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ENVIRONMENTAL ENGINEERING VOLUME II



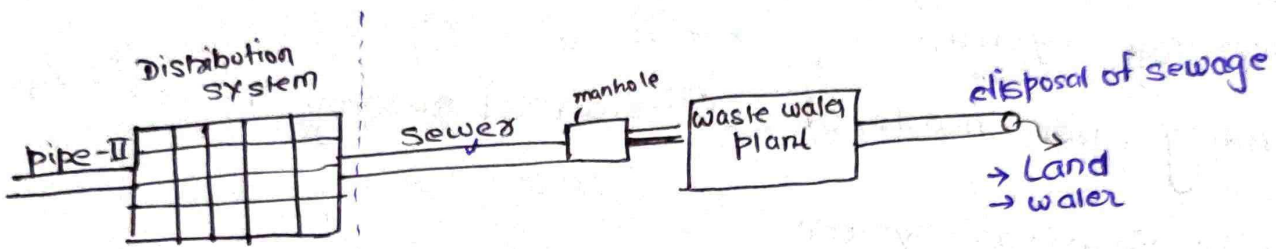
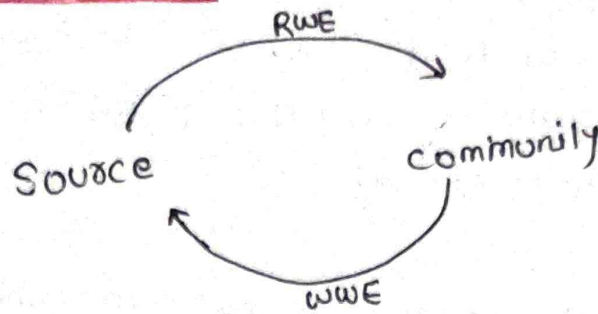
BY JASPAL SIR

TABLE OF CONTENTS

I	Quality of Waste Water	03
II	Disposal of Sewage Effluent	29
III	Design of Sewer	53
IV	Treatment of Sewage	105
V	Disposal of Solid Waste	218
V	Air Pollution	237
V	Noise Pollution	266

Waste - Water Engineering.

Quality of waste water



Sewage/waste water: It is a mixture of water and liquid waste generated due to different activities in the community.

→ Depending upon mode of generation it is further classified as:-

(i) Domestic Sewage: It is a mixture and liquid waste generated due to domestic activity.

eg.

(ii) Industrial sewage: If it is produced after industrial activity it is termed as industrial sewage.

(iii) Drainage or runoff: It is a mixture of water and liquid waste produced after heavy rainfall.

Note → Kitchen and bathroom waste termed as "Sullage".

→ Domestic sewage and Industrial sewage taken together is called "Sanitary Sewage".

→ Drainage discharge is approximately 20-30 times more than sanitary sewage discharge.

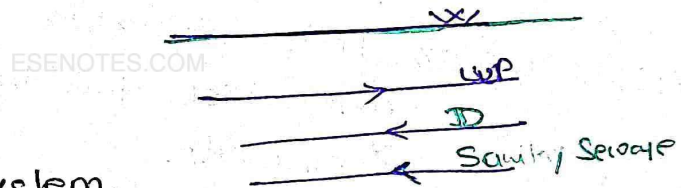
II Sewer: Pipe or conduit required to carry the sewage from one point to another point in sewerage system is termed as sewer.

III Sewerage system: The entire process in which sewage is collected, treated and disposed is termed as Sewerage system.

→ Depending upon mode of collection of sewage it is classified into:-

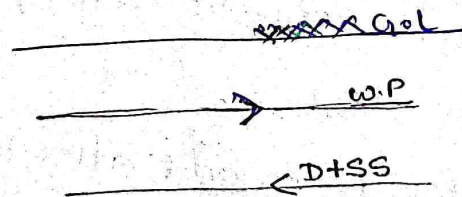
(i) Separate Sewerage System;

It is type of sewerage system in which two separate sewer are provided to carry drainage and sanitary sewage.



(ii) Combined Sewerage system.

It is type of system in which only sewer is provided to carry both drainage and sanitary sewage.



→ A separate system will require laying of two set of conduit whereas combined system requires only one set of bigger sized conduit, thus "separate system cost of conduit is more.

→ Moreover the separate conduit can not be laid in congested street and locality making it physically

- The sewer pipe in combined system are liable to frequent sitting during non-monsoon system (only sanitary sewage discharge will run) hence they are to be laid at steeper slope which makes them deeper, requiring more excavation and pumping making them costlier.
- In a combined system the less foul drainage water get mixed with highly foul sanitary sewage thus increases cost of treatment where as in seprate system only sanitary sewage is treated and drainage is disposed of without any treatment.
- In case flooding and backing up of sewer or drains occur due to excessive rain more foul and insanitary condition will prevent in case of combined system.
- Since sewer lines are generally laid deep & at steeper slope as compared to the strong water surface drain pumping of sewage & often no pumping of drainage is required in seprate system. where as the entire discharge will have to be pumped in combined system which makes them costly.
- Seprate sewerage system is found more economical than combined system.

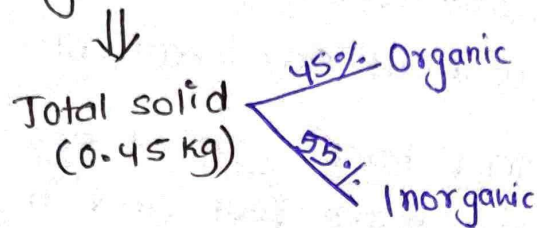
(iii) Partially Seprate Sewerage system.

- It is type of system in which particular portion of drainage is allowed to get mixed with sanitary sewage. [ie the drainage obtained from roof & paved courtyard]

→ About 80% of water being supplied into the community gets converted in sewage.

→ Sewage is more than 99.9% water & less than 0.1% solid, such that if 1000 kg of sewage sample is considered it consist of only of 0.45 kg of total solid.

1000 kg of sewage

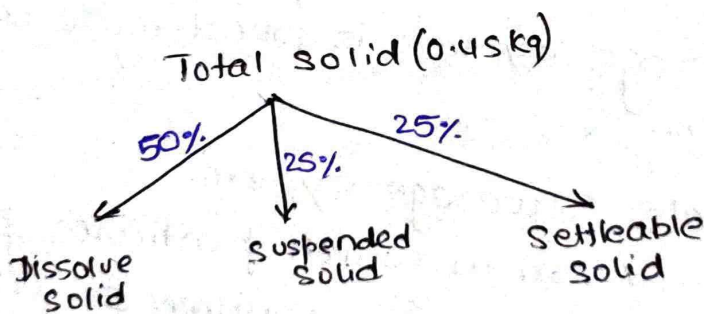


$$\text{Sewage} = \frac{0.5 (\text{organic solid})}{0.0002} = 0.21 \text{ kg}$$

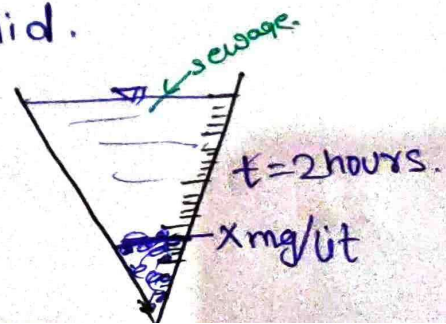
$$\text{Raw water} = 0.01 \times \frac{1}{100} = 0.0002 \text{ kg}$$

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$$\frac{0.2}{0.0002} = 1000 \text{ times}$$



Note:- If the sample of sewage is allowed to stand in "Imhoff cone" for two hours the amount of solids settled in it are termed as settleable solid.



Types of Solids

Settleable

Suspended

Colloids

Dissolved

Size (μm) $\times 10^{-6} m$

> 10

$> 10^{-5} m$

> 1

~~$< 10^{-3}$~~ (10^{-3})

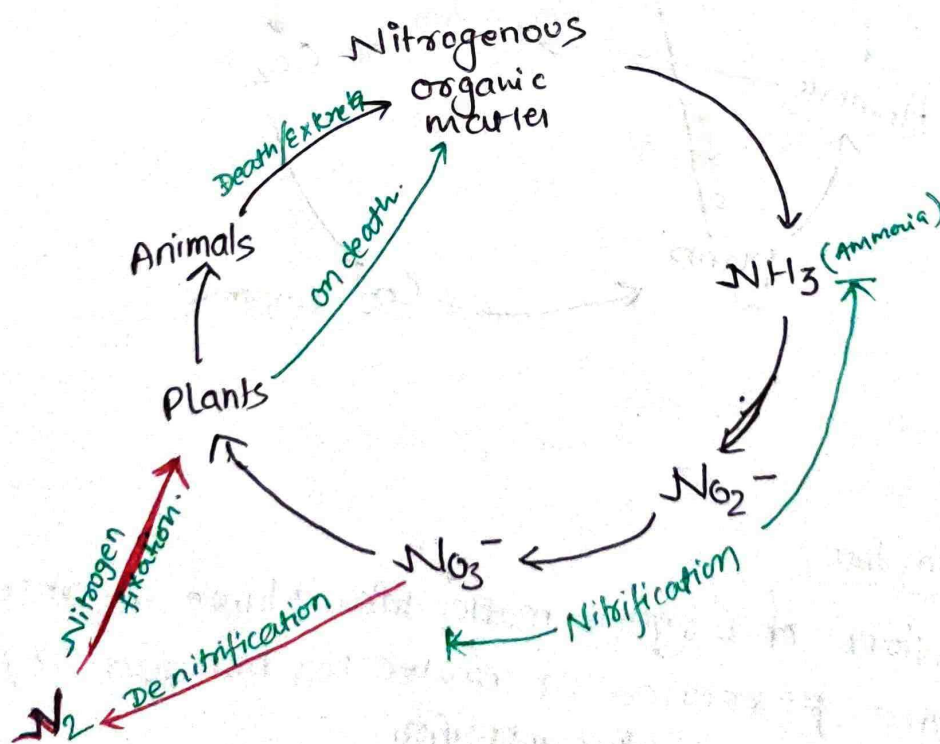
$< 10^{-9}$

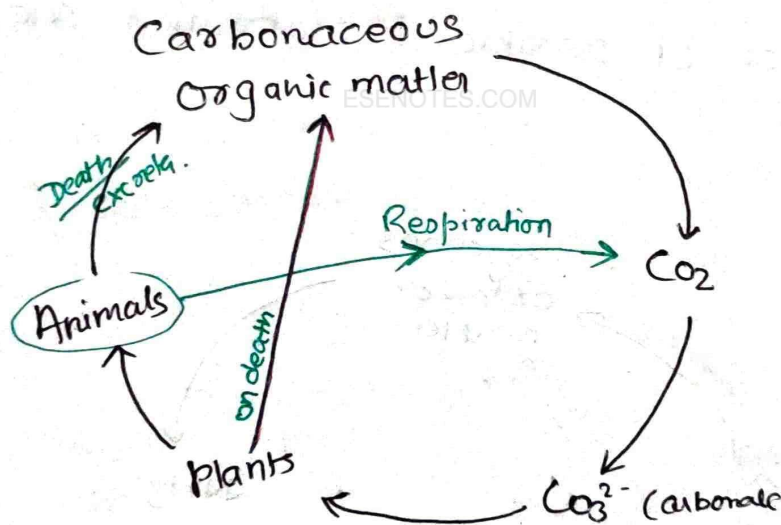
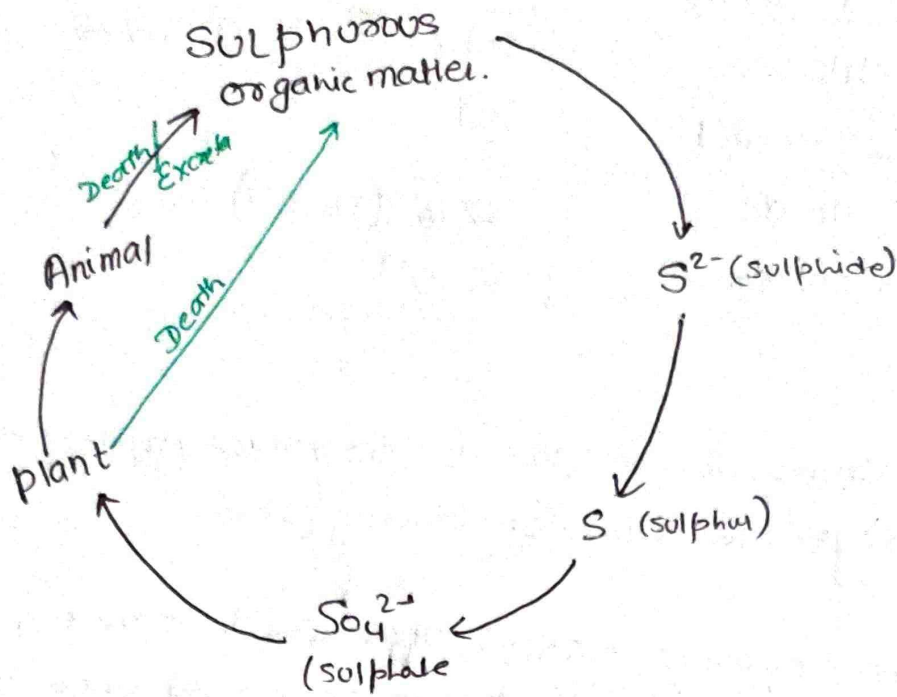
→ Prime impurities of sewage is organic matter that can be either biodegradable or non-biodegradable.

→ Biodegradable organic matter is that which can be decomposed by action of M/O either in presence or absence of oxygen.

→ If this decomposition takes place in presence of O_2 (dissolved oxygen) it is termed as aerobic decomposition.

→ End products of aerobic decomposition are CO_2 , NO_3^- , SO_4^{2-} .

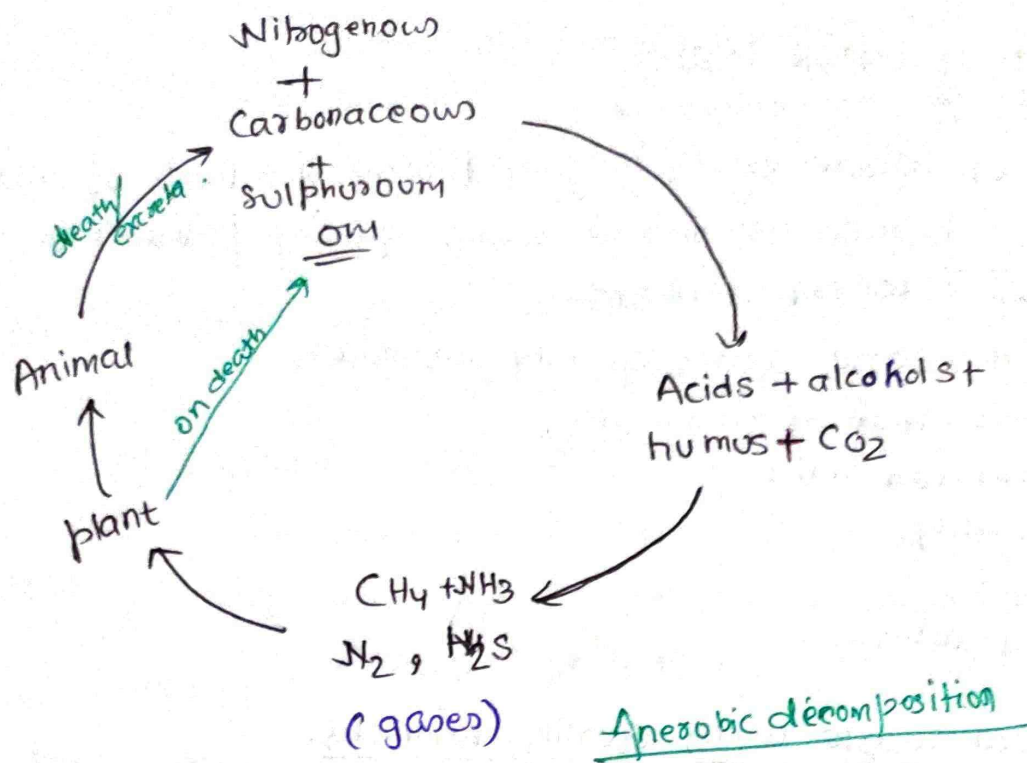




Aerobic decomposition

if decomposition of organic matter takes place in absence of oxygen or in the presence of molecular bonded oxygen it is termed as aerobic decomposition.

→ The end product of aerobic decomposition are comparatively unstable and includes, acid, alcohol and gases.



* Rate of aerobic decomposition $\approx$ 3X Rate of Anaerobic decomposition
 (It use molecular bonded oxygen.)

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Quality of Waste Water.

→ Quality of waste water is found with the help of certain indicators termed as waste water quality parameters. which are classified into:-

I Physical waste water quality parameter.

→ same as that of raw water.

(i) Suspended solid.

(ii) turbidity.

(iii) Colour

(iv) Temperature.

II Chemical waste water quality parameter.

(i) Dissolved oxygen (DO)

→ Maximum oxygen that can be dissolved in water @ any given temperature is termed as saturation dissolved oxygen. (DO_{sat})

→ As per Henry's law solubility of gases is inversely depended on temperature.

<u>Temperature (T°C)</u>	<u>DO_{sat}</u>
0°C	14.6 mg/lit
5°	12.75
10	11.27
15	10.07
20	9.12
25	8.24
30	7.54
35	6.93

*

Note

→ Minimum oxygen required for survival of fishes is 4 mg/lit

→ The solubility of oxygen in sewage is 95% of that in distilled water.

→ With increase in concentration of salt solubility of O_2 (and decrease in water. (Sea water has 20% less DO as compare to ~~river~~ ^{river} water).

* Any deficiency of oxygen in water with respect to saturation dissolved oxygen indicates presence of organic matter in system.

$$D = DO_{sat} - DO \propto OM$$

Deficiency.

→ It depends upon temp and is a standard parameter.

→ DO is found ~~while~~ using Winkler's equation.

~~DO is found while using~~

SAMPLE	OM	DO	D
Raw water	negligible	$\approx DO_{sat}$	≈ 0
Waste water	Excess	≈ 0	$\approx DO_{sat}$

(ii) Chemical Oxygen Demand (COD)

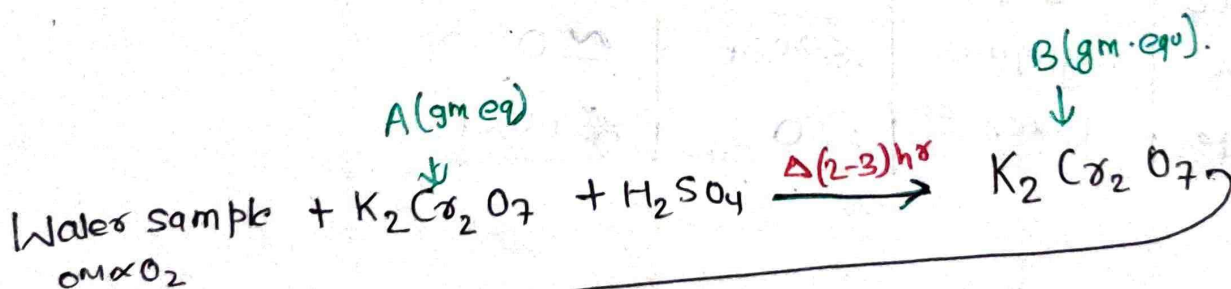
→ It is the amount of oxygen required to carry out the decomposition of total organic matter (biodegradable and non-biodegradable) organic matter present in the system (wastewater).

→ In order to find COD waste water sample is mixed with strong oxidising reagent like $K_2Cr_2O_7$ or $KMnO_4$ (generally $K_2Cr_2O_7$ is used) & sulphuric acid. (to make the medium ^{potassium dichromate} acidic in order to expedite the oxidation of complex organic matter. and sample is heated.

→ $K_2Cr_2O_7$ (Potassium dichromate) after carrying out the decomposition of organic matter within 2-3 hrs is left in the water sample.

→ This ~~potassium~~ $K_2Cr_2O_7$ left is further determined by titrating the sample with (ferrous ammonium sulphate) using ferroine as an indicator.

Note:- Fresh sewage sample is always alkaline.



Procedure

→ $K_2Cr_2O_7$ $B(\text{gm. eq})$ $\xrightarrow{\text{Ferroine Indicator}}$ BLUE GREEN Sample $\xrightarrow{\text{Titrated with ferrous Ammonium Sulphate.}}$ Vml of FAS required to turn the sample Red/Brown.

gm eq. of FAS used in titration = g.m. eq. of $K_2Cr_2O_7$ left after decomposition of organic matter.

gm. eq. of ~~Potassium~~ $K_2Cr_2O_7$ used for decomposition of OM = $A - B =$ gm eq. of OM.
= gm eq. of oxygen (O_2) required for decomposition of organic matter.

⇒ Oxygen required for decomposition of organic matter
= gm eq. of oxygen required $\times$ Equivalent wt. of O_2

→ ~~Red~~ $K_2Cr_2O_7$ is a such a strong oxidising reagent that along with decomposition of OM it also carries out decomposition of certain Inorganic matter resulting in higher oxygen demand than actual termed as di-chromate demand.

$$\text{Dichromate Demand (DOD)} = O_2 \text{ (COD)} + O_2 \text{ (Inorganic matter)}$$

Note:- $K_2Cr_2O_7$ decompose inorganic matter along with organic matter but it not able to decompose organic matter completely.

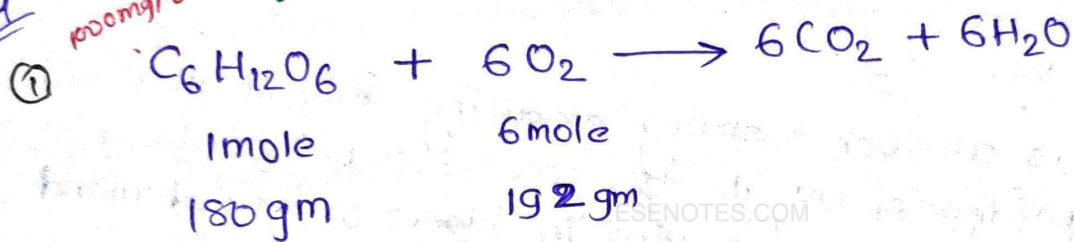
(iii) Theoretical Oxygen Demand. (ThOD)

→ If the type & concentration of OM present in system is known then the amount of oxygen required for its decomposition can be found theoretically and is termed as theoretical oxygen demand. (ThOD)

→ Generally COD comes out less than ThOD. as complete decomposition of organic matter does not take place while performing COD test. practically. But COD may be taken equal to ThOD for calculation purposes.

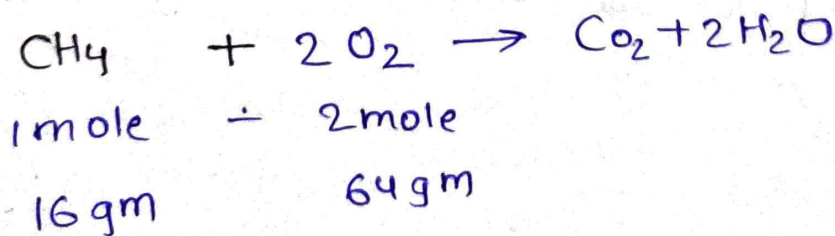
— [ThOD > COD, $\text{ThOD} = \text{COD}$]
for calculation

eg ^{1000mg/lit of glucose}



$$\text{1000mg/lit of glucose} = \frac{192}{180} \times 1000 \text{ mg/lit (ThOD) of oxygen.}$$

$\textcircled{2}$ 1000mg/lit of methane



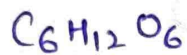
$$\begin{aligned} \text{1000mg/lit of methane require} &= \frac{64}{16} \times 1000 \text{ ThOD.} \\ &= 4000 \text{ mg/lit) of oxygen} \end{aligned}$$

IV Total Organic Carbon (TOC)

→ TOC is just another way to report the concentration of OM in terms of carbon contain.

→ If type and concentration of organic matter present in system is known it can be found theoretically or else if it is not known it can be computed by converting the organic carbon into carbon dioxide and then by analysing it by infrared rays. (This method is not used / ~~for~~ popular in India due to its high cost).

eg. 1000 mg/lit of glucose.



1 mole of glucose has 6 mole of carbon.

180 gm of glucose — 72 gm of carbon.

1000 mg/lit of — $\frac{72}{180} \times 1000 \text{ mg/lit} = \text{TOC}$

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eg 1000 mg/lit of methane, (CH₄)

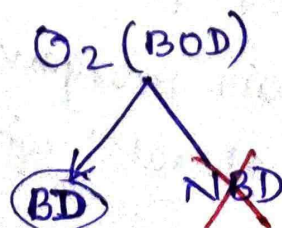
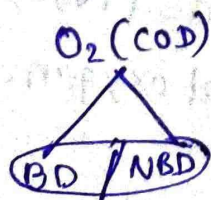
1 mole of methane has 1 mole of carbon.

16 gm of methane — 12 gm of carbon

1000 gm/lit of methane = $\frac{12}{16} \times 1000 \text{ mg/lit of carbon.}$

V Bio-chemical Oxygen demand.

→ It is amount of oxygen required for the decomposition of bio-degradable organic matter / biologically active OM present in the system.

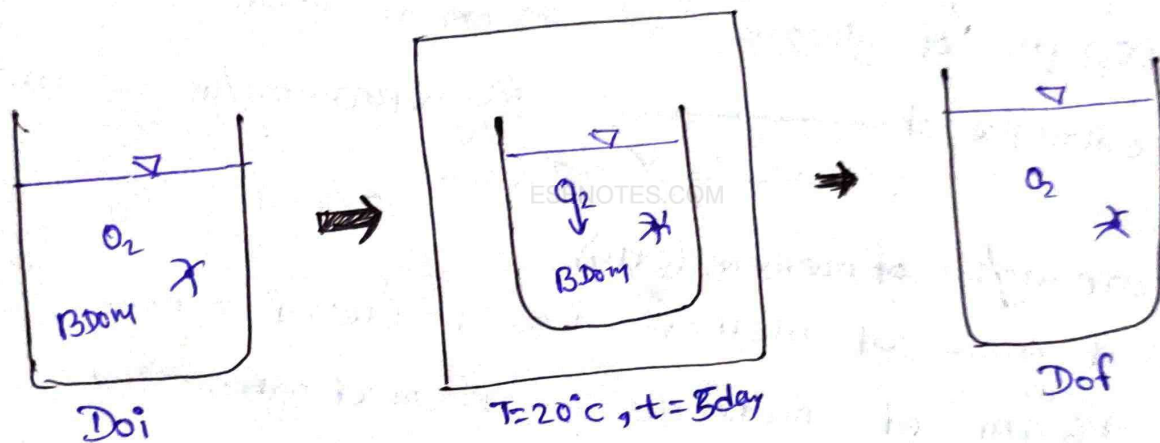


→ BOD during 5 days @ 20°C is taken as standard BOD and is approximately 68% of ultimate BOD.

→ BOD is determined by diluting the known volume of test sample with known volume of aerated/Blank water sample and noting the dissolved oxygen of diluted sample before and after incubation for 5 days at 20°C.

→ All the sources of light must be excluded from the incubator in order to avoid photosynthesis during which oxygen is released that results in higher DO final and lower BOD than actual.

(i) When no dilution is done. (OM, χ , O_2)



$$BOD_5 @ 20^\circ C = DO_i - DO_f$$

(ii) When sample is diluted. (OM, χ)

The ^{sample} ~~diluted water~~ used for dilution [Aerated/Blank sample] must possess following properties:-

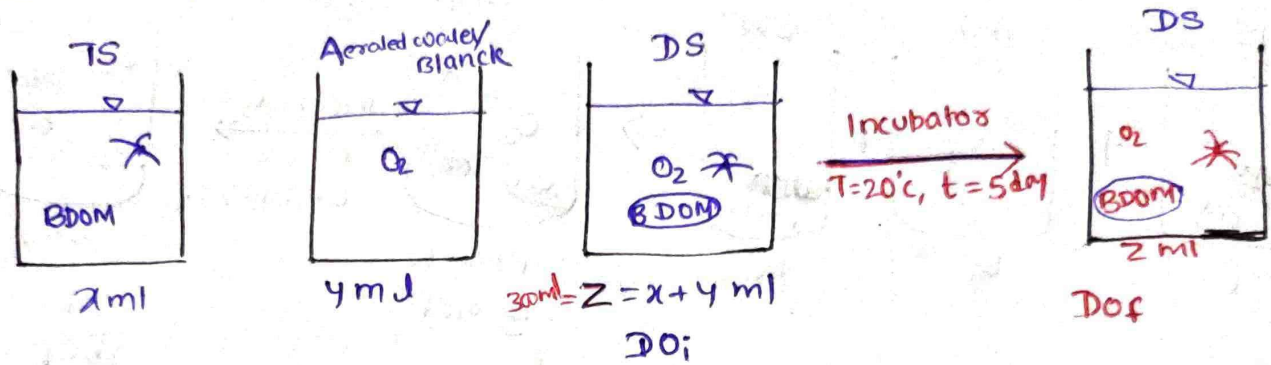
- (i) It should be free from organic matter.
- (ii) It is prepared from distilled water.
- (iii) It should contain inorganic nutrients. (N, P, S)
- (iv) It should be saturated with dissolved oxygen.

→ Consumption of oxygen must be atleast 2 mg/lit (i.e. DO_i must be atleast 2 mg/lit less than DO_i) for reliability of the result.

→ Final oxygen in the sample must be atleast 1 mg/lit

→ Hence ~~BOD of sample~~ if BOD of sample is more than 7 mg/lit then dilution of sample becomes necessary

→ Dilution



$$\text{BOD}_5 @ 20^\circ\text{C} = (\text{DO}_i - \text{Dof}) \times \text{Dilution factor or dilution ratio.}$$

$$\text{Dilution factor} = \frac{\text{Final volume of DS (300 ml)}}{\text{Initial volume of TS}}$$

(iii) When sample is diluted and seeded (om).

Seeding:- It is the process of adding microorganism in the sample during BOD test.

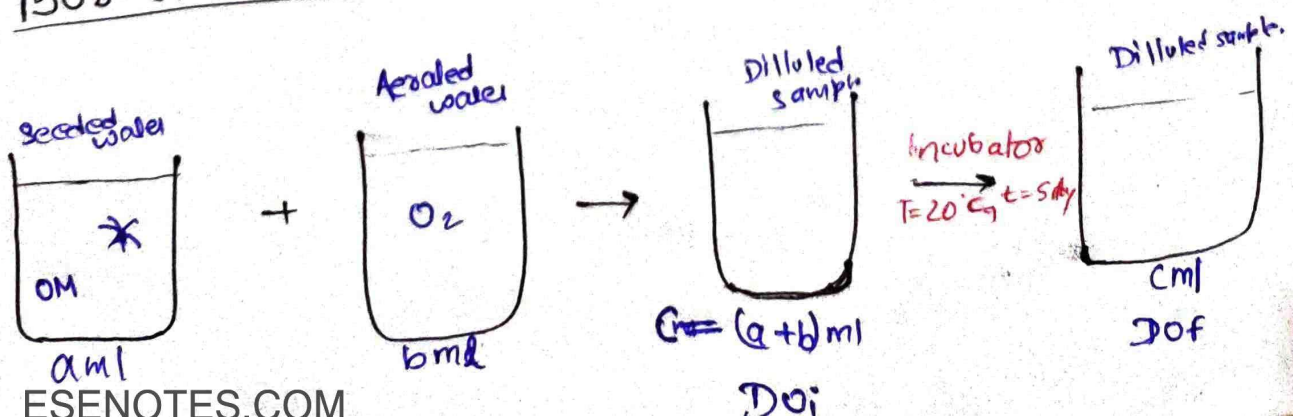
Source of seed:-

(a) Activated Sludge.

(b) Soil culture.

(c) Untreated waste water.

BOD of seeded water (Controlled sample)



$$BOD_{ss} = (DO_i - DO_f) \times DF$$

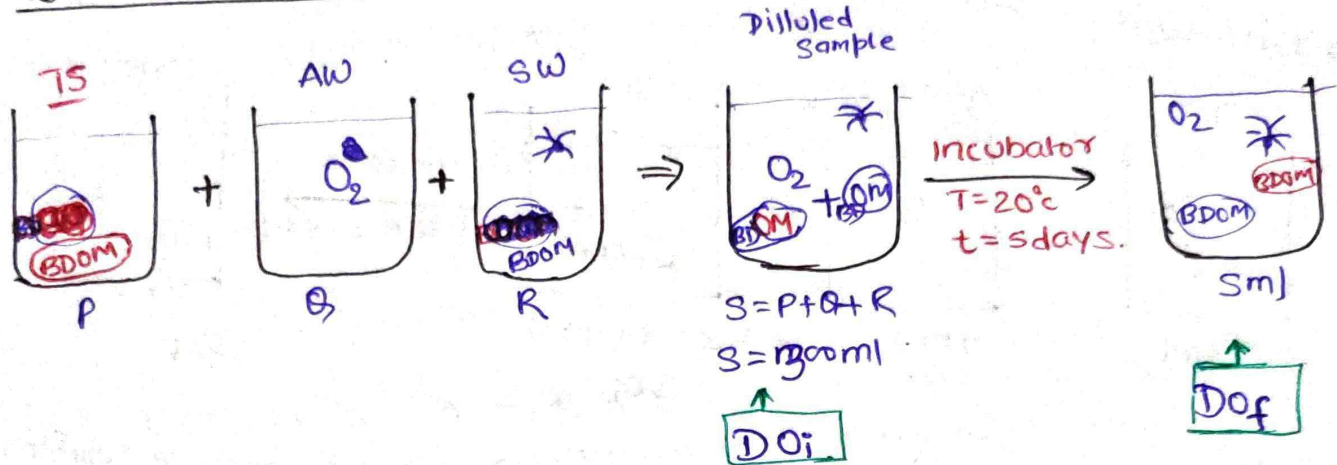
$$= (DO_i - DO_f) \frac{C}{a}$$

$C = 300 \text{ ml}$

Note If dilution of seeded water is not done. $\therefore$

$$BOD_{ss} = DO_i - DO_f$$

BOD of test sample.



$$BOD = \left[DO_i - DO_f - (DO_i - DO_f) \frac{C}{a} \times \frac{R}{S} \right] \times \frac{S}{P}$$

Here, we can say, $(DO_i - DO_f) \frac{R}{a}$ is termed as seed correction factor

$$SCF = 0.6 - 1$$

Here, $\frac{R}{a} = f = \frac{\% \text{ of seed in diluted sample}}{\% \text{ of seed in controlled sample}}$

Q. Calculate the BOD_5 at 20°C of a waste water sample of following data. BOD bottle capacity = 300 ml. = S

Seed control

Vol of seed added to bottle = 6 ml = a

DO_i of seed & dilution water = 8.2 mg/lit

DO_f of seed & dilution water = 4.8 mg/lit

Sample

Volume of seed added to bottle = 2 ml. = R

Volume of sample added to bottle = 30 ml. = P

DO_i of seed, sample and dilution water = 6.1 mg/lit.

DO_f of seed " " " " = 3.2 mg/lit

Solⁿ

$$BOD = \left[(DO_i - DO_f) - (DO_i - DO_f) \frac{a}{A} \times \frac{R}{S} \right] \times \frac{S}{P}$$

$$= \left[(6.1 - 3.2) - (8.2 - 4.8) \times \frac{2}{6} \right] \times \frac{300}{30}$$

$$= 17.66 \text{ mg/lit.}$$

Again full

$$BOD \text{ of seed sample} = (8.2 - 4.8) \times \frac{300}{6} = (DO_i - DO_f) \times D.F.$$

$$= 170 \text{ mg/lit.}$$

$$BOD \text{ of test sample} = \left[(6.1 - 3.2) - 170 \times \frac{2}{300} \right] \times \frac{300}{30}$$

$$= 17.66 \text{ mg/lit.}$$

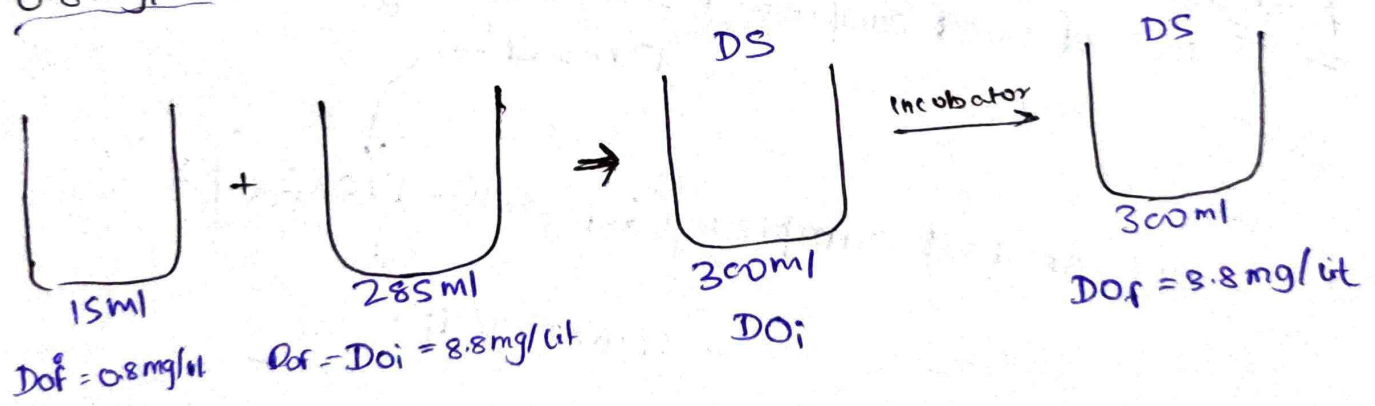
Q The following observation is available for a seeded 5 day BOD test conducted on a waste water sample 15ml of waste water sample was added directly to 300ml BOD incubation bottle the initial DO of the diluted sample was 8.8 mg/lt and $D_{of} = 1.9 \text{ mg/lt}$. The corresponding initial and final DO of seeded dilution water 9.8 and 7.9 mg/lt respectively. Compute BODs of waste water sample.

~~$BOD_s = \frac{[(8.8 - 1.9) - (9.8 - 7.9) \times \frac{300}{300} \times \frac{(300 - 15)}{300}] \times \frac{300}{15}}{1}$~~

$$BOD_s = \frac{[(8.8 - 1.9) - (9.8 - 7.9) \times \frac{300}{300} \times \frac{(300 - 15)}{300}] \times \frac{300}{15}}{1}$$

$$= 115.2 \text{ mg/lt}$$

Q In a BOD test using 5% dilution of sample, dissolved oxygen values for the sample and dilution water, ~~bottle~~ blank bottle after 5 days of incubation at 20°C were 3.8 and 8.8 mg/lt respectively DO originally presented in undiluted sample was 0.8 mg/lt. find BOD of sample.

