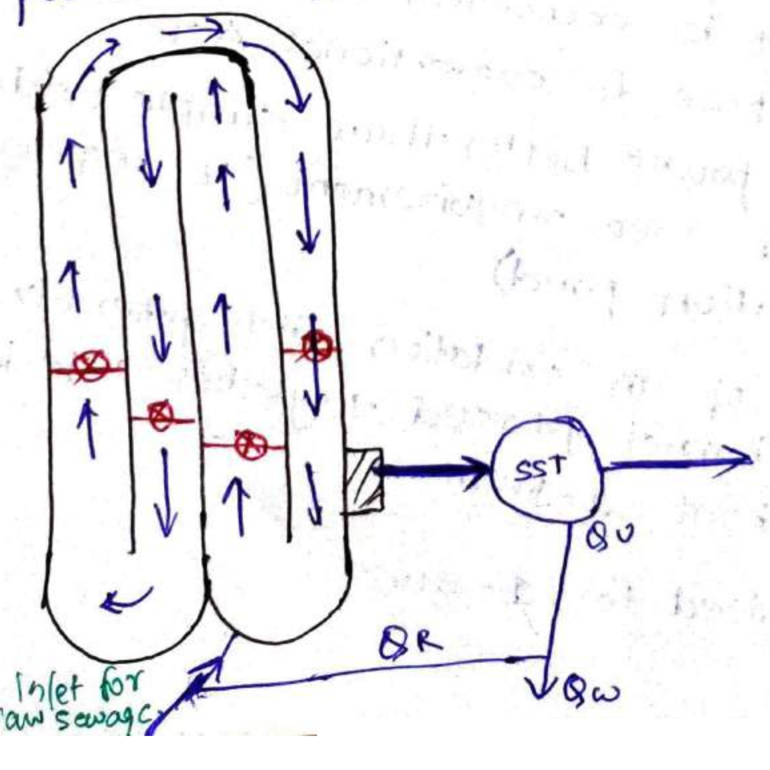
→ The rotor not only gives oxygen ~~to~~ supplied to sewage but also helps in keeping the sewage solids in suspension.

Design data:-

→ The quality of effluent obtained is comparatively very good with SS removal of 95% & BOD removal of 98%.

→ The power consumption is comparitively more than ordinary ASP. (which is compensated by elimination of PST and anerobic digestor)

→ This process requires skilled supervision.



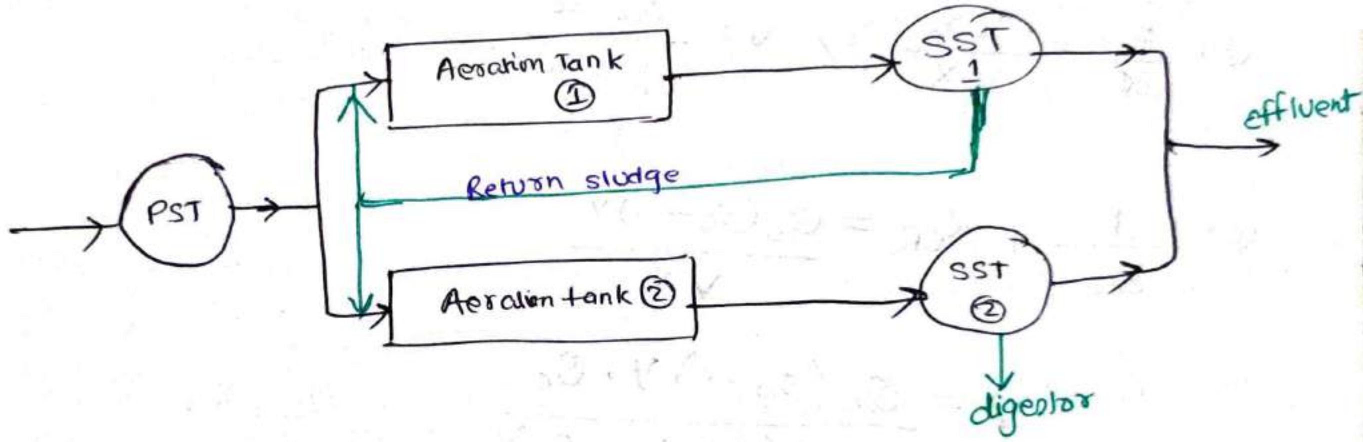
Activared Aeration Process.

→ In the activaled aeration process, a pair of conventional activaled sludge plant operating in parllel are used

→ Excess activated sludge from SST of one unit will supply to the aeration tank of both unit.

→ whereas sludge from second unit is pumped to final disposal

→ This arrangement although ~~helps~~ has no modification over conventional process in strict sense but reduces the construction cost and operating cost, but does not modify the operating result.



Design data:- for ASP

Process type	Flow Pattern	MLSS (mg/l)	F/M (kg/kg/d)	HRT (hrs)	θc (days)
(i) Conventional Process (η↑)	PLUG FLOW	1500-3000	0.4-0.3	4-6	5-8
(ii) Complete Mix (rate↑)	Complete mix	3000-4000	0.5-0.3	4-5	5-8
(iii) Extended Aeration (η↑, rate↑)	Complete mix	3000-4000	0.1-0.18	12-24	10-25

Types of process	Q_R/Q_0	η_{BOD}	MLVSS / MLSS	Volumetric loading kg BOD/m^3
① Convention	0.25-0.5	85-92	0.8	0.3-0.7
② Complete Mix	0.25-0.28	85-92	0.8	0.8-0.2
③ Extended aeration	0.5-1	95-98	0.5-0.8	0.2-0.4.

(i) Volume of aeration tank = $[V = L \times B \times H]$

ⓐ $HRT = \dfrac{V}{Q_0} \Rightarrow V = Q_0 . HRT$ $\{HRT\}$

(b) $\dfrac{F}{M} = \dfrac{Q_0 S_0}{VX} \Rightarrow V = \dfrac{Q_0 . S_0}{\frac{F}{M} . MLSS}$ $\left\{\frac{F}{M}, X\right\}$

(c) $\dfrac{1}{\theta_c} + K_{ER} = \dfrac{Q_0(S_0 - S)Y}{VX}$

$$V = \frac{Q_0(S_0-S)Y \cdot \theta_c}{X(1+K_{ER}\theta_c)}$$ $[\theta_c, X, \eta(\%)]$

→ For conventional plant depth should be range of

$H = 3-4.5$ (It control aeration efficiency)

$B = 5-10\,m$ (It contol mixing efficiency)

$\dfrac{B}{H} = 1.2 - 2.2$

$L = 30-100\,m$

Free board $= 0.3 - 0.5\,m$

→ Oxygen requirement of the aeration tank

$$\boxed{O_2 \text{ required for } BOD_I = \frac{Q_0(S_0 - S)}{0.68} - 1.42\, Q_w \cdot X_U} \quad (i)$$

If oxygen is also to be supplied to satisfy second stage BOD_{II}, additional oxygen O_2 required is.

$$\boxed{O_2 \text{ required for } BOD_{II} = 4.56 \text{ kg of } O_2/\text{kg } NH_3 \rightarrow NO_3^-} \quad (ii)$$

Total O_2 required = (i) + (ii)

Type of Process	Conventional Process	Complete Mix	Extended Aeration.
kg of O_2 required per kg BOD removed	0.8 - 1.0	0.8 - 1	1 - 1.2

→ This oxygen is supplied in the aeration tank with the help of unit termed as aerator.