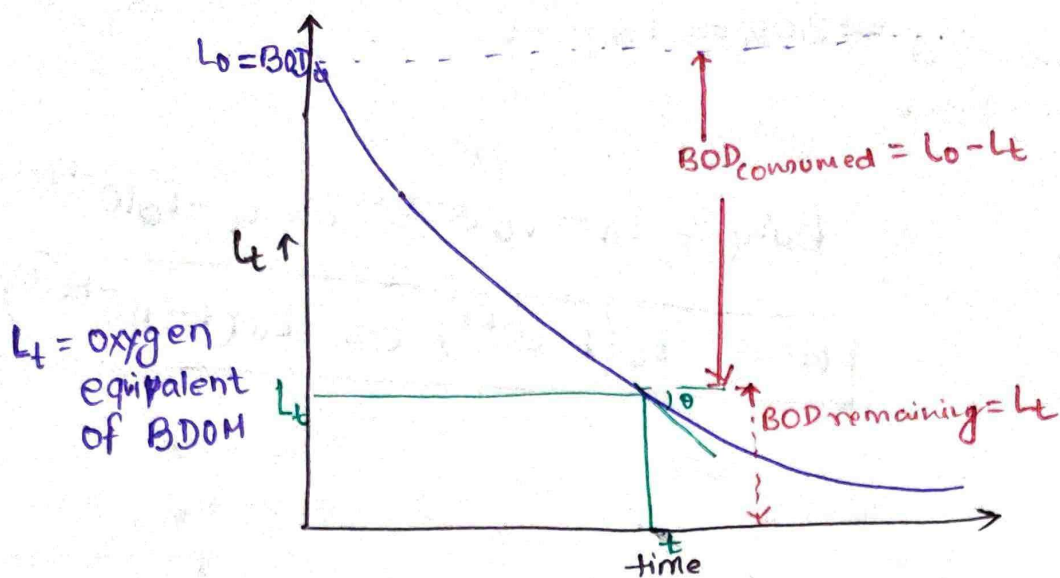


$$BOD = (DO_i - DO_f) \times \frac{300}{15}$$

$$DO_i = \frac{DO_{TS} \cdot V_{TS} + DO_{AW} \cdot V_{AW}}{V_{TS} + V_{AW}} = 8.4 \text{ mg/lt}$$

$$BOD = (8.4 - 3.8) \times \frac{300}{15}$$

$$= 92 \text{ mg/lt}$$



$$\frac{dL_t}{dt} \propto -L_t$$

($\theta_x \propto y$) first order kinetics.

$$\frac{dL_t}{dt} = -K L_t$$

$$\int_{L_t=L_0}^{L_t=L_t} \frac{dL_t}{L_t} = \int_{t=0}^{t=t} -K dt$$

$$[\ln L_t]_{L_0}^{L_t} = -K[t]_0^t$$

$$\ln L_t - \ln L_0 = -Kt$$

$$\ln \left(\frac{L_t}{L_0} \right) = -Kt$$

$$L_t = L_0 \cdot e^{-Kt}$$

$$2.303 \log_{10} \left(\frac{L_t}{L_0} \right) = -Kt$$

$$\log_{10} \left(\frac{L_t}{L_0} \right) = \frac{-K}{2.303} t$$

$$\log_{10} \left(\frac{L_t}{L_0} \right) = -K_D \cdot t$$

$$L_t = L_0 10^{-K_D \cdot t} \quad (y=f(x))$$

$$\left[\frac{K}{2.303} = K_D = 0.434K \right]$$

- $BOD_{\text{consumed during time } t} = BOD_t = L_0 - L_t$

~~BOD_t~~

$$BOD_t = L_0 - L_0 e^{-kt} \text{ or } L_0 - L_0 10^{-K_D \cdot t}$$

$$BOD_t = L_0 (1 - e^{-kt}) \text{ or } L_0 (1 - 10^{-K_D \cdot t})$$

- $BOD_{\text{remaining after time } t} = L_t$

→ Here k and K_D are terms are as "Deoxygenation Constant" at base e and base 10 respectively, that signify the rate of BOD reaction without affecting ultimate BOD.

→ The value of these constant depend upon type of organic matter undergoing decomposition. For simple organic matter like sugar it is more & for complex OM like phenol it is less.

→ With increase in temperature rate of decomposition increases, so the value of deoxygenation constant which @ any other temperature than 20°C can be found as follows:

$$K_D \text{ at } T^\circ\text{C} = K_{D@20^\circ\text{C}} (K_T)^{T-20}$$

$$K_T = 1.047 \text{ if } T \geq 20^\circ\text{C}$$

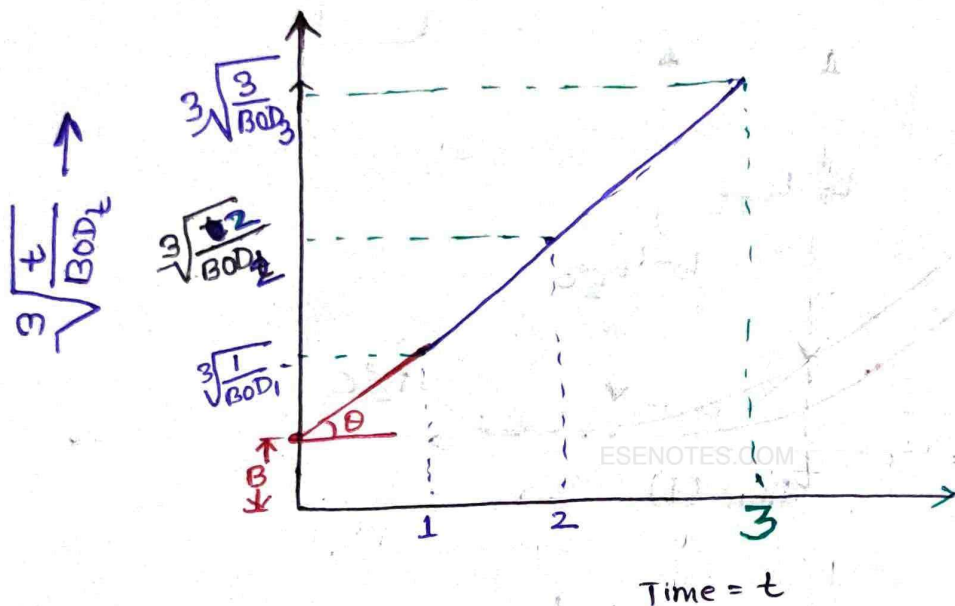
$$K_T = 1.056 \text{ if } T < 20^\circ\text{C}$$

→ For municipal sewage deoxygenation constant at base 10 @ 20°C is $K_D = 0.05 - 0.2 (\text{day})^{-1}$

→ Deoxygenation constant is calculated as follow.

Thomson Graphical Method.

Time	D_{oi}	D_{of_t}	$D_{O_t} = (D_{oi} - D_{of_t}) D.F$
1	D_{oi}	D_{of_1}	BOD_1
2	"	D_{of_2}	BOD_2
3	"	D_{of_3}	BOD_3
4	"	"	"
5	"	"	"
n	D_{oi}	D_{of_n}	BOD_n



$$\tan \theta = A$$

$$K_D = 2.61 \frac{A}{B}$$

$$\Rightarrow BOD_t = L_0(1 - e^{-kt}) \quad \text{or} \quad L_0(1 - 10^{-K_D t})$$

For $BOD_{\infty}, t = \infty$
 $BOD_{\infty} = L_0(1 - 10^{-K_D \cdot \infty}) = L_0$

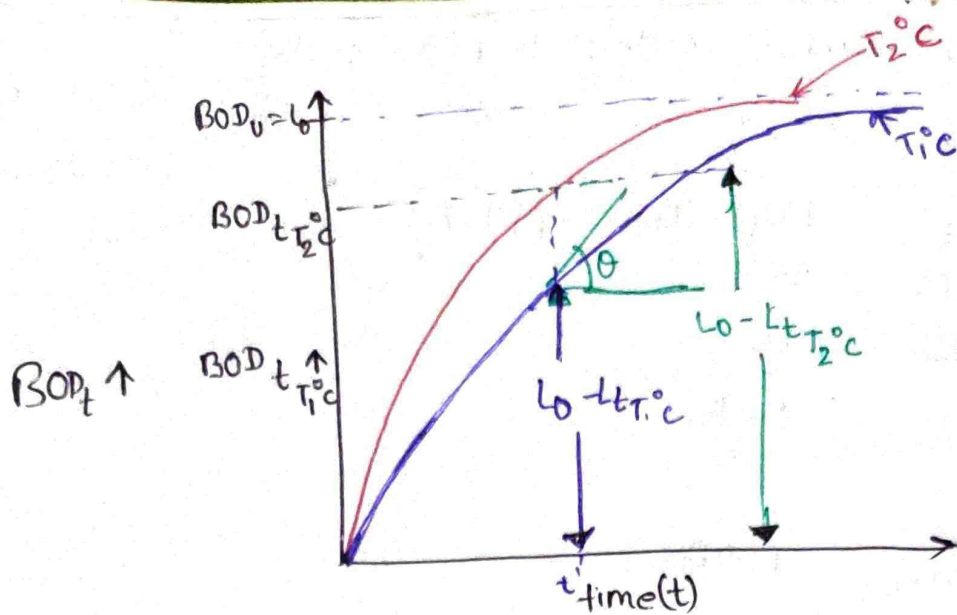
For $BOD_5, t = 5$, assume $K_D = 0.1 d^{-1}$
 $BOD_5 = L_0(1 - 10^{-0.1 \times 5}) = 0.68 L_0$

$$BOD_{10} = 0.9 BOD_{\infty}$$

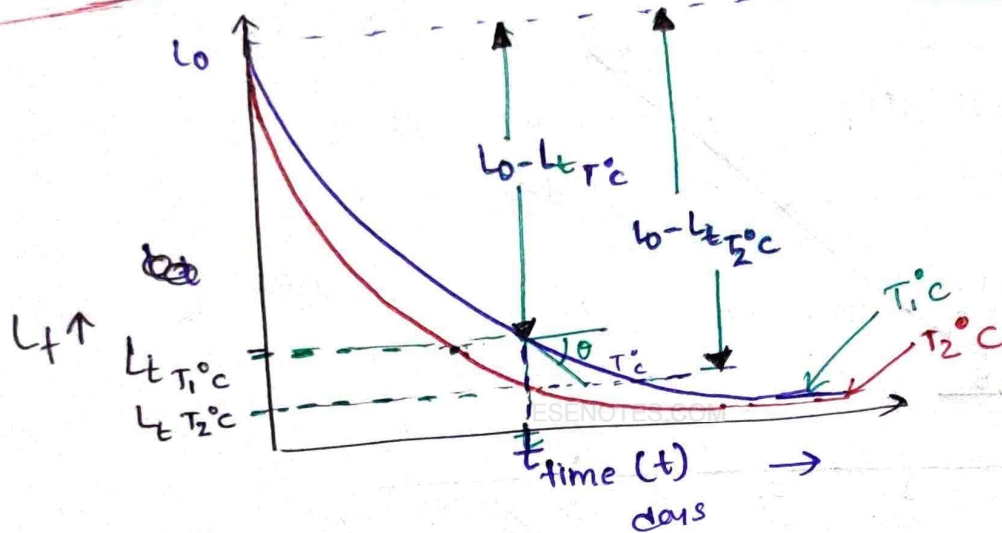
$$BOD_{15} = 0.96 BOD_{\infty}$$

$$BOD_{20} = 0.99 BOD_{\infty}$$

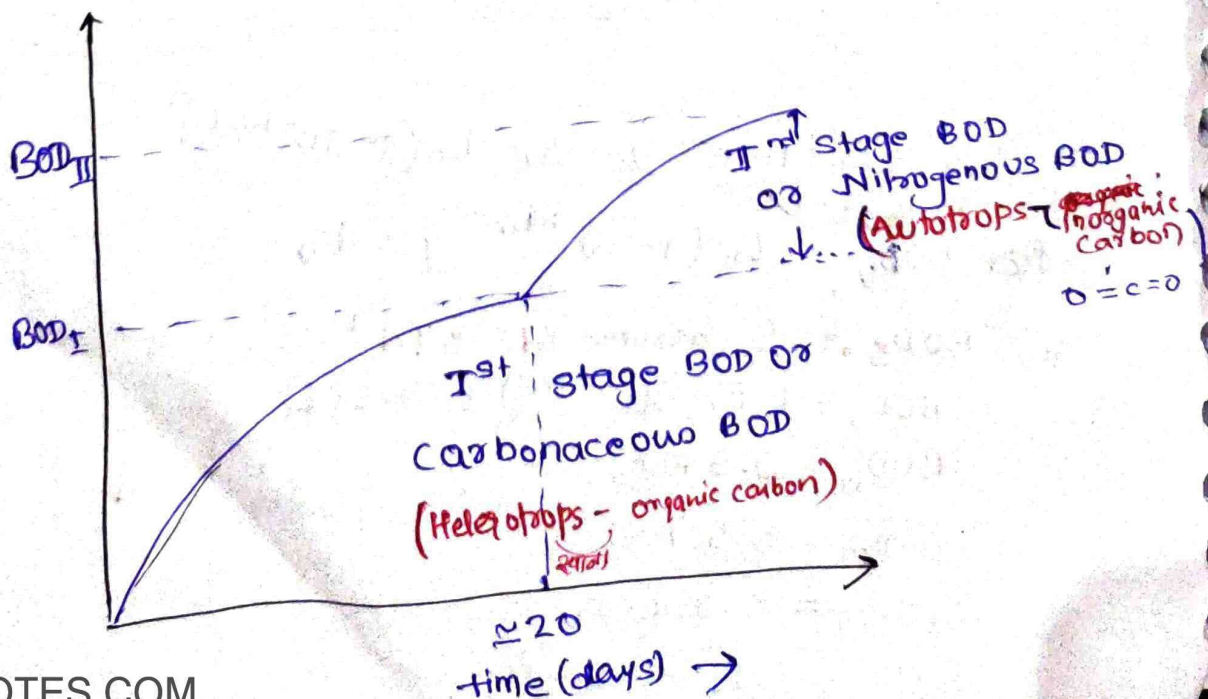
$$\downarrow \frac{dB}{dt} \propto L_t \downarrow$$



$$\begin{aligned}
 Q_{BOD_t} &= \frac{dBOD_t}{dt} \\
 &= \frac{d}{dt} [L_0 (1 - e^{-kt})] \\
 &= 0 - e^{-kt} (-k) L_0 \\
 &= k L_0 e^{-kt} \\
 &= k L_t \\
 \frac{dBOD_t}{dt} &= + K L_t \downarrow
 \end{aligned}$$



$$Q_t = \frac{dL_t}{dt} = -K L_t \downarrow$$



→ BOD_5 for municipal sewage is in range of 100-500 mg/lit.

VI $\frac{\text{Ultimate BOD}}{\text{COD}} = \frac{BOD_u}{\text{COD}}$

→ This ~~ratio~~ parameter indicates the quality of waste water with respect to type of organic matter present in it. i.e., either biodegradable and non-biodegradable.

→ This ratio varies between 0-1

→ This ratio helps in deciding the type of treatment which can be given to the waste water sample.

→ If $\frac{BOD_u}{\text{COD}} > 0.8$, waste water can be treated biologically.

→ If $\frac{BOD_u}{\text{COD}} < 0.5$, waste water can't be treated biologically, & if in such case treatment is required seeding of waste water is done, as it is toxic for treatment plant.

eg $BOD_u = 200 \text{ mg/l}$ $BOD_0 = 200 \text{ mg/l}$

(i) $\text{COD} = 2000 \text{ mg/l} \Rightarrow \frac{BOD_u}{\text{COD}} = \frac{200}{2000} = 0.1$ **toxic.**

(ii) $\text{COD} = 220 \text{ mg/l} \Rightarrow \frac{BOD_u}{\text{COD}} = \frac{200}{220} = 0.9$ - can be treated biologically
 $0.9 > 0.8$

VII $\frac{\text{COD}}{\text{TOC}}$

→ This ratio also indicates the quality of waste water with respect to the type of organic matter present in it. i.e., either simple or complex.

→ The value of this ratio is in range of 0-5.33, ~~is~~

→ 5.33 for methane

→ 0 for those organic matter which could not be decomposed even by the action of potassium dichromate.

$$\frac{\text{COD}}{\text{TOC}} = C$$

(i) COD ↓ TOC ↓
Simplest.

(ii) ^{CH₄ methane}

$$\frac{\text{COD}}{\text{TOC}} = \frac{\text{ThOD}}{\text{TOC}} = \frac{4000}{750} = 5.333$$

$$\boxed{\text{ThOD} \geq \text{COD} \geq \text{BOD}_u > \text{TOC}}$$

Q. Calculate the max^m upper limit of BOD of Benzene solution of concentration 1000 mg/lit. ~~calculate~~

$$\text{BOD}_{\text{max}} = \text{BOD}_u = \text{COD} = \text{ThOD}$$

3026

1000 mg/l C₆H₆ ESENOTES.COM



1 mole 7.5 mole
78 gm → ~~240gm~~ 240gm.

$$\begin{aligned} 1000 \text{ mg/lit of Benzene.} &\rightarrow \frac{240}{78} \times 1000 \\ &= 3076.92 \text{ mg/l} \end{aligned}$$

VIII Population Equivalent.

- Avg. standard BOD of domestic sewage is 80 gm/c/d .
- No. of individual producing the BOD @ the rate of avg standard BOD of domestic sewage equivalent to that produced by an industry is termed as Population Equivalent.
- Population equivalent signify the strength of industrial sewage thereby helps in finding the cost of treatment of sewage.

$$P.E. \times BOD_{avg DS} = BOD_{IS}$$

$$P.E. = \frac{BOD_{IS}}{BOD_{avg DS}}$$

eg. $BOD_{IS} = 10^5 \text{ kg/day}$
 $BOD_{DS} = 800 \text{ mg/lt}$

Avg. per capita water demand = 300 l/c/d .

Avg. per capita sewage = 0.80×300
 $= 240 \text{ l/c/d}$.

$$BOD_{DS} = 240 \times 300 \times 10^{-3}$$

$$= 72 \text{ gm/c/day}$$

$$P.E. = \frac{10^5 \text{ kg/day} \times 10^3}{72 \text{ gm/c/day}}$$

$$= 1388889 \text{ nos.}$$

	Q	SS	COD
Sewage.	10^7 l/d	10^3 kg/d	10^6 kg/d
SEWAGE	200 l/c/d	500 mg/c/d	750 gm/c/d
INDUSTRIAL			
DOMESTIC			
P.E. =	$\frac{10^7}{200}$	$P.E. = \frac{10^3 \times 10^6}{500}$	$P.E. = \frac{10^6 \times 10^3}{750}$
	$= 5 \times 10^4$	$= 2 \times 10^6$	$= 1.33 \times 10^6$

IX Relative Stability (S_r)

→ It is defined as ratio of oxygen available in effluent in any form (as DO or as molecular bonded oxygen) to the total amount of oxygen required to satisfy BOD_T .

	A	B
DO	5	7
BOD	10	10
R_s	5 unstable	3 Relatively Stable

$$S_r = \frac{\text{Amount of oxygen available (as DO or MBO)}}{\text{Amount of oxygen required for } BOD_T} \times 100$$

→ It can be computed by any of the following

$$S_r = 100 [1 - (0.793)^{t_{20}}]$$
$$\text{or}$$
$$= 100 [1 - (0.605)^{t_{37}}]$$

Here, t_{20} and t_{37} is the time required in days by sewage sample to decolourised standard methylene blue coloured solution when incubated @ 20°C & 37°C respectively.

- This decolourisation of methylene blue coloured solution is done by the enzyme which are released during metabolism of anaerobic microorganism.
- Hence earlier the decolourisation take place, sooner anaerobic condition develops, indicating the deficiency of oxygen in the system. thereby represents its instability.
- If sample decolourised in less than four days @ 20°C , it is termed unstable.

⑧ If sample decolourises in 5.5 days. After addition of methylene blue in it @ 20°C. In how many days the sample of same relative stability would decolourised at 37°C.

$$x = 100 [1 - (0.794)^{t_{20}}] = 100 [1 - (0.605)^{t_{37}}]$$

$$0.794^{5.5} = 0.605^{t_{37}}$$

$$5.5 \log 0.794 = t_{37} \times \log 0.605$$

$$t_{37} = 2.53 \text{ day.}$$

DISPOSAL OF SEWAGE EFFLUENT

- Disposal of sewage effluent is to be considered prior to designing the method of treatment as it governs the desired quality of treated sewage
- Disposal of sewage can be done by any of the following methods:-
 - (i) Disposal by dilution. (in water)
 - (ii) Disposal on land.

★ Disposal By Dilution.

- It is the process in which sewage effluent is disposed in large water bodies which may either be static or running after which it undergoes the process of self purification.
- Degree of treatment required to be given to the sewage and the amount of sewage which can be disposed depends upon self-purification capacity of water body.
- It is the most common method used for disposal of sewage which is adopted if following cases:-
 - (a) When sewage is comparatively fresh (4-5 hr old)
 - (b) Free from floating and settleable solid.

- (b) When the diluting water has high concentration of dissolved oxygen in it.
- (c) Where diluting water is not used for purpose of navigation.
- (d) Where the flow currents of diluting water causing no deposition - or destruction of aquatic life.
- (e) When the outfall sewer of city or treatment plant is situated near some natural water having large volume.

→ Disposal by dilution can be done using :-

- I) River water.
- II) Lake water
- III) Sea water.

I Disposal by dilution in River stream.

→ When the sewage effluent is disposed in river stream it undergoes self purification by following natural mechanism:-

(i) Sedimentation: When the sewage is disposed in river stream according to turbulence available in it organic suspended particle get settled at the bottom of river bed, thereby helps the river to get self purified.

(ii) Sunlight: In the presence of sunlight photosynthesis reaction takes place in river resulting in release of oxygen which is used by micro. organism for decomposition of organic matter thereby helping the river to get self purified.

(iii) Oxidation and Reduction.

Decomposition of organic matter ~~at~~ at surface of river stream by aerobic M/O & over the by anerobic micro-organism collectively removes organic matter from river stream, thereby helps it to self purify.

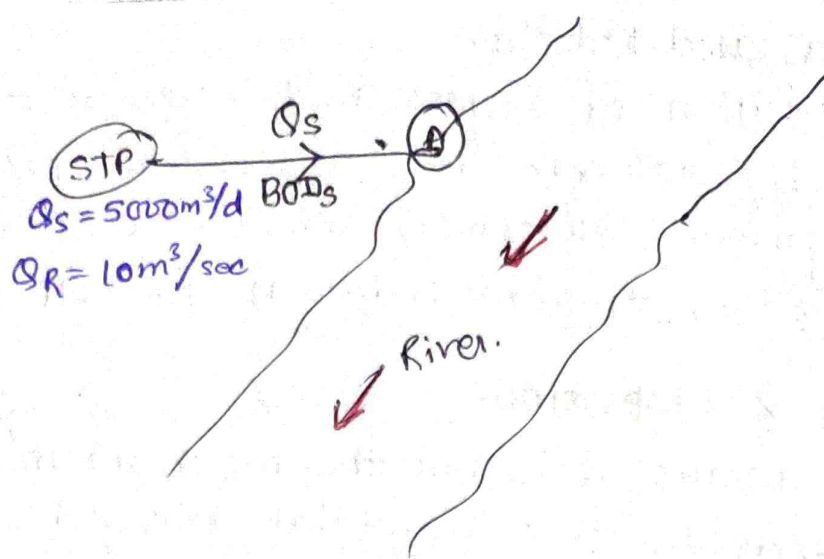
(iv) Dillution & Dispersion:

In real terms it is not the mechanism of self-purification it only reduces the potential hazard of the sewage into the river stream.

Note:- Dillution & dispersion, sedimentation and sunlight are physical forces of self purification. whereas Oxidation & reduction are chemical forces aided by biological forces (bio-chemical) of self-purification.

→ Degree of treatment required to be given on the basis of dillution available in river is as follow:-

Dillution factor	Degree of Treatment Required.
① > 500	No treatment Required.
300-500	Primary treatment Required (plain sedimentation is required such that suspended solid are less than < 150 ppm in effluent.
150-300	Treatment such as sedimentation, screening, coagulation are required such that suspended solid is less than 50 ppm in effluent.
< 150	Complete Treatment is required such that suspend solid < 30 ppm, BOD < 20 ppm in the effluent.



$$\text{Dilution factor} = \frac{\text{Final Volume / Discharge}}{\text{Initial Volume / Discharge}}$$

Q: $Q_s = 5000 \text{ m}^3/\text{day}$

$Q_R = 10 \text{ m}^3/\text{sec} = 10 \times 60 \times 60 \times 24 \text{ m}^3/\text{day}$

$$\frac{Q_s + Q_R}{Q_s} = D.F. = \frac{10 \times 60 \times 60 \times 24 + 5000}{5000}$$

$$= 172.8$$

150 - 300 range

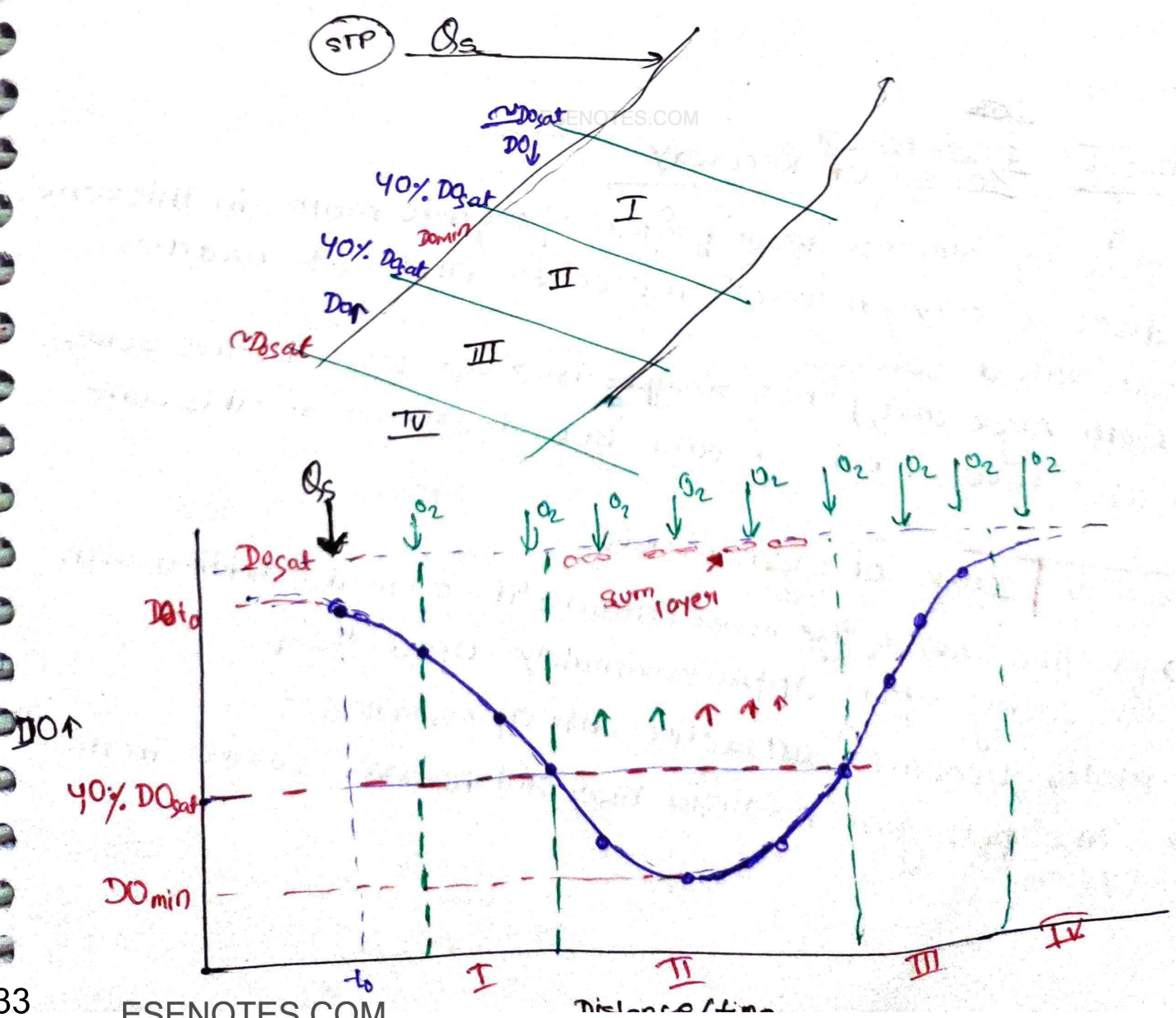
Zones of Pollution in River Stream.

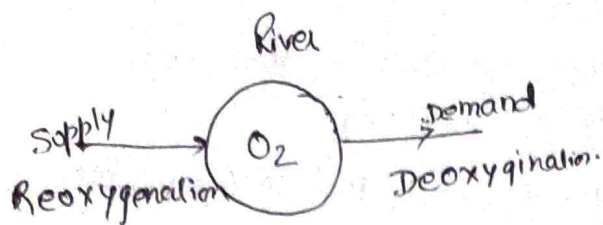
Zone-I - Zone of Degradation.

- It is the zone which is found certain distance downstream of point of disposal of sewage in the river stream.
- Water in this zone is dark and turbid in comparison to water prior to it.
 - Due to decomposition of organic matter by micro-organism dissolved oxygen level decreases in river stream.
 - Algae is not found in this zone but fishes may survive.
 - End of this zone is characterised by 80% depletion of O_2 with respect to its saturated value.

Zone-II : zone of Active Decomposition.

- It is the zone of heavy pollution, water in this zone is even dark & turbid than water in zone-I.
- Aerobic micro-organism are replaced by anaerobic M/O, which produces acids, alcohol & gases during decomposition of OM, that rises to the surface & lead to the formation of dirty scum layer.
- DO level reduces upto its minimum value in this zone (which may be 0 also).
- Both algae and fishes are not found in this zone.
- DO level increases back upto 40% of its saturated value at the end of this zone.



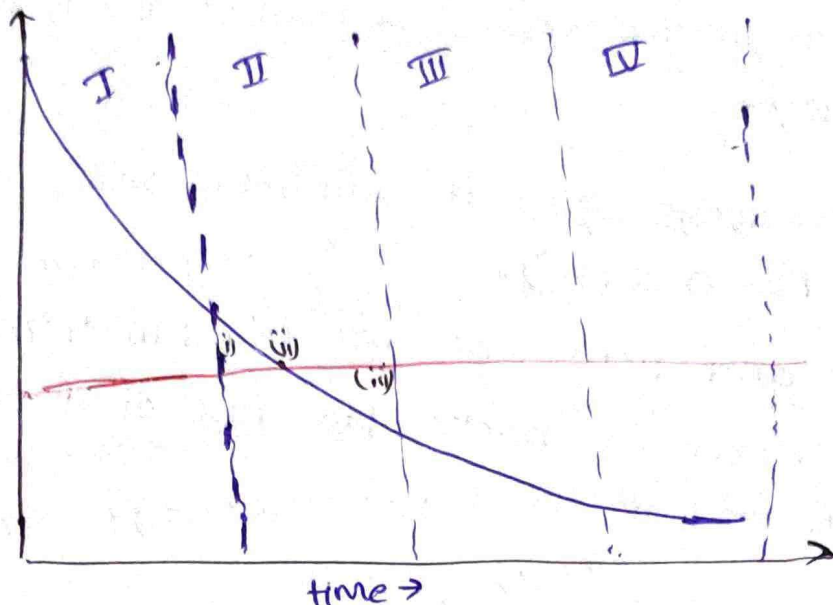


	I	II		
		(i)	(ii)	(iii)
ROD	MORE	MORE	Equal	Less
ROR	LESS	Less	Equal	more
DO	DECREASE	Decrease	min	Increase

$$ROD = \frac{dBOD_t}{dt} \times K_4 \uparrow$$

ROR $\uparrow$

Rate $\uparrow$



Zone-III ~~Due to~~ Zone of Recovery.

- $\rightarrow$ Due to almost decomposition of organic matter in this zone dissolve oxygen level increases and try to matches saturated value.
- $\rightarrow$ Both Algae and fishes reappear in river water in this zone.
- $\rightarrow$ NO_3^- level increases and BOD decreases in this zone.
 remaining

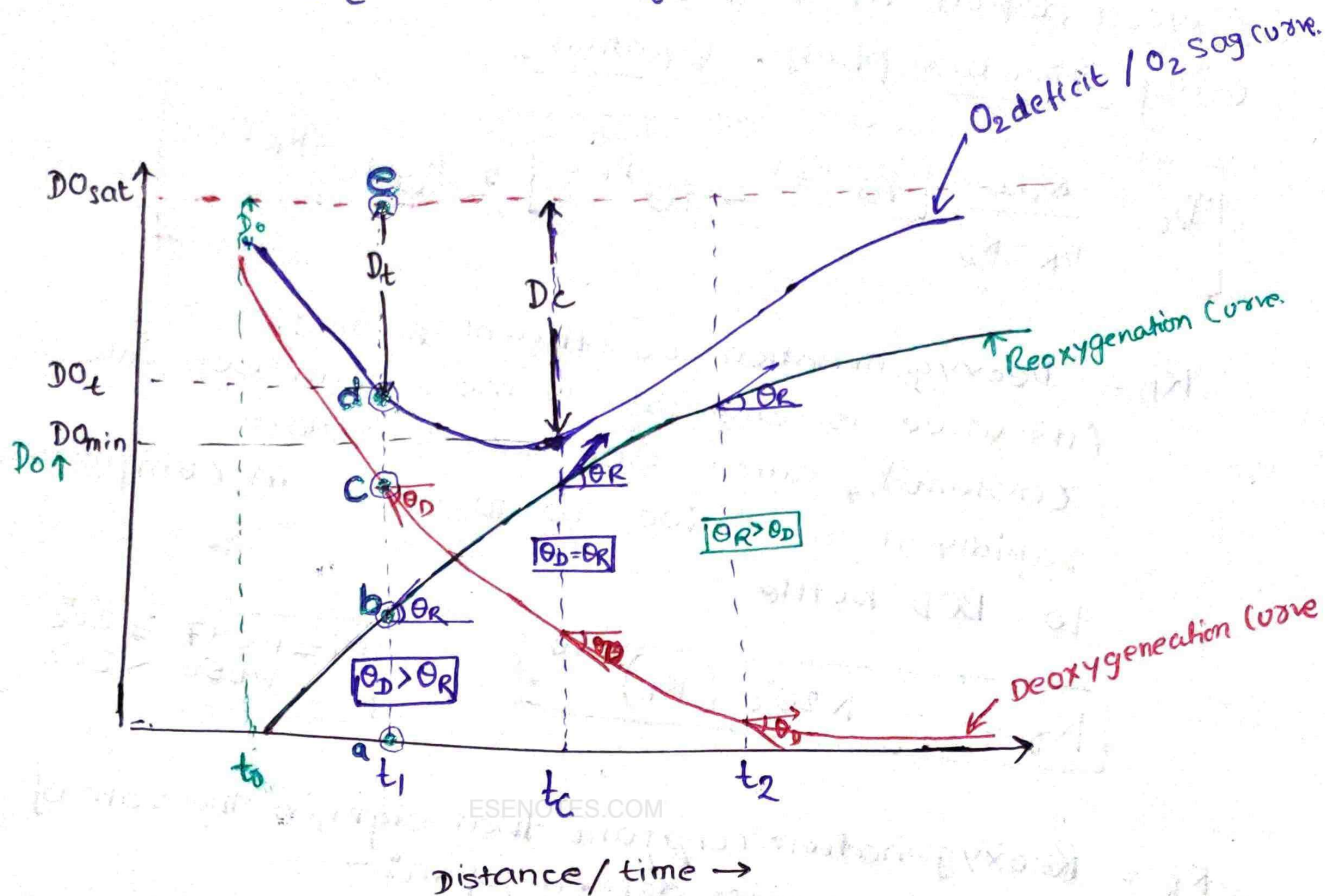
Zone-IV Zone of clear water

- $\rightarrow$ In this zone the river attains its original condition with DO rising upto approximately upto DO_{sat} .
- $\rightarrow$ Water becomes attractive in appearance.
- $\rightarrow$ Some pathogenic bacteria may still however survive in the system.

Oxygen Deficit in River Stream (D_t)

Mix water = River water + waste water.

$$D_t = D_{\text{sat}} - DO_t$$



a_b = oxygen consumed by river from the atmosphere in time t_1 .

C_e = oxygen consumed by micro-organism in time t_1
out of $DO_{sat}(ae)$

$a_c =$ oxygen left in river @ time t , after consumption of C_e from a_e by M/O.

$d_e =$ Oxygen deficit at time t , out of DO_{sat} (ae)

de = Oxygen deficit at time t
ad = Net oxygen available in river at time t, availability.

Net deficiencies
 $ce - ab = de$
 $ce - cd = de$

$$cd = ab$$

or, ^{Net availability.}
 $ac + ab = ad$
 $ac + cd = ad$

$$cd = ab$$

$$D_t = DO_{sat} - DO_t$$

→ Oxygen deficit in river stream can be computed using Streeter Phelps Equation.

$$D_t = \frac{K_D L}{K_R - K_D} [10^{-K_D t} - 10^{-K_R t}] + D_0 10^{-K_R t}$$

K_D = Deoxygenation coefficient for river.
(Its value is comparatively more than BOD rate constant), since BOD exerted more rapidly in river due to turbulence in comparison to BOD bottle

$$K_D T^{\circ}C = K_{20^{\circ}C} (K_T)^{T-20}$$

$$K_T = 1.047 \geq 20^{\circ}C$$

$$1.056 < 20^{\circ}C$$

K_R = Reoxygenation constant that signifies the rate of reoxygenation which depend upon:-

- width of river ($K_R \propto B$)
- depth of river. ($K_R \propto \frac{1}{D}$)
- turbulence of river. ($K_R \propto T$)
- Wind velocity. ($K_R \propto \frac{1}{V_w}$)
- Temperature ($K_R \propto T$)
- oxygen deficiency. ($K_R \propto DO_t$)

$$K_R T^{\circ}C = K_R 20^{\circ}C (K_T)^{T-20}$$

$$K_T = 1.016$$

For deficiency to be critical $D_t = D_c$, $\frac{dD_t}{dt} = 0$

$$t_c = \frac{1}{K_R - K_D} \log_{10} \left\{ \frac{K_R}{K_D} \left[\frac{K_D L - (K_R - K_D) D_0}{K_D \cdot L} \right] \right\}$$

Here, $\frac{K_R}{K_D} = f$ = self purification constant
 $= H$ signifies the rate of self purification.

$$t_c = \frac{1}{K_D \left(\frac{K_R}{K_D} - 1 \right)} \log_{10} \left\{ f \left[1 - (f-1) \frac{D_0}{L} \right] \right\}$$

$$t_c = \frac{1}{K_D (f-1)} \times \log_{10} \left\{ f \left[1 - (f-1) \frac{D_0}{L} \right] \right\} \quad f = \frac{K_R}{K_D}$$

For, $D_t = D_c$
 put $t = t_c$

$$D_c = \frac{K_D}{K_R} L \cdot 10^{-K_D t_c} \quad \text{or} \quad \frac{L}{f} 10^{-K_D t_c} \quad \text{--- (iii)}$$

or,

$$\left(\frac{L}{D_c f} \right)^{f-1} = f \left[1 - (f-1) \frac{D_0}{L} \right] \quad \text{--- (iv)}$$

L = ultimate BOD of mixture. (river + waste water)

D_0 = Initial oxygen deficit.

$D_0 = DO_{sat} - DO$

$$DO_{mix} = \frac{DO_s \cdot Q_s + DO_R \cdot Q_R}{Q_s + Q_R}$$

$DO_s \approx 0$ mg/lite (assumption)

$DO_R \approx DO_{sat}$ (assume).

Q A waste water treatment plant disposes of its effluent into a stream at a point A. characteristics of the stream at a location fairly upstream of A and of the effluent are as follow:

Item	Units	effluent	stream (River)
Flow	m ³ /s	0.20	0.50
Dissolved oxygen	mg/l	2.00	8
Temperature	°C	26	22
BOD at 20°C	mg/l	40	3

Assume that the deoxygenation constant K_1 @ 20°C (base e) = $0.20d^{-1}$ and the re-aeration constant K_2 at 20°C (base e) = $0.40d^{-1}$ for the mixture. Equilibrium concentration of dissolved oxygen C_s for the free water is as follow.

Temperature °C	18	20	22	23	24	25	26
C_s (mg/lit)	9.54	9.17	8.99	8.83	8.53	8.38	8.22

The velocity of the stream ~~mm/s~~ downstream of the point A is 0.2 m/s. Determine the critical oxygen deficit and its location. [use temperature co-efficients of 1.04 for K_1 and 1.02 for K_2].

$$(i) t_c = \frac{1}{K_2(f-1)} \log_{10} \left[f \left\{ 1 - (f-1) \frac{D_0}{L} \right\} \right]$$

$$D_0 = DO_{sat} - DO_{to}$$

$$\begin{aligned}
 DO_{to} &= \frac{DO_s \cdot Q_s + DO_R \cdot Q_R}{Q_s + Q_R} \\
 &= \frac{2 \times 0.20 + 8 \times 0.5}{0.2 + 0.5} \\
 &= 6.285 \text{ mg/lit}
 \end{aligned}$$

$$T_{mix} = \frac{T_s Q_s + T_R Q_R}{Q_s + Q_R}$$

$$= \frac{22 \times 0.6 + 22 \times 0.5}{0.2 + 0.5} = 23.142^\circ\text{C}$$

temperature	23	23.142	24
Cs	8.83	<u>8.78</u>	8.53

By Interpolation.

$$= \frac{24 - 23}{8.53 - 8.83} = \frac{24 - 23.142}{8.53 - ?}$$

$$? = 8.78$$

Now

$$D_0 = D_{0sat} - D_{0to}$$

$$= 8.78 - 6.285$$

$$= 2.495 \text{ mg/lit}$$

$$\begin{aligned} K_D @ 23.14^\circ\text{C} &= K_D @ 20^\circ (1.04)^{23.14 - 20} \\ &= 0.434 \text{ K} (1.04)^{3.14} \\ &= 0.434 \times 0.2 \times (1.04)^{3.14} \\ &= 0.098 \text{ d}^{-1} \end{aligned}$$

$$\begin{aligned} K_R @ 23.14^\circ\text{C} &= K_R @ 20^\circ (1.02)^{23.14 - 20} \\ &= 0.434 \times 0.4 \times (1.02)^{3.14} \\ &= 0.184 \text{ d}^{-1} \end{aligned}$$

$$f = \frac{K_R}{K_D} = \frac{0.184}{0.098} = 1.87$$

②

$$\begin{aligned} \text{BOD}_5 \text{ mix @ } 20^\circ\text{C} &= \frac{\text{BOD}_5 \times Q_S + \text{BOD}_{5r} \times Q_R}{Q_S + Q_R} \\ &= \frac{40 \times 0.2 + 3 \times 0.5}{0.2 + 0.5} \\ &= 13.57 \text{ mg/Lit} \end{aligned}$$

$$\begin{aligned} \text{BOD}_5 \text{ mix @ } 26^\circ\text{C} &= L (1 - e^{-K_d \times 5}) \\ 13.57 &= L (1 - e^{-0.2 \times 5}) \\ L &= 21.467 \text{ mg/Lit} \end{aligned}$$

$$\begin{aligned} t_c &= \frac{1}{K_d(F-1)} \log_{10} \left[f \left\{ 1 - (f-1) \frac{D_0}{L} \right\} \right] \\ &= \frac{1}{0.098 \times (1.88-1)} \log_{10} \left[1.88 \left\{ 1 - (1.88-1) \frac{2.495}{21.47} \right\} \right] \\ &= 2.635 \text{ days.} \end{aligned}$$

$$\begin{aligned} d_c &= t_c \cdot V \\ &= 2.635 \times 0.2 \text{ m/sec} \\ &= 0.2635 \times 0.2 \times 86400 \times 10^3 \\ &= 45.53 \text{ km.} \end{aligned}$$

$$\begin{aligned} \text{(ii) } D_c &= \frac{L}{f} 10^{-K_d \cdot t_c} \\ &= \frac{21.47}{1.88} 10^{-0.098 \times 2.635} \\ D_c &= 6.3 \text{ mg/L} \end{aligned}$$

③ ^{Ground water} with fishes will able to survive in this river system. If no compute the degree of treatment required to be given to sewage. consider 10% expansion of sewage effluent.

$$DO_{min} = DO_{sat} - D_c$$

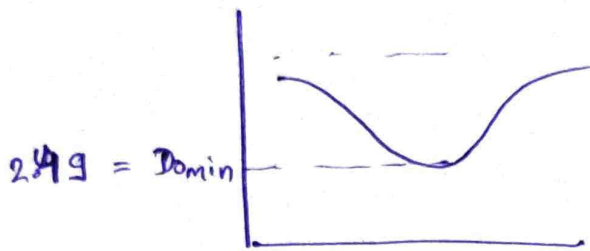
$$= 8.78 - 6.29$$

$$= 2.49 \text{ mg/l} < 4 \text{ mg/l}$$

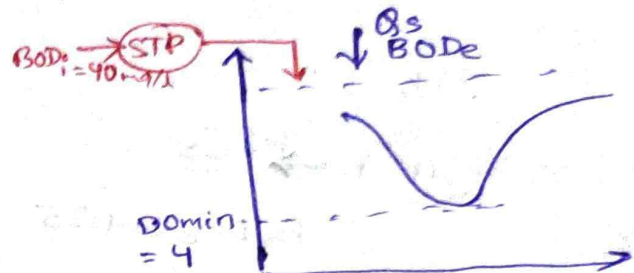
So in this case cold water fish can't survive.

Given situation,

$$\frac{Q_s = 0.2 \text{ m}^3/\text{s}}{BOD = 40 \text{ mg/lit}} \downarrow$$



→ Desired Situation



$DO_{min} = 4 \text{ mg/l}$ for survival of cold water fish

$$D_c = DO_{sat} - DO_{min}$$

$$= 8.78 - 4$$

$$= 4.78 \text{ mg/l}$$

$$\left(\frac{L}{D_c f} \right)^{f-1} = f \left[1 - (f-1) \frac{D_o}{L} \right]$$

$$\left(\frac{L}{4.78 \times 1.88} \right)^{1.88-1} = 1.88 \left[1 - (1.88-1) \frac{2.495}{L} \right]$$

$$L = 2.69 \text{ mg/lit}$$

$$BOD_{5 \text{ mix}} @ 20^\circ\text{C} = L (1 - e^{-K_{20^\circ\text{C}} \times 5})$$

$$= 2.69 (1 - e^{-0.2 \times 5})$$

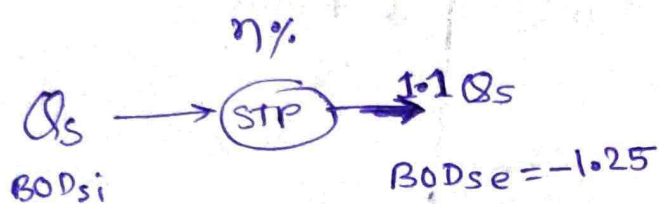
$$= 1.7 \text{ mg/lit}$$

$$BOD_{5\text{ mix}}/20^\circ = \frac{BOD_{5e} \times 1.1 Q_s + BOD_R \cdot Q_R}{1.1 Q_s + Q_R}$$

$$1.7 = \frac{BOD_{5e} \times 1.1 \times 0.2 + 3 \times 0.5}{1.1 \times 0.2 + 0.5}$$

$$BOD_{5e} = \cancel{0.2} - 1.25 \text{ mg/l}$$

Even after 100% degree of treatment BOD_{5e} can't be attained as $umgl$



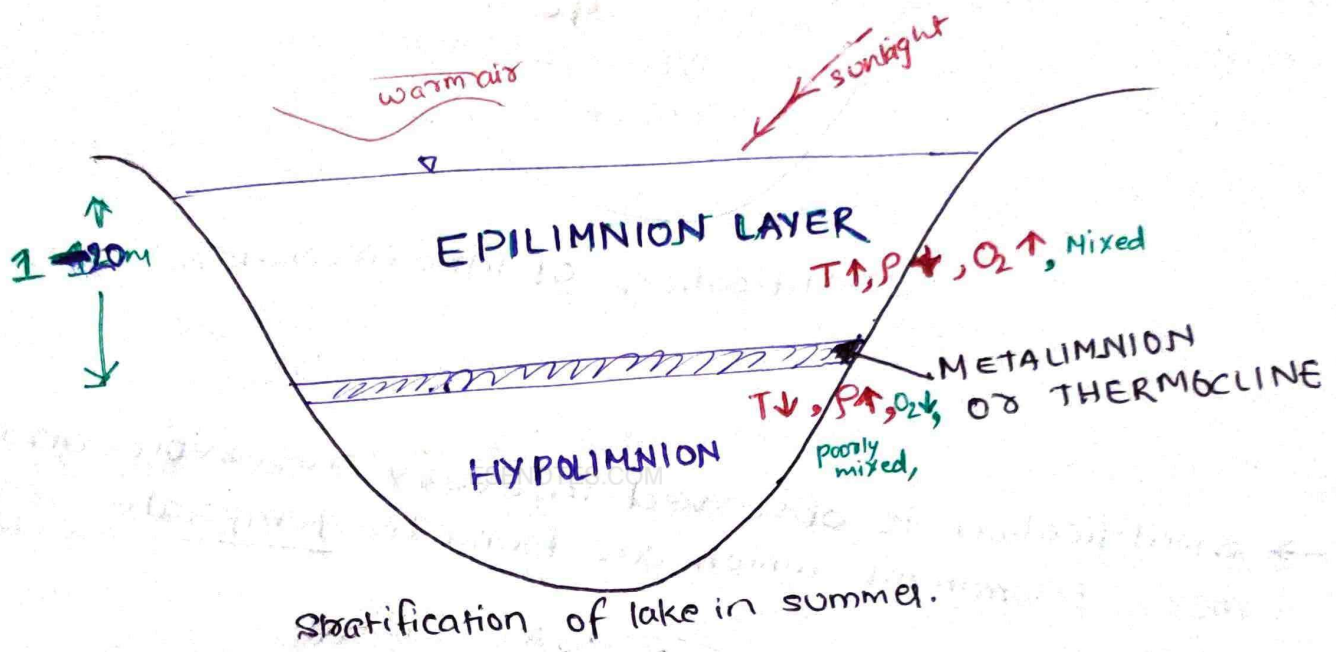
II Disposal By dilution in Lake.

- Disposal of waste water in confined lake or reservoir is much more harmful than its disposal in flowing streams or river as its purification does not take by same mechanism.
- Prime pollutant of lake is "phosphorus" (which act as a nutrient for algae). which is present in both industrial and domestic sewage.
- The study of lake system is essential to understand the role of phosphorus in lake pollution.
- Study of lake is called "Limonology".

(i) Stratification in Lakes [formation of layers].

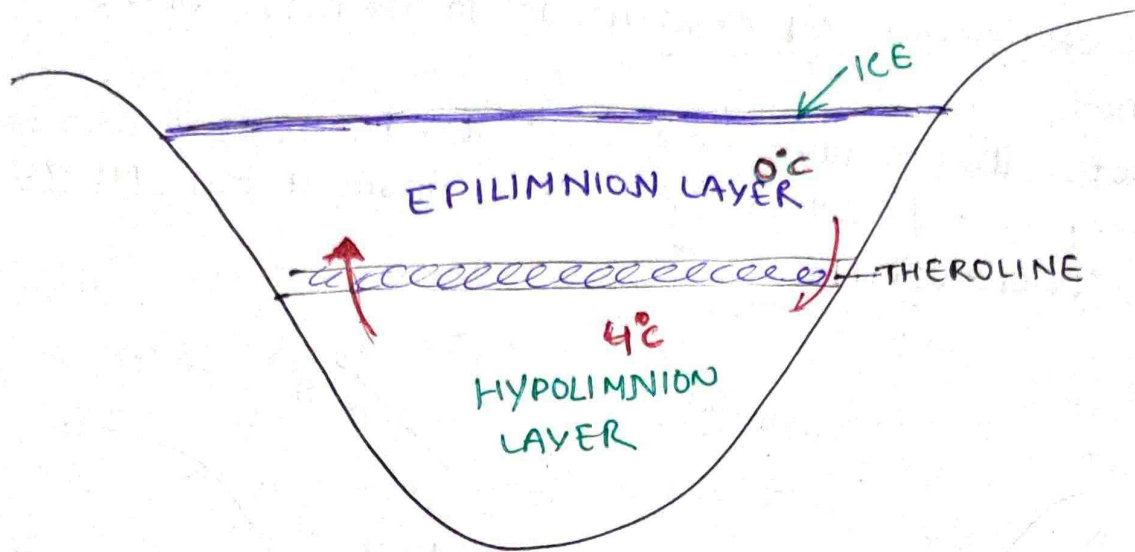
- During summers, the surface water of lakes gets heated up by sunlight and warm air. This warm water being lighter remain in upper layer near to the surface.

- This warmer, well mixed water at sufficient depth has oxygen present in it and is termed as EPIIMNION ZONE.
- The lower depth which remain colder poorly mixed and has deficiency of oxygen is termed as HYPOLIMNION zone.
- Both these layer are separated by a narrow layer of steep temperature gradient. termed as METALIMNION or THERMOCLINE.



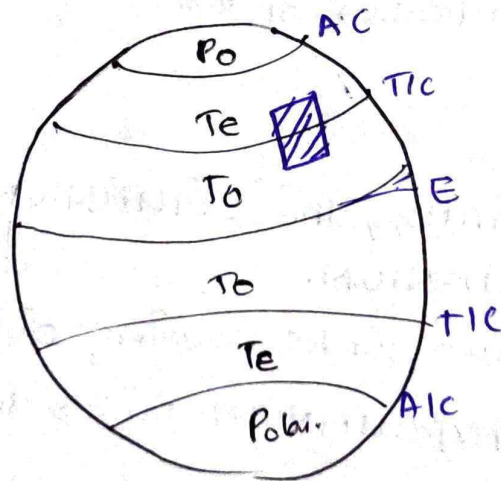
- With onset of winter, the EPIIMNION cools, until is more dense than HYPOLIMNION.
- The surface water sinks causing overturning.
- The water of hypolimnion rises to surface where it cool & again sinks.
- The lake thus become completely mixed making it aerobic.
- When temperature corresponding to freezing of water is reached, this stratification is interrupted at 4°C. when water is most dense

→ Further cooling or freezing of water surface results in winter stratification.



Stratification of lake in winter.

→ Stratification is observed in every reservoir and is more prominent which are found in temperate region.



II Biological zones in Lake.

→ Lakes have been found to exhibit distinct zones of biological activity, largely determined by the availability of light and oxygen. as follows:-

(i) EUPHOTIC ZONE.

→ The upper layer of lake water through which sunlight can penetrate is called EUPHOTIC ZONE.

→ All plant growth occurs in this zone only.

→ In deep water algae growth as the most important plant, while rooted plants grow in shallow water near the surface.

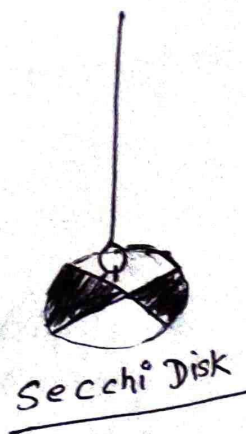
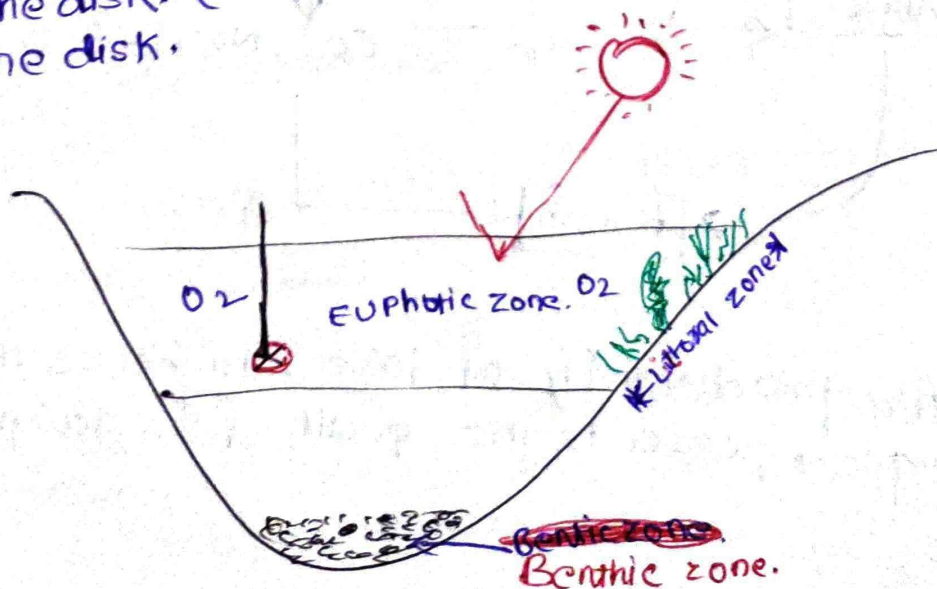
→ Depth of EUPHOTIC zone is reduced by turbidity which blocks the penetration of sunlight.

→ This turbidity is due to presence of algae.

→ At the bottom of Euphotic zone only rarely coincide with thermocline. (Depth of Euphotic zone $<$ Euphotic zone).

→ The depth of Euphotic zone can be approximately determined by a simple device termed as "SECHI DISK".

→ This disk is lowered into the lake until the observer can no longer distinguish between the quadrants of the disk. (between the black and white markings on the disk).



(ii) Littoral Zone

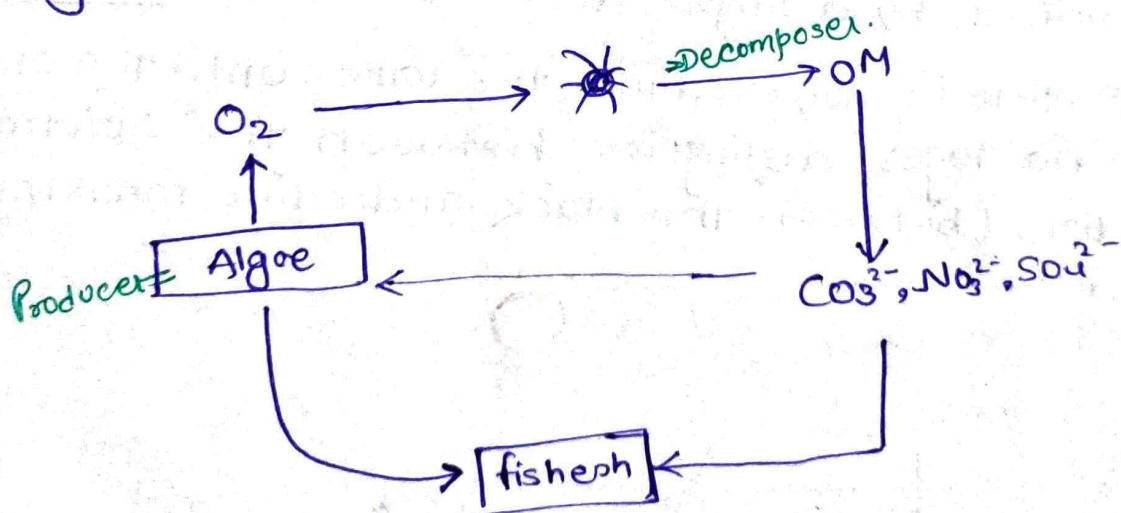
- The shallow water near the shore, in which rooted plant grows is called littoral zone.
- The Extent of the littoral zone depends upon the Slope of the lake bottom and depth of Euphotic zone.
- This zone can't extend deeper than Euphotic zone.
-

(iii) Benthic Zone:

- The bottom sediment in the lake comprise that is called Benthic zone.
- As the organism living in the overlying water die, they settle down to the bottom where they are decomposed by the organism, leaving in the benthic zone.

III Productivity of lake

- It is the ability of lakes to support the food chain or may also be define as ability to support the growth of algae.



- Higher is the productivity of lake, higher is the presence of algae, poorer is the quality of water in the lake.

→ Lakes can also be classified on the basis of their productivity level as follows:-

(i) OLIGOTROPIC LAKES.

- These are the lakes of least productivity levels hence supports no or negligible growth of algae.
- Sunlight is able to penetrate upto the hypolimnion layer in these lakes.
- Euphotic zone often extends upto the hypolimnion layer in these lakes.
- Due to sufficient availability of oxygen, these are termed as aerobic lakes.

(ii) MESOTROPIC LAKES.

- These are the lakes of intermediate productivity level hence support intermediate growth of algae.
- Though deficiency of oxygen is found in these lakes these are termed as aerobic lakes.

(iii) EUTROPHIC LAKES.

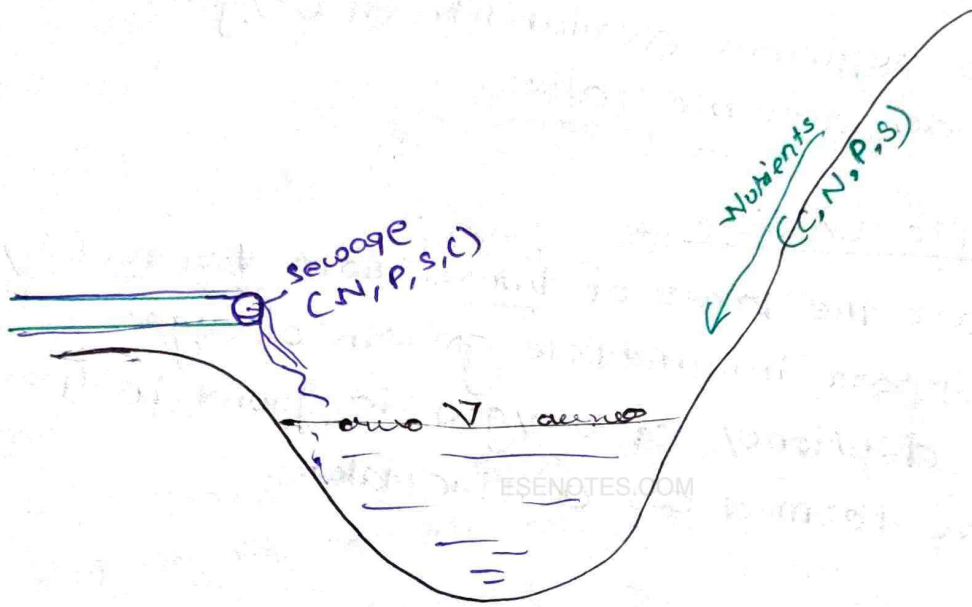
- These are the lakes of high ^{level of} productivity ~~level~~ hence support high growth of algae.
- Penetration of sunlight in these lake limited to epilimnion layer.
- Euphotic zone extends only partially in ~~the~~ Epilimnion zone.
- Due to high deficiency of oxygen, these are termed as anaerobic lake.

(iv) SENESCENT LAKE.

- These are the lakes of highest productivity levels which represents very old lake that is practically converted into marshy land.

IV Eutrophication of Lakes

- It is the natural process in which lakes get infested by algae and gradually silts up to become more shallow, due to continuous entry & recycling of nutrients in it.
- In order to avoid the growth of algae in lakes copper sulphate or lime can be added and in order to avoid eutrophication, entry of nutrients must be restricted in it.



Disposal by Dillution in Sea Water.

- Sea water consist of 20% less dissolved oxygen than river water at any given temperature due to high concentration of salt in it
- When sewage solid are disposed into the sea they chemically react with dissolved matter of sea water. Resulting in precipitation of some of the sewage. giving milky appearance to it and forming SLUDGE BANKS which produces hydrogen sulphide thereby leading to unsanitary condition.

→ Since specific gravity of sea water is greater than that of sewage & temp. of sea water is lower than that of sewage, it floats over surface of sea water leading to the formation of layer at top. termed as "SLEEK".

→ But dilution offered by sea water is much more than river water hence it is better source of disposal than river but it obselete method nowadays.

→ Following condition are to be ensured while disposing sewage into the sea:-

- (i) The sewage should be discharged in deep sea only (1.5 km - 3 km from shore).
- (ii) In order to mix the sewage properly with sea water it should be disposed at minimum depth of 3-5m.
- (iii) Before deciding the position of outfall sewer the sea current, wind direction & velocity should be checked.
- (iv) Disposal should be done only @ time of low tides the outfall sewer should be placed firmly in rock foundation.

II Disposal on Land.

→ In this method the sewage effluent either treated or diluted is generally disposed by applying it over the land.

→ The percolating water may either join water table or is collected by system of underdrain.

→ Disposal on land can be done by following method:-

- (i) Effluent Irrigation
- (ii) Sewage farming.

→ In effluent irrigation prime concern in disposal of sewage, whereas in sewage farming prime concern is the growth of crops. hence quality of effluent in the later case should be comparatively better.

→ Effluent irrigation method of disposal of sewage is adopted under following conditions

- (i) When some natural river are not located in the vicinity.
- (ii) When irrigation water is scarcely available.
- (iii) When large area of open land are available.
- (iv) The method of effluent irrigation will prove useful in area of low rainfall as this will help in maintaining good absorption capacity of the soil.
- (v) The area of land treatment or sewage farming should preferably porous ^{and permeable} such as sandy or loamy soil.

→ This method of disposal poses problem during periods of no irrigation water is required for crop. eg. rainy season.

→ This method is preferred in areas of low water table where rate of percolation may be high.

SEWAGE SICKNESS.

→ It says when sewage is disposed continuously on the piece of land, the soil pores or voids may get filled up choked with sewage solid retain in them leading to accumulation of sewage over the land and avoiding the entry of oxygen in it. Thereby leading to anaerobic decomposition of organic matter which develop unsanitary condition.

Q A town disposes sewage by land treatment. It has a sewage farm area of 150 hectares. the area included an extra provision of 50% for rest of rotation. The population of town is 50,000 and rate of supply 140 l/c/d. If 75% then calculate the consuming capacity of the soil.

$$\text{Rate of production of sewage} = 0.80 \times 140 \times 50000 \times 10^{-3} \\ = 5600 \text{ m}^3/\text{day}.$$

$$\text{farm area used for disposal} = \frac{0.80 \times 150}{1.5} = 100 \text{ hectare}$$

$$\text{consuming capacity of soil} = \frac{5600 \text{ m}^3/\text{day}}{100 \times 10^4 \text{ m}^2}$$

$$= 5.6 \times 10^{-3} \frac{\text{m}^3/\text{day}}{\text{m}^2/\text{day}}$$

$$= 5.6 \text{ l/m}^2/\text{day}$$

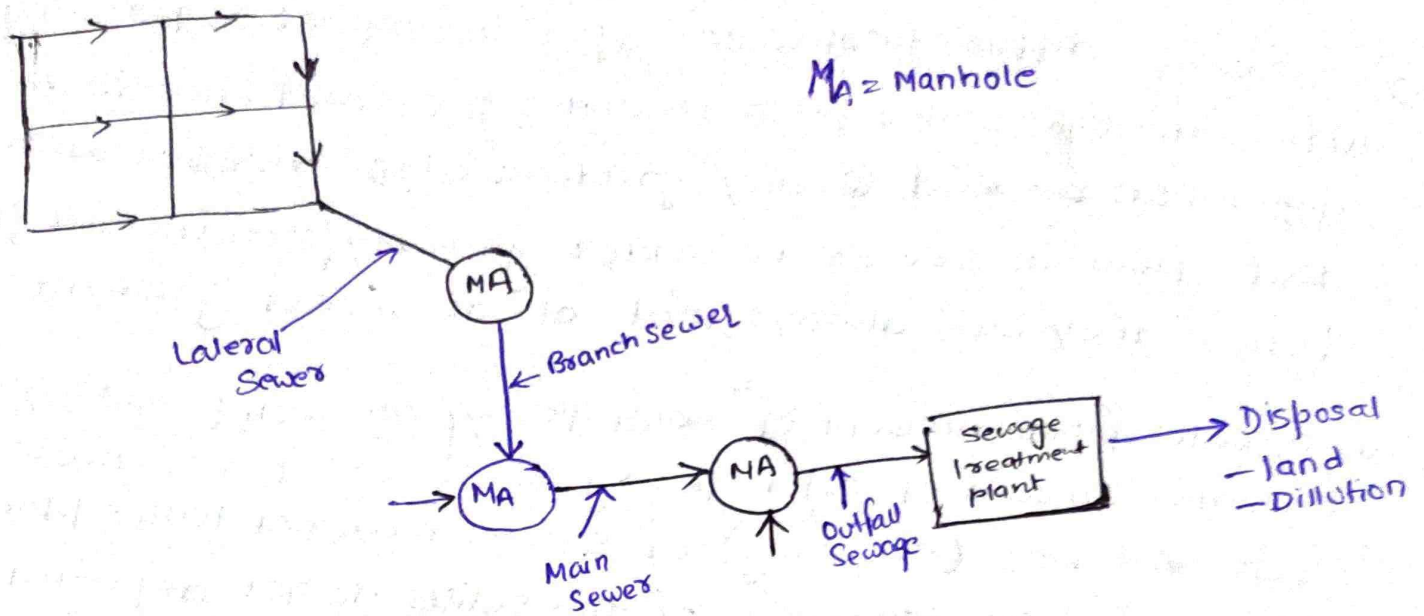
$$2 \times 2 \times 50$$

→ For most of the Indian town, the land disposal method is better choice as:-

- (i) Due to hot climate, DO contain of river water are low while the fishes and aquatic life require higher DO for their survival.
- (ii) There are only a few coastal town in India, which have strong tidal currents moving in forward direction and necessary depth of water at disposal site.
- (iii) Most of the inland river flowing ~~flowing~~ in vicinity of our cities do have very small amount of dry wheather flow, most of them run dry during summer
- (iv) Except for few major city the water supply are very low in the range of (90-120 l/c/d) thus resulting in production of highly concentrated sewage which on travelling in our hot climate become stale & septic by the time it reaches point of disposal.

Characteristic of effluent	(For Land) Tolerance limit as per IS 3007- 1965	Marine.
1. BOD ₅ at 20°C	500 mg/l	BOD - 100 mg/l
2. pH value	5.5 to 9.0	COD = 250 mg/l
3. Total dissolved solid	2100 mg/l	pH = 5.5 to 9
4. Oil and grease	30 mg/l	T.S.S = 100 mg/l
5. Chlorides (as Cl)	600 mg/l	Oil & grease = 20 mg/l
6. Boron	2 mg/l	Fluorides = 15 mg/l
7. Sulphates	1000 mg/l	Ammoniacal Nitrogen = 50 mg/l
8. Percentage of sodium with respect to total content of sodium, calcium, magnesium and potassium	60%	
9. Radioactive material		
(i) α-emitters	10 ⁻⁹ H c/ml	
(ii) β-emitters	10 ⁻⁸ H c/ml	

Design of Sewers



Type of Component Structure.	Properties for deciding design period	Design life
→ Lateral sewer	Requirement may changes faster in limited area.	full development.
→ Branch, Main, outfall Sewer	These are difficult & costly to enlarge.	40-50
→ Treatment plant	Interest rate being high/moderate.	15-20
→ Pumping plant	Technological advancement	5-10

→ Sewers are pipes or conduits required to carry the sewage from one point to another in sewerage system.

→ Sewers differ from water pipes in respect to following:-

(i) flow through water pipes is under pressure hence these pipes can be laid @ any gradient (upward ~~down~~ or downward) but flow in sewer is under gravity (open channel flow) hence they are always laid at downward gradient.

(ii) In water pipes amount of solid flowing are comparatively less then the amount of solid flowing in sewers hence more wear and tear (scouring) of sewer material takes place. And has higher tendency of the solids to get deposited & lead to the chocking of sewers.

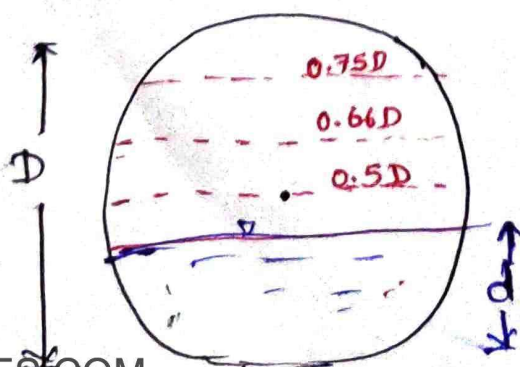
→ About 80% of water being supplied into community flows into the sewer.

Assumption: Avg discharge = 0.8 Avg. water demand.

→ Sewers are never designed to run at full depth.

if	D(mm)	$\frac{d}{D}$	FOS
	< 400	$\frac{1}{2}$	2 2
	400-900	$\frac{2}{3}$	1.5 1.5
	≥ 900	$\frac{3}{4}$	1.33

FOS = $\frac{\text{Max}^m \text{ depth at which sewer can run without overflow (D)}}{\text{Actual flow depth (d)}}$



(i) Additional space provided above the depth of flow helps to accommodate the additional sewage being produced over the assumed design flow of sewage.

(ii) This additional space also helps in accumulating methane produced during anaerobic decomposition in sewer. This helps in reducing the chances of explosion in the sewer.

→ Unless given fluctuation in the rate of flow of sewage in branch sewer is taken as follow:-

$$\text{Max}^m \text{ daily flow} = 2 \times \text{Avg daily flow.}$$

$$\begin{aligned} \text{Max}^m \text{ hourly flow} &= 1.5 \times \text{Max}^m \text{ daily flow} \\ &= 1.5 \times 2 \times \text{Avg daily flow} \end{aligned}$$

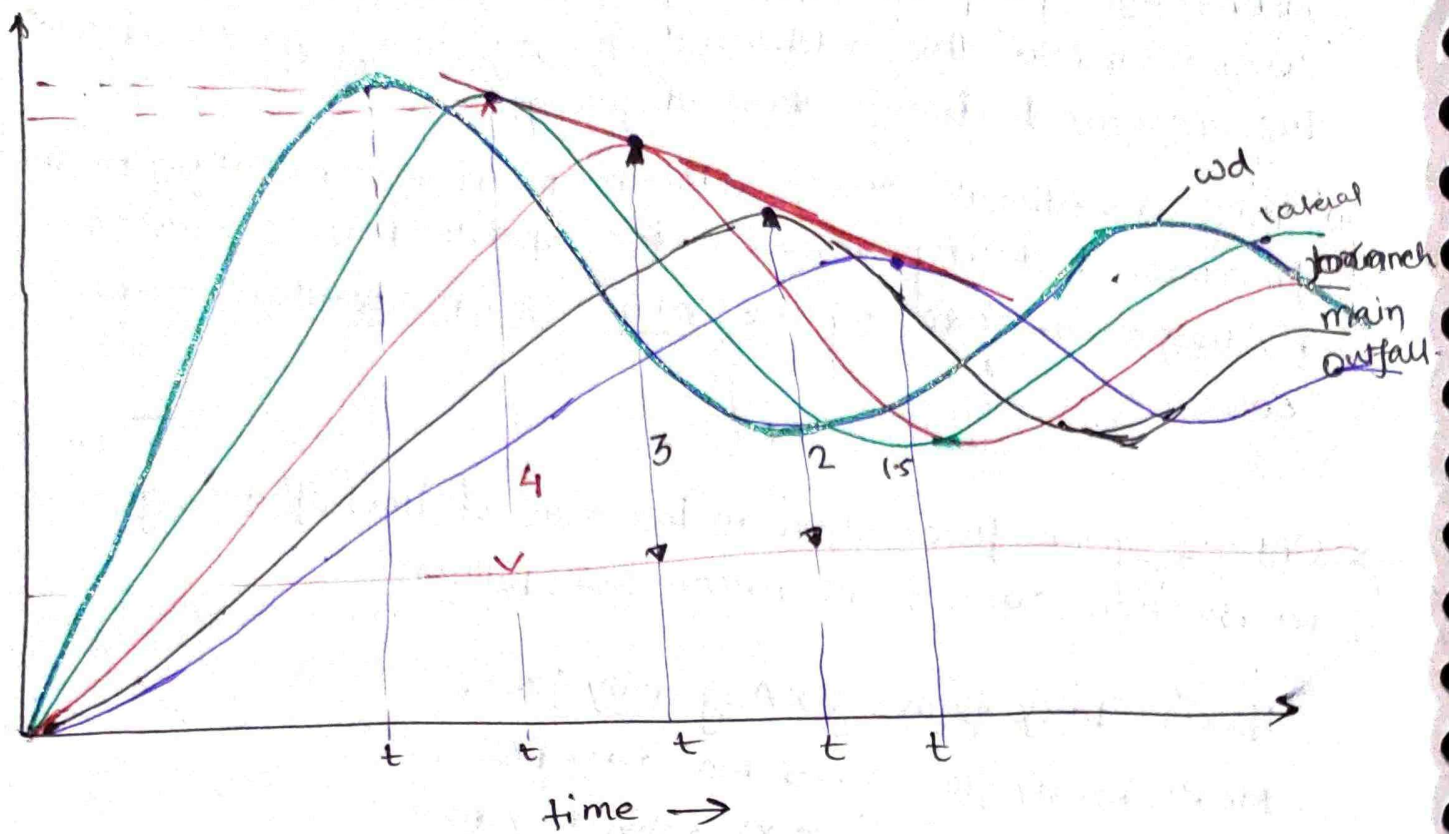
$$\text{Max}^m \text{ hourly flow} = \boxed{3} \text{ Avg daily flow.}$$

$$\text{Minimum daily flow} = \frac{2}{3} \text{ avg. daily flow.}$$

$$\text{Minimum hourly flow} = \frac{1}{2} \times \text{minimum daily flow.}$$

$$= \frac{1}{2} \times \left(\frac{2}{3} \times \text{Avg daily flow} \right)$$

$$\text{Min}^m \text{ hourly flow} = \boxed{\frac{1}{3}} \text{ avg daily flow.}$$



→ Fluctuation in lateral sewer is observed to be maximum & it tapper off from lateral to branch, branch to main, main to outfall sewers due to increase in sample size.

Type of Sewer	(m) diameter	fluctuation
Lateral	0.25	4
Branch	0.50	3
Main	1	2
Outfall	1.25	1.5

Note Fluctuation i.e., $\frac{Q_{\max}}{Q_{\text{avg}}} = \frac{18 + \sqrt{P}}{4 + \sqrt{P}}$

P = population of community in thousand.

Design:

→ Sewers are design to carry max^m hourly discharge and are checked at minimum discharge for development of self cleansing velocity.

→ Self cleansing velocity is that which do not permit the silting of solid in sewer moreover also carry out removal of solid which have already settled in it.

→ Self cleansing velocity for 1mm in organic particle and 5mm organic particles is 0.45 m/s

→ These are the size of particles which are generally present in sewage

→ Self cleansing velocity in sewer is given by

$$V_{sc} = \sqrt{\frac{8K}{f}(G-1)gd} \quad \text{or} \quad V_s = \frac{1}{n} g^{1/6} \sqrt{K(G-1)d}$$

Chezy's equation Manning equation

K = constant which depend on the type of solid in sewage.

f = friction factor.

n = Mannings constant

d = hydraulic mean depth.