→ The recommended DO concentration in the aeration tank is 0.1 - 1 mg/l for conventional plant & is 1-2 mg/l for extended aeration plant and when nitrification is also required it is kept above 2 mg/l.

→ Aerators are rated on the basis of amount of oxygen that they can transfer to the tap water under standard condition @ 20°c, 760 mm of Hg barometric pressure, and zero DO in it, per unit of energy consumed.

→ The oxygen transfer capacity (N) under field condition can be calculated from the standard oxygen transfer capacity (N_s)

$$N = \frac{N_s (D_s - D)(1.024)^{T-20} \cdot \alpha}{9.17}$$

N = oxygen transfered under field condition (kg O_2/kwh)

N_s = oxygen transfered capacity under standard conditions. (kg O_2/kwh)

D_s = Dissolved oxygen saturation value for sewage @ operating temperature.

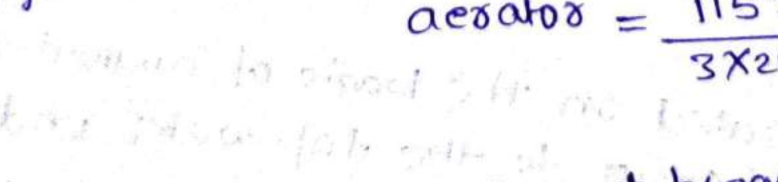
D = operation DO level in aeration tank.

T = Temp (°c)

α = correction factor (0.8 - 0.85)

Q BOD loading rate in aeration tank 1152 kg/d. This is to be satisfied by 3 aerator installed in aeration tank. Compute the power requirement of each of these aerator if there oxygen transfered capacity is 0.8 kg/HP/Hr.

BOD = 1152 kg/d



Oxygen demand to be full filled by each aerator $= \frac{1152}{3 \times 24} = 16$ kg/hr

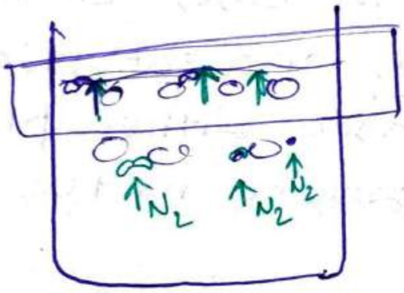
Power requirement by each aerator $= \frac{16}{0.8} = 20$ HP

Nitrification in ASP

→ ASP is designed for satisfaction of I^{st} stage BOD also termed as [carbonaceous BOD], but if nitrification takes place in it oxygen available for first stage BOD decrease which reduces the efficiency of BOD_I.

→ Nitrification in aeration tank leads to subsequent denitrification in ASP, that further reduces the effeciency of ASP.

→ **Note:-** The process in which nitrogen gas is released in SST during de-nitrification and reduces the settelability of sludge solid and carries them to surface of tank is termed as "Blanket Rising".



$$NH_3 \xrightarrow[\text{Aeration tank}]{\text{Nitrification.}} NO_3^- \xrightarrow[\text{SST}]{\text{De-nitrification}} N_2\uparrow$$

$$O_2\uparrow$$

$$N_2\uparrow \longrightarrow \eta_{SST}\downarrow \rightarrow X_u\downarrow \longrightarrow X\downarrow \rightarrow \frac{F}{M}\uparrow \rightarrow \eta^{ASP}\downarrow$$

→ Nitrification is further aided by long aeration, high sludge age, & low $\frac{F}{M}$ ratio.

$$Q_c \uparrow \rightarrow Q_w \downarrow \longrightarrow Q_R \uparrow \longrightarrow X \uparrow \rightarrow \frac{F}{M} \downarrow$$

→ Nitrification can be avoided by reducing the sludge age, that is achieved by increasing the wasted sludge discharge

$$\boxed{\downarrow \theta_c = \frac{V X \downarrow}{\uparrow Q_w X_u}}$$

→ In some cases nitrification followed by denitrification is required to be carried out in cases when effluent is to be disposed in the lakes and eutrophication is to be avoided. In such cases either plug flow process or complete mix process in two stages is adopted.

Sludge Bulking.

→ Sludge having poor settling characteristic is termed as "bulking sludge".

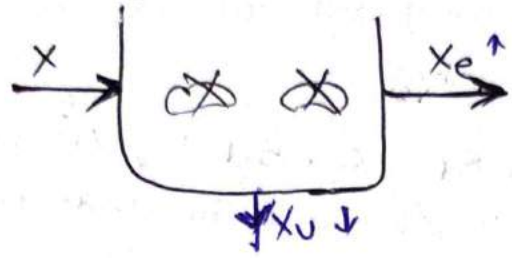
→ It is formed due to the growth of filamentous micro-organism in the system.

→ These MO are found due to low nutrient content high sludg age, & low $\frac{F}{M}$ ratio. ($< 0.3 d^{-}$)

→ Sludge Bulking result in excessive loss of MLSS out from the system thereby reduces its effeciency.

→ In order to avoid sludge bulking either SST is Chlorinated or nutrient content is increased in the system such that

$$BOD_5 : N : P = 100 : 5 : 1.$$



Following are the Properties of ASP.

(i) Advantages.

→ lesser land area required.

→ The head loss in the plant is comparatively less.

→ There is no fly and odour nuisance.

→ capital cost is less.

→ It offers high flexibility in operation

Disadvantages

→ High operational cost.

→ A lot of machinery is too be handled.

→ The sudden change in the quantity and quality of sewage may produce adverse effect on the working process thereby reducing efficiency.

→ Bulking of sludge and blanket rising.

→ The quantity of return sludge has to be adjusted everytime as and when there is quantity of sewage flow.

Q An activated sludge system is to be used for secondary treatment of 10000 m³/day of municipal waste water. After primary clarification the BOD is 150 mg/l and it is desired to have not more than 5 mg/l of soluble BOD in the effluent. A completely mixed reactor is to be used and pilot-plant analysis has established the following kinetic values.

$Y = 0.5 kg/kg$, $K_d = 0.05 d^{-1}$, Assume MLSS concentration = 3000 mg/l and an underflow concentration of 10000 mg/l from SST. Determine.

(i) Volume of Reactor.

(ii) Mass and volume of Excess sludge wasted.

(iii) Recycle Ratio

(iv) Sludge recirculation pump capacity.

(v) O_2 required to be supplied.

Assume sludge age = 10 days.