→ This velocity must technically develop once in a day to avoid silting of solid in sewer and removal of those solid which have already settled. But for designing purpose it is considered at minimum discharge

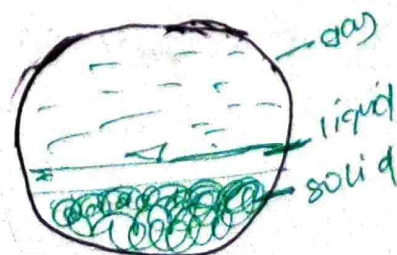
→ The self cleansing velocity of 0.45 m/sec will be attained at minimum discharge if sewer are designed for design flow conditions and velocity at design discharge is in range of 0.8-0.9 m/s.

- Max^m velocity of flow in sewer is also constrained as higher velocity leads to wear and tear (scouring) of sewer material thereby reduces their life span and discharge carrying capacity.
- This limiting or non-scouring velocity mainly depends upon the material of the sewer. as
- The problem of controlling high velocity in sewer mainly rises in hilly areas where available ground slope may be very large or steep.
- Non-scouring velocity in sewer are as follow:-

Sewer material	Limiting velocity (m/s)
→ Vitrified tile & glazed brick	- 4.5-5.5
→ Cast iron	- 3.5-4.5
→ Stone ware	3-4
→ Cement concrete	2.5-3
→ Ordinary brick-lined	- 1.5-2.5
→ Earthen channel	0.6-1.2

✓ Note:- Vitrified tiles & glazed bricks are much more resistant to wear and tear compared to ordinary brick or concrete.

Hence the ordinary brick or concrete sewer are sometime coated with vitrified clay or glazed brick lining at their bottom (where abrasion is max^m due to solid).



* Hydraulic Properties of flow in sewers running partially full and completely full.

(i) Depth of flow

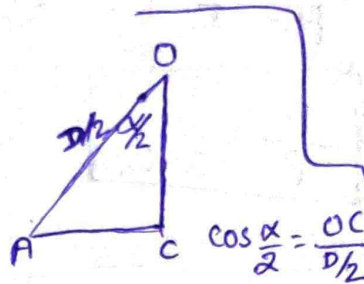
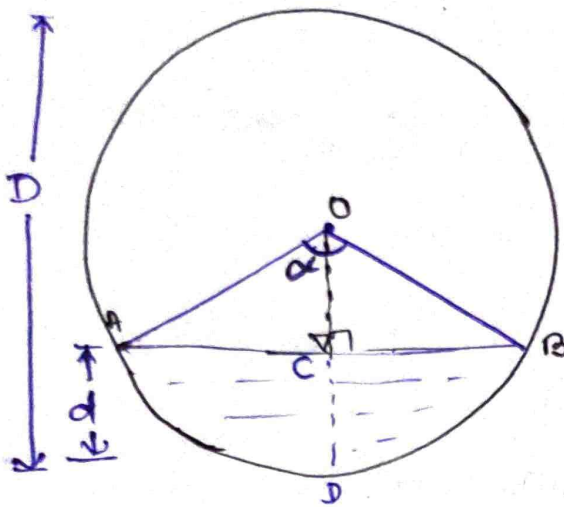
$$= d = CD = OD - OC$$

$$= \frac{D}{2} - OC$$

$$= \frac{D}{2} - \frac{D}{2} \cos \frac{\alpha}{2}$$

$$d = \frac{D}{2} (1 - \cos \frac{\alpha}{2})$$

$$\frac{d}{D} = \frac{1 - \cos \frac{\alpha}{2}}{2}$$



$$\sin \frac{\alpha}{2} = \frac{OC}{D/2}$$

$$OC = \frac{D}{2} \sin \frac{\alpha}{2}$$

$$\cos \frac{\alpha}{2} = \frac{AC}{D/2}$$

$$\sin \frac{\alpha}{2} = \frac{AC}{D/2}$$

(ii) Area of flow

$$Q = \text{area ADBCA} = \text{area ADBO} - \text{Area OAB}$$

$$\text{If } \alpha = 360^\circ, \text{ area OADB } = \frac{\pi D^2}{4}$$

$$\alpha = \alpha^\circ, \text{ area OADB, } Q = \frac{\pi D^2}{4} \times \frac{\alpha}{360}$$

$$= \frac{A\alpha}{360}$$

$$\text{Area OAB} = \frac{1}{2} \times AB \times OC$$

$$= \frac{1}{2} \times \frac{D}{2} \cos \frac{\alpha}{2} \times \frac{D}{2} \sin \frac{\alpha}{2}$$

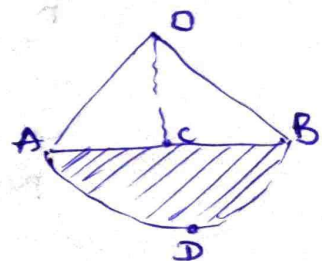
$$= \frac{D^2}{4} \cos \frac{\alpha}{2} \sin \frac{\alpha}{2}$$

$$= \frac{D^2}{8} \sin \alpha$$

$$= \frac{A \sin \alpha}{2\pi}$$

$$\text{area of flow under partial flow condition} = \frac{A\alpha}{360} - \frac{A \sin \alpha}{2\pi}$$

$$\frac{Q}{A} = \frac{\alpha}{360} - \frac{\sin \alpha}{2\pi}$$



(AB = 2AC)

(iii) Perimeter of flow

$p = \text{length ADB.}$

if $\alpha = 360^\circ$, $p = 2\pi r = \pi D$

$\alpha = \alpha$ $p = \frac{\alpha}{360} \times \pi D$

$p = \frac{p \alpha}{360}$

Proportional perimeter: $\frac{p}{P} = \frac{\alpha}{360}$

(iv) Hydraulic Mean Depth of flow, r_h

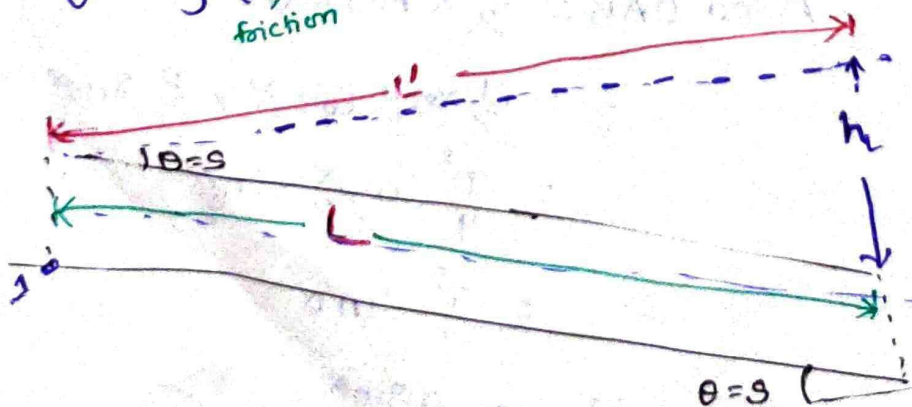
$r_h = \frac{\text{wetted area}}{\text{wetted perimeter}} = \frac{A}{P}$, $R = \frac{A}{P}$

$\frac{r_h}{R} = \frac{\frac{a}{P}}{\frac{A}{P}} = \left(\frac{a}{A}\right) \left(\frac{P}{P}\right)$

$\frac{r_h}{R} = \frac{\frac{\alpha}{360} - \frac{\sin \alpha}{2\pi}}{\alpha/360}$

(v) Velocity of flow.

$v = f(f, r_h, \text{slope})$



$i = \frac{h_L}{L} = \sin \theta$

$S = \frac{h_L}{L} = \tan \theta$

when θ is small
 $\theta = \sin \theta = \tan \theta$.

$$\boxed{i = S}$$

Manning's flow, $v = \frac{1}{n} g^{2/3} \cdot S^{1/2}$ or $V = \frac{1}{N} R^{2/3} \cdot S^{1/2}$

$$\frac{v}{V} = \frac{n}{N} \left(\frac{g}{R} \right)^{2/3} \cdot \left(\frac{3}{5} \right)^{1/2}$$

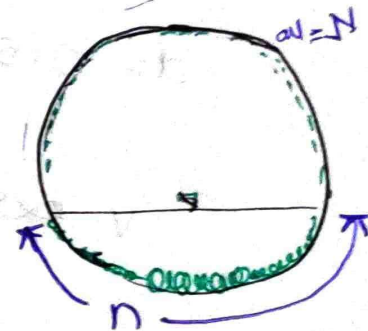
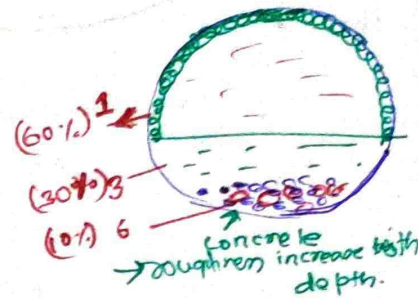
$$[S = S, N \neq n, N < n]$$

Overall roughness observed
 roughness in lower part

we assume, $n = N$

$$\frac{v}{V} = \left(\frac{g}{R} \right)^{2/3}$$

$$\frac{v}{V} = \frac{\left(\frac{\alpha}{360^\circ} - \frac{\sin \alpha}{2\pi} \right)^{2/3}}{\alpha/360^\circ}$$



(vi) Rate of flow,

$$q = a v, Q = A \times V$$

$$\frac{q}{Q} = \left(\frac{a}{A} \right) \left(\frac{v}{V} \right)$$

$$\frac{q}{Q} = \left(\frac{\alpha}{360^\circ} - \frac{\sin \alpha}{2\pi} \right) \left(\frac{\alpha}{360^\circ} - \frac{\sin \alpha}{2\pi} \right)^{2/3}$$

Assuming
 $n = \text{constant}$.

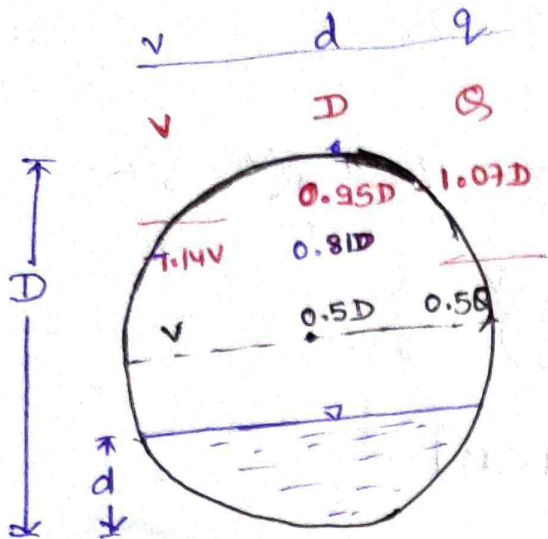
Assume $n = \text{constant}$
if sewer is running half full,

$$\frac{d}{D} = \frac{1}{2}$$

$$\frac{d}{2} = \frac{1 - \cos \alpha/2}{2} = \frac{1}{2} \Rightarrow \alpha = 180^\circ$$

$$\frac{a}{A} = \frac{1}{2}, \quad \frac{b}{P} = \frac{1}{2}, \quad \frac{g}{R} = 1$$

$$\frac{V}{V} = 1, \frac{q}{Q} = \frac{1}{2}$$



$$v = f(f, r, s)$$

$d = 0.5d \uparrow \rightarrow S = \text{constant}$
 $\rightarrow f \downarrow$ (Assume $f = \text{constant}$)

$$V \propto r^2 = \frac{\text{wetfield area} \uparrow \uparrow}{\text{wetfield perimeter} \uparrow}$$

For, $v_{\text{max}} = \frac{d(v/v)}{d\alpha} = 0$

$$\rightarrow \frac{d}{dx} \left[\left(\frac{\frac{x}{360^\circ} - \frac{\sin x}{2\pi}}{x/360^\circ} \right)^{2/3} \right] = 0$$

$$\tan \alpha = \alpha$$
$$\alpha = 256^\circ$$

$$\frac{d}{D} \Big|_{\alpha=256^\circ} = 0.81$$

D

$$a \uparrow, b \uparrow \Rightarrow g \downarrow$$

0.81^D

$$a \uparrow \uparrow, p \uparrow \Rightarrow q \uparrow$$

0.5D

$$\frac{V_{\max}}{V} \Big|_{\alpha=256^\circ} = 1.14$$

$V_{\max} = 1.14V \Rightarrow V_{\max}$ velocity is only 14% more than velocity at full depth of flow.

$$q = av$$

$$q = f[f, \alpha, s, a]$$

$$q \propto a \cdot a = \frac{a}{P} \cdot a = \frac{a^2}{P}$$

$$\text{For } q_{\max} = \frac{d(q/\alpha)}{d\alpha} = 0$$

$$\frac{d}{d\alpha} \left[\left(\frac{\alpha}{360} - \frac{\sin \alpha}{2\pi} \right) \left(\frac{\frac{\alpha}{360} - \frac{\sin \alpha}{2\pi}}{\alpha/360} \right)^{2/3} \right] = 0$$

$$\alpha \approx 308^\circ$$

$$\frac{d}{D} \Big|_{\alpha=308^\circ} = 0.95$$

$$\frac{q_{\max}}{Q} \Big|_{\alpha=308^\circ} = 1.07$$

$$q_{\max} = 1.07Q$$

It means maximum discharge is only 7% more than full discharge.

$$D \text{ ————— } a^2 \uparrow, P \uparrow \uparrow, \Rightarrow \frac{a^2}{P} \downarrow$$

$$0.95D \text{ ————— } a^2 \uparrow \uparrow, P \uparrow \Rightarrow \frac{a^2}{P} \uparrow$$

$$0.5D \text{ ————— }$$

If $\Delta d = 100\%$ ($0.5D - D$)

$$V_{\max} = 1.08 V_{\text{avg}}$$

$$Q_{\max} = 1.25 Q_{\text{avg}}$$

- This is the greatest advantage of circular section that fluctuation in velocity & discharge (8%, 25%, respectively) are least amongst other geometry of section available. Hence sewers are designed to be circular to avoid shock loading (sudden change in flow properties (V, Q) . To avoid shock loading in sewage treatment plant which is disaster for micro-organisms carrying out treatment in the plant.
- This advantage of circular section is available only if depth of flow is greater than half full, hence minimum depth of flow for which circular section design is $0.5D$.
- The above statement ~~are~~ ~~not~~ precisely correct only as long as roughness (n) is supposed to be independent of depth.
- However roughness varies with depth. as much as by 20% or more
- The effect of variation of n is to reduce the proportionate velocity and discharge at lower depth of flow because roughness increases with lower depth.
- If these variation of " n " are also considered more precise values of proportionate velocity & discharge are computed.
- If roughness is also considered, velocity equal to ~~more~~ more than these produced in section flowing full, will be produced so long as the circular section flow 80% of depth as against half the depth when roughness is not considered.

→ However if depth of flow is in between 50% to 80% full, sewer need not be placed on steeper gradient

→ If two sewer have degree of self cleansing then shear stress or tractive stress acting on the sewerage flowing in these sewers is also same.

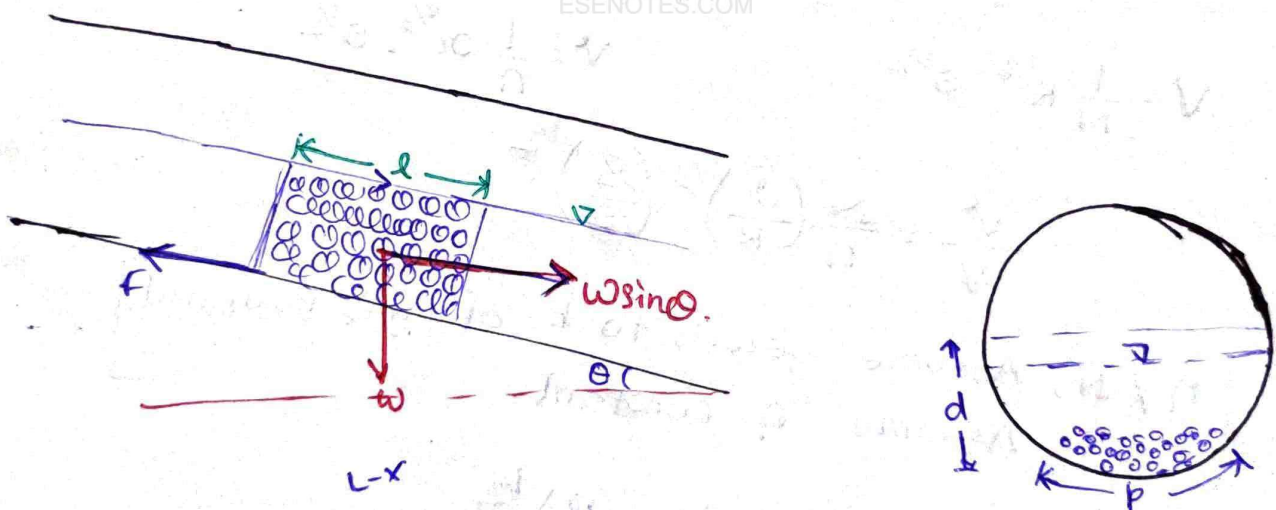
Note:- Shear stress or tractive stress acting on the fluid flowing in open channel is given by

$$\tau = \gamma \cdot R \cdot S$$

γ = unit weight of fluid

R = Hydraulic mean depth of flow.

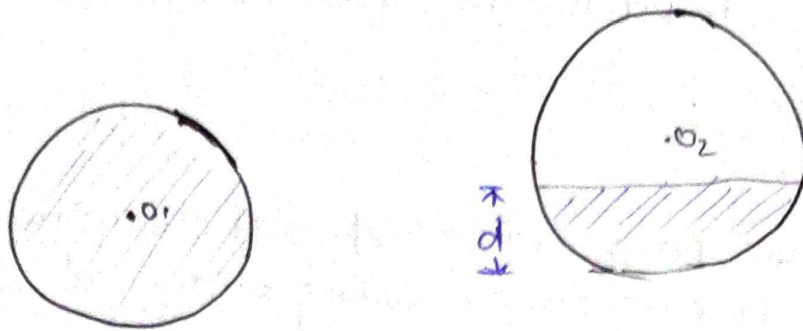
S = slope of channel.



$$\tau = \frac{f}{a} = \frac{W \sin \theta}{a} \approx \frac{W \tan \theta}{a}$$

$$\tau = \frac{\gamma \cdot V \tan \theta}{a} = \frac{\gamma \cdot a \cdot l \cdot S}{p \cdot l} = \gamma \cdot R \cdot S$$

$$\begin{aligned} \tan \theta &= S \\ R &= \frac{a}{p} \end{aligned}$$



* For some degree of self cleansing

$$\boxed{Z = z}$$

$$\gamma \cdot RS = \gamma \cdot r \cdot s$$

($\gamma \neq \gamma$, Assume, $\gamma \approx \gamma \approx \gamma_w$)

$$\boxed{\frac{S}{s} = \frac{r}{R}}$$

$$V = \frac{1}{N} R^{2/3} \cdot s^{1/2}$$

$$v = \frac{1}{n} r^{2/3} \cdot s^{1/2}$$

$$\frac{v}{V} = \frac{N}{n} \left(\frac{r}{R} \right)^{2/3} \left(\frac{s}{s} \right)^{1/2}$$

[$\eta \neq N$, Assume sewers to be of same material.
Assume $\eta = \text{constant}$.]

$$\frac{v}{V} = \left(\frac{r}{R} \right)^{2/3} \left(\frac{R}{r} \right)^{1/2}$$

$$\boxed{\frac{v}{V} = \left(\frac{r}{R} \right)^{1/6}}$$

r = hydraulic mean depth of bigger sewer with partial flow.

R = hydraulic mean depth of smaller sewer with full flow.

$$\frac{r}{R} = \frac{\sin \alpha}{\sin 2\pi} = \frac{\sin \alpha}{2 \sin \alpha \cos \alpha} = \frac{1}{2 \cos \alpha}$$

Note: - All the relation between hydraulic properties of particles & full depth are valid for same sewer

→ But can also be applied to two different sewer in one condition:-

- (i) Sewage in both sewer is same.
- (ii) Material of both sewer is same.
- (iii) n is constant.
- (iv) Sewers are of same size.

Q A 350mm dia sewer is to flow at 0.35m depth on a grade ensuring a degree of self cleansing equivalent to that obtained at full depth at a velocity of 0.8m/sec.
compute

- (a) Required grade (slope) ✓
- (b) associated velocity.

Assume $n = 0.014$

$$\frac{a}{A} = 0.315$$

$$\frac{n}{R} = 0.7705$$

$$\frac{P}{P} = 0.472$$

Solⁿ

$$V = 0.8 \text{ m/s}$$

$$V = \frac{1}{n} R^{2/3} \cdot S^{1/2}$$

$$0.8 = \frac{1}{0.014} \cdot \left(\frac{\pi D^2 y}{4 \pi D} \right)^{2/3} \cdot S^{1/2}$$

$$0.8 = \frac{1}{0.014} \times \left(\frac{0.35}{4} \right)^{2/3} \cdot S^{1/2}$$

$$S = 3.29 \times 10^{-3}$$

If two sewers have same degree of self cleansing

$$\frac{S}{S} = \frac{R}{R} \Rightarrow S = \left(\frac{R}{R} \right) \times S$$

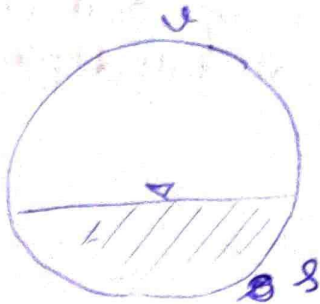
$$= 3.29 \times 10^{-3} \times \frac{0.7705}{0.7705}$$

$$= 4.19 \times 10^{-3}$$

$$\frac{V}{V} = \left(\frac{31}{R}\right)^{1/6}$$

$$V = (0.7705)^{1/6} \times 0.8$$

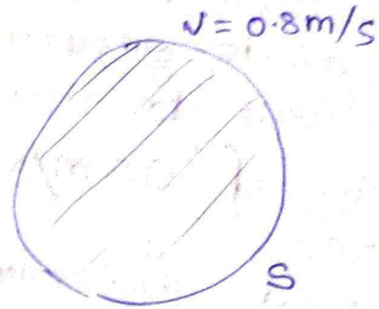
$$= 0.765 \text{ m/s}$$



I



=



III

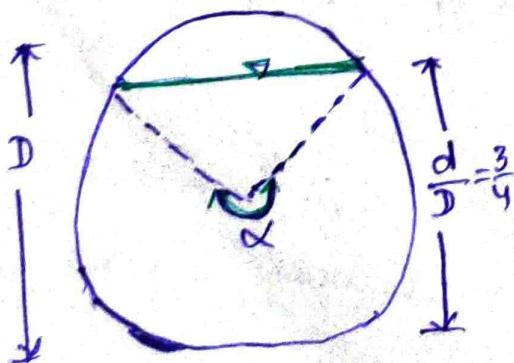
Q Design a sewer running at depth of $\frac{3}{4}$ th full to serve the population of 36000 having water demand of 150 l/c/d at slope of $\frac{1}{600}$ assume $N=0.012$

Design discharge through sewer = $Q_{\max}$ = fluctuation of avg discharge

Assuming sewer to be branch sewer, $F=3$

$$Q_{\max} = 3 \left[\frac{0.8 \times 36000 \times 150 \times 10^{-3}}{86400} \right]$$

$$= 0.15 \text{ m}^3/\text{s}$$



$$\frac{d}{D} = \frac{1 - \cos \alpha/2}{2}$$

$$\frac{3}{4} \times 2 = \frac{1 - \cos \alpha/2}{2}$$

$$\frac{\alpha}{2} = 120^\circ$$

$$\alpha = 240^\circ$$

$$\frac{q_{max}}{Q} = \left(\frac{a}{A}\right) \left(\frac{v}{V}\right)$$

$$= \left(\frac{\alpha}{360^\circ} - \frac{\sin \alpha}{2\pi}\right) \left(\frac{\frac{\alpha}{360^\circ} - \frac{\sin \alpha}{2\pi}}{\alpha/360^\circ}\right)^{2/3}$$

For $\alpha = 240^\circ$

$$\frac{q_{max}}{Q} = (0.80)(0.13) = 0.911$$

$$Q = \frac{q_{max}}{0.911} = \frac{0.15}{0.91} = 0.164 \text{ m}^3/\text{s}$$

$$Q = A \times V$$

$$0.164 = \frac{\pi D^2}{4} \times \frac{1}{N} \cdot R^{2/3} \cdot S^{1/2}$$

$$= \frac{\pi D^2}{4} \times \left(\frac{\pi D^2}{4 \pi D}\right)^{2/3} \cdot S^{1/2} \cdot \frac{1}{N}$$

$$= \frac{\pi D^2}{4} \times \left(\frac{D}{4}\right)^{2/3} \cdot \left(\frac{1}{600}\right)^{1/2} \cdot \frac{1}{0.012}$$

$$0.164 = \frac{\pi}{4} \times \frac{D^{8/3}}{4^{2/3}} \cdot \left(\frac{1}{600}\right)^{1/2} \cdot \frac{1}{0.012}$$

$$D = 0.495$$

$$= \sim 0.5 \text{ m}$$

Check:-

$$q_{min} = \frac{1}{F} q_{avg}$$

$$= \frac{1}{3} \times 0.05 = 0.0167 \text{ m}^3/\text{s}$$

$$\frac{q_{min}}{Q} = \frac{0.0167}{0.164} = 0.1016$$

$$\frac{q_{min}}{Q} = \left(\frac{a}{A}\right) \left(\frac{v}{V}\right)$$

$$0.1016 = \left(\frac{\alpha}{360^\circ} - \frac{\sin \alpha}{2\pi}\right) \left(\frac{\frac{\alpha}{360^\circ} - \frac{\sin \alpha}{2\pi}}{\alpha/360^\circ}\right)^{2/3}$$

$$\alpha = 110.5^\circ$$

$$\frac{V_c}{V} = \left(\frac{\frac{\alpha}{360} - \frac{\sin \alpha}{2\pi}}{\alpha/360} \right)^{2/3} = 0.64$$

$\alpha = 110.5^\circ$

$$V = 0.64 \times V_c$$

$$V = \frac{1}{N} R^{2/3} \cdot S^{1/2} = \frac{1}{N} \times \left(\frac{D}{4} \right)^{2/3} \cdot S^{1/2} = 0.85 \text{ m/s}$$

$$= \frac{1}{0.012} \times \left(\frac{0.5}{4} \right)^{2/3} \times \left(\frac{1}{600} \right)^{1/2}$$

$$V = 0.64 \times 0.85$$

$$= 0.54 \text{ m/s} < 0.45 \text{ m/s}$$

$$= 0.54 > \underline{0.45 \text{ m/s}}$$

Self cleaning velocity for 1mm or 5mm particle.

~~Redesign~~

Q Calculate the ratio of head loss per unit length in sewer running half full and completely full and having same mean velocity.

(a) 0.5

(b) 2

(c) 1

(d) $2^{2/3}$

$$i = \frac{\text{head loss}}{\text{per unit length}}$$

$$\frac{d}{D} = \frac{1}{2} \quad \therefore \quad \frac{V}{V} = 1 \quad ; \quad \frac{Q}{R} = 1$$

$$v = V$$

$$\frac{1}{N} \cdot Q^{2/3} \cdot I^{1/2} = \frac{1}{N} R^{2/3} \cdot I^{1/2}$$

$$\frac{i}{I} = \left(\frac{N}{N} \right)^2$$

so, if n is constant

$$\frac{i}{I} = 1$$

Egg Shaped Sewer

The circular sections are generally preferred to all other shape, of sewer because of following properties.

- (i) They can be manufactured most easily and conveniently.
- (ii) A circular sewer offers maximum area for a given parameter. thus provides the maximum hydraulic mean depth when running full or half full. Hence is the most hydraulically efficient section.
- (iii) Circular section utilises minimum quantity of material and is therefore most economical.
- (iv) It being uniform in curvature all around offers less opportunity of deposits.

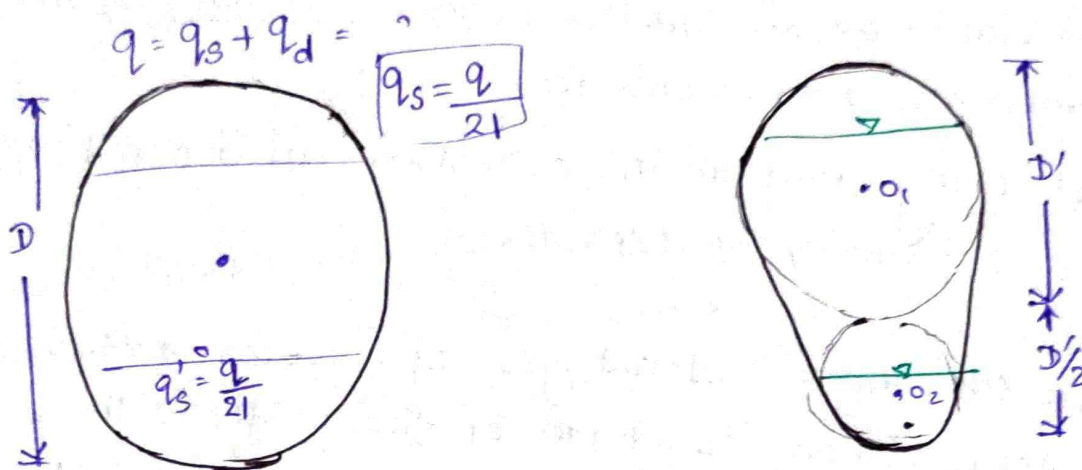
But all these advantages of circular section are available when its depth of flow is more than 0.5D. Hence, these circular sections are suitable to be provided in separate sewerage system where depth of flow remains above $0.5D$ for most part of operation.

→ But in combined sewerage system, where both drainage and sanitary sewage is collected by single sewer, advantage of circular section would be available only during rainy season as for most part of year the sewer would run less than half full.

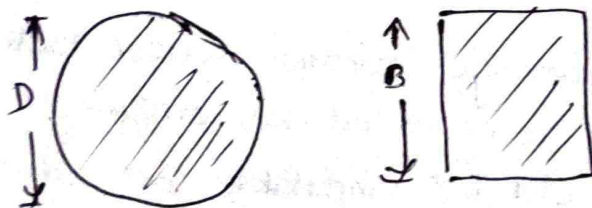
→ Hence in combined sewerage system, in order to take the advantage of circular section through out the year two such circular section are placed one over each other such that upper bigger circular section is effective during rainy season and lower smaller section is effective during dry weather flow, since the section appears like egg in shape it is termed as "egg shaped Sewer".

Note! If two sections are hydraulically equivalent then they carry same discharge while running full on the same grade & are of same material.

→ If a circular section of diameter "D" is hydraulically equivalent to Egg shaped section of total height " $\frac{3}{2}D'$ " then $D' = 0.84D$



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$$Q_o = Q_{\square}$$

$$A_o V_o = A_{\square} V_{\square}$$

$$\frac{\pi D^2}{4} \times \frac{1}{N} \times \left(\frac{D}{4}\right)^{2/3} s^{1/2} = B^2 \cdot \frac{1}{N} \left(\frac{B}{4}\right)^{2/3} \cdot s^{1/2}$$

$$D = 1.1B$$

2 Design the sewer for following data

$$P = 70000$$

$$APWP = 150 \text{ l/c/d}$$

$$N = 0.012$$

$$S = \frac{1}{500}$$

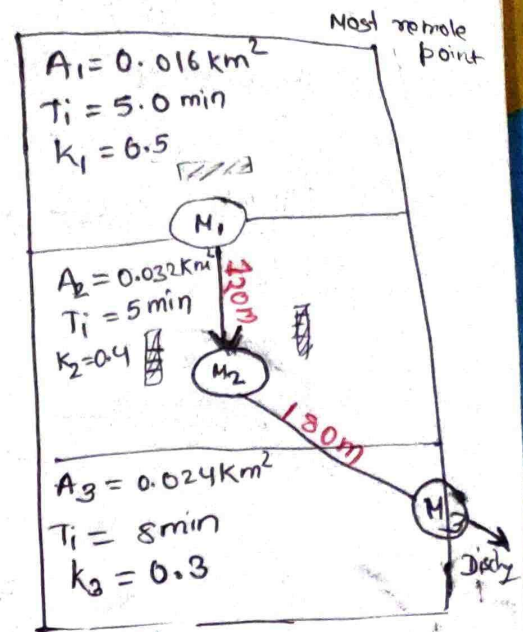
Assume any data not given.

Design sanitary sewage discharge:-

$$Q_{ss} = F Q_{avg}$$

$$Q_{ss} = 3 \times \left[\frac{0.8 \times 150 \times 70000 \times 10^{-3}}{86400} \right]$$

$$= 0.29 \text{ m}^3/\text{sec.}$$



Design drainage discharge, $Q_D = \frac{1}{36} k P A$ (m³/s)

coefficient of runoff $\frac{\text{runoff}}{\text{rainfall}} < 1$

critical design rainfall intensity = cm/hr

$P_c = f(t_c)$

t_c = time of concentration = time taken by water from remotest point to reach up to concerned section.

$$t_c = t_i + t_f$$

t_i = Inlet time;

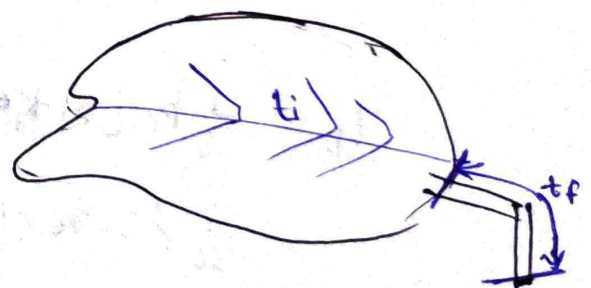
t_f = flow time.

If $t_c = (5-20) \text{ min}$

$$P_c = \frac{75}{10 + t_{c \text{ min}}} \text{ (cm/hr)}$$

if $t_c = (20-100) \text{ min}$

$$P_c = \frac{100}{20 + t_c} \text{ (cm/hr)}$$



For area 1

$$t_i = 5 \text{ min}, \quad t_f = \frac{L_1 + L_2}{V_f} = \frac{120 + 180}{V_f}$$

$$\text{Assume } V_f = 0.75 \text{ m/sec}, \quad t_f = \frac{300}{0.75 \times 60} = 6.67 \text{ min}$$

$$t_c = t_i + t_f = 5 + 6.67 = 11.67 \text{ min}$$

For Area-2

$$t_i = 5 \text{ min}, \quad t_f = \frac{L_2}{V_f} = \frac{180}{0.75 \times 60} = 4 \text{ min}$$

$$t_c = 5 + 4 = 9 \text{ min}$$

For Area-3

$$t_i = 8 \text{ min}, \quad t_f = 0$$

$$t_c = 8 + 0 = 8 \text{ min}$$

t_c for peak discharge, $t_c = 11.67 \text{ min} < 20 \text{ min}$,
 $p_c = \frac{75}{10 + 11.67} = 3.46 \text{ cm/m}$

$$Q_D = \frac{1}{36} p_c [A_1 K_1 + A_2 K_2 + A_3 K_3]$$

$$= \frac{1}{36} \times 3.46 \times [0.5 \times 0.016 + 0.4 \times 0.032 + 0.3 \times 0.024] \times 10^6 \times 10^{-4}$$

$$= 0.268 \text{ m}^3/\text{s}$$

$$\text{Total discharge} = 0.29 + 0.268 \\ = 0.558 \text{ m}^3/\text{sec}$$

Assuming sewer to be running full,

$$Q = 0.5587 \text{ m}^3/\text{s}$$

$$Q = A \times V$$

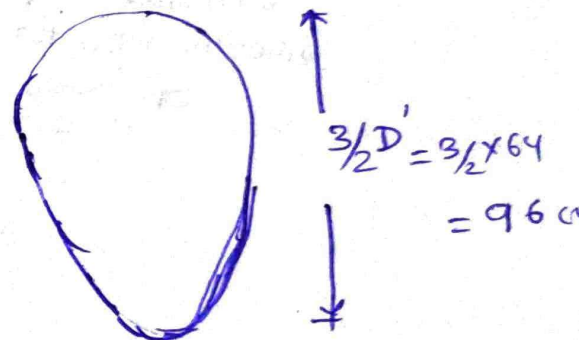
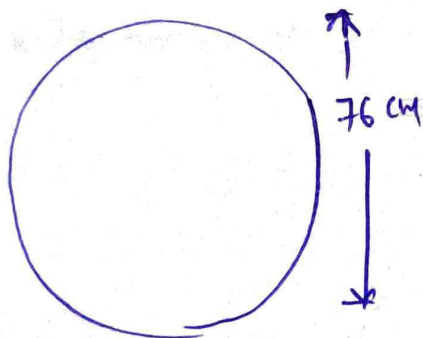
$$0.5587 = \frac{\pi D^2}{4} \times \frac{1}{N} \left(\frac{D}{4} \right)^{2/3} \cdot S^{1/2}$$

$$0.5587 = \frac{\pi D^2}{4} \times \frac{1}{0.012} \times \left(\frac{D}{4} \right)^{2/3} \cdot \frac{1}{500}$$

$$D = 76 \text{ cm.}$$

This circular section is hydraulically equivalent to an egg shaped section, of total height $\frac{3}{2}D'$

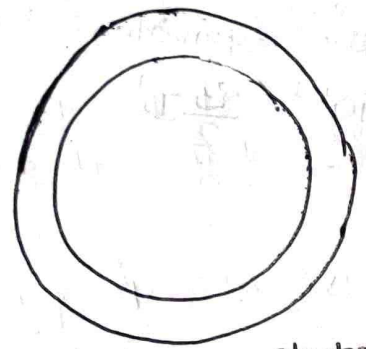
$$\begin{aligned} \text{where, } D' &= 0.84 \times D \\ &= 0.84 \times 76 \\ &= 64 \text{ cm.} \end{aligned}$$



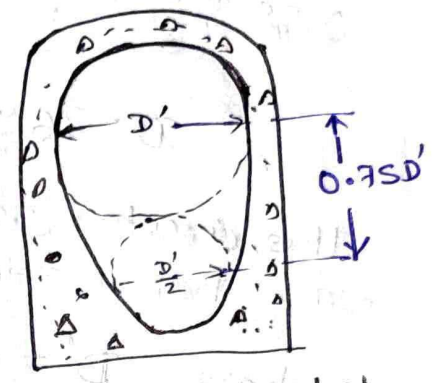
Sewer, its construction, Maintenance and its appostenance

(i) Shape of Sewer

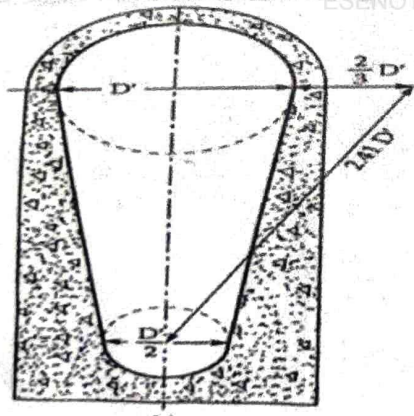
→ The sewer pipes are normally designed as circular in section. in some cases section provided is egg shaped also, however other geometry of section is also available to be used in special cases.



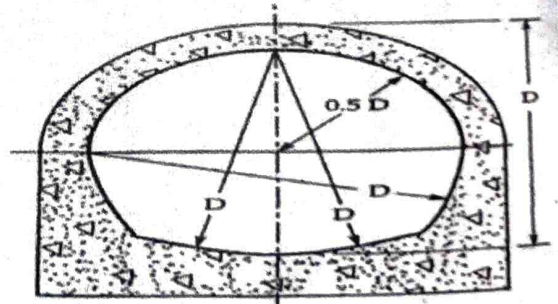
Circular shaped sewer
(Mostly used for all type of sewer)



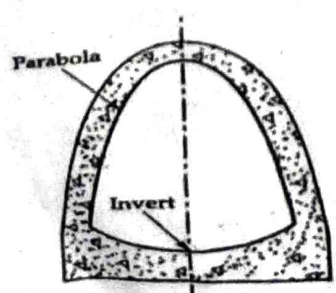
Standard Egg shaped
(mostly used for combined sewer).



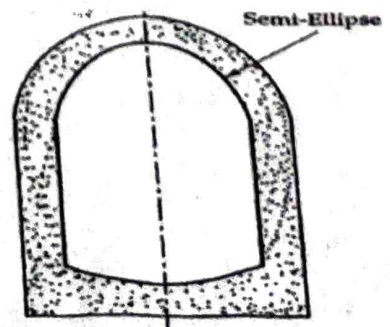
(c)
New Egg-shaped sewer
(may be preferred for combined sewers)



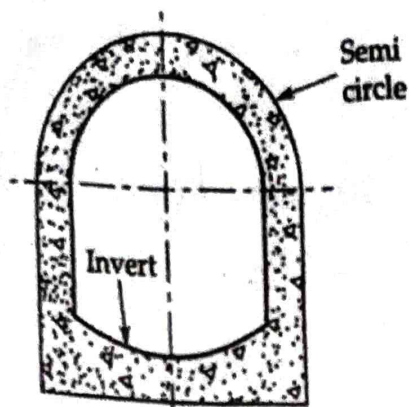
(d)
Horse shoe shaped sewer
(may be used for large sewers with heavy discharges, such as for Trunk and Outfall sewers)



(e)
Parabolic shaped Sewer
(may be used for carrying comparatively smaller quantities of sewage.)

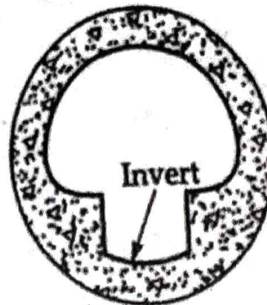


(f)
Semi-Elliptical Section
(may be used for soft soils as it is more stable, but useful only for carrying large amounts of sewage, in sewers greater than about 1.8 m in a diameter.)

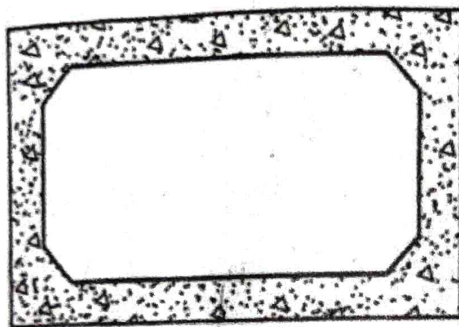


(i)
Semi-circular shaped sewer
(out-dated)

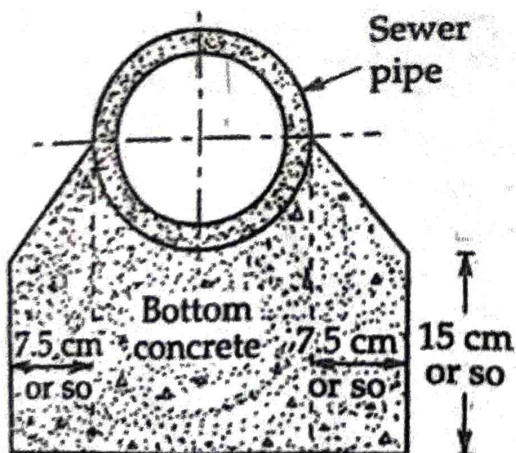
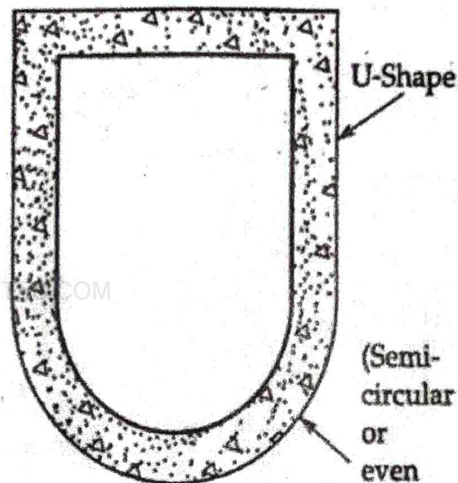
U-shaped section
(may be used for larger sewers
and especially in open cuts)



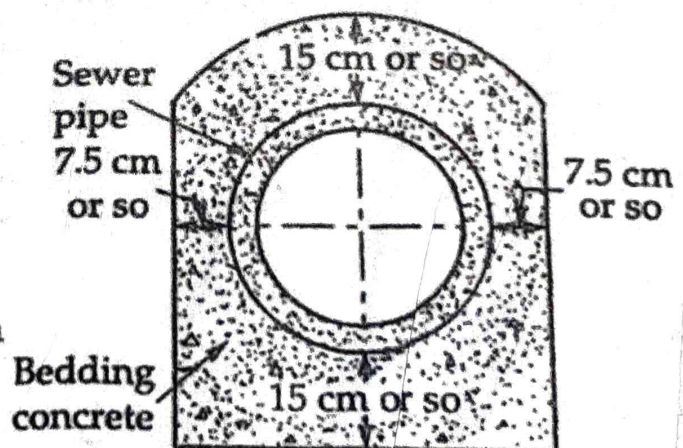
(j)
Basket handle shaped sewer
(out-dated)



(g)
Rectangular shaped section
(generally used for covered
storm water drains)



(a) Bedding of sewers
in ordinary soils



(b) Bedding of sewers
in softer soils

Forces Acting on Sewers.

(i) Internal Pressure.

- The pressure exerted by the sewage from inside the pipe when running full is called internal pressure.
- Such a pressure may be exerted due to some faulty surcharge or due to overflow of the sewer flowing under gravity, or it may be exerted in outfall sewer which are designed to run at full depth.
- In most of the cases since flowing sewer is under gravity the internal pressure is not acting.

(ii) Pressure due to External load.

- Sewer pipes are mostly laid below the ground and placed in trenches and are backfilled. The weight of the pipe, wt of backfill & superimposed load due to traffic if any will be transferred over the pipe. which would produce compressive stresses.

(iii) Flexural Stress

Sometimes the sewer pipes have to be carried and supported between the restles, piers like beams or in some case rain water may wash off the ground from below the pipe resulting in the pipe being exposed like beams.

- In such cases flexural stresses are also developed in the pipes.



(iv) Temperature Stress.

When pipes are laid ~~below~~ ^{above} the ground they are exposed to atmosphere and are therefore subjected to temperature changes.

→ They expand during day time and contract during night time.

→ If expansion and contraction is prevented due to fixation or friction over the support it results in development of temperature stresses.

Sewer Material

To decide the sewer material following factors must be considered :-

- (i) Resistance to Corrosion.
- (ii) Resistance to abrasion/wear & tear.
- (iii) Should be strong & durable.
- (iv) Should be light weight (to easy to transport).
- (v) Should be impervious.
- (vi) Should be cheap.
- (vii) Should be hydraulically efficient.

Different types of sewer Material Used are as follows: -

I) ASBESTOS CEMENT SEWER

→ Asbestos Cement pipes are manufactured from asbestos ~~sheet~~ fibre, silica & cement, mixed & converted under pressure to a dense homogenous material, possessing sufficient strength.

→ These sewers are ⁽ⁱ⁾ light in weight and easy to transport

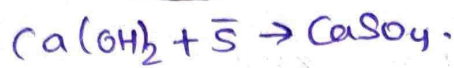
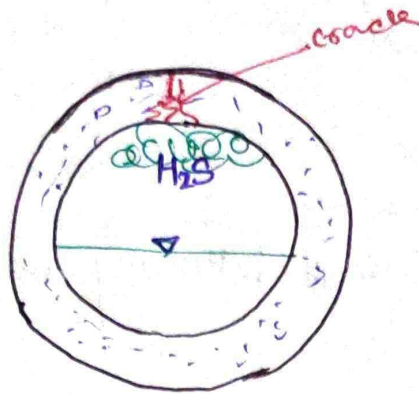
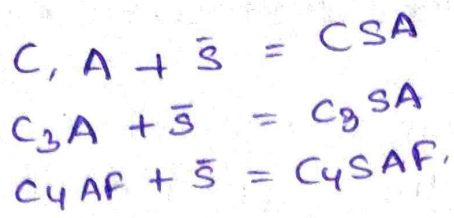
(ii) Can be easily cut and assembled without skilled labour

(iii) Are hydraulically efficient.

(iv) Are not strong enough to bear heavy compressive load.

(v) Are susceptible to hydrogen sulphide ^(H₂S) corrosion at crown.

Due to these drawbacks of AC sewer they are used as verticals or to carry less foul sewage from kitchen and bathroom.



- (ii) Plain cement Concrete or Reinforced Cement Concrete Sewer.
- PCC pipes are manufactured for small size upto 0.45 m diameter while they are reinforced with steel for larger diameter sewers.
 - These sewers are strong in tension as well as in compression hence it take both high internal pressure and overburden pressure.
 - These pipes are resistant to erosion and abrasion.
 - They can be easily moulded and manufactured either at site or in casting yard.
 - They can be made up of any desired strength by proper designing.
 - They prove economical ~~and~~ in medium and large size hence are widely used for branch and main sewer.
 - ^{weight} These pipes are susceptible to H_2S corrosion at crown.
 - In order to avoid hydrogen sulphide corrosion following major can be adopted: -

- (i) Lining the exterior surface with vitrified clay.
- (ii) Prohibiting the entry of waste containing sulphide.
- (iii) Reducing sulphate content by pre-treatment.
- (iv) Aerating and chlorinating the sewage.
- (v) By adequately ventilating the sewer.
- (vi) By making sewer to run full.
- (vii) By adding such chemical to sewage which neutralises already present sulphur compound.

Note:- If a thin steel shell pipe is applied with concrete to increase its resistance against corrosion is termed as "HUME STEEL PIPE".

(iii) Vitrified clay Pipe or/stoneware/Salt Glazed ~~Pipe~~ Sewer

- These pipes are widely used for carrying the sewage and drainage as house connections as well as lateral pipes sewer.
- These pipes are manufactured from clay and shales of special qualities which are first pulverised and mixed thoroughly with water.
- This mixture is then used for casting standard pipe by heating it at high temperature ($T \approx 1200^\circ\text{C}$) where vitrification of clay takes place.
- These pipe offers maximum advantage of being highly resistant to sulphide corrosion.
- Their interior are very smooth, therefore ~~big~~ offer very high discharge carrying capacity.
- They are highly impervious.
- They are though weak in tension but are strong in compression.

- Hence can be used to carry sewage under gravity but not under pressure.
- These are economical, durable and easily available.
- They are made of non-absorbant material hence is resistant against action of chemicals and salt.
- They are heavy, bulky and brittle, difficult to transport hence are cased smaller in size. therefore can't be used for branch and main sewer.

(ii) Brick Sewers

- Brick sewers were used in the earlier days which is now being replaced by cement concrete sewers.
- They are still used at the places where the raw ingredient required for manufacturing of concreting is not available.
- They are preferred for constructing the combined sewers.
- These sewers are generally plastered on their outer surface & coated with vitrified clay / ceramic block from inside to increase the resistance against sulphide attack.

(iii) Cast-Iron Sewers

- These pipes are structurally strong & stable and capable of withstanding both tensile and compressive stresses upto reasonable extent.
- These are ~~cost~~ comparatively costly than concrete or stone ware pipes.
- They are used for manufacturing of outfall sewers or sewers to be laid below heavy overburden pressure.

- They are also used for sewer to be carried over trestles.
- They are 100% leak proof hence they are used where Contamination of nearby water pipe is to be avoided.

(vi) LEAD SEWER

- These pipes are smooth and can be bent to take odd shape.
- They are not effected by acid or alkaline sewage & can resist H_2S Corrosion.
- They are very costly. hence are manufactured in small sizes.
- They are used as downtake pipe of flushing cisterns or waste pipe from wash basin or gysers connections.

(vii) Plastic Sewer

- The use of plastic sewers can be done upto dia of 1.5m for carrying sewage.

Laying & testing of Sewers

→ All sewer pipes are generally laid starting from their outfall end (tail end) towards their starting end (Head End) as starting from tail end gives the advantage of utilisation of tail length even during the initial period of its construction. thus, insuring the functioning of sewerage system without waiting for its completion.

→ Laying of the sewer is carried out in following sequence of operation:→

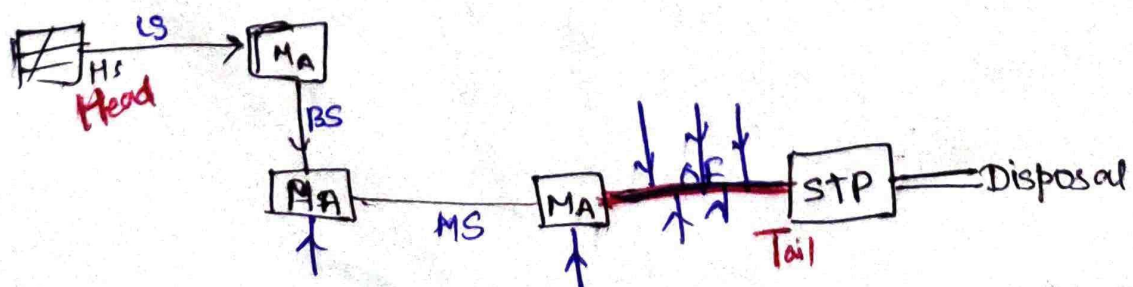
① Marking the center-line.

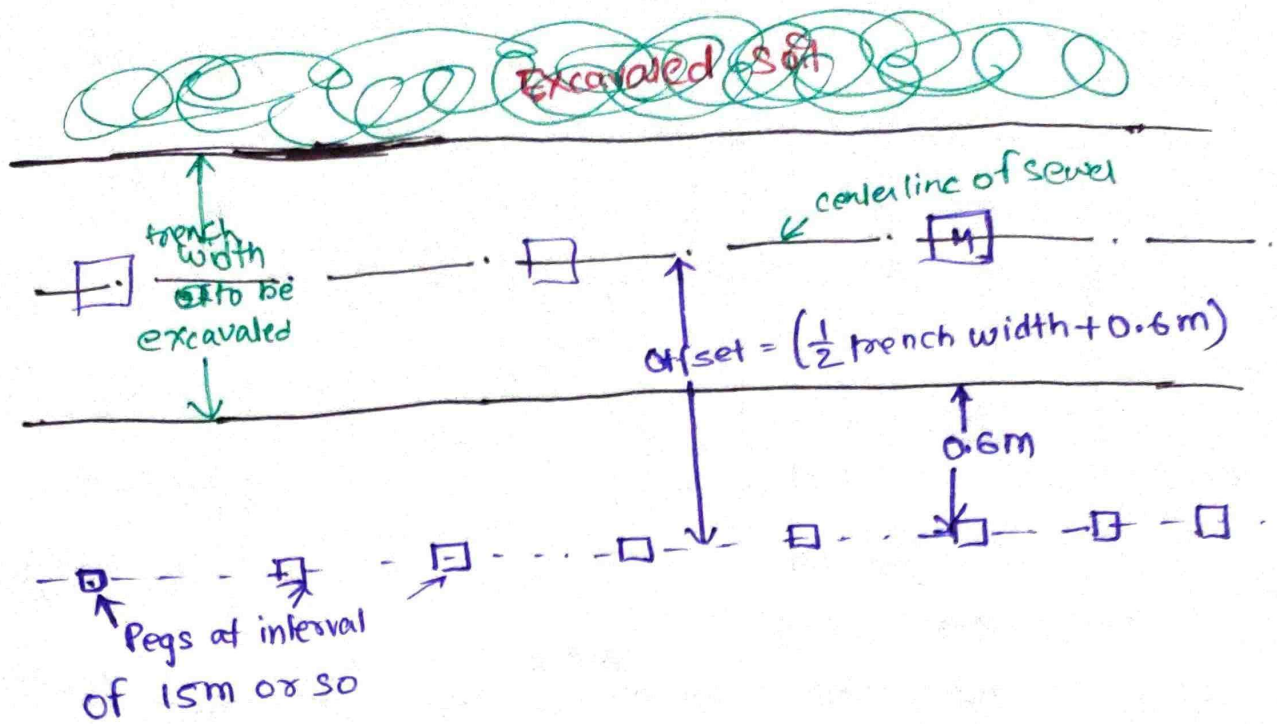
Marking the center line of sewer is done by locating the points where manholes are required to be constructed and then laying of pipes between the two manholes is done.

→ It is done by offset line method in which offset line on the ground is drawn parallel to the center line of sewer.

→ This offset line, usually is offsetted from the center line of sewer by a distance of (half of trench width + 0.6m) and pegs are marked on the ground at a interval of 15m on this line.

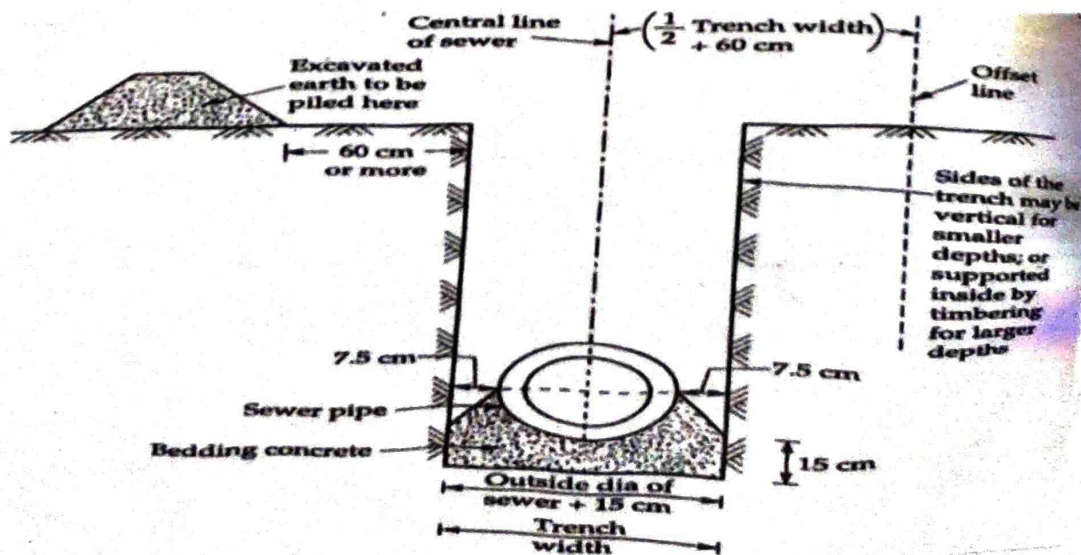
→ This offset line is marked on that side of center line where excavated soil is not to be disposed off.





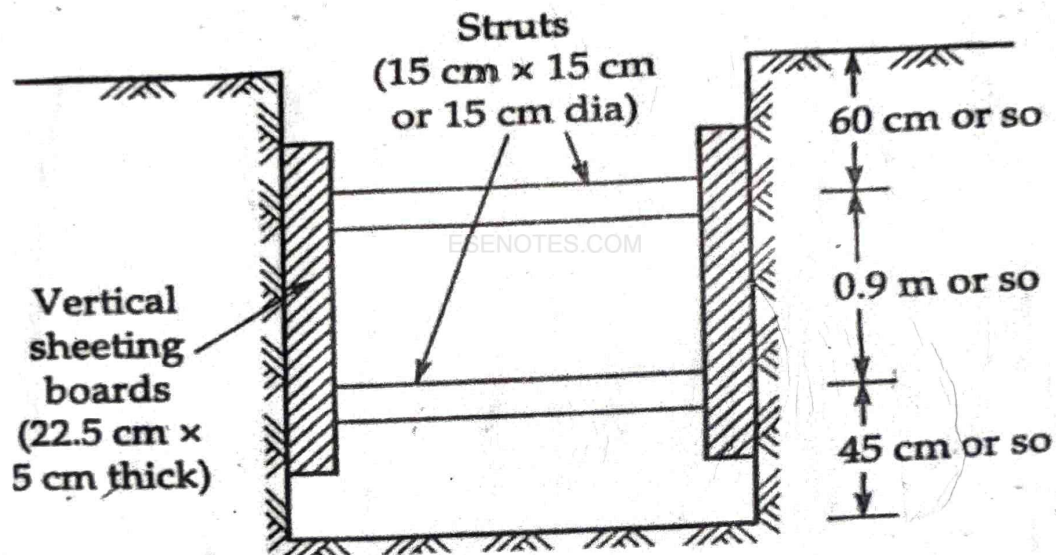
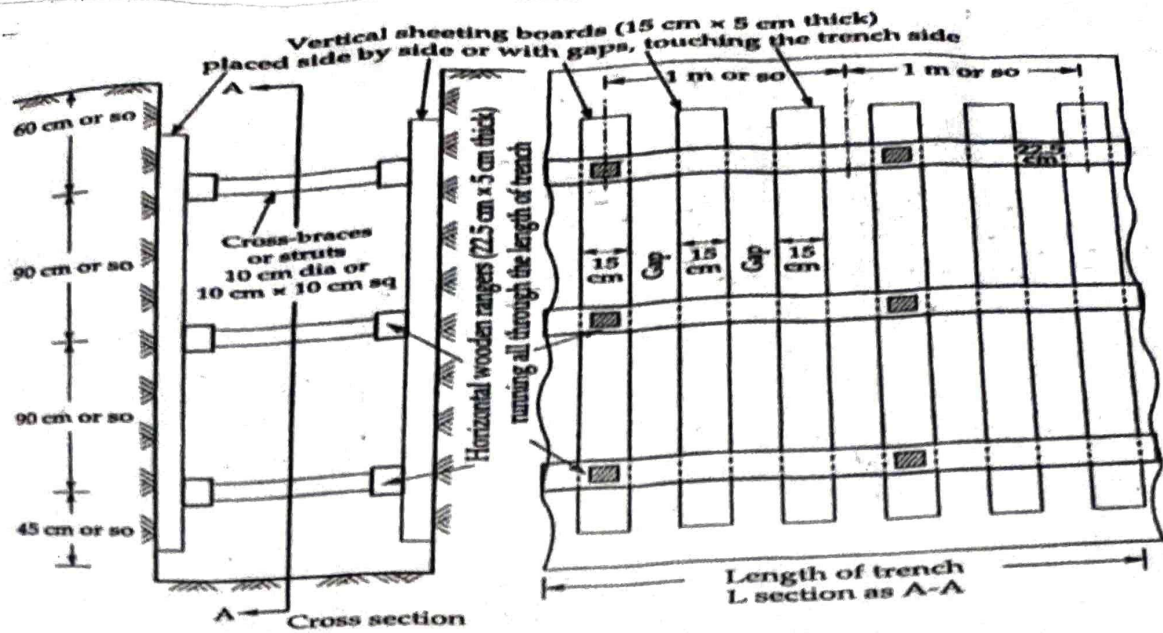
(ii) Excavation of Trench.

- Once center line is marked excavation work is carried out either manually or mechanically for laying the sewer.
- Width of the trench is kept 15cm more at the bottom than the diameter of sewer, so as to facilitate laying & joining of sewer.
- Depth of trench must be upto bottom of the embedded concrete.



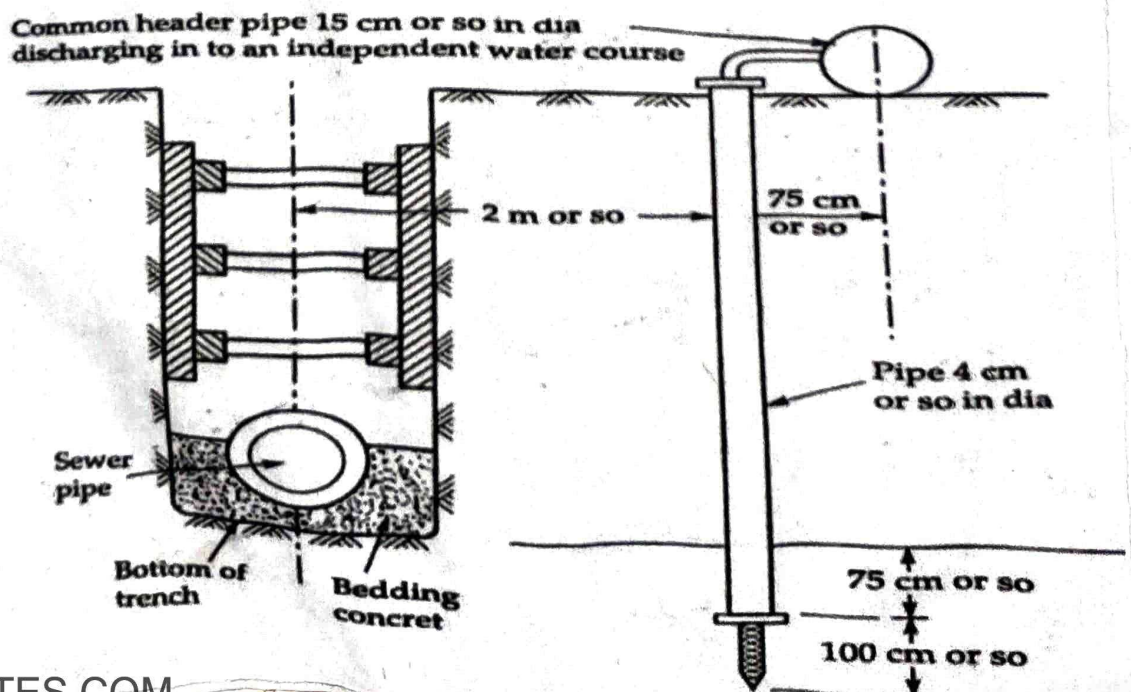
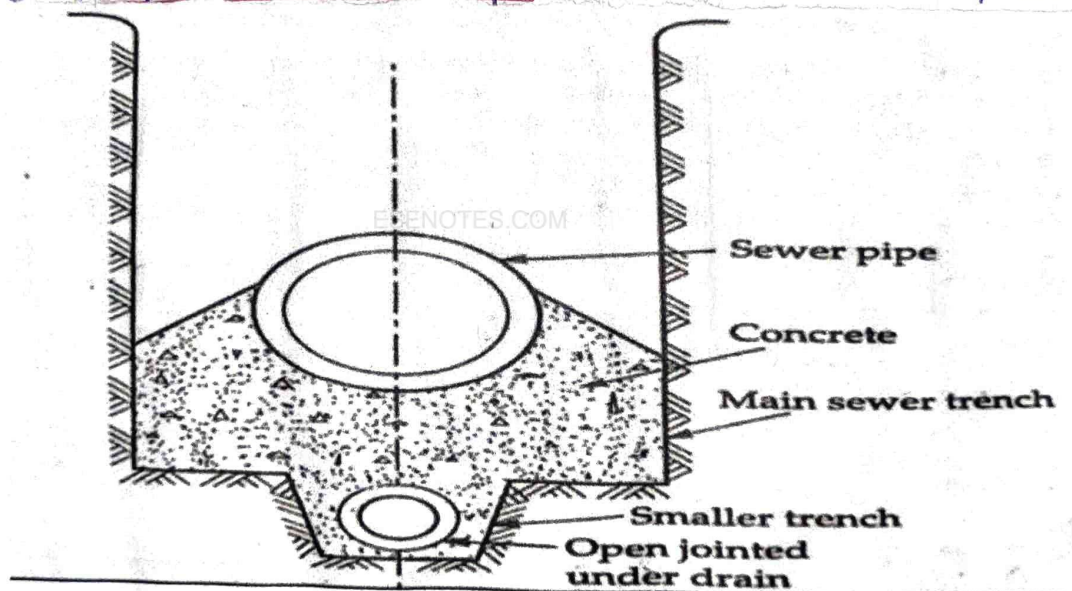
(iii) Timbering or sheeting the trenches:

- The trenches are excavated with vertical or sloping sides keeping in view the fact that soil may collapse or fall in.
- Hence when trenches are excavated in depth more than 2m or in ordinary soil it is necessary to support the soil by sheeting and ^(bracing) timbering the trenches.
- The sheeting boards are the wooden pieces of plank which are placed in actual contact with the trench side, either horizontally or vertically.
- Braces are the cross-wooden pieces extending from one side of trench to the other side and may also be called as struts.
- The rangers are the timber planks which transfer the load from the sheeting board to the struts.

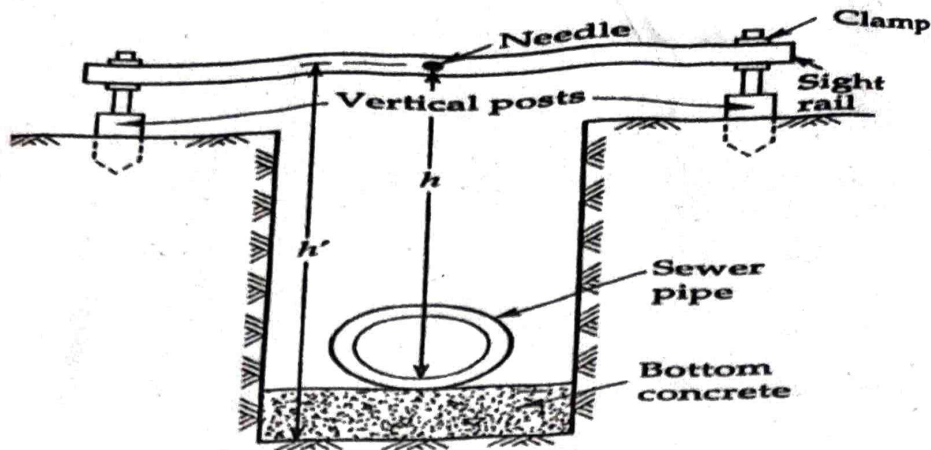


(iv) Dewatering of trench.

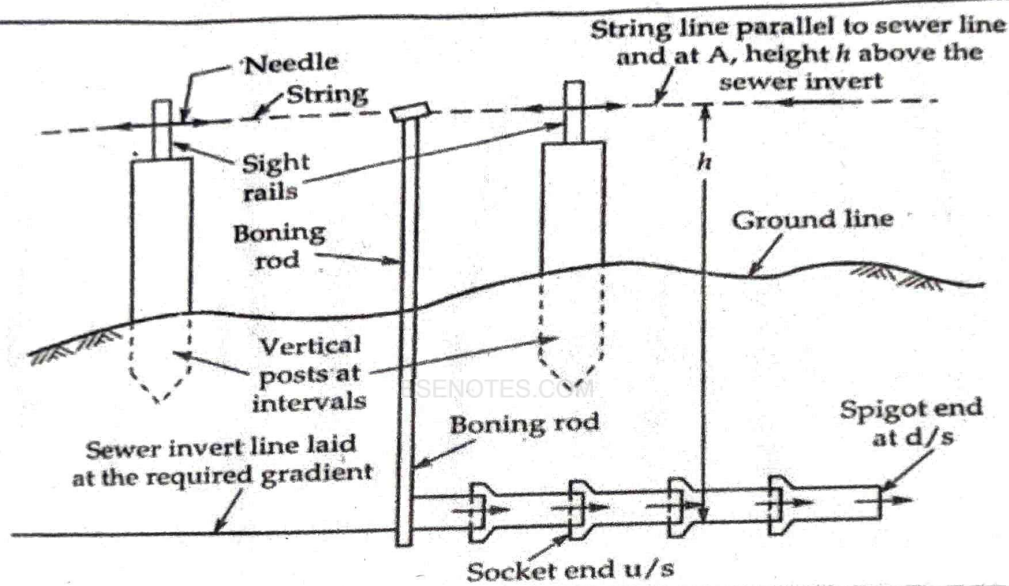
- While excavating the trench water may enter into it, either when excavation is too deep or ground water table level is high.
- This water creates the problem in further ~~excavation~~ excavation and laying of sewer.
- This water may also seep into the sewer there by increases the load over it.
- Hence, to avoid this, water must be removed from the trench. either by open jointed under drain which meet some surface source of water, or pumping the water by excavating ~~some~~ a sump in trench or with the help of well point discharge in case of sandy soil.



SHAPING THE TRENCH BOTTOM



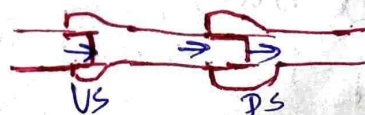
LAYING OF SEWER PIPES



Laying of sewer:-

After the bedding concrete has laid into position, the sewers are lowered in trench either manually (for small sewers) or mechanically (for large sewer)

- The sewer pipe lengths are usually laid from the lowest point with the socket end facing upstream as in this way the spigot end of each pipe can be easily inserted in the socket of already laid pipe.
- After fitting the socket of spigot into each other the proper jointing is carried out.

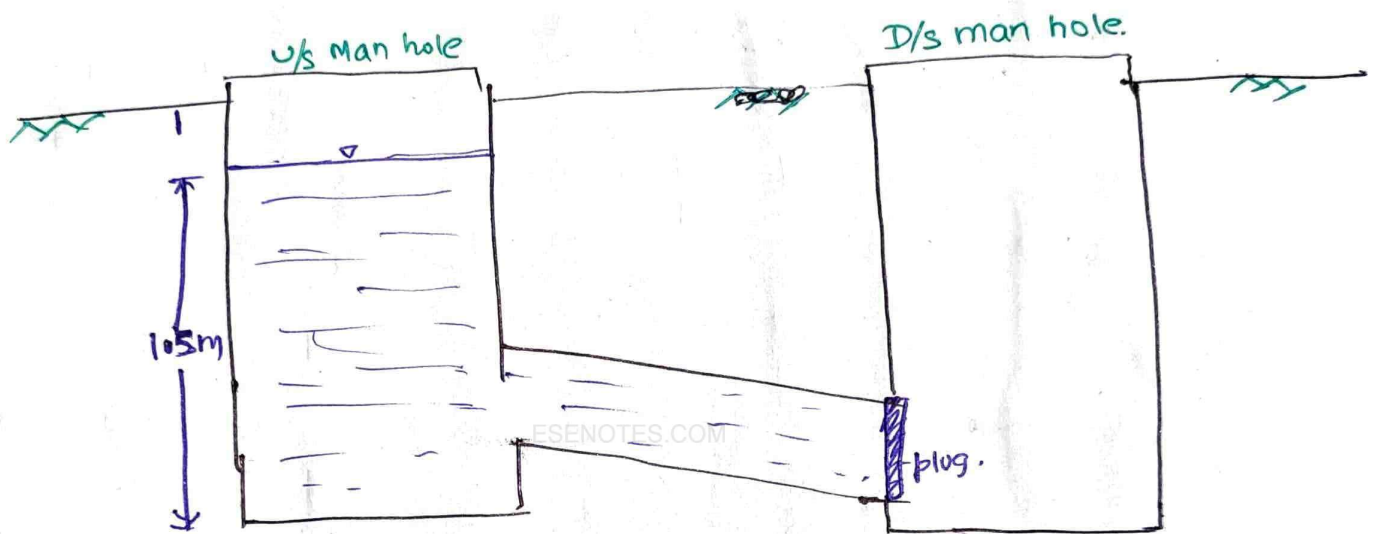


Testing of Sewers

(i) Test for leakage (water test)

- The sewers are tested so as to ensure no leakage through their joints after giving sufficient time to these joint to set in.
- for this purpose the sewer pipe section are tested between manhole to manhole under a test pressure of 1.5m of water head.

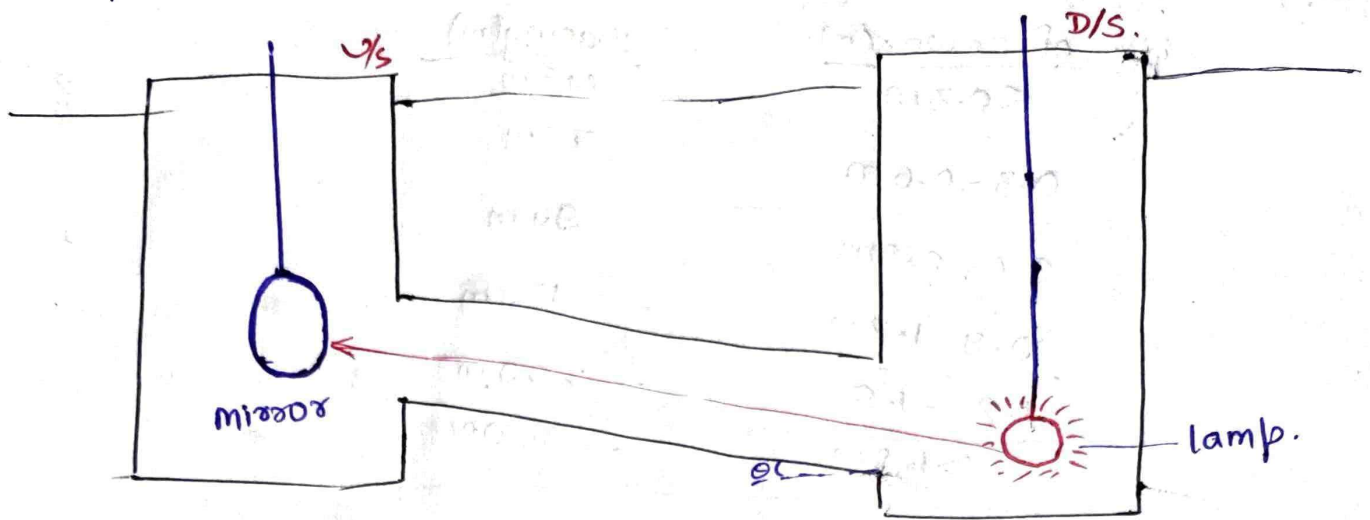
→



(ii) Test for straightness of alignment and obstruction.

- The straightness of the sewer pipe can be tested by placing a mirror at one end of the sewer line and a lamp on the other end.
- if the pipeline is straight the full circle light will be observed. however if the pipe line is not straight this would be apparent not full circle and mirror will also indicate any obstruction in the pipe.
- Any obstruction present in the pipe can also be tested by inserting a ball/yarn of diameter 19mm smaller than the diameter of the sewer. at upper end, if this ball

rolls down to the other end there is no obstruction in pipe.



Sewer Appurtenances:

→ Sewer Appurtenances are those structure which are constructed at suitable interval along a sewerage system which helps in efficient operation and maintenance.

→ Sewer Appurtenances includes :-

(i) Manhole.

→ These are masonry or RCC chamber constructed at suitable interval along the sewer line for providing access into them.

→ These manholes are provided at the junction of two sewers and helps in their inspection, cleaning and maintenance. They are also provided in situations where ~~some~~:-

(i) ~~Size~~ of sewer changes.

(ii) Slope of sewer changes.

(iii) alignment of sewer changes.

(iv) for joining two different type of sewer.

→ As per IS 1742, Manholes spacing generally adopted on straight sewer reaches are:-

Size of sewer (m)	Spacing (m)
≤ 0.3 m	45 m
0.3 - 0.6 m	75 m
0.6 - 0.9 m	90 m
0.9 - 1.2 m	120 m
1.2 - 1.5 m	250 m
> 1.5 m	300 m

→ As per IS - 411 (Part-1), spacing is recommended are as follows.