(i) $$\text{Volume} = \frac{Q_0(S_0 - S)\, Y\, \theta_c}{(1 + K_{ER}\, \theta_c)\, X}$$

$$= \frac{10000\,(150 - 5)\, 0.5 \times 10}{3000(1 + 0.05 \times 10)}$$

$$= 1611.11 m^3$$

(ii) $$\theta_c = \frac{VX}{(Q_0 - Q_w)X_e^{\,0} + Q_w X_u} = \frac{VX}{Q_w X_u}$$

$$Q_w X_u = \frac{1611.11 \times 3000 \times 10^{-6} \times 10^3}{10}$$

$$= 483.3 \text{ kg/day}.$$

volume of excess sludge wasted, $Q_w = \frac{Q_w X_u}{X_u}$

$$Q_w = \frac{483.3 \times 10^{-3}}{10^4 \times 10^{-6}} = 48.33 m^3$$

(iii) Recycle ratio, $R = \frac{Q_R}{Q_0} = \frac{X}{X_u - X} = \frac{3000}{10^4 - 3000} = 0.428$

(iv) sludge recirculation pump capacity Q_R $= R\,Q_0 = 0.428 \times 10^4 = 4280 m^3/day$

(v) O_2 required $= \frac{Q_0(S_0 - S)}{0.68} - 1.42\, Q_w X_u$

$$= \frac{10000(150-5) \times 10^3 \times 10^{-6}}{0.68} - 1.42 \times 483.3$$

$= 1446.02 kg/day$

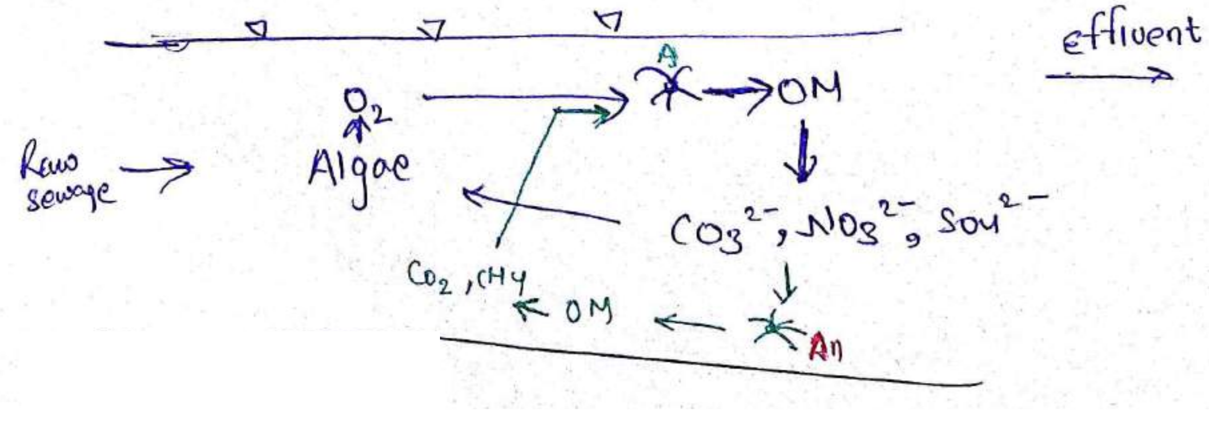
(iv) Oxidation / Stabilisation / Symbiotic Pond
(Aerobic suspended growth system)

→ Oxidation pond is an open flow through an earthen channel which provide comparatively long detention time, during which MO carry out decomposition of organic matter in suspension.

→ Special type of relationship exist in this pond in between algae and aerobic micro-organism.
ie, algae during photosynthesis produces oxygen which is utilize by m/o for the decomposition of organic matter leading to the formation of bio-mass which is further uses nutrient by ~~aerobic micro-organism~~ algae

→ This type of relationship also exist between aerobic and anerobic micro-organism, anerobic micro-organism during decomposition of organic matter produces gases CO_2, methane which rises to surface to be used as food by aerobic MO & biomass formed by decomposition of organic matter by aerobic MO settle to be used as nutrient by anerobic MO.

→ Such mutual beneficial relationship is termed as "symbiotic" relationship.



(V) → Stabilisation pond may be classified as aerobic, facultative or anerobic depending upon the mechanism of purification.

(i) Aerobic Pond:

Here decomposition is carried mainly in the presence of oxygen, for which depth is to be kept below 0.5 m & occasional stirring is required to prevent anerobic conditions of settled sludge.

(ii) Anerobic Pond

Here stabilisation of waste is mainly brough about by anerobic condition leading to formation of enstable end product (acids, CO_2, CH_4)

→ there depth varies in the range of 2.5–4m

(iii) Facultative Pond

→ In upper layers, here work is under aerobic condition while anerobic condition prevails in bottom layers.

→ Here upper layer act as a check for evolution of foul odours from bottom.

→ Depth of these ponds is in range of 1 - 1.5m

→ The term oxidation pond is referred to that stabilisation pond which receives partially treated sewage, where as the pond ~~~~ that received raw sewage is termed as "sewage lagoon".

But presently both terms are used collectively for all ~~types~~ the pond.

→ Detention time = 2-6 weeks

→ Depth = 1 - 1.5m

→ Free board = 1m may also be provided above the capacity corresponding to 20-30 day of detention period.

→ Area of each pond ~~~~ = 0.5 – 1 hectare.

→ $\frac{L}{B} = 2$

→ Organic loading rate of pond depends upon the temperature of location of pond. that further depends upon latitude of that location.

Latitude (°N)	OLR (kg/hac/d)
8	325
12	300
16	275
20	250
24	225
28	200
32	175
36	150

→ Efficiency of this pond is approximately 90% for removal of BOD and 99% for removal of micro-organism.

→ These pond are suitable to be provided for small commonity or villages or having ~~less~~ no or less power supply.

→ Removal of sludge from the pond is done approximately once in 6year for 1.2m deep ponds. where rate of accumulation of sludge is 2-5cm/~~year~~ depth/year.

→ 6

→ Oxidation pond are quite suitable for communities which are hot and dry. having 200 or more sunny days.

→ The maintenance cost is also compaitively low. No skilled man power is required, they are quite flexible and do not get upset by ~~the~~ BOD fluctuation.

→ Their major draw back is nusience due to mosquito breeding and bad odours.

Note: (1) Maturation Pond

Where land is available, facultative stabilization ponds may be followed by one or more maturation pond.

→ These pond are wholly aerobic & remove further solid & BOD & micro-organism and makes the effluent safe & easy to utilize on land or to be disposed in water.

→ These pond provide excellent ground for breeding of edible fish

→ These ponds are designed for a retention period of 5 days and are constructed with earthen embankment to contain liquid depth of 1m.

(ii) Polishing Pond.

→ It is the tertiary and final effluent waste water treatment stage before the waste water can eventually be discharged into natural water bodies.

→ It involves removal of remaining SS and BOD that may be left after secondary treatment.

Q. Design an oxidation pond for treating sewage from a colony of 5000 person in Kalikomaram contributing sewage @ 120 l/c/d. BOD of this waste water is 300 mg/l.

Kerala, latitude - 8°

Design discharge = $120 \times 5000 \times 10^{-3} = 600\ m^3/d$

Assuming organic loading Rate = 300 kg/hectare/day.

$$SA = \frac{Q_o . S_o}{OLR}$$

$$= \frac{600 \times 300 \times 10^{-6} \times 10^3}{300} \quad \left[\frac{m^3}{d} \times \frac{mg}{l} \times \frac{10^{-6} \times 10^3\ \text{hec-day}}{\text{kg}}\right]$$

$= 0.6$ hectare (0.5 - 1 hectare)

Assume, $\frac{L}{B} = 2$

$SA = L \times B$

$0.6 \times 10^4 = 2B \times B$

$B = \sqrt{\frac{0.6 \times 10^4}{2}}$

$B = 54.77\ m \sim 55\ m$

$L = 2 \times 54.77 = 109.54\ m \sim 110\ m.$

Assume, $H = 1.5m$

size of tank $= 110m \times 55 \times 1.5 m.$

$$\text{detention time} = \frac{\text{Volume}}{Q.}$$

$$= \frac{110 \times 55 \times 1.5}{600} = 15.125 \text{ days.}$$

$= 2.16$ weeks

$(2-6)$ weeks.

Note: Design of inlet pipe.

Assume avg. velocity of sewage $(0.8 - 1m/s) = 0.9 m/sec$

operating time be 10 hr.

$$Q = AV$$

$$A = \frac{Q}{V} = \frac{600}{10 \times 60 \times 60 \times 0.9} = 0.01851 m^2$$

$$\frac{\pi D^2}{4} = 0.0185 \times 10^4 cm^2$$

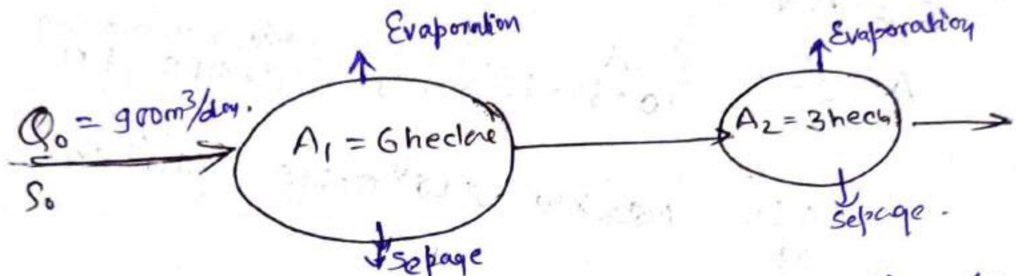
$$D = 15.35 cm$$

Q Stabilisation pond of 3000 population are provided to operate in series the larger cell has an area of $60,000 \, m^2$ & smaller one has area of $30,000 \, m^2$. The avg. daily waste flow is $900 \, m^3/d$ containing 200 kg of BOD.

$\frac{m^3}{ha} \cdot \frac{mg}{l}$

(i) For series operation calculate BOD loading based on both total & larger cell area only.

(ii) Estimate the no. of days of winter storage available between 0.6 m and 1.5 m water levels assuming seepage loss of 2.5 mm of an evaporation and seepage loss of water per day.



$$OLR_T = \frac{Q_0 S_0}{A_1 + A_2} = \frac{200}{9} = 22.22 \text{ kg/hac/day}$$

$$OLR_L = \frac{Q_0 S_0}{A_1} = \frac{200}{6} = 33.33 \text{ kg/hac/day}$$

Net rate of flow of sewage $= 900 - 2.55 \times 10^{-3} \times 9 \times 10^4$
$= 675 \, m^3/d$

Volume available for winter storage $= (1.5 - 0.6) \times 9 \times 10^4$
$= 8.1 \times 10^4 \, m^3$

No. of days of winter storage

$$= \frac{8.1 \times 10^4}{675} \approx 12 \text{ days}$$
$= 4$ month

V. Mechanically Aerated Lagoon

→ It is deeper oxidation pond with oxygen supplied by mechanical aerators rather than relying on photosynthesis oxygen or atmospheric O_2

→ As these pond are deeper than oxidation pond and as they are artificially aerated, less detention time and area is required.

→ Depth of these tanks is in range of 2.4–3.5 m.

→ Detention time is = 4–10 hr.

→ Land are required = 5 to 10% of that required in conventional oxidation pond.

→ $\eta_{BOD} = 65\text{-}90\%$

→ It is used for treating industrial waste water as well as city sewage.

VI Septic tank (Anerobic suspended growth system).

→ A septic tank may be defined as primary sedimentation tank with longer detention period of 24 to 48 hr. in compairision to 2 to 2.5 hr of PST

→ raw sewage is directly fed into these tank and is allow to settle leading to formation of sludge at the bottom of the tank which is also digested in it over a period of time anerobically.

→ Digestion of sludge leads to development of foul gases hence it is complelely covered with vents provided for escape of gases.

→ A septic tank is thus a horizontal continuous flow type of sedimentation tank, removing 60-70% of dissolve organic matter. from it.

→ Gases produced during digestion carries oil & greece to the surface leading to the formation of scum layer at the top, which act as an insulator, there by avoid the release of odourus gases & heat out from the system.

→ Septic tanks are generally provided in areas where sewer have not been laid and for catering to the sanitary disposal of sewage produced from isolated communities.
eg. school, hospital, hotels etc.

→ These tanks are designed to prevent direct currents between the inlet and outlet of tank.
In order to avoid the carrying ~~of~~ away of settled solid out from the tank along with the ~~tank~~ effluent.

→ In order to achieve this T-shaped pipes are provided at the inlet and outlet ~~to~~ of the tank and hanging baffle wall is provided at the outlet of the tank. in front of the inlet.

→ $t_d = 24\text{-}48\ hr$

→ Cleaning period = 6 to 12 month. ($\not>$ 3 years)

→ Rate of flow of sewage = 90-150 l/c/d.

→ Rate of accumulation of sewage = 30-70 l/c/yr.

→ ~~Minimum capacity of septic tank~~

→ It is generally designed for = 8/10 - 300 people

→ $\frac{L}{B} = 2\text{-}3$

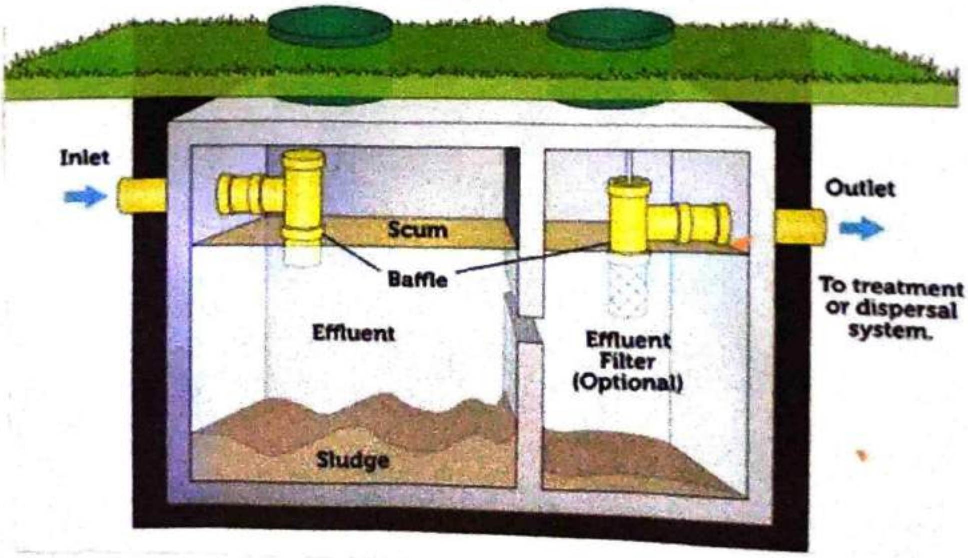
⇒ width, $B \not< 0.75\ m$.