Size of sewer (mm)	Spacing (m)
900 - 1500	90-150
1500 - 2000	150-200
> 2000	300

→ Allowance of 100m for every 1m dia of sewer is general rule in case of very large sewer.

→ Manholes can be ^{classified} as follow :-

- (i) Shallow : depth - 0.7 - 0.9 m
- (ii) Normal : ~~normal~~ depth upto 1.5 m
- (iii) Deep : depth more than > 1.5 m

→ Covers of manholes are generally designed to be circular because of following reasons:-

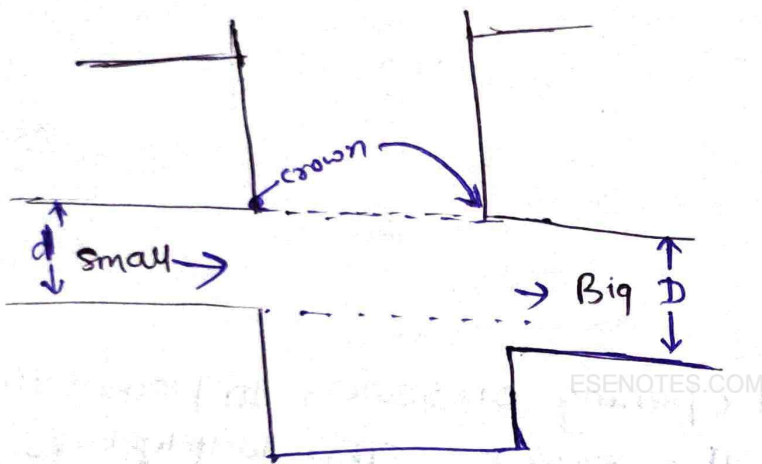
- (i) Round shape are easier to manufacture
- (ii) Round shape manholes are easy to roll in order to carry them from one point to another.
- (iii) Round covers can easily slip into place and need not be lined up with the opening of manhole

(iv) Round Cover manhole can't fall through the circular opening.

(v) Circular shape can resist compressive pressure from the surrounding uniformly.

(vi) It can't be unlocked easily.

→ In manholes crown of smaller incoming sewer is always matched with bigger outgoing sewer in order to avoid chocking of smaller incoming sewer in case of backing up of bigger outgoing sewer.



(ii) Drop Manhole

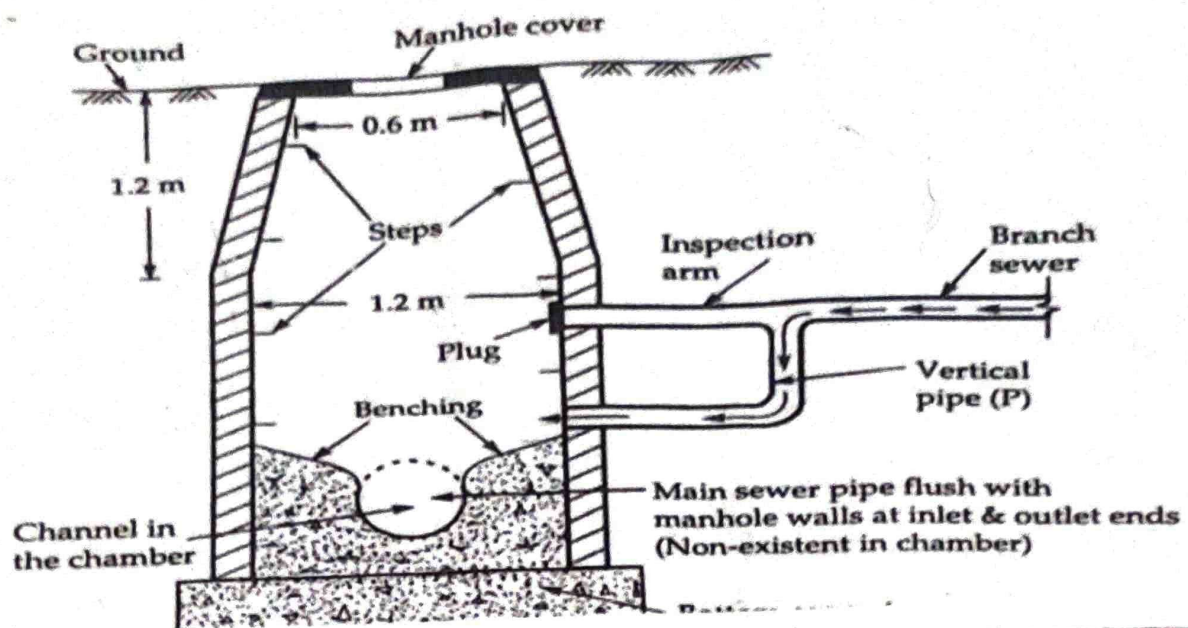
→ If sewer enters a manhole by more than 0.5 to 0.6m above the other ~~sewer~~ outgoing sewer the sewage is generally not allowed to fall directly into the manhole but is brought into it through a down pipe taken from incoming sewer, to the bottom of manhole.

→ If the drop is only of few meters, the down pipe can be kept sloping at an angle of 45° (ramp) & if the drop is more vertical pipe is found economical. (drop manhole)

→ The height of fall of sewage is reduced in order to avoid separation of solids

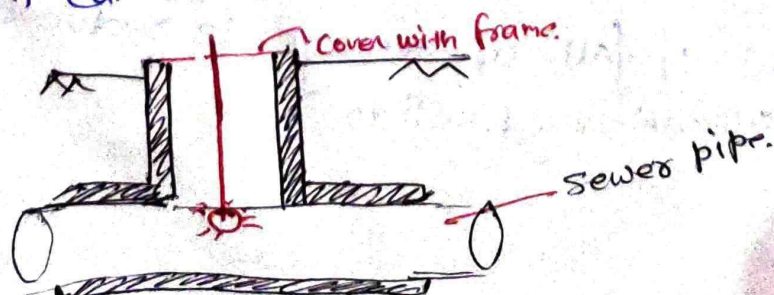
(ii) Lamp holes

→ Lamp holes are sma



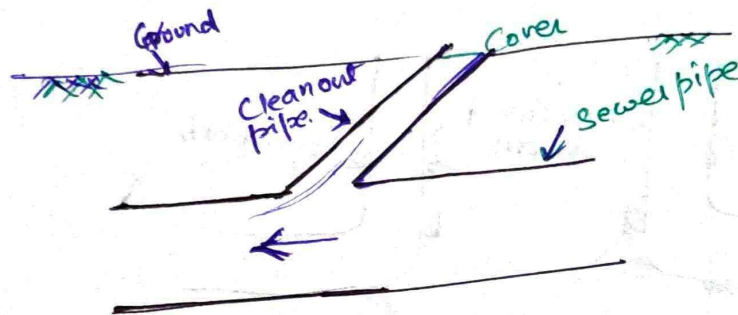
(iii) Lamp holes

- Lamp holes are small opening on sewers to permit the ingestion of **lamp** into the sewer. The **lamp light** is then viewed from upstream and downstream manholes
- The obstructed light confirms the obstruction in the sewer.
- These lampholes are specially adopted:-
 - (i) when a bend in sewer is necessary.
 - (ii) Construction of manhole is difficult.
 - (iii) Spacing of manhole is more than usual.
 - (iv) They can also be used as flushing device & its cover is perforated it can be used for ventilation of sewer.



(iv) Clean Out

- A clean out is inclined pipe extending from the ground surface and is connected to the underground sewer that is used for cleaning the sewer pipe.
- It is generally provided at the upper end of lateral sewer in place of manholes.
- Its functioning involves forcing water through the cleanout pipe to the lateral sewer in order to remove the obstruction in it. If obstruction is large enough a flexible rod may be inserted through the clean out pipe and forward and backward action is imparted to it.



(v) Street Inlet / Gullies

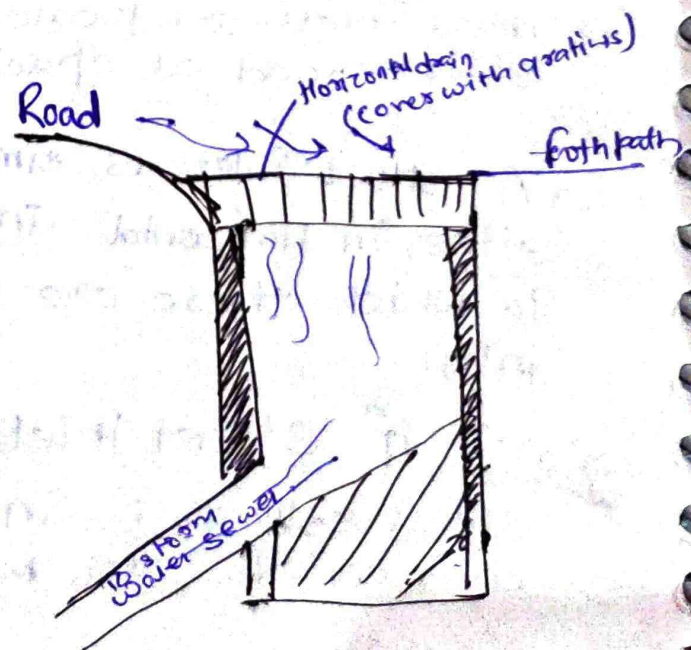
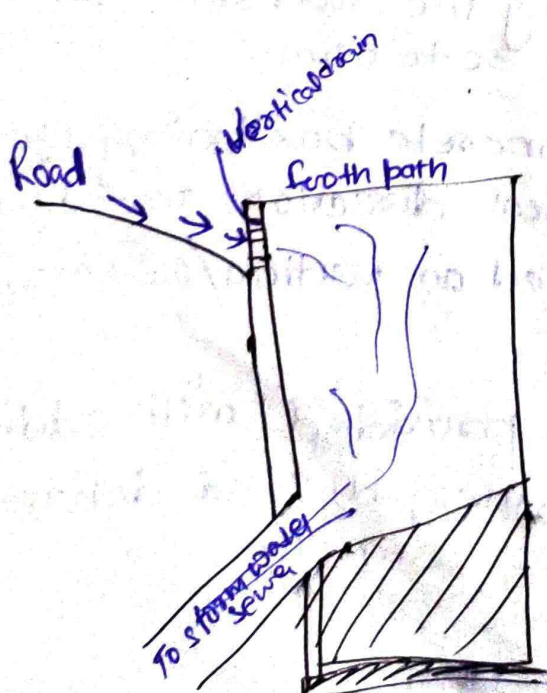
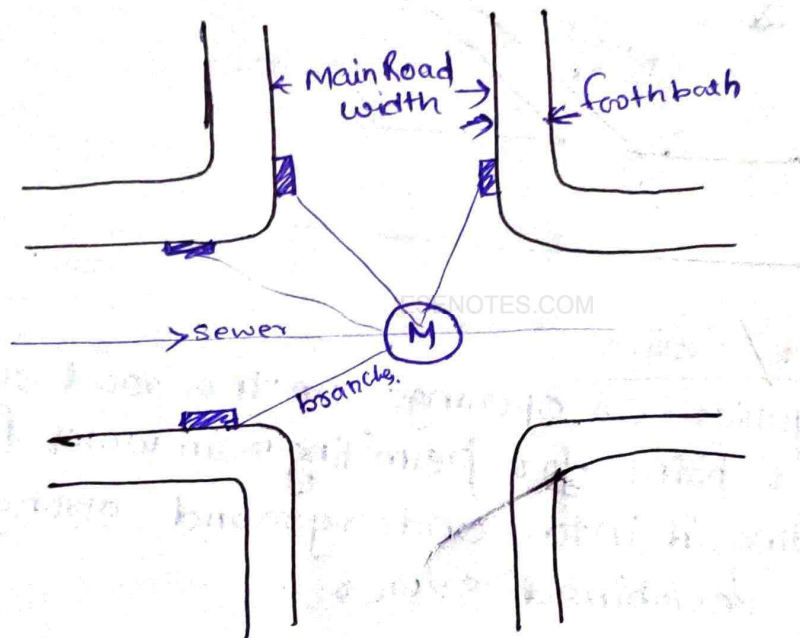
- Inlet are gullies or openings on the road surface at the lowest point for permitting rain water from roads and admitting it into underground storm water sewer (drains) or combined sewers.
- These inlets are located along the road sides on straight road at spacing of 30 to 60 m.
- A street inlet is simple concrete box having opening either in horizontal and vertical direction. according to which these are classified as vertical/or horizontal inlet.

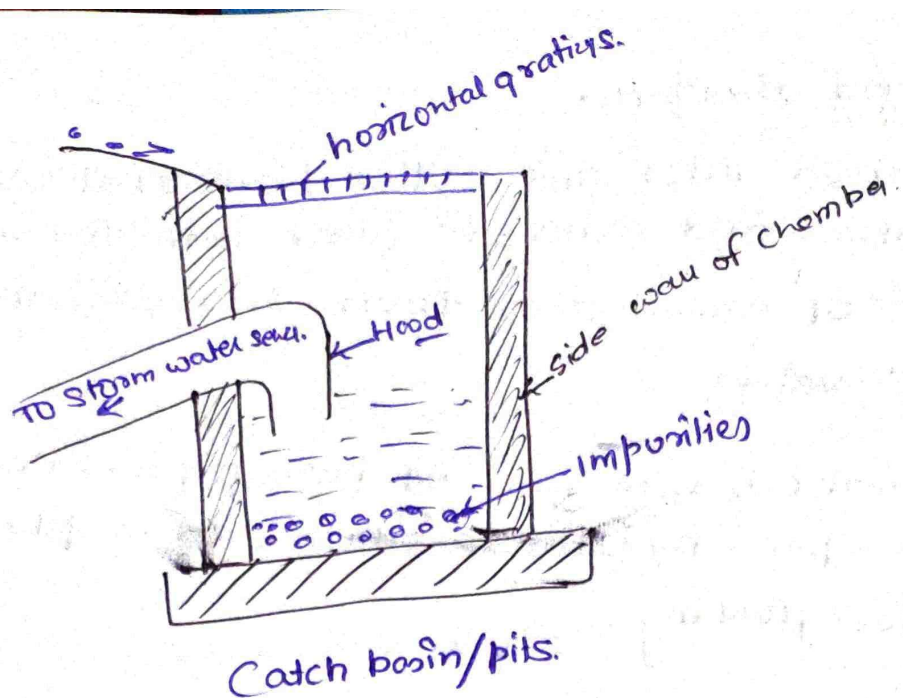
Note:- If street inlets are provided with additional small settling basin to settle grit, sand debris etc it is called catch basin or catch pit.

→ In addition to it a hood is also provided in this to prevent escape of gases from sewer into the catch basin.

→ These are obsolete nowadays as:-

- (i) Quality of road is improved over the period of time hence it is goit free.
- (ii) Sewers are designed at suitable gradient to ensure self cleaning velocity.
- (iii) They lead to evolution of foul gases thereby develop unsanitary condition.





(vi) Flushing tank

→ Whenever there are any chances of blockage of sewer such in case of sewer laid on flat gradient not producing self cleaning velocity or near by to the dead end point of sewer flushing devices are installed.

→ These devices store water temporary, and throw it into the sewer for the purpose of flushing and cleaning the sewer.

→ These flushing devices are normally of two types

- (i) Hand operated flushing operation.
- (ii) flushing operation using automatic flushing tank.

Hand Operated Flushing Operation

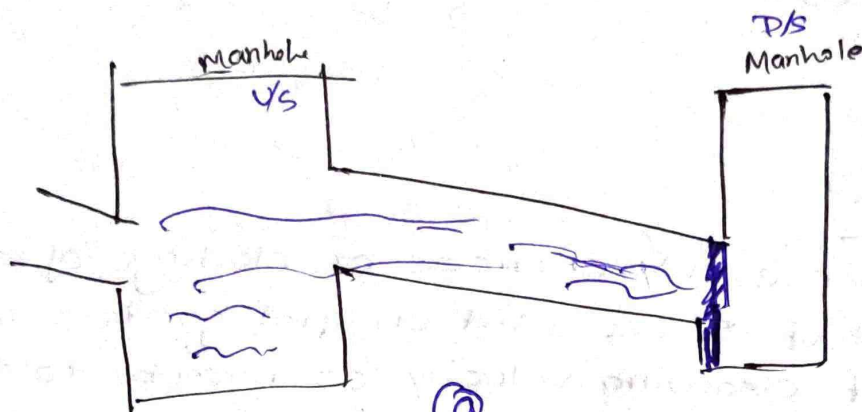
→ It is carried out in following ways:-

- In this method one end of manhole is closed by sluice valve. The sewage entering into the manhole from inlet end will start collecting into it. When sufficient quantity of sewage gets accumulated

The outlet end is open.

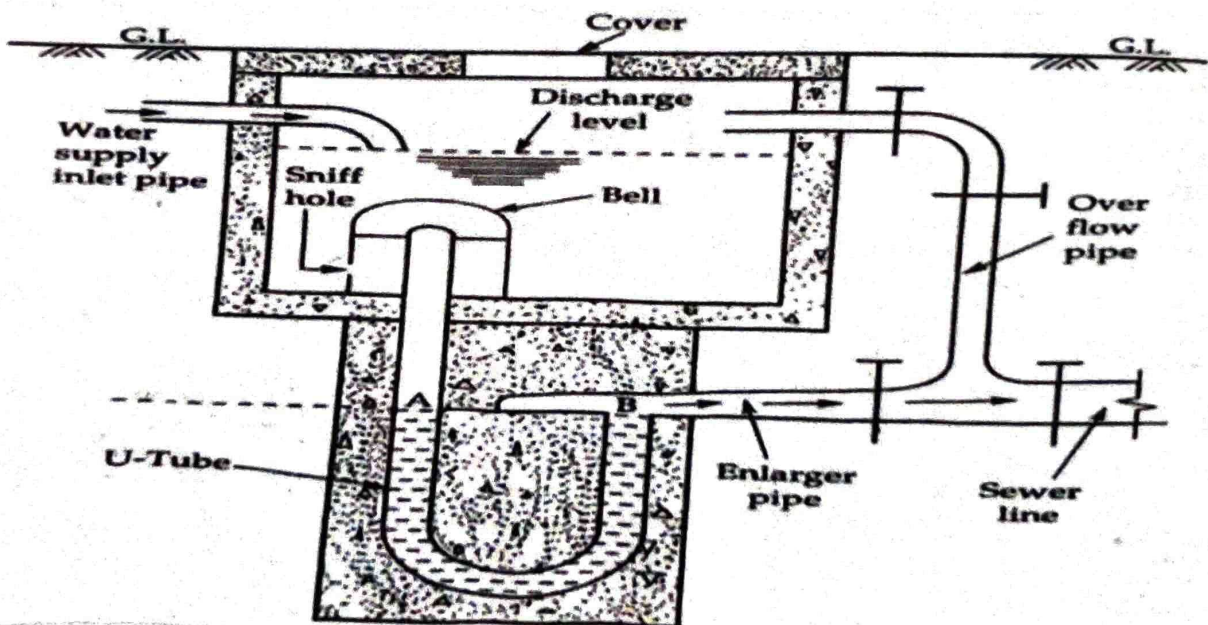
(b) In this method inlet and outlet both end are closed by sluice valve and water is filled into the manhole. The flushing of sewer can then be carried out by opening the valves.

(c) In this method, one end of hose pipe is connected to a nearby fire hydrant & other end is place in manhole for flushing.



(a)

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(vii) Oil & Grease trap.

→ Grease and oil traps are those traps which are constructed in a sewerage system to remove oil and grease from sewage before it enters into sewer line.

→ Such traps are located near to the source of oil & grease. such as automobile repair shop, workshop, garages, kitchen of hotels, oil and grease industries.

→ Oil & grease if not removed would lead to following:-
(i) It will enter the sewer and will stick to the sewer sides thus reduce the sewer capacity

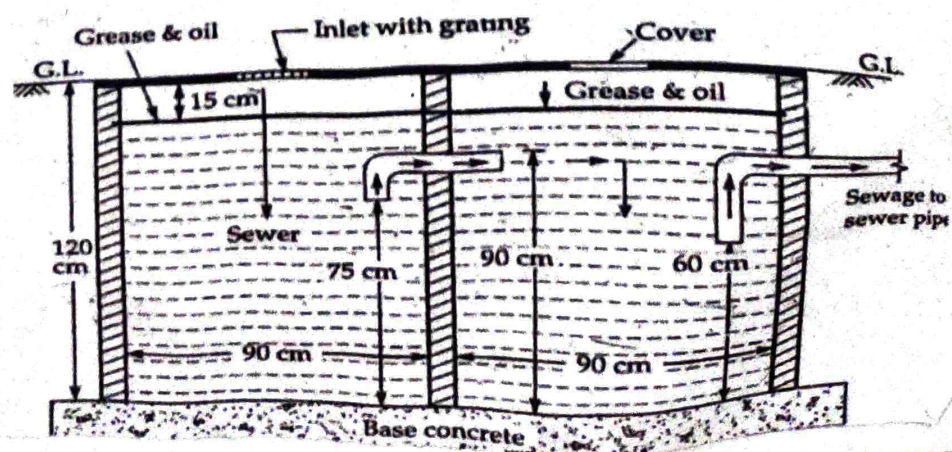
(ii) The suspended solid which would have, otherwise flow along with sewage would stick to the surface of sewer and reduce its discharge carrying capacity.

(iii) The presence of oil & grease leads to possibility of explosion in the sewer.

(iv) It hampers the growth of micro-organism in treatment plant required for biological treatment.

→ The principle on which oil & grease trap work is that grease and oil are lighter than sewage hence float over the surface of sewage thereby sewage is collected from bottom of these traps.

→ If sand is also desired to be removed from the sewage, dead space is provided at bottom of these traps.



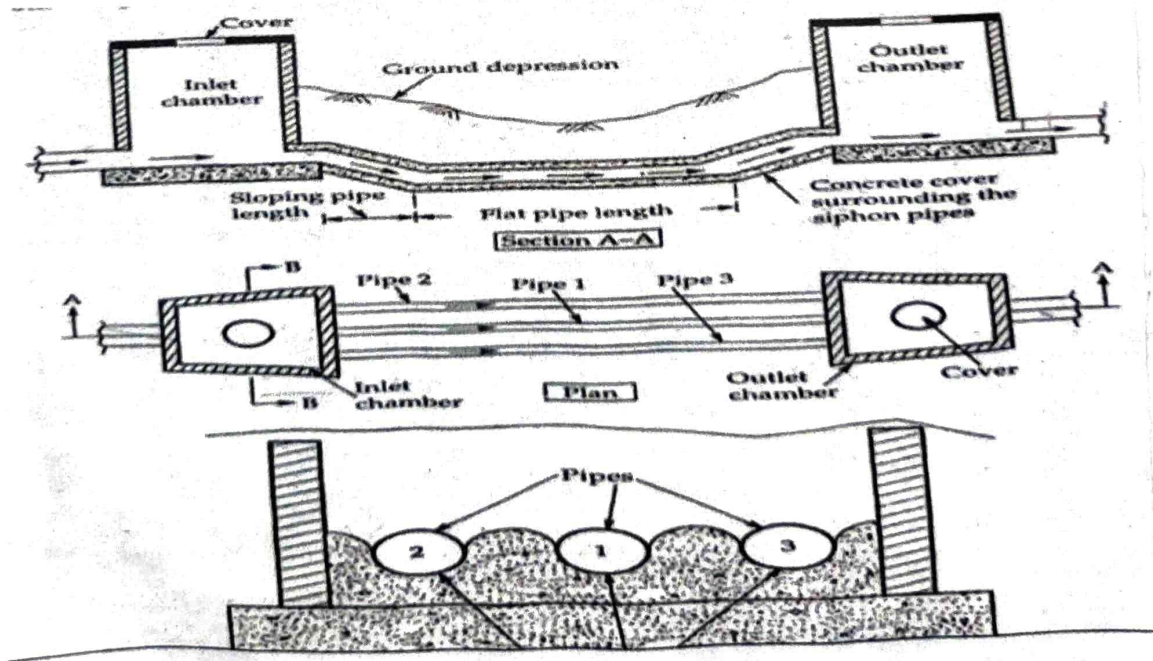
VIII Inverted Siphon

→ Whenever a sewer pipe is to be dropped below the hydraulic gradient line for passing it beneath the valley road, railway track stream or canal or any other obstruction in the earth surface or where it passes beneath some other obstruction in its path it is termed as "inverted syphon." / Depression sewer or sag pipe.

→ The sewage through such a pipe line flows under pressure & not under gravity.

→ An inverted syphon is thus a sewer section constructed lower than adjacent sewer section & it runs full under gravity with pressure greater than Atmospheric pressure.

→ An inverted siphon is usually made up of siphon tubes made of cast iron or concrete



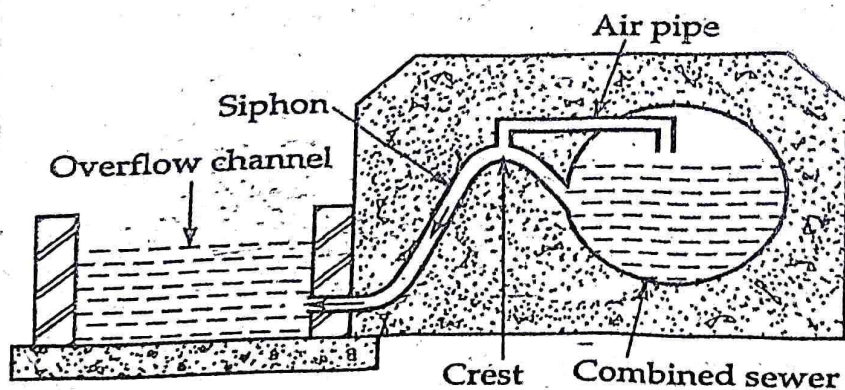
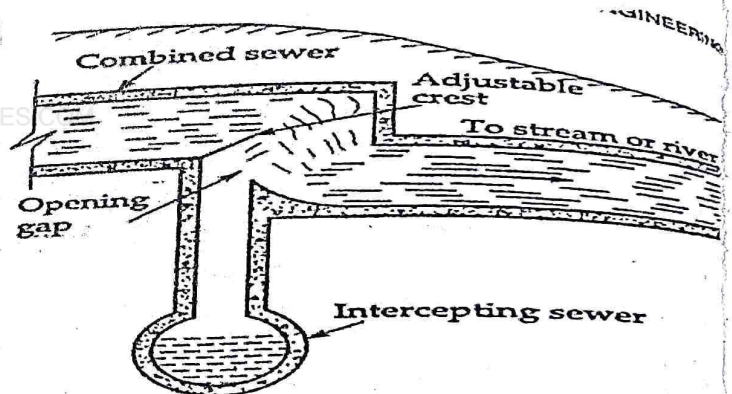
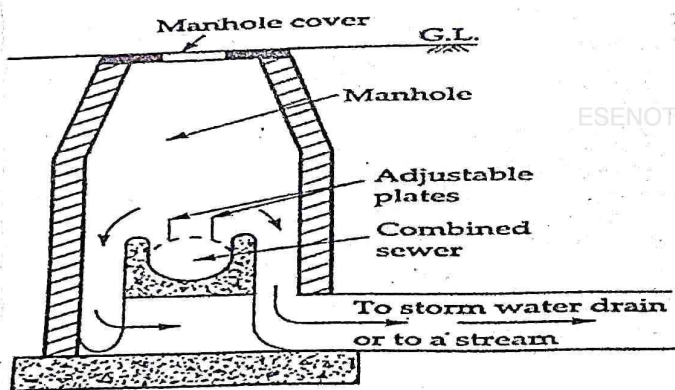
Storm Water Regulator / Storm Relief Work

→ Storm Water Regulator are provided in combined sewerage system and permits the diversion of excess storm water into the nearby stream.

→ This excess sewage will mainly comprise of storm water and therefore it will not be affecting the quality of nearby stream.

→ Storm regulators may be of following types:-

- (i) Leaping weirs.
- (ii) Overflow weir
- (iii) Siphon spill way.



Maintenance, cleaning and ventilation of sewer.

Clean → Sewers are maintained, generally in terms of their cleaning to keep them free from any clogging & to carry out the repairs to the damaged portion if any, so as to increase their life and ensure efficient functioning.

→ The sewer maintenance include, their frequent inspection & supervision measuring the rate of flow, cleaning & flushing and repairing the leakage joints or any other damaged portion, protecting them against ~~their~~ explosion.

→ Sewer should be ~~clean~~ practically clean so as to avoid their complete clogging.

→ Depending upon the type of sewer they can be cleaned manually or mechanically.

→ In order to clean the sewer commonly used device is steel cylinder of 20-25 cm diameter and 60-75 cm length provided with sharp cutting edge. termed as cane rod.

→ Pills can also be used for cleaning of sewer. (These are light floating wooden or rubber hollow balls).

Ventilation → Various poisonous gases and explosive gases which are generally found in sewer are, H_2S , CO_2 , CH_4 , petrol vapours. hence proper precaution must be taken while entering the sewer as follows: -

(i) Before entering sewer through manhole its cover must be open about an hour before entering into it.

(ii) Test should be performed for the presence of different gases in the sewer.

→ H₂S gas may be detected by exposing a sheet of paper moistened with lead acetate for five minutes near the sewer entry.

→ If the paper turns black H₂S is present.

→ Presence of carbon dioxide may be detected by lowering a minor safety lamp near the level of sewage in manhole. If the flame extinguishes within 5 minutes CO₂ is present.

→ The presence of methane may be detected by lowering the minor's safety lamp in upper layers of sewer, methane being lighter than air is generally present in upper region, when gas is present it forms an explosive mixture with air. & gauge cylinder of the safety lamp gets filled with flame.

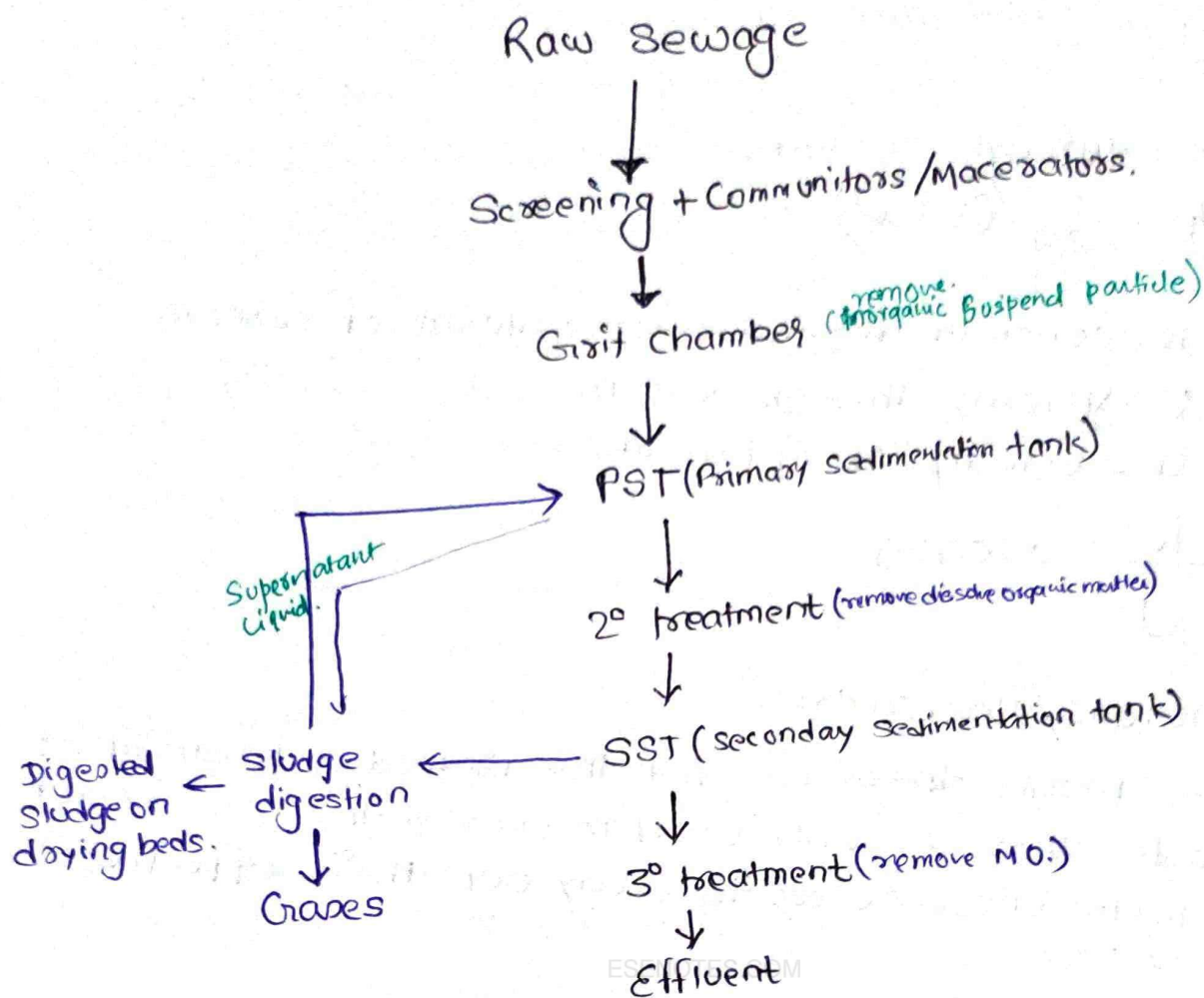
→ Hence ventilation of sewer must be carried out due to following reason:-

- (i) To eliminate the gases accumulated in the sewer.
- (ii) To ensure continuous flow of sewage into the sewer, by avoiding air blocks.

→ It can be achieved by use of ventilating columns, use of ventilating manhole.

- (iii) Proper design of sewer.
- (iv) Use of mechanical devices.
- (v) Unobstructed outflow.
- (vi) House vents & soil pipes.
- (vii) Artificial ventilation

Treatment of Sewage.



(i) 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 50mm 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 material of screen.

V = Velocity through screen.

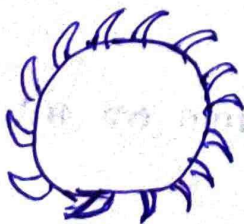
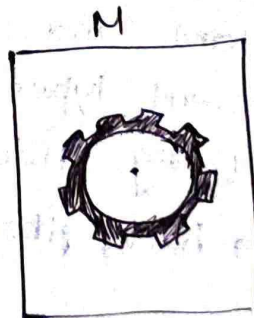
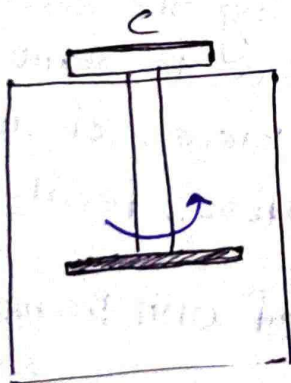
u = velocity of approach.

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

(iv) Comminutors/Macerators.

→ In real terms these are not the method of treatment of waste water. They are only used to convert the heavier solid into the lighter ones, so as to carry out their effective disposal.

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



Notes:- Disposal of screenings.

- Material separated by screen is called screenings.
- It can be disposed by any of following method:-
 - (i) Burying 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 and to pass forward the organic suspended particle in PST.
- Grit quantity produced may vary between $0.004 - 0.037 \text{ m}^3$ / 1000 m^3 of sewage for separate sewerage system and $0.004 - 0.180 \text{ m}^3 / 1000 \text{ m}^3$ 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 chambers are designed to give a horizontal constant velocity of flow or discharge which is achieved by providing velocity control devices such as proportional weirs or sub weirs & partial flume at the effluent end of the section.

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

→ Surface overflow rate for 0.2mm sized particle is ~~0.25-0.3 m/sec~~. $0.025 - 0.03 \text{ m/s}$

→ The important point in the design 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 ~~two~~ 2- or 3 chambers for average discharged or dry weather flow.

→ $t_d = 40 - 60 \text{ sec}$.

→ $H = 1 - 1.8 \text{ m}$

→ Free Board = 0.3 m

→ length of tank is increased by 25-30% over the design length to account for the turbulence 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

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

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

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

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

Design

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

$$V = Q_0 \cdot t_d$$

(LBH)

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

$$L = V_f \cdot t_d$$

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

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(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 low organic matter may also settle down.
- Air feed rate is kept in range of 0.15 to 0.45 m³/min/m length of the tank. (0.3 m³/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 m³/m³ of sewage in combined sewerage system & 30 m³/m³ in separate sewerage system.



Q A rectangular grit chamber is designed to remove particle with diameter 0.2 mm, $G_s = 2.65$. 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 m³/day

~~12000~~ $Q_0 = 12000 \text{ m}^3/\text{day}$.

Assume $V_s (\text{set}) = 0.025 \text{ m/sec}$

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

Assume, $t_d = 50 \text{ sec}$.

$$V = Q_0 \cdot t_d$$

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

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

Provide free board of $= 0.35 \text{ m}$
depth of sludge zone $= 0.9 \text{ m}$

ESENOTES.COM $L = V_f \cdot d$

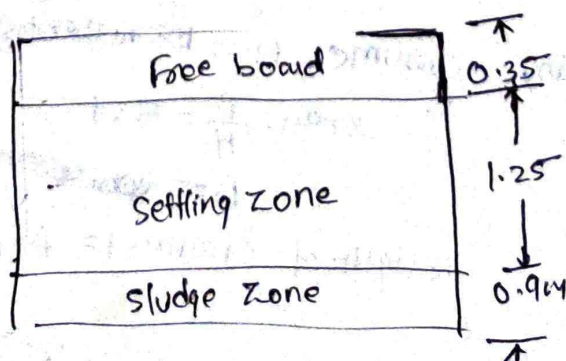
$$= 0.3 \times 50 = 15 \text{ m}$$

$$\text{width of tank} = \frac{\text{area}}{\text{length}} = \frac{5.55}{15} = 0.37 \text{ m}$$

Increase the length by 30%,

$$L' = 19.5 \text{ m}$$

$$\text{total height of tank} = 0.35 + 1.25 + 0.9 = 2.5 \text{ m}$$



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

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

$$\text{Assume } t_d = 3 \text{ min}$$

$$\begin{aligned} \text{Volume} &= Q_0 \times t_d \\ &= 2.25 \times 3 \times 60 = 405 \text{ m}^3 \end{aligned}$$

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

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

$$\text{Assume, } H = 1 \text{ m}$$

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

$$B = 2 \text{ m} = 2 \times 1$$

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

length of tank is increased by 25%.

$$\begin{aligned} L' &= 1.25 \times 101.25 \\ &= 126.56 \text{ m} \end{aligned}$$

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

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

$$\text{Again, assume, } H = 1.2$$

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

$$B = 6 \text{ m}$$

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

$$\frac{L}{B} = \frac{126.56}{6} = 21.09$$

length of tank is increased by 25%.

$$L' = 1.25 \times 28.125 = 35.156 \text{ m}$$

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

$$\text{Air supply requirement} = 0.03 \text{ m}^3/\text{min}/\text{m}$$

$$\begin{aligned} \text{Air supply requirement} &= 0.3 \times 35.15 \\ &= 10.54 \text{ m}^3/\text{min} \end{aligned}$$

Volume of grit produce @ $30 \text{ m}^3/\text{M m}^3$ of sewage treated

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

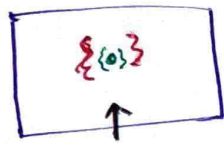
$$= \frac{5832}{5832} \text{ m}^3/\text{day}$$

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IV Detritus tank

- Detritus tank are grit chambers designed to flow with the smaller flow velocity of 0.09 m/s and longer detention period of 3-4 min.
- They are designed to separate 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 use of controlled aeration of flow through the tank in which air bubbles will then separate the lighter organic matter from settling grit or by control of current in the tank through baffle wall.