→ H, 1.2 - 1.8 m.

→ $F_B = 0.3 - 0.5\ m$

⇒ Volume of tank = $V_1 + V_2$

$V_1 = Q_0\, t_d$; $V_2 =$ RAS × CP.



Q Design a septic tank for a small colony of 150 people having water demand of 120 l/c/d

Design discharge, $Q_0 = 0.8 \times 150 \times 120 \times 10^{-3}$

$= 14.4 \, m^3/day$

Assume $t_d = 24 \, hr.$

cleaning period = 1 yr.

rate of accumulation of sludge = 30 l/c/year.

$V = V_1 + V_2$

$V_1 = Q \times t_d$

$= 14.4 \times \times \frac{24}{24} = 14.4 m^3.$

$V_2 = RAS \times CP$

$= 30 \times 150 \times 10^{-3} = 4.5 m^3$

$V = 14.4 + 4.5 = 18.9 \, m^3$

Assume $H = 1.5m$

$SA = \frac{V}{H} = \frac{18.9}{1.5} = 12.6 m^2.$

Assume, $\frac{L}{B} = 2$

$L = 2B.$

$SA = L \times B$

$12.6 = 2B^2$

$B = 2.5 m \not> 0.7m.$

$L = 5 m.$

$F_B = 0.3m.$

→ Disposal of efflvent from the seplic tank.

→ The effluent coming out from a septic tank is no betler than the effluent of an ordinary sedimentation tank.

→ It contains large amount of organic matles (BOD = 100-200mg/l)

→ This effluent should therefore be disposed carefully, so as to minimise the nuisance created by it.

→ Disposal of effluent in this case can be done by any of the following method:-

(i) Biological filler.

(ii) Up-flow - Anaerobic filter.

(iii) Soil - absorption system.

Note: In case sufficient porous and permeable ground is not available the effluent is ~~disposed~~ treated in Biological filters or in upflow prior to its disposal Anerobic filters.

(iii) Soil absorption system.

→ It involves the disposal of effluent on land and can be adopted only when sufficient land is available. & soil is porous and permeable to allow easy percolation of effluent in it.

Note: Percolation rate is defined as time required in "minute" by sewage effluent to seep into the ground by "1cm".

→ This system is of two types:-

(i) soakpit/seepage pits.

(ii) Dispersion Trench

(i) Soak pit

→ A soak pit is circular cover pit through which effluent is allowed to be soaked or absorbed into the surrounding soil.

→ It may be either filled with stones or may be empty.

→ It is used when percolation rate is less than 30 minutes

(ii) Dispersion trench

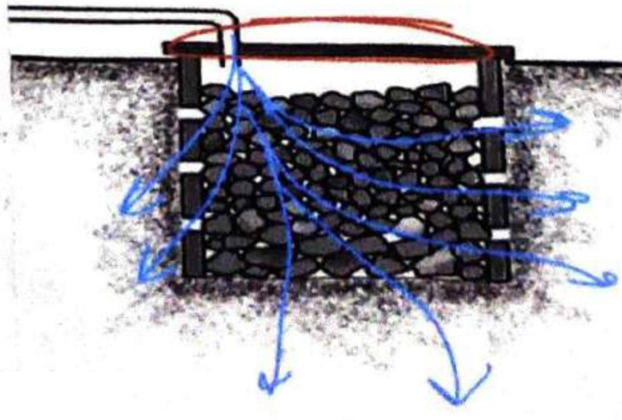
→ In this effluent is allowed to enter into a masonry chamber called distribution box from where it is uniformly distributed through an underground network of open-jointed pipes into absorption trenches. Called Dispersion trench.

→ It is used when percolation rate is in range of 30-60 minutes.

→ Maximum rate of disposal of effluent is given by

$$Q^* = \frac{204}{\sqrt{t}} \quad (l/m^2/d)$$

t = percolation rate (min).



Q Design the absorption field system for disposal of septic tank effluent for a population of 100 person with sewage flow rate of 135 l/c/d. The percolation rate for percolation test is carried out at the site of the absorption field may be taken as 3 min.

Q_0 Design discharge = $135 \times 100 = 1.35 \times 10^4$ l/c/d.

Max^m rate of disposal,

$$Q^* = \frac{204}{\sqrt{t}}$$

$$= \frac{204}{\sqrt{3}} = 117.779 \ l/m^2/d$$

$$\text{Area of Soak pit} = \frac{Q_0}{Q^*} = \frac{1.35 \times 10^4}{117.779}$$

$$\frac{\pi}{4} D^2 = 114.62 m^2$$

$$D = 12m.$$

Properties of Septic tank:-

→ Septic tank can be easily constructed and do not required any skilled supervision.

→ The cost is reasonable compare to the advantages and sanitation they provide.

→ An excellently functioning septic tank can considerably reduced the suspended solid & BOD from the sewage.

→ The sludge volume to be disposed off is quite less as compared to that in normal sedimentation tank.

→ The quantity is reduced due to digestion taking place in the tank itself.

→ The reduction in volume is about 60% & reduction is weight is about 30%.

→ The effluent from the septic tank can be disposed on land without much trouble.

→ These are best suited for isolated rular areas. and isolated insitutes.

drawback:-
→ It requires proper maintenance.

→ They require too large size for serving many people.

→ Its working is unpredictable. and non-uniform.

VII Inhoff tank

→ Inoff tank is also anaerobic suspended growth system.

→ It is a modification over the septic tank in which incoming sewage is not allowed to get mixed up with the sludge produced in the tank. Hence in these case effluent is not able to carry large undigested solid out from the tank, there by offers high efficiency.

→ It is a two storeyed tank in which sedimentation is carried out in upper sedimentation chamber and digestion is carried out in lower digestion chamber.

→ For sedimentation chamber, (i) detention time = 2-4 hr.

(ii) V_f = 0.3m/min

(iii) SLR = $30 m^3/m^2/d$.

(iv) L $\ngtr$ 30m

(v) $\frac{L}{B}$ = 3-5

(vi) Depth of ~~tank~~ = 3-3.5m

(vii) Freeboad = 0.45m.

→ for digestion chamber

(i) It is generally designed for a minimum capacity of 57 liler/capila.

(ii) Cleaning period = 1 – 1.5 month.

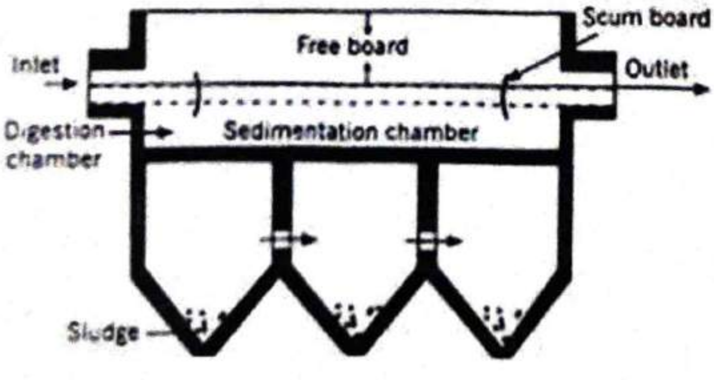
(iii) Sides slopes of hopper = 1 : 1

Nole: Slot slope (1.25V : 1H)

→ In todays time it is practically obsolete

Nole:- Claridigesler

→ These are small palenled circular imhoff tank type double storey tank. without bottom hoppers and fitted with mechanically sludge ^ ~~removed~~ & scum removal unit which helps in reducing its over all height. to 6m.



VIII High Rate Anaerobic System

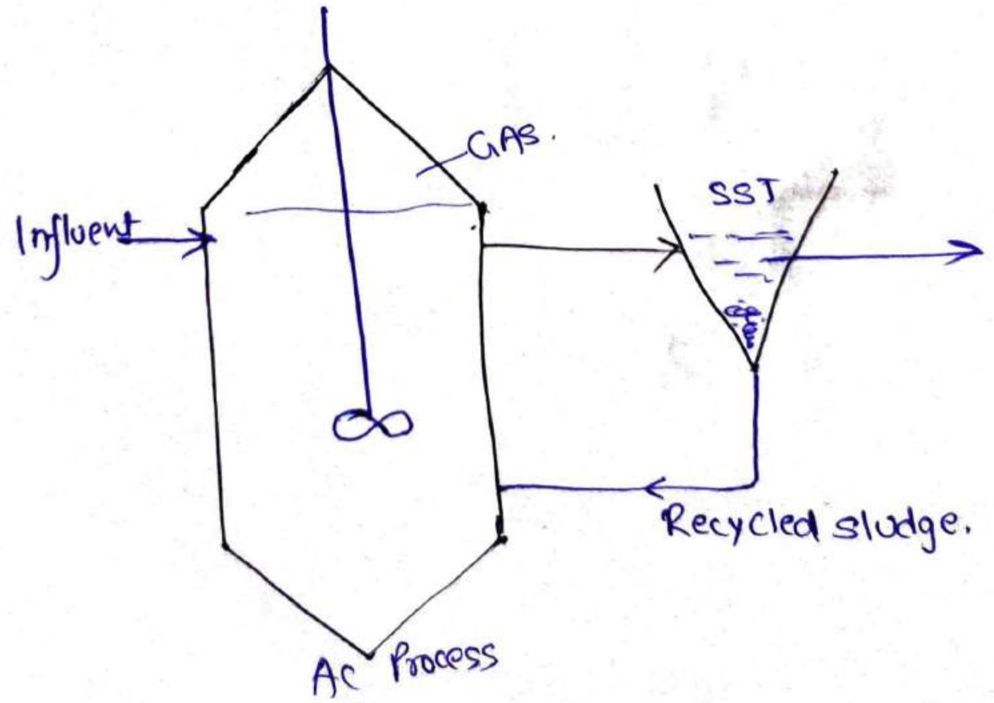
→ In anaerobic process rate of decomposition of organic matter is comparitevely less requiring higher detention time, bigger treatment unit, thereby more cost, hence in order to ~~increase the rate~~ reduce the time and area required for treatment high rate anerobic system are being designed.

→ These high rate unit provide a little bit of incomplete treatment, bringing down BOD and suspended solid by 50-70% hence efflvent may therefore need post treatment by aerobic filters, maturation pond.

→ The various high rate anerobic system:-

(i) Anaerobic contact Process.

→ The system involves a closed stirred tank reactor, in which the biomass leaving with the reactor effluent, is settled in a sedimentation tank is recycled to the stirred tank to increase solid retention time (SRT)



AC Process

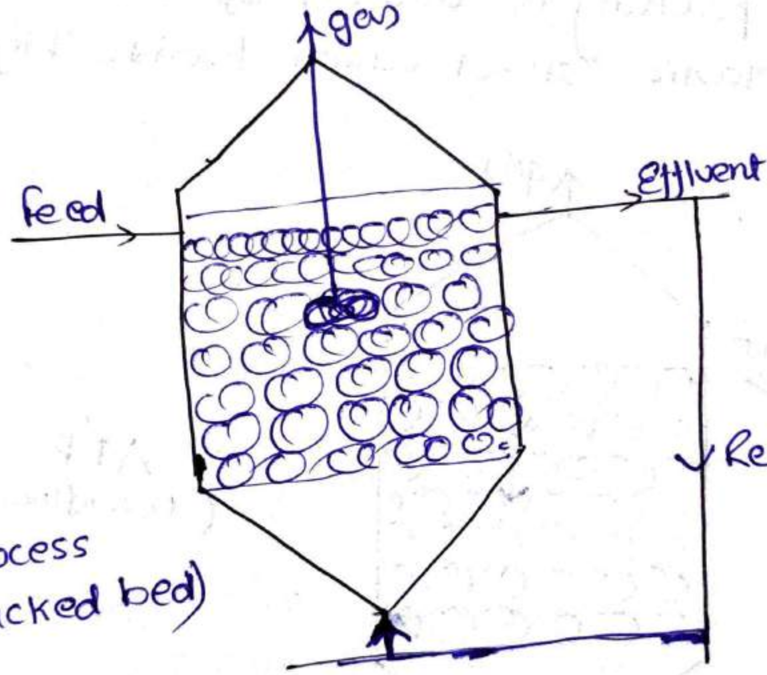
(ii) Anaerobic filter

→ In an anerobic filter a stationary filter media consist of crushed stone or/rocks of 15-25mm size, or plastic beads etc are used as packed medium in closed tank and the waste water is entered from the bottom to move up the packing mediam.

→ The sludge is entrapped in void space between the packing material.

→ The effluent is recycled to increase SRT of the reactor.

→ This reactor is operated as an upflow submerged bed reactor.