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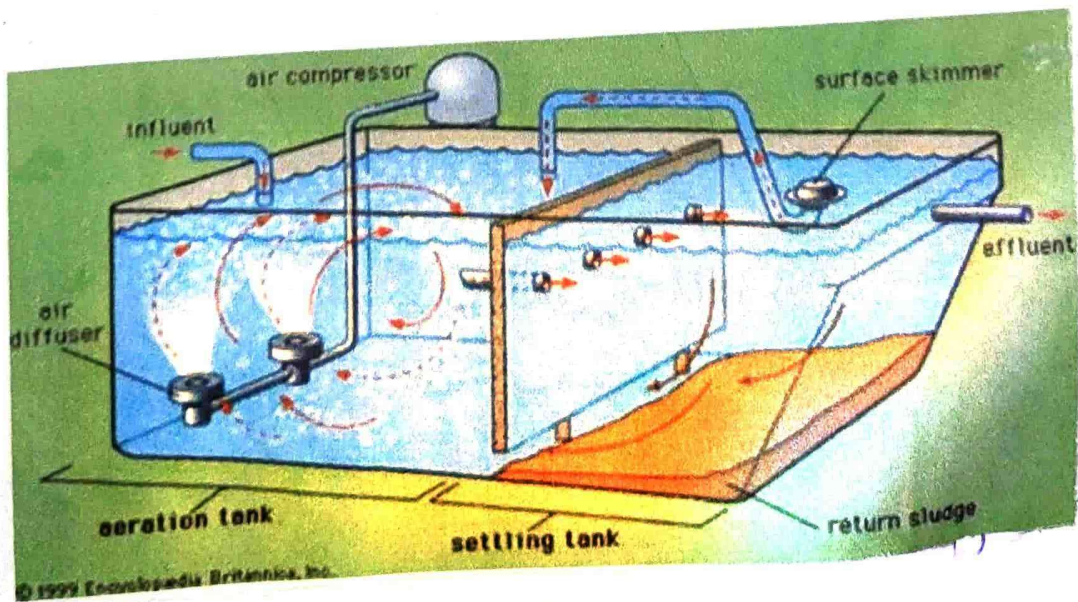
V Skimming Tank

- These tank are employed to remove oil and grease from waste water and are placed before PST
- These oil and grease enters into the sewage from kitchen, restaurant, 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.
- $t_D = 3-5$ minutes.
- Compressed air required is $300-6000 \text{ m}^3/\text{ML}$ of sewage
- Area of tank is given by, ~~area~~

$$A = 0.00622 \frac{Q}{V_r}$$

Q = rate of flow of sewage in m^3/day
 V_r = minimum rising velocity of greasy material to be removed.
 $= 0.25 \text{ m/min.}$



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:-

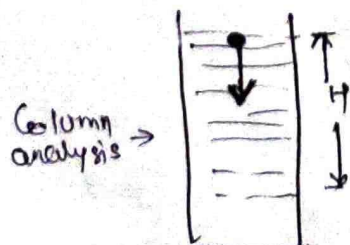
(i) TYPE-I : DISCRETE SETTLING

concentration of particles $< 1000 \text{ mg/l}$

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

(ii) TYPE-II FLOCULANT SETTLING

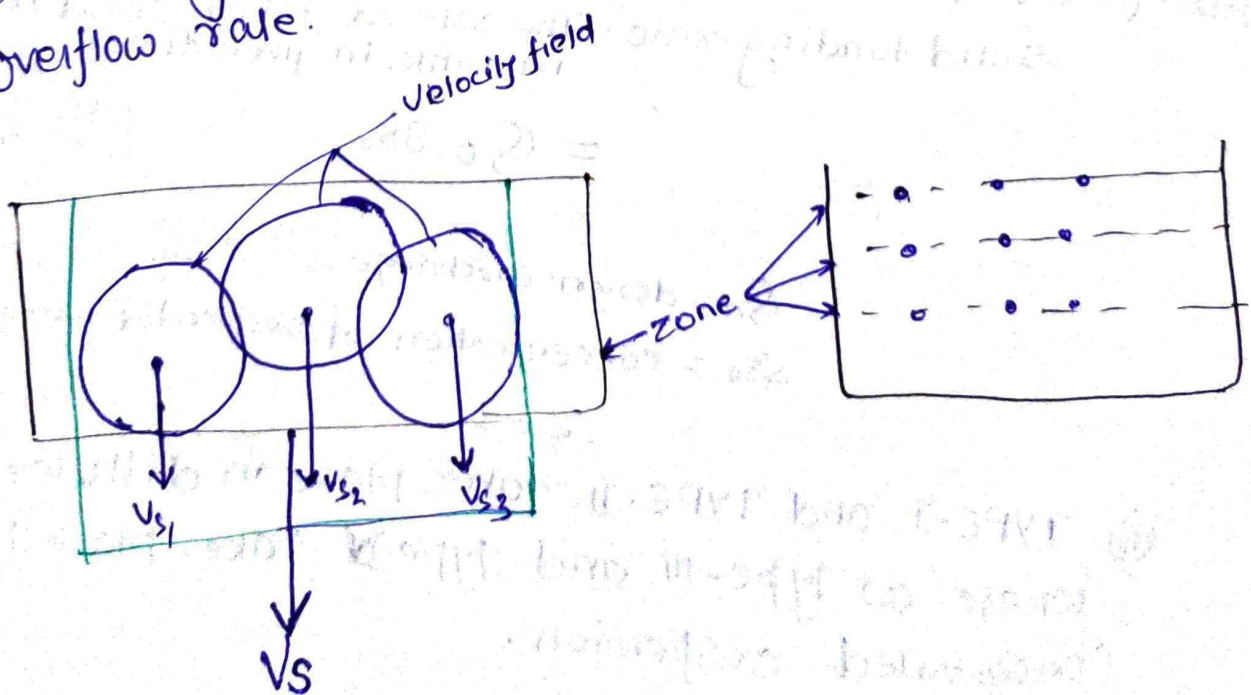
- It is the type of settling in which shape, size and mass of particles changes during settlement due to formation of flocs.
- It takes place when concentration of particle is comparatively low. 1000 mg/lit
- There is no mathematical relationship to find settling velocity of particle in this case.
- This settling is found in clarifloculators. (RWP) or in primary sedimentation tank. (PST) (WWP) or in upper portions of SST (secondary sedimentation tank).



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

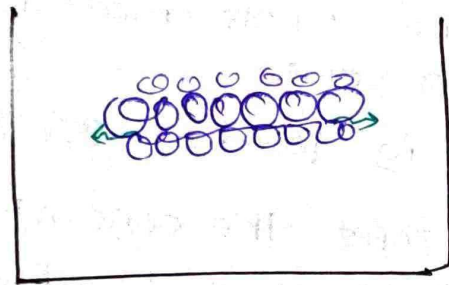
Zone-III : ZONE/HINDERED SETTLING.

- 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 their 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 thickening 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 accompanied 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).



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- Such secondary clarifiers are also designed for overflow rate & solid loading rate.

Note: (i) Overflow rate = V_s

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

$$= Q_0 \cdot SS_0 \quad \left[\frac{w \cdot v}{V \cdot T} \right]$$

Q_0 = design discharge

SS_0 = concentration of suspended particle,

- (ii) TYPE-I and TYPE-II takes place in dilutes 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 ($\text{m}^3/\text{m}^2/\text{day}$)		Depth of tank (m)	Detention time hr.
	Avg	Peak		
→ 1 st Settling only	25-30	50-60	2.5-3.5	2-2.5
→ 1 st Settling followed by TF.	35-50	80-120	2.5-3.5	2-2.5
→ 1 st settling followed by ASP.	25-35	50-60	3.4-4.5	2-2.5

$$\rightarrow V_f = 0.3 \text{ m/min}$$

$$\rightarrow B = 5-6 \text{ m}$$

$$\rightarrow \frac{L}{B} = 4-5$$

$$SA = \frac{Q_{\text{avg}} \text{ or } Q_0}{OFR_{\text{avg}}} \quad \& \quad SA = \frac{Q_{\text{PEAK}}}{OFR_{\text{PEAK}}} \quad \left. \vphantom{\frac{Q_{\text{PEAK}}}{OFR_{\text{PEAK}}}} \right\} \text{Maximum}$$

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

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

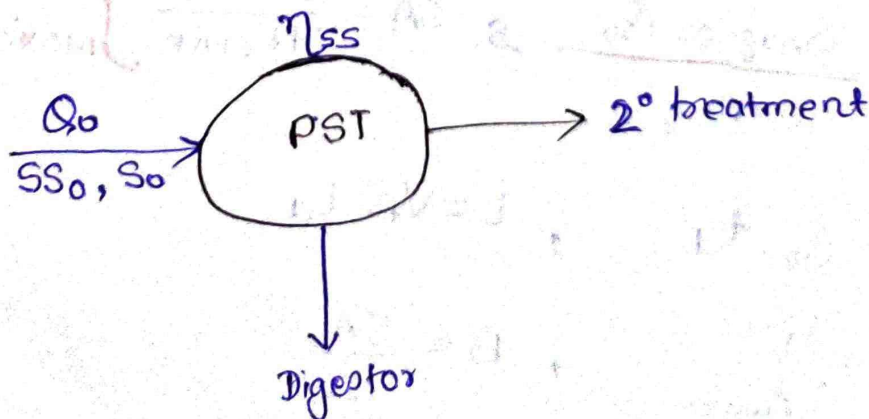
Coagulation aided sedimentation.

→ 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 biological 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 ~~involve~~^{involved} are complete in nature, hence do not require coagulation seprately.

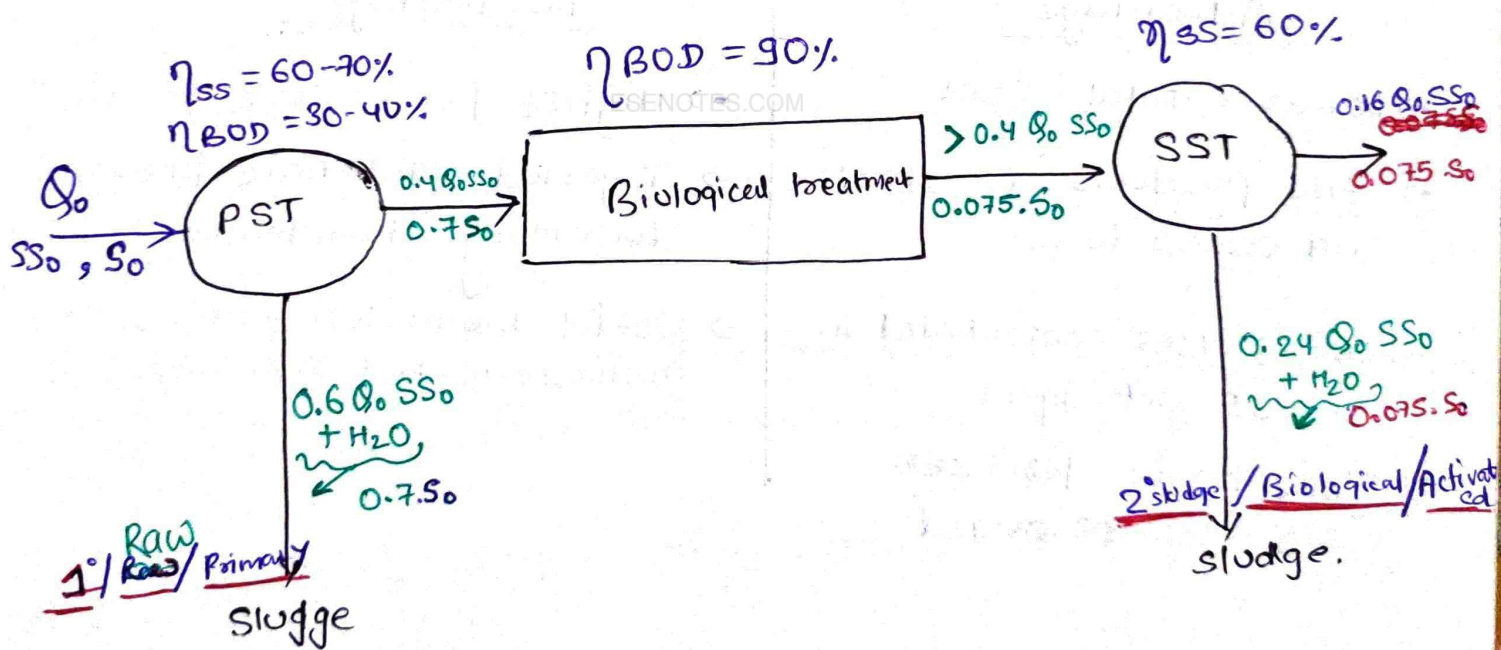
→ But in some extreme condition it can be used by addition of iron salts. (FeCl_3 , $\text{Fe}_2(\text{SO}_4)_3$) as in case of:-

- 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.

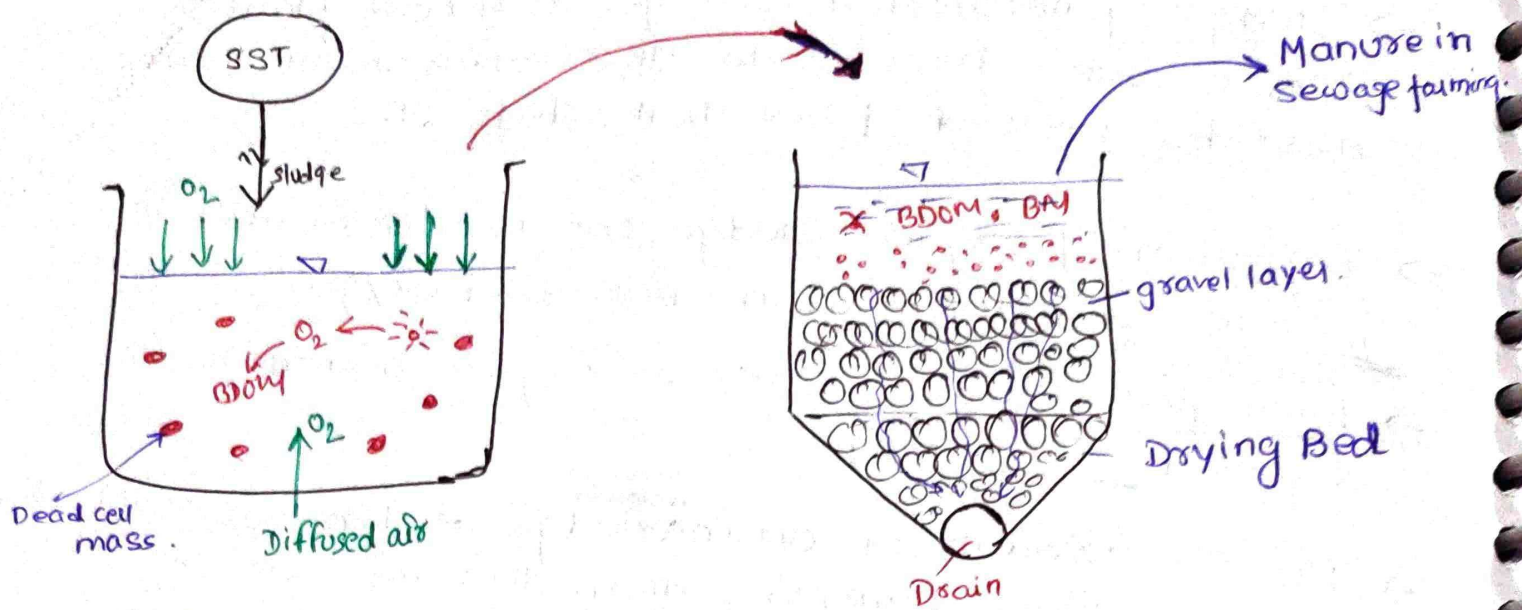


Sludge Digestion.

- 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 ^{from SST} Biological sludge 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

- lower capital cost
- End products are stable and odour less.
- Lower BOD concentrated in supernatant liquor.
- More basic fertilizer values are recovered.

Disadvantage

- Higher power cost.
- Digested solid have poorer dewatering characteristics.
- Useful bio-product such as methane is not recovered.

Anaerobic Digestion.

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

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

→ During anaerobic digestion sludge is reduced into following constituents: →

(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) Supernatant 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.

→ Anaerobic digestion is being carried out by following set of micro-organisms.

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

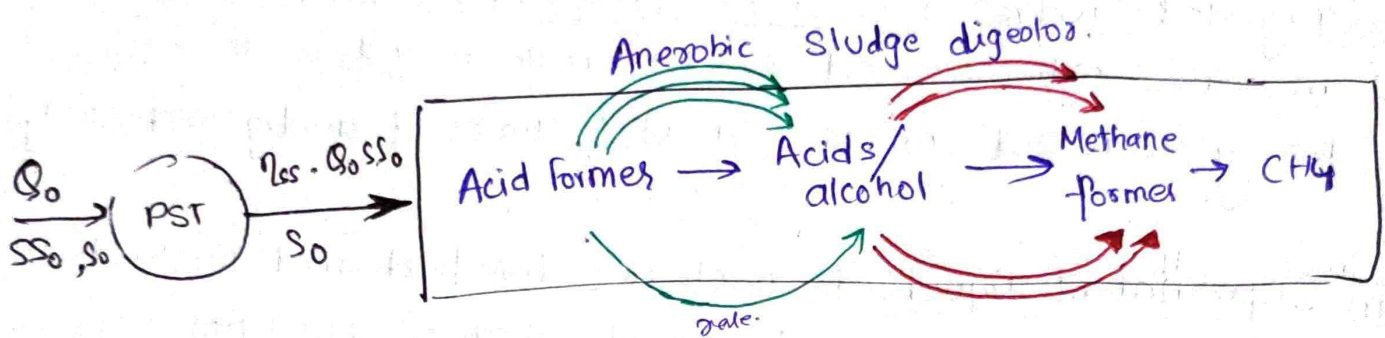
→ These are highly competitive M/O as they respond very quickly to the food given to them.

(ii) Methane formers:

These are strictly anaerobic 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 delicate 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 anaerobic & facultative micro-organism called acid formers.

→ These MO solubilize the organic solid through "HYDROLYSIS".

→ The soluble 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 up to 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

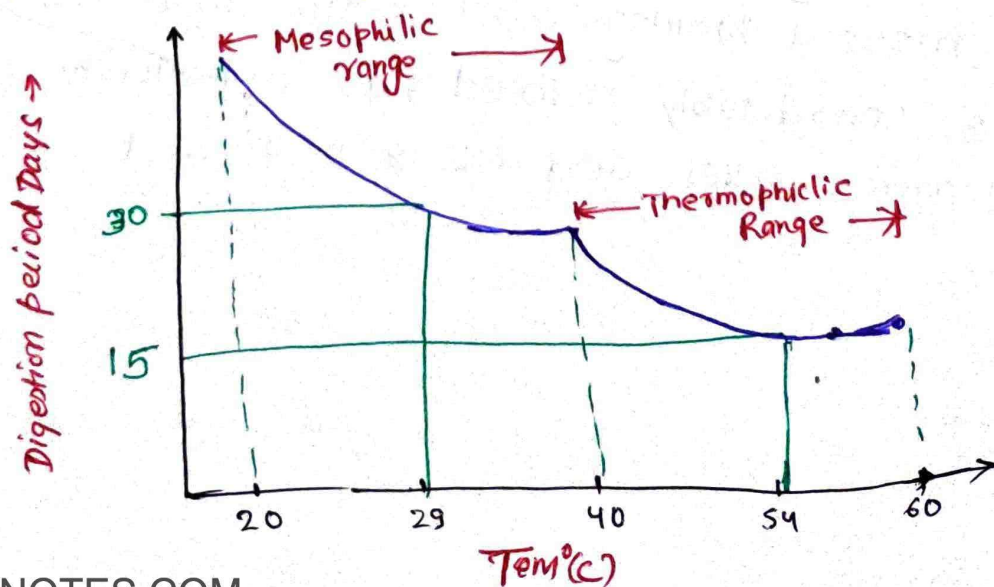
→ Digested sludge is alkaline, hence pH is slightly greater than 7.

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

* Factors affecting Anaerobic Digestion.

(i) Temperature

The process of digestion is greatly influenced by temp, rate of digestion increases with increase in temperature due to the weakening 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 $\rightarrow$ overdosing of raw sludge.

$\rightarrow$ with overwithdral of digested sludge.

$\rightarrow$ 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.

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

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

$\rightarrow$ Proper seeding will help attain quick balance condition of reaction. in the digester.

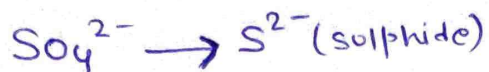
(iv) Mixing and stirring of the raw sludge with digested sludge.

$\rightarrow$ if incoming fresh raw sludge is thoroughly mixed with the digested sludge by some effective method of agitation so as to make a homogenous 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 ~~SS~~



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

Gases produced during Anaerobic Digestion.

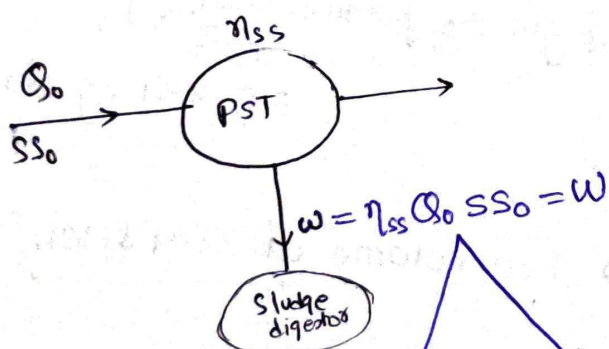
→ Of all the solids that comes into digester from PST, (70%) are volatile, & 30% are fixed.

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

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

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

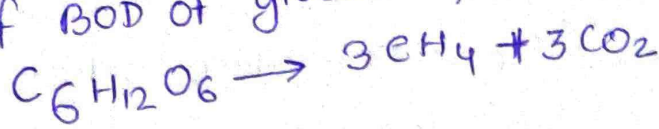
→ Calorific value of methane is 8600 Kcal/m^3



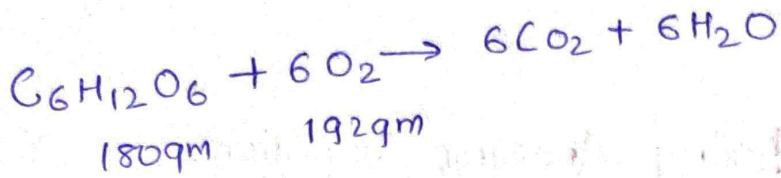
$$\begin{aligned} &0.6 \text{ m}^3/\text{kg} \text{ of VS present} \\ &= 0.6 \times VS \\ &= 0.6 \times 0.7 w \\ &= 0.42 w \end{aligned}$$

$$\begin{aligned} &\text{OR} \\ &0.9 \text{ m}^3/\text{kg} \text{ VS reduced to gases.} \\ &= 0.7 w \times 0.65 \times 0.9 \\ &= 0.41 w \end{aligned}$$

Note:- 1 kg of BOD of glucose produce 0.35 m³ of CH₄

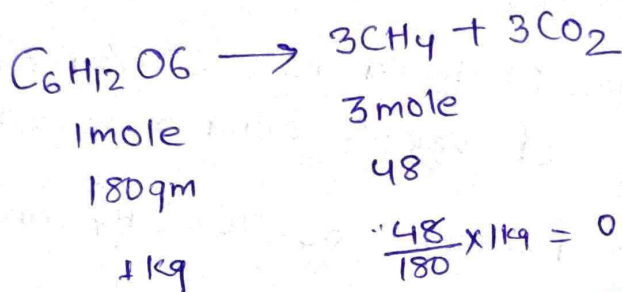


$$BOD_{max} = COD = ThOD$$



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

→ 1 kg of glucose has 1.06 kg of BOD. (i)



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

→ 1 kg of glucose produce 0.267 kg of methane (ii)

1.06 kg of BOD of glucose produces 0.267 kg of methane

$$1 \text{ kg of BOD of glucose produce} = \frac{0.267}{1.06} \times 1 = 0.251 \text{ kg of methane}$$

1 mole of CH₄ at STP has volume of 22.4 liter.

$$16 \text{ gm methane} \text{ ————— } 22.4 \text{ liter}$$

$$0.25 \text{ kg of methane} \text{ ————— } \frac{22.4}{16} \times 0.25 \times 10^3 \times 10^3 = 0.35 \text{ m}^3 \text{ of methane}$$

1 kg of BOD of glucose produce 0.35 m³ of CH₄ (STP) (adm, D)

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) Efficiency of PST for removal of suspended solid.

Soln

$$(i) \text{ BOD of waste water} = (2000 - 400) \times 0.6 \times 10 \times 10^6 \times 10^{-6} \\ = 9600 \text{ Kg/day}$$

$$\text{Volume of methane produced} = 9600 \times 0.35 = 3360 \text{ m}^3/\text{day} \quad (i)$$

$$\text{fuel value of waste} = 3360 \times 8600 \\ = 2.889 \times 10^7 \text{ Kcal/day.}$$

$$(ii) \text{ Volume of methane} = 0.6 \text{ m}^3 \text{ kg of volatile solid} \\ \left(\text{assuming it to be 70\% of total gases produced} \right) = 0.6 \times [0.7 \times \eta_{ss} \times 10 \times 10^6 \times 1300 \times 10^{-6}] \times 0.7 \\ = 3822 \cdot \eta_{ss} \quad (ii)$$

from (i) and (ii)

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

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

Types of Anaerobic Digesters.

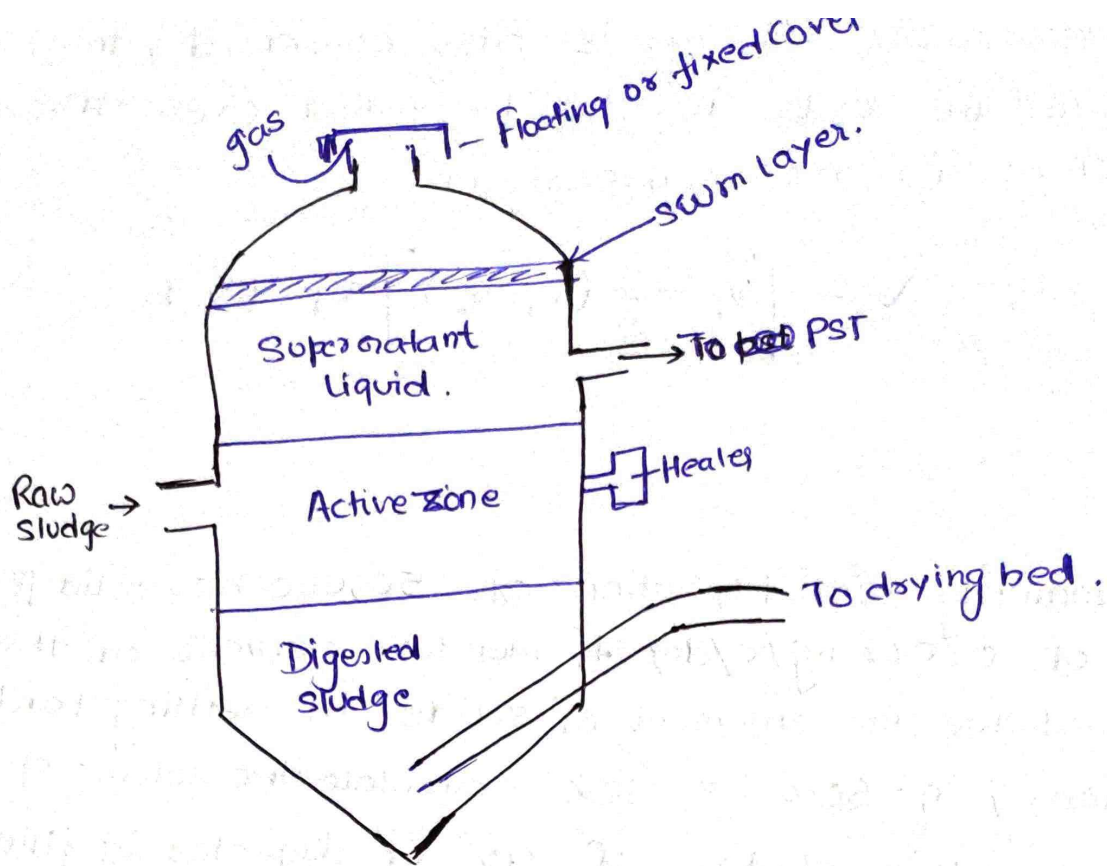
→ Anaerobic digestion can be carried out in any of the following units :-

- (i) Standard Rate Sludge Digester (SRSD)
- (ii) High Rate Sludge Digester (HRSD)

(i) SRSD

Raw sludge is directly fed into these digester 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 constituents 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.
- Supernatant liquid being highly organic (due to rising of organic suspended particles from active zone into it by the gases) it is send back to PST ~~for~~ 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), thereby 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 anaerobic digester:-

- (i) Dia of tanks = 3-12m (~~18m~~ may extend upto 18m)
- (ii) Depth of tank = 6m (may extend upto 12m)
- (iii) Bottom hoppers floor of tank is given a ~~stone~~ slope of 1:1 to 1:3 (1H:3V)

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

(v) Volume of Digester (v)

(a) If variation in quantity of sludge is assumed 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 sludge 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 considered, 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 stabilizing this sludge, if 4% of digester is filled with fresh sludge daily. Assume density of sludge = 1.03.

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

$$P = 90\%$$

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

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

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

$$4\% V = V_1 \times 1$$

$$V = \frac{20.97}{0.04}$$

$$V = 524.25 \text{ m}^3$$

(ii) HRSD

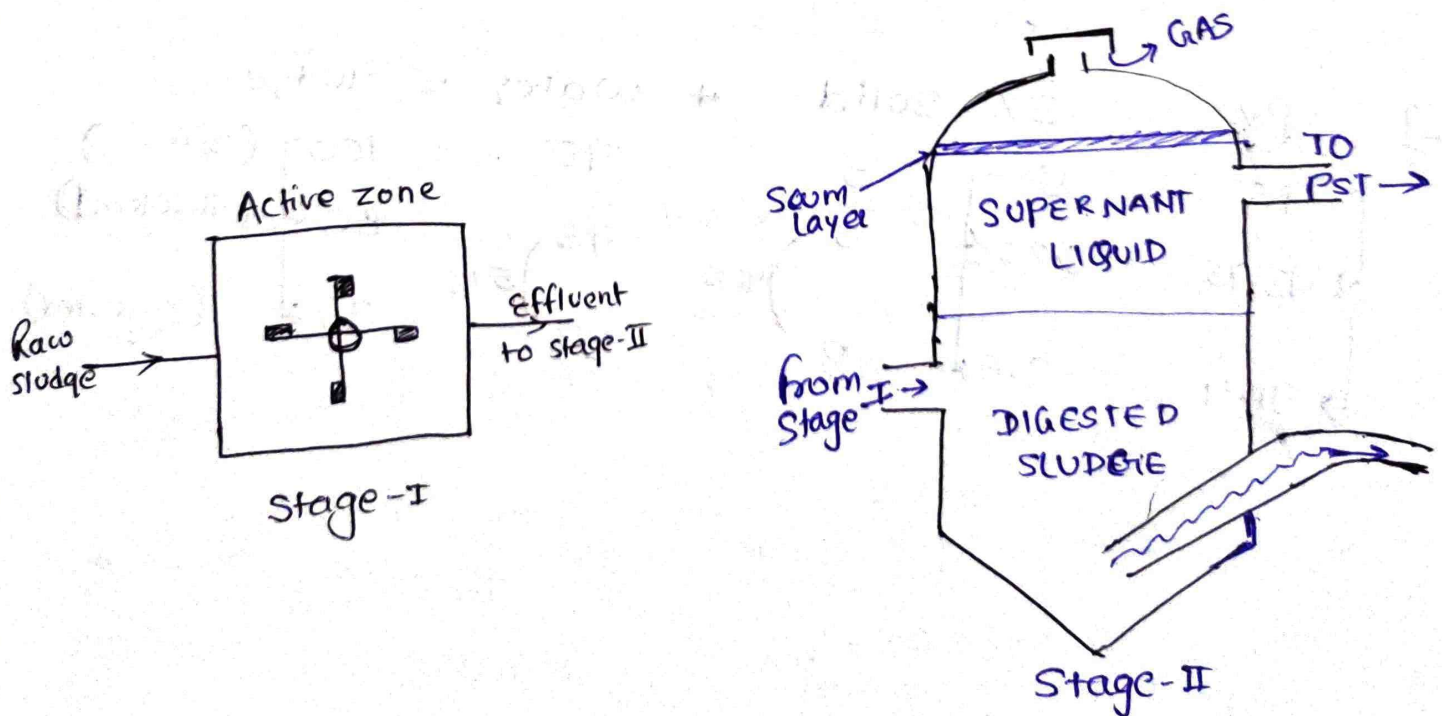
→ In this case treatment is carried out in two stages where in the first stage complete ~~to~~ mixing of incoming raw sludge is done ~~mechanically~~ mechanically to increase the ^{opportunity} of contact betⁿ organic matter and M/O. that in turn increase the rate of digestion.

→ 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 homogenous environment created by mixing.

$$V_I = V_1 t_{D1} \quad (t_{D1} = 10-15 \text{ days})$$
$$V_{II} = \left(V_1 - \frac{2}{3} (V_1 - V_2) \right) t_{D2} + V_2 T \quad (t_{D2} = 10-12 \text{ days})$$



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 = 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.

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

$\downarrow 40\%$
 $\downarrow 57\%$

Q Raw sludge (V_A, P_A), Digested sludge (V_B, P_B)

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

Solid + water = sludge

$$100 - P_A \quad P_A = 100 \text{ kg of raw sludge}$$

$$w = \eta_{ss} Q_0 \cdot SS_0 = \left(\frac{100}{100 - P_A} \times w \right) \text{ of raw sludge.}$$

$$\text{Volume of raw sludge} = \frac{100w}{100 - P_A} \times \frac{1}{P_A} \quad \text{--- (i)}$$

Digested sludge

Solid + water = sludge

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

$$w' = NVS + 0.35VS$$

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

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

Note:- Volume of sludge = $f(w, P/s, P)$

Assume, $w = w' \quad (w > w')$

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

Assume, $P_A = P_B \quad (P_B > P_A)$

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

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

$$P_B = 80\%$$

$$P_A = ?$$

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

$$\text{Assume, } \frac{V_B}{3} = \frac{V_A}{3} \quad \text{or}$$

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

$$3(100 - \cancel{P_A}) = (100 - \cancel{80})$$

$$P_A = 93.33\%$$

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) Degritting - 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, thereby 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.

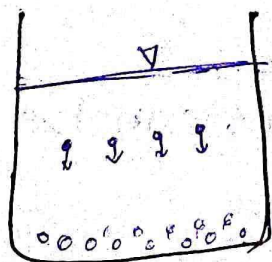
- It is done for primary sludge or primary sludge combined with ASP sludge.
- It is never adopted for ASP sludge independently.
- It is not effective, if in combined sludge the ASP sludge portion exceed 40% of total sludge.
- It is designed for surface loading rate of approx $25-30 \text{ kg/m}^2/\text{day}$.
- In this the working is same as that of sedimentation tank. in which sludge is ~~about~~ allowed to stand to carry out settlement of suspended solid.
- Depth of tank 3-4 m.
- $t_d = 4 \text{ hours}$.

(b) Air floatation

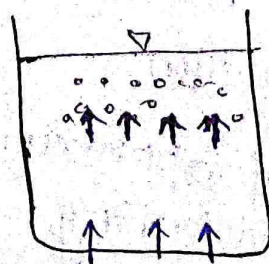
- This unit employs floatation of sludge by air under pressure or vacume ~~g~~ is normally used for thickening of activated sludge.
- The supernatant is pressurised at $3-5 \text{ kg/cm}^2$ and saturated with air in pressure tank.
- The excess dissolved air then rises up in the form of bubbles attaching themselves to the particle.

(c) Centrifugation.

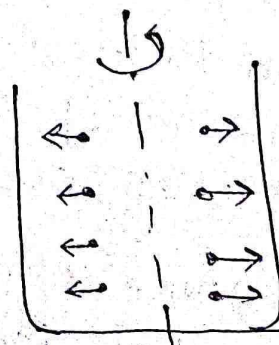
It is done where availability of space is limited.



Gravity thickener



Air floatation



Centrifugation.

III Stabilization/Digestion:

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

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 oxidised end product by burning it at high temp. in presence of oxygen.

VIII Sludge Disposal

It can be carried out by land fill method

- (b) sewage farming.
- (c) Dumping into sea.

Q A 10MLD waste water plant treats 250 mg/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 sludge 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 thickened 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 $0.05 \text{ m}^3/\text{ha}/\text{day}$.

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

$$= \eta_{ss} \cdot Q_o \cdot SS_o$$

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

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

Assuming 70% of total solids to be volatile.

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

Assuming 30% of total solid to be NVS.

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

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

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

$$V_{RS} = (V_{VS} + V_{NVS}) + V_w \\ = 1.115 + 0.195 + 39 = 40.31 \text{ m}^3/\text{d}.$$

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

$$\text{Volume of water in thickened sludge, } V_w = \frac{90}{10} \times \frac{1625}{10^3} \\ = 14.625 \text{ m}^3/\text{day}$$

$$V_{TS} = (V_{VS} + V_{NS}) + V_w \\ = (1.115 + 0.195) + 14.625 \\ = 15.935 \text{ m}^3/\text{day}.$$

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

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

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

$$\text{Volume of water in digested sludge, } V_w = \frac{80}{20} \times \left[\frac{0.7 \times 0.3}{10^3} + 0.3 \right] \times 1625 \\ = 3.315 \text{ m}^3/\text{day}.$$

$$V_{DS} = 0.334 + 0.195 + 3.315 \\ V_{DS} = 3.844 \text{ m}^3/\text{day}.$$

(iv) $P = 50\%$

50 kg of solid + 50 kg of water

$$V_w \text{ in dewatered sludge} = \frac{50}{50} \times \frac{828.75}{10^3} = 0.82875 \text{ m}^3/\text{day}$$

Volume of dewatered sludge

$$V_{DWS} = \overset{V_s}{(0.334 + 0.195)} + \overset{+ V_w}{0.828} = 0.529 + 0.828 = 1.357 \text{ m}^3/\text{day}$$

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

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

$$\begin{aligned} \text{Volume of dried sludge} &= V_w + V_s \\ &= 0.092 + 0.529 \\ &= 0.621 \text{ m}^3/\text{day} \end{aligned}$$

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

$$\begin{aligned} \text{(vii) Area of land} &= \frac{0.195 \text{ m}^3/\text{day}}{0.05 \text{ m}^3/\text{hae}/\text{day}} \\ &= 3.9 \text{ hact} \end{aligned}$$

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

- (i) Average flow = 20MLD
 - (ii) Total suspended solid in raw sewage = 300mg/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%. $\rho = 1000 \text{ kg/m}^3$

$$\text{Volume of raw sludge, } V_1 = \frac{100 \times Q \times \theta_0 \times SS_0}{(100 - P) \rho} = \frac{100 \times 20 \times 0.6 \times 300 \times 10^6}{(100 - 95) \times 1000}$$

$$V_1 = 72 \text{ m}^3/\text{day}$$

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

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

$$V_2 = 24 \text{ m}^3/\text{day}$$

Assume, $t_d = 30 \text{ days}$.