AF process
(Up flow packed bed)

(iii) Anerobic fixed film Reactor (AFF)

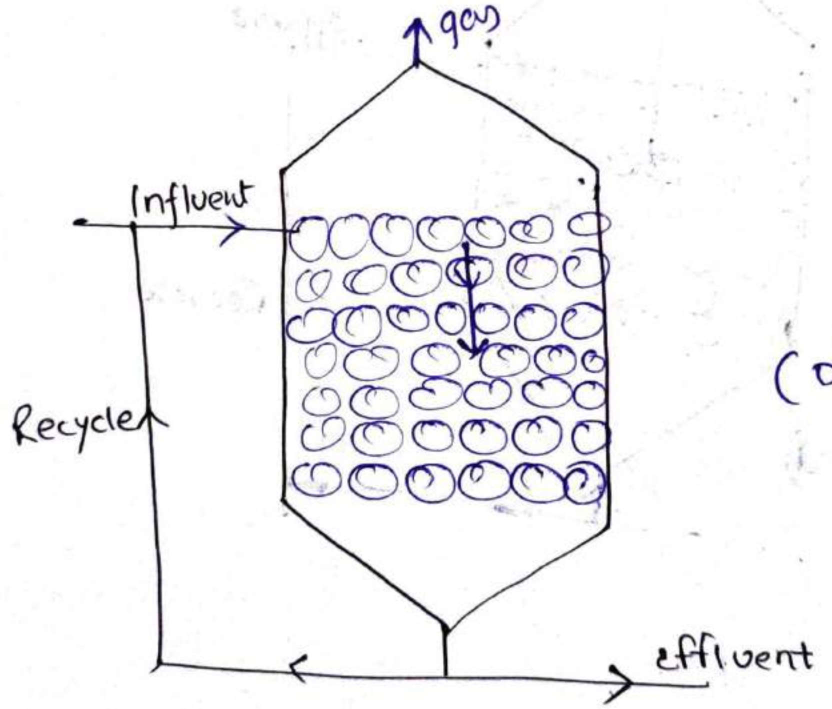
→ like an anerobic filter this process is also characterised by the presence of stationary packing maleiial in the reactor.

→ But here the aim is to ~~provide to~~ avoid entrapment of suspended solid in the void of packing maleiial

→ In order to prevent accumulation of solid in the reactor, the AFF reactor is worked with downflow mode.

→ The reactor may be operaled either in submerged or in unsubmerged condition.

→ The reactor packing is usually of modular construction, consisting of plastic sheet which provide high void ratio.



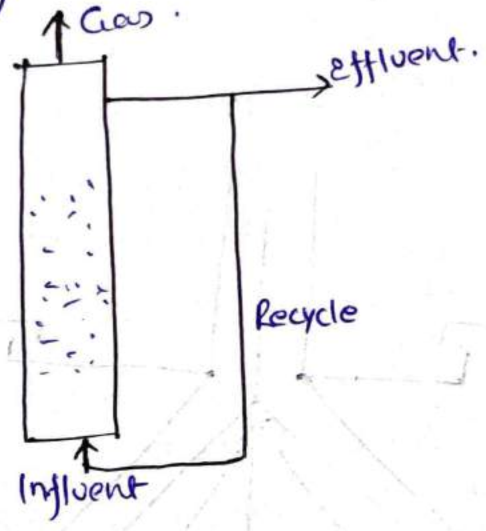
AFF
(Downflow packed bed)

(iv) Fludized and Expanded Bed Reactor

→ A fludized bed reactor as well as an expanded bed reactor are both characterised by the presence of mobile packing material such as sand, clay, coal etc.

→ The organics & microbes in the reactor get attached to the particle with which reactor tank is filled only to a part height.

→ The waste water is entered from the bottom of the tank with an upflow velocity to move (fluidize) the medium particle, which act as a mobile bio-mass carrier, causing digestion of organic matter.



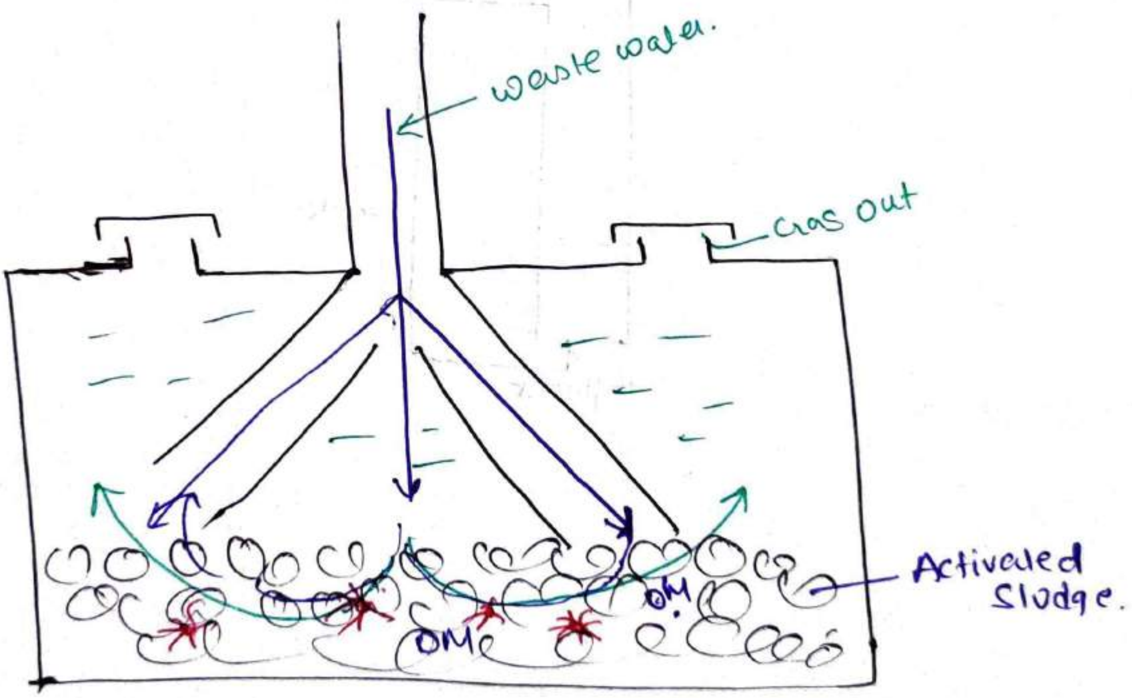
(v) UASBR (Upflow anerobic sludge blanket reactor).

→ It maintains a high concentration of bio-mass through the formation of highly settelable microbial sludge aggregates.

→ The waste water is then allow to flow upward through a layer of very activated sludge to cause anerobic decomposition of OM present in the waste water.

→ At the top of the reactor. three phase sepration of the consituents takes place as mixing is not induced in it.

→ This process is suitable for both soluble waste water as well as containing particulate matter.



IX Moving Bed Biofilm Reactor (MBBR)

→ The MBBR system consist of an aeration tank (similar to an activated sludge tank) with special plastic carrier that provide a surface where a bio-film can grow

→ The carriers are made of material with density close to that of water eg. polyethelene ($\rho = 0.95 g m/cm^3$)

→ The carrier will be mixed in the tank by the aeration system. thus will have good contact between the substrate in the effluent waste ~~waste~~ water & biomass on the carriers

→ To prevent the plastic carrier from escaping the aeration tank it is necessary to have a sieve on the outlet of the tank.

→ To achieve higher concentration of biomass in the bio-reactor, hybrid MBBR system have also been developed which takes the advantage of both attached and suspended growth system.

→ It is suspended growth system requiring less space than ASP. and is independent of the final sludge produced (ie no recirculation is required in this.

→ There is no clogging of medium in this unit.

→ MBBR is often installed as a retrofit of existing ASP to increase the capacity of existing system.

→ It has lower sludge production.

→ It is more resistant to shock loading.

Secondary Sedimentation Tank. (SST)

- The prime function of SST is to produce the effluent which is comparatively clarified and to concentrate the sludge formed in it. So as to reduce the volume of sludge to be handle in the plant.
- These tank are designed for both over flow rate (OFR) and solid loading rate (SLR). ~~Surfac~~
- Surface area of these tank is computed by ~~con~~ using both avg. value of over flow rate & SLR and max^m area computed is adopted.
- This max^m area is further used to ~~find the~~ check the peak value of both OFR and SLR

Types of SST	Over flow Rate ($m^3/m^2/day$)		Solid Loading Rate (kg/m²/d)		Depth (m)	Detention time (hr)
	Avg	Peak	Avg	Peak		
→ SST after TF	15-25	15-35	70-120	190	2.5-3.5	1.5-2
→ SST after ASP	15-35	40-50	70-140	210	3.5-4.5	1.5-2
→ SST after extended area.	8-15	25-35	25-120	170	3.5-4.5	1.5-2

(1) $SA = \dfrac{Q_0 + Q_R}{OFR_{avg}}$ or $\dfrac{Q_0}{OFR_{avg}}$

& $SA = \dfrac{(Q_0 + Q_R)X}{SLR_{avg}}$ or $\dfrac{Q_0 X}{SLR_{avg}}$ $\Big\}$ $\max^m$.

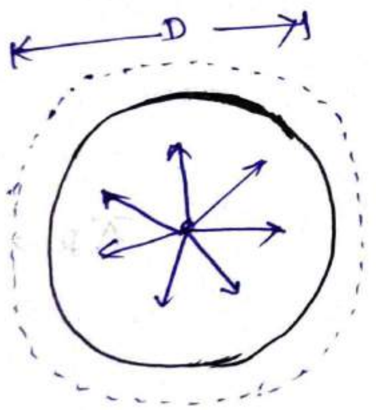
(2) $OFR_{PEAK} = \dfrac{Q_{PEAK} + Q_R}{SA_{max}}$, $SLR_{PEAK} = \dfrac{(Q_{PEAK} + Q_R)X}{SA_{max}}$

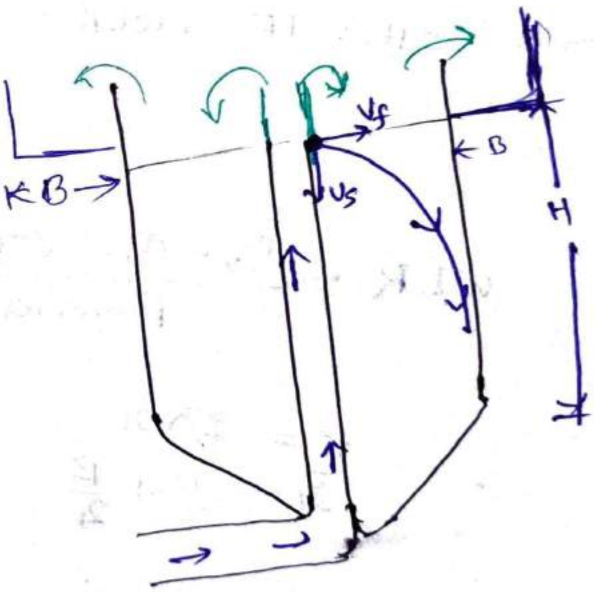
$$\frac{\pi D^2}{4} = S_{max} \Rightarrow \left(\frac{4}{\pi} \times S_{max}\right)^{1/2} = D$$

(3) $V = Q_0 \cdot t_d$

$= D^2(0.011D + 0.785H)$

$H = 2$





→ Solid entering into SST are considerable light in weight hence are significantly effected by turbulence and current of outgoing effluent

→ Hence in order to prevent the loss of solid out from the tank sufficient length of overflow weir is provided along the circumference of the tank at the top

→ There by these tank after being designed are also checked for "weir loading rate"

$$\text{Weir Loading rate} = \frac{\text{Rate of flow of effluent}}{\text{length of over flow weir.}}$$

→ For SST after ASP WLR = 185 $m^3/m/day$ &

For SST after TF, WLR = 125 $m^3/m/day$.

$$WLR = \frac{Q_0 - Q_w \text{ or } Q_0}{\text{Perimeter.}}$$

$\pi D \rightarrow \pi(D+2B)$

$\bar{P} = 2\pi \bar{r}$

$\bar{r} = \frac{D}{2} + \frac{B}{2}$

Q Design SST to treat effluent from ASP from following data.

Average sewage inflow = 20MLD

MLSS in influent = 3000 mg/l

Peak flow factor = 2.25

Assume $OFR_{avg} = 20 m^3/m^2/d$

$SLR_{avg} = 80\ kg/m^3/day.$

$$SA = \frac{Q_0}{OFR_{avg}} = \frac{20 \times 10^6 \times 10^{-3}}{20} = 1000 m^2$$

$$S.A = \frac{Q_0}{SLR_{avg}} = \frac{Q_0 \cdot X}{SLR_{avg}} = \frac{20 \times 10^6 \times 300 \times 10^{-6}}{80} = 750 m^2$$

$\left.\right\} = 1000 m^2$ max

Check $OFR_{PEAK} = \frac{Q_{PEAK}}{SA_{MAX}} = \frac{2.25 \times 20 \times 10^6 \times 10^{-3}}{1000} = 45 m^3/day$ $(40-50 m^3/m^2/d)$

$$SLR_{PEAK} = \frac{Q_{PEAK} \cdot X}{SA_{max}} = \frac{2.25 \times 20 \times 10^6 \times 3000 \times 10^{-6}}{1000} = 135 kg/m^2/day$$

$< 210 kg/m^2/d$ ✓

$SA_{max} = 1000 m^2$

$$\frac{\pi D^2}{4} = 1000$$

$D = 35.68$

$D = 36m.$

Assume, $t_d = 2 hrs.$

$$V = Q_0 \cdot t_d = 20 \times 10^6 \times 10^{-3} \times \frac{2}{24} = 1666.7\ m^3$$

$$V = D^2(0.011 D + 0.785 H)$$

$$1666.7 = 36^2(0.011 \times 36 + 0.785 H)$$

$H = 1.17m$

$\sim 1.2m.$

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Check $WLR = \dfrac{Q_0}{P} = \dfrac{20 \times 10^6 \times 10^{-3}}{\pi(36)}$

$= 176.8\ m^3/m/d < 185\ m^3/m/day.$

No. requirement of weir.