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

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

$$V = 1200 \text{ m}^3$$

Assume, $H = 6 \text{ m}$.

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

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

$$D = 16 \text{ m} > 12 \text{ m}$$

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.

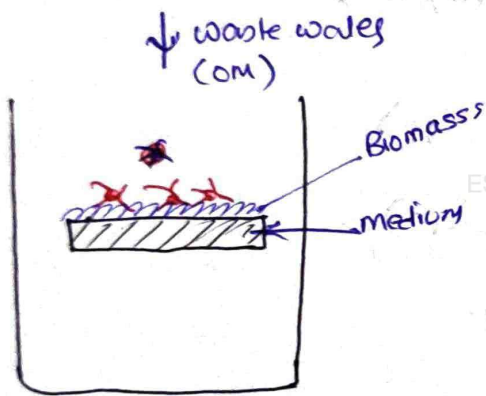
→ To establish the contact between organic matter and micro-organism following mechanism are being developed:-

(i) Attached growth system.

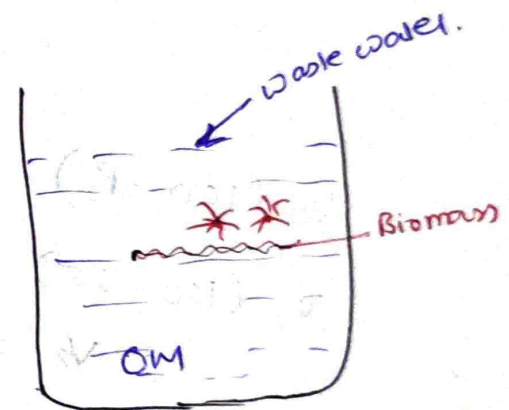
→ In this a medium is provided for attachment of biomass and waste water is passed through it. (m/o)

(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. Rotatory Biological reactor	"	"
3. Activated sludge process	Suspended	"
4. Oxidation Pond	"	"
5. Septic tank	"	Anaerobic
6. Inflow tank	"	Anaerobic.

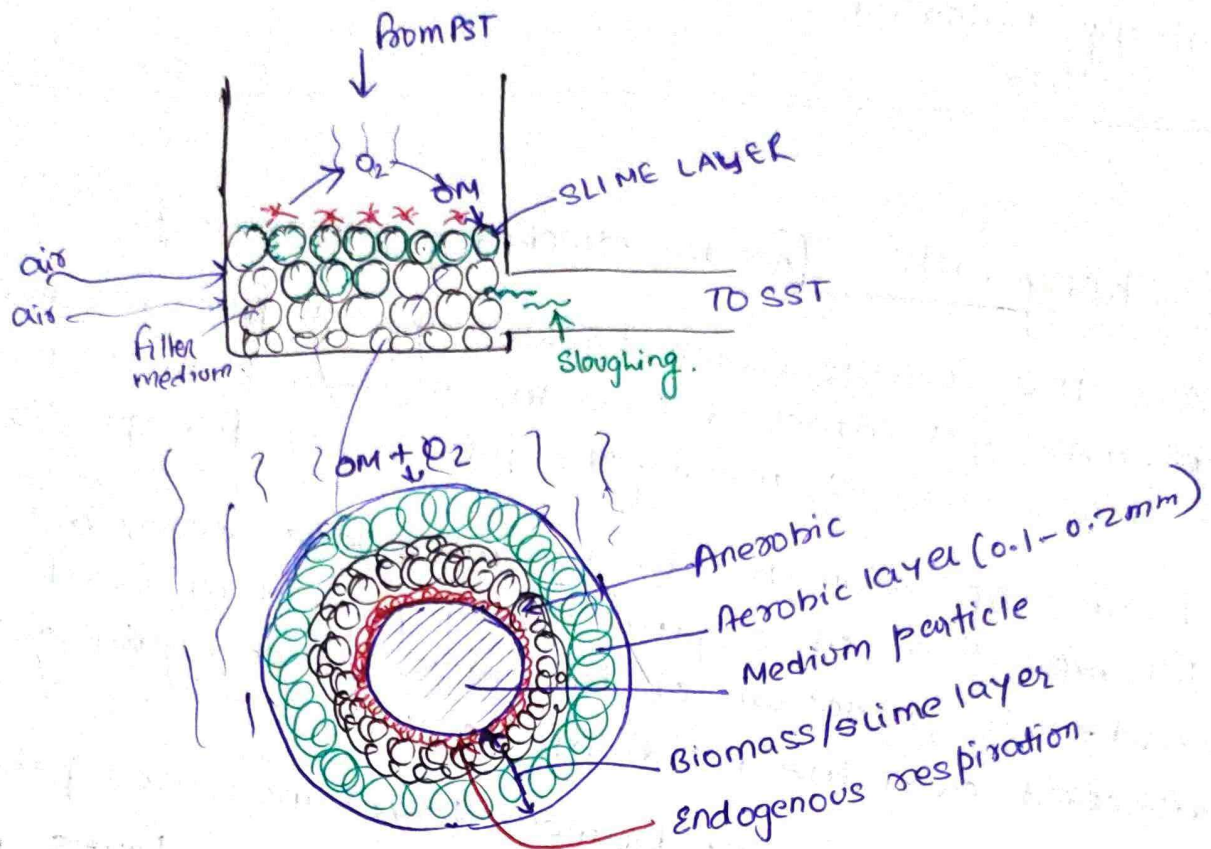
→ upflow anaerobic
sludge balanced
reactor

suspended

Anaerobic

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 bio mass 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 throughout volume of medium particles.



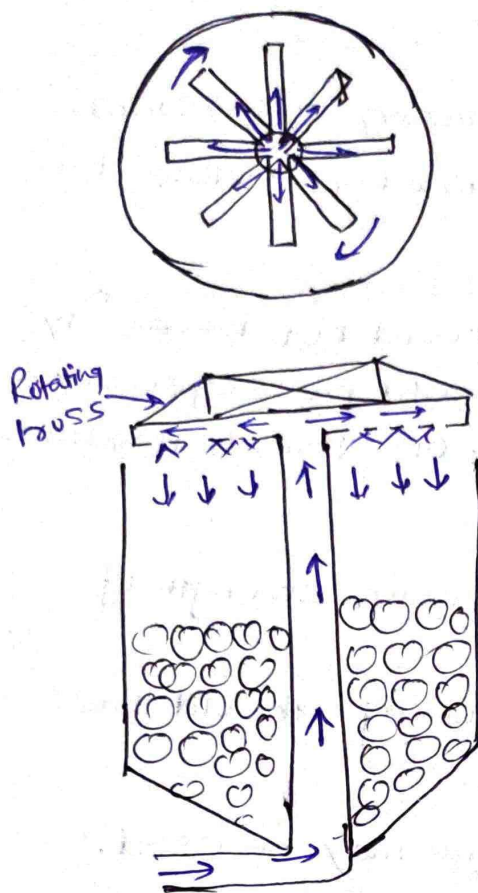
Application of waste water in trickling filter can be done by any of following method.

- (i) Rotatory distribution system.
- (ii) spray nozzle method.

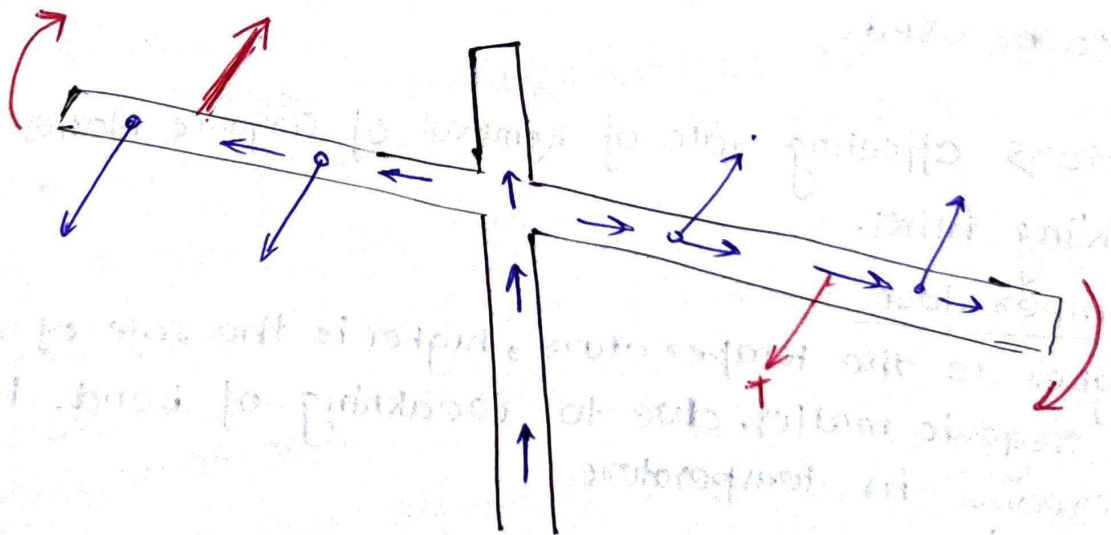
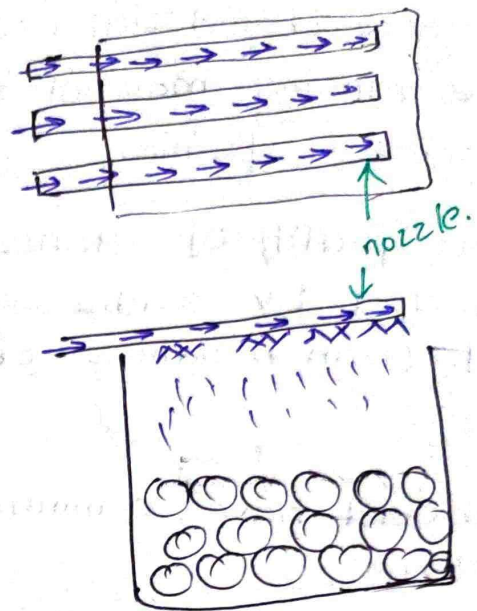
→ Application of waste water is uniform throughout the medium in rotatory method moreover application ~~is also~~ ~~continuous~~ of waste water is also continuous.

→ 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 100N/mm².
- Its resistance to freezing and thawing should be comparatively 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.

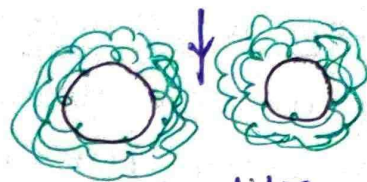
- Higher is the temperature, higher is the rate of removal of organic matter. due to weakning of bonds by increase in temperature.

(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 filter.

- Rate of filter loading is high, hence area required is comparatively less.
- Effluent obtain from trickling filter is sufficiently nitrified and has BOD removal 75 to 80%.
- Working of trickling filter is simple and does not require any supervision.
- They are flexible in operation and can treat different sewage having different concentration of BOD.
- They are self cleaning.
- Mechanical wear and tear is small.
- Moisture content of sludge obtained from trickling filter is in range of 96-98%.
- They operate more efficiently in warm weather.
- The head loss through these filters make automatic dosing of filter 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 nuisance: 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 (larvae) which act as a energy for 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, Chlorange, 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 choking 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.

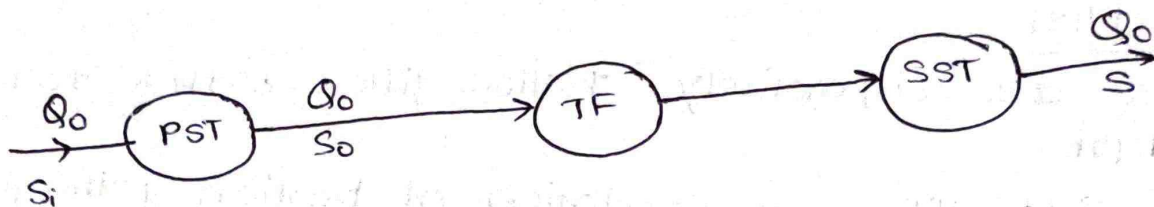
⇒ Trickling filters are generally of two types:-

(i) standard Rate trickling filter. (SRTF)

(ii) High rate trickling filter. (HRTF)

(i) SRTF

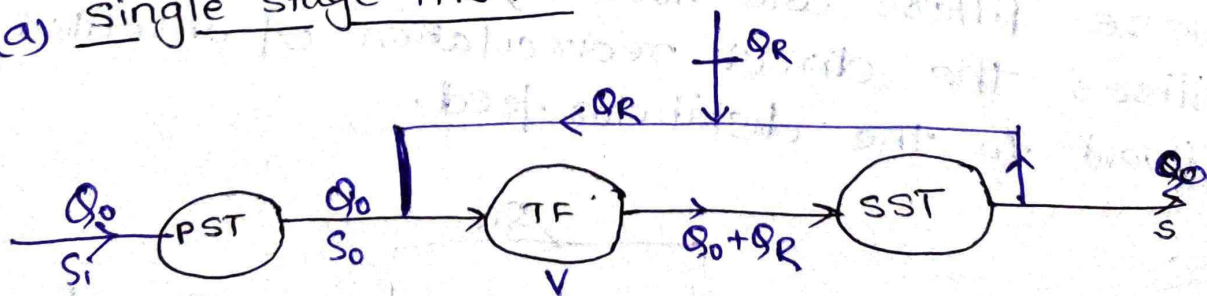
It is type of trickling filter in which rate of removal of organic matter is comparatively less, due to its lower hydraulic loading rate. as there is no provision of recirculation in it.



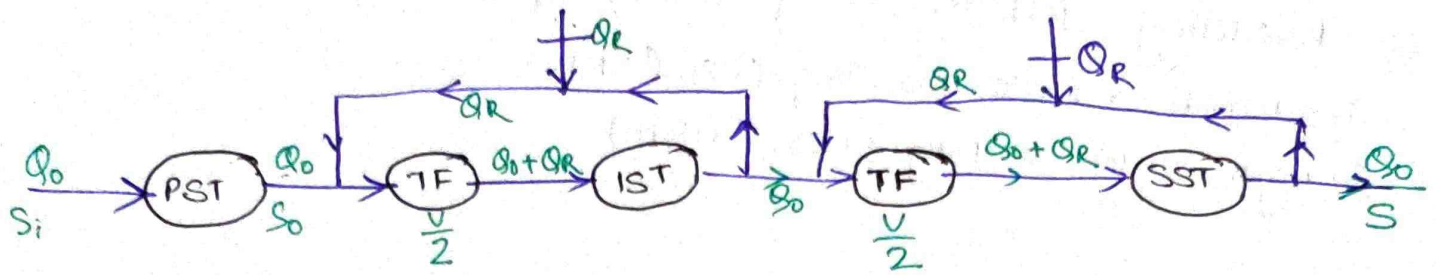
(ii) HRTF

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



⑤ Two Stage HRTF

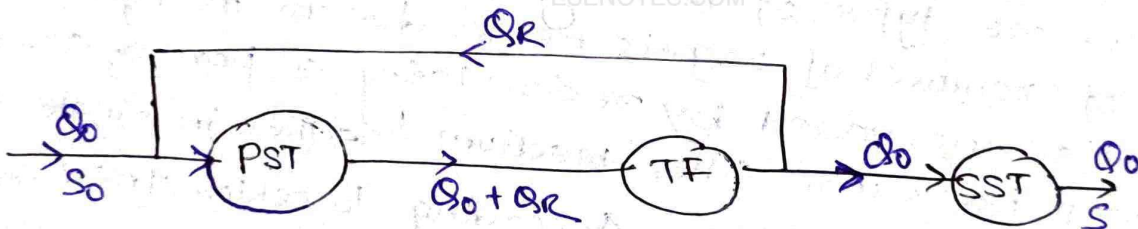


Types of High Rate trickling filter.

High rate trickling filter are further classified as:-

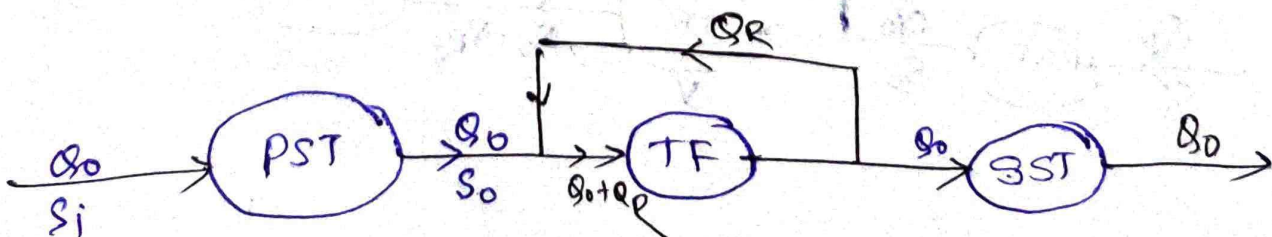
(i) Bio-filter

- 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.



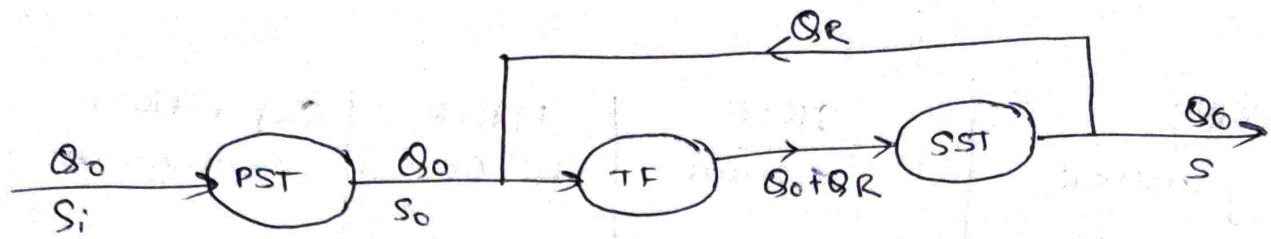
(ii) Accelu Filters

- These filters are normally 2.1.8 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 - 0.85 (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

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

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

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

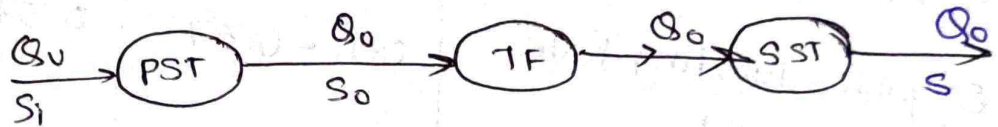
$$④ H = \frac{V}{PA}$$

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

PROPERTIES	SRTF	HRTF	Super HRTF
1. Dosing Interval	$\leq 5 \text{ min}$	15-60 sec	Continuous
2. Sloughing.	Intermediate	Continuous	Continuous
3. Effluent Quality.	Fully nitrified	Partially nitrified	Nitrified

Efficiency of Trickling filter.

i) SRTF

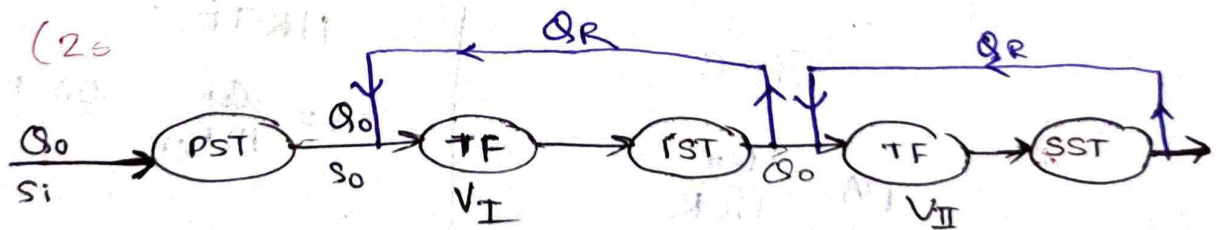


$$\eta = \frac{100}{1 + 0.0044 \sqrt{U}}$$

$$U = \text{OLR (kg/hac-m/day)}$$

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

ii) HRTF (2s)



1st stage

$$\eta_I = \frac{100}{1 + 0.0044 \sqrt{\frac{W_I}{V_I F_I}}}$$

$$W_I = Q_0 S_0 \text{ (kg/day)}$$

$$V_I = \text{Volume of TF in stage I - (hac-m)}$$

$$F_I = \text{recirculation factor}$$

$$F_I = \frac{1 + R}{(1 + 0.2R)^2}, \quad R = \frac{Q_R}{Q_0}$$

IInd stage

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

$$W_2 = (1 - \eta_I) W_I \text{ (kg/d)}$$

$$V_2 = \text{Volume}_{II} \text{ (ha-c-m)}$$

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

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

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

(i) SRTF

$$\frac{S_e}{S_0} = e^{-\left(\frac{K D}{Q_L^n}\right)} e^{-\left(\frac{K D}{Q_L^n}\right)}$$



S_e = effluent BOD in (mg/l)
 S_0 = influent BOD (mg/l)

K = Rate constant

D = depth of filter (m)

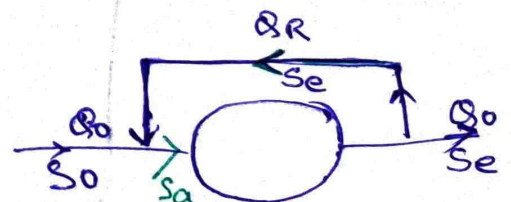
Q_L = hydraulic loading rate ($m^3/m^2/day$)

η = filter media constant

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

(ii) HRTF

$$\frac{S_e}{S_0} = \frac{e^{-\left(\frac{K D}{Q_L^n}\right)}}{(1+R) - R e^{-\left(\frac{K D}{Q_L^n}\right)}}$$



$$S_a = \frac{Q_0 S_0 + Q_R S_e}{Q_0 + Q_R} = \frac{S_0 + R S_e}{1+R}$$

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

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/hae/day including recirculation

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

$$Q_0 = 120 \times 30000 \times 10^{-3} + 3 \times 10^5 \times 10^{-3}$$

$$= 3600 + 300$$

$$= 3900 \text{ m}^3/\text{day}$$

$$BOD_i = \frac{3600 \times 200 + 300 \times 800}{3900}$$

$$= 246.15 \text{ mg/l}$$

Since PST remove 35% BOD = $246.15 - 0.35 \times 246.15$

$$= 159.99$$

$$= 160 \text{ mg/l}$$

$$n_I = \frac{100}{1 + 0.0044 \sqrt{\frac{W_i}{U_i F_i}}}$$

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

$$F_i = \frac{1+R}{(1+0.R)^2}$$

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

$$= 1.652$$

$$SA = \frac{Q_0 + QR}{HLR} = \frac{2 \times 3.9 \times 10^6}{170 \times 10^6} = 0.458 \text{ hectare}$$

$$= 458 \text{ m}^2$$

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

$$D = 24.14 \text{ m}$$

$$V = \frac{Q_0 S_0}{Q L R_{eff}} = \frac{3.9 \times 10^6 \times 160 \times 10^{-6}}{10^4} \times 10^9 = 624 \text{ m}^2$$

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

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

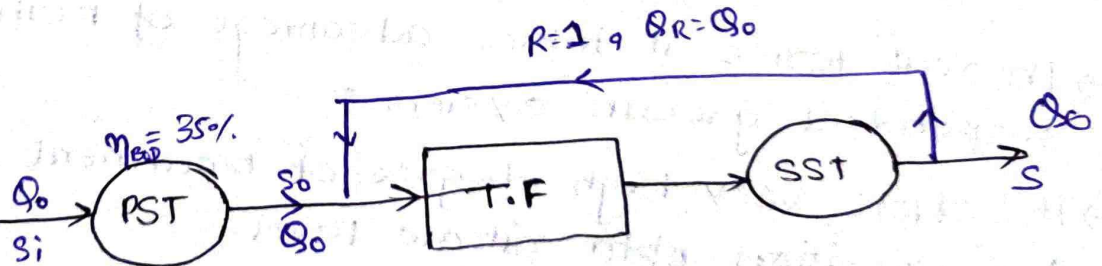
$$\eta = \frac{100}{1 + 0.0044 \sqrt{\frac{\omega}{VF}}} = \frac{100}{1 + 0.0044 \sqrt{\frac{10^4}{1.65}}} = 74.48.$$

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

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

$$S = 40.8 \text{ mg/l}$$

$Q_{DS} = 120 \text{ l/c/d}$
 $P = 3 \times 10^4$
 $BOD_{DS} = 200 \text{ mg/l}$
 $Q_{IS} = 3 \times 10^5 \text{ l/d}$
 $BOD_{IS} = 800 \text{ mg/l}$



ROTATORY BIOLOGICAL REACTOR/ CONTRACTOR (RBR/RBC)

(Aerobic attached growth system).

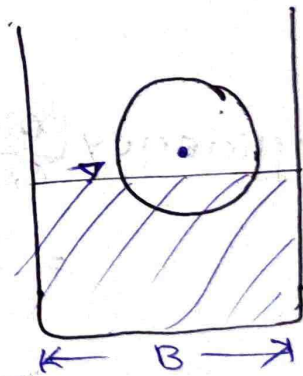
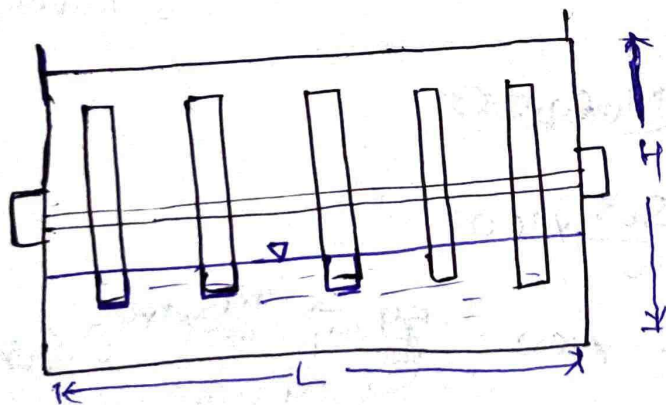
- In this case closely spaced rotating disk is used as a medium for the growth of bio-mass layer (slime layer)
 - These disk are kept emerged in waste water upto 40% of their size. and are exposed alternatively to the atmosphere and the waste water due to which micro-organism in the bio-mass layer comes 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 weakens the bond between the two.
 - This biomass layer gets removed off from the disk by the turbulence 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
- ~~after~~

→ 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 $20 \text{ gm BOD/m}^2/\text{day}$ and 3m diameter disc spaced 5cm apart on center @ 54gm, use BOD at 54 gm/c/d and sewage flow of 200 l/c/d . Assume 0.2 m total clearance for the disk and depth of the tank is 2 m . Permissible BOD in effluent = 4 mg/l . Determine:-

- No. of disk require. and tank volume
- Hydraulic loading rate.
- Surface loading rate.
- Efficiency
- Excess sludge at the rate of 0.6 kg/kg of BOD removed.



(i) Total BOD to be removed = 54×10000
 $= 5.4 \times 10^4 \text{ gm/day.}$

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

Area of one disk available for removal of BOD.

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

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

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

(iii) Height of tank = 2m

$$\text{width} = d + 0.2 = 3.2 \text{ m.}$$

$$\begin{aligned} \text{length} &= \text{no. of disk} \times \text{spacing} \\ &= 191 \times 5 \times 10^{-2} = 9.55 \text{ m.} \end{aligned}$$

$$\begin{aligned} \text{Volume} &= L \times B \times H \\ &= 2 \times 3.2 \times 9.55 = 61.12 \text{ m}^3. \end{aligned}$$

$$\begin{aligned} \text{(iii) Hydraulic loading rate} &= \frac{Q_0}{N \times \frac{\pi d^2}{4} \times 2} = \frac{2000 \times 1000}{191 \times \pi \times \frac{3^2}{4} \times 2} \\ &= 74.12 \text{ m}^3/\text{m}^2/\text{day.} \end{aligned}$$

$$\begin{aligned} \text{(iv) Surface loading rate} &= \frac{Q_0}{X\text{-area}} = \frac{2000 \times 1000 \times 10^{-3}}{3.2 \times 2 \times 0.4} = 78.1 \text{ m}^3/\text{m}^2/\text{d.} \end{aligned}$$

$$\text{(v) Efficiency} = \frac{Q_0 S_0 - Q_0 S}{Q_0 S_0} \times 100$$

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

$$= \frac{S_0 - S}{S_0} \times 100 = \frac{54 \times 10^3 - 49 \times 10^3}{54 \times 10^3} \times 100$$

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

(vi) BOD removed,

$$= Q_0 (S_0 - S)$$

$$= 2000 \times 1000 \left(\frac{54}{200} \times 10^3 - 49 \right) \times 10^{-6}$$

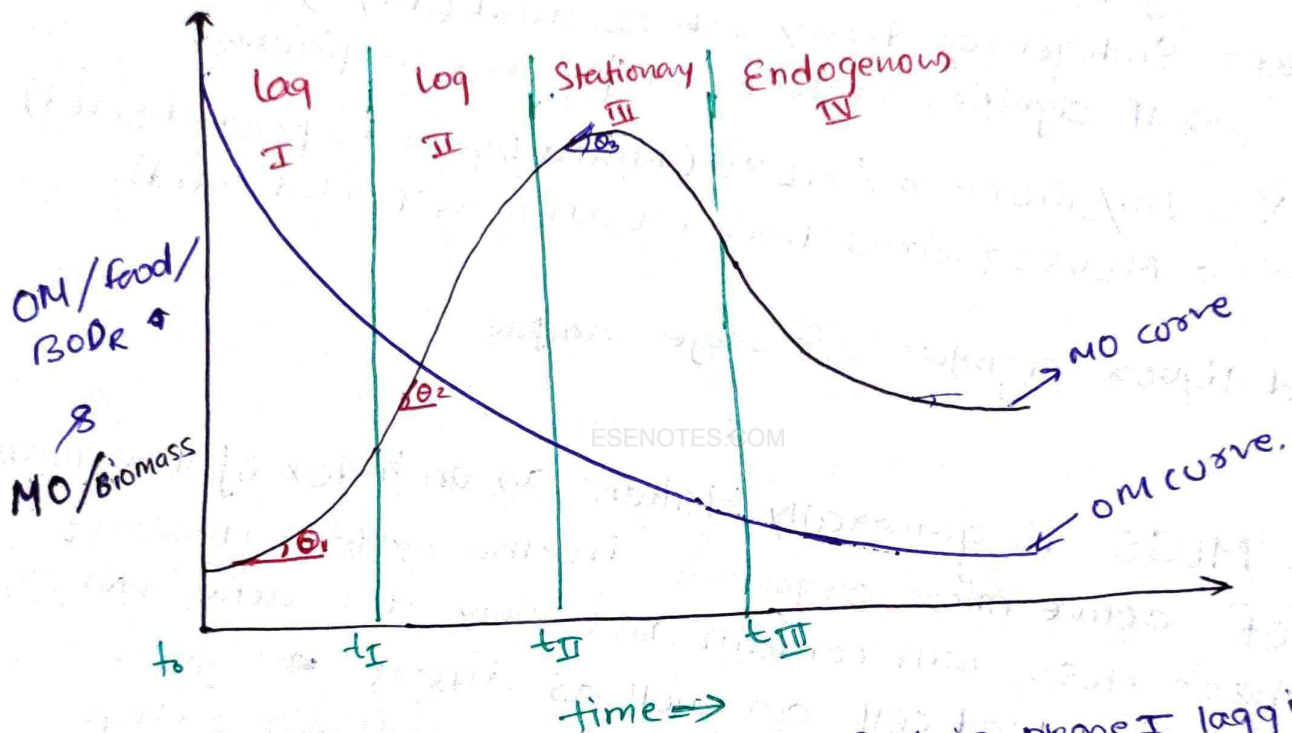
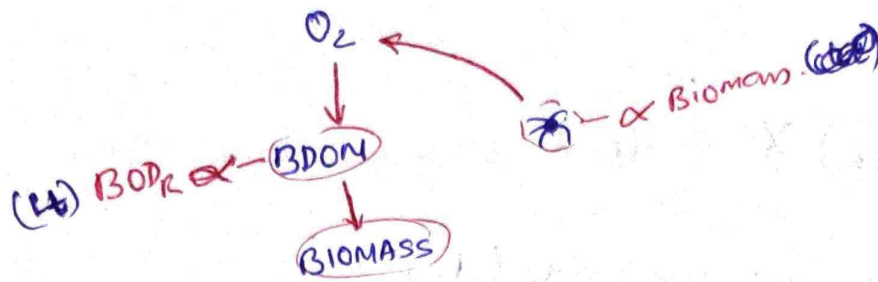
$$= 44.2 \text{ kg/day.}$$

$$\begin{aligned} \text{Excess solid produced} &= 0.6 \times 44.2 \\ &= 26.52 \text{ kg/day.} \end{aligned}$$

II

Activated Sludge Process (ASP)

(Aerobic suspended growth system)



$\theta_1 \ll \theta_2 \rightarrow$ Rate of growth of MO in phase I lagging.

(i) $OM/Feed/BODR$

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

$$\frac{ds}{dt} = -K_d s \quad (A)$$

K_d = deoxygenation constant (d^{-1})
 s = Remaining BOD (mg/l)

(i) M_0 / Biomass.

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

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

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

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 = M_0 / Biomass / MLSS (Mixed liquor suspended solid)

= MLVSS (Mixed liquor volatile suspended solid).

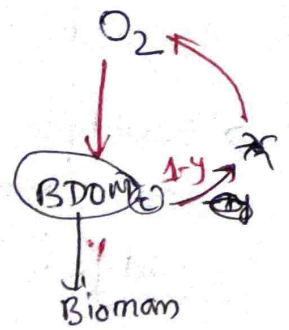
Mixed liquor signifies sewage sample.

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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-organisms 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 preferred than MLSS as it eliminates the effect 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 Bioman.



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

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

eg

t	OM	Bio mass.
3pm	2 kg	200 gm
3:30pm.	1.5 kg	400 gm.

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

$$ds = 1.5 - 2 = -0.5 \text{ kg} = -500 \text{ gm}$$

$$dx = 400 - 200 = 200 \text{ gm.}$$

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

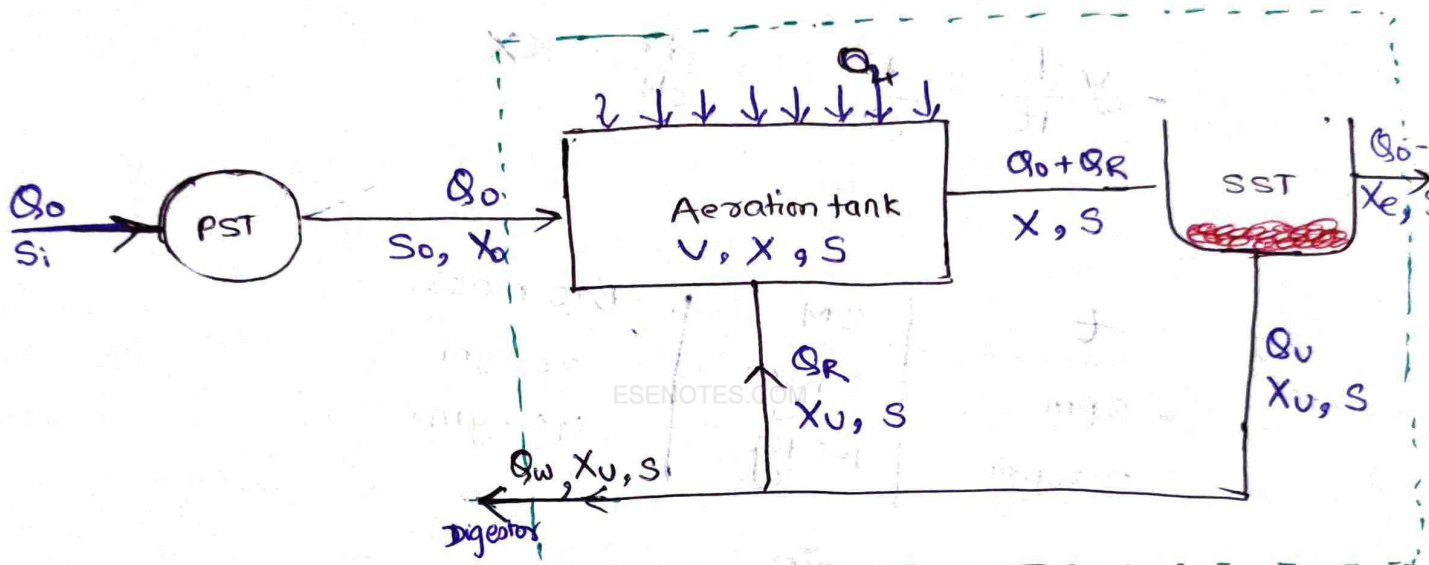
$$-0.4 \frac{(-500) \text{ gm}}{30 \text{ min}} = \frac{200 \text{ gm}}{30 \text{ min.}}$$

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

Activated sludge process.

→ Sludge generated in SST must 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.



$$S_0 > S$$

$$X > X_0$$

$$Q_0 > Q_R$$

$$X_u > X$$

$$X_e > X$$

(i) Biomass.

Biomass IN + BIOMASS GROWTH = BIOMASS OUT

$$Q_0 X_0 + \left(\frac{dx}{dt} \right) V = (Q_0 - Q_w) X_e + Q_w X_u$$

$$(K - K_{ER}) X V = Q_w X_u$$

$$K = \frac{Q_w X_u}{X V} + K_{ER}$$

(ii) Food

FOOD IN - FOOD CONSUMED = FOOD OUT

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

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

$$Q_0 S_0 - \frac{K X V}{Y} = Q_0 S$$

$$K = \frac{Q_0 (S_0 - S) Y}{V X} \quad \text{--- (iii)}$$

From (i) and (ii)

$$\frac{Q_w X_u}{X V} + K_{ER} = \frac{Q_0 (S_0 - S) Y}{V X}$$

Assumption made in above analysis:-

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

(ii) BOD removal is not considered in SST.

(iii) Mixture is considered to be homogeneous inside the aeration tank (It is achieved by mixing induced mixing in aeration tank).

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

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

$$Q_0 S_0 - K_d S V = Q_0 S$$

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

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

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

→ 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 reactors results in following equation.

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

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.

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

(ii) Organic Loading Rate (OLR)

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

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

(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.

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

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

(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.

$$\theta_c = \frac{VX}{(Q_0 - Q_w)X_e + Q_w X_0}$$

If X_e is neglected,

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

Note:-

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

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

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

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

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

$$\text{Unit} = (\text{kg/kg-d}) \\ = \text{day}^{-1}$$

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$$\begin{cases} \text{(i) } \frac{F}{M} \uparrow (F \uparrow, M \downarrow) \Rightarrow \eta \downarrow \\ \text{(ii) } \frac{F}{M} \downarrow (F \downarrow, M \uparrow) \Rightarrow \eta \uparrow \end{cases}$$

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

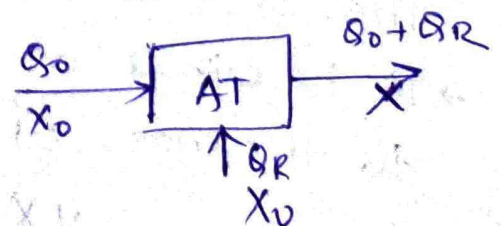
(vi) Recirculation Ratio $(R = \frac{Q_R}{Q_0})$

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

$$Q_0 X_0 + Q_R X_u = (Q_0 + Q_R) X$$

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

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



(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 1gm 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)".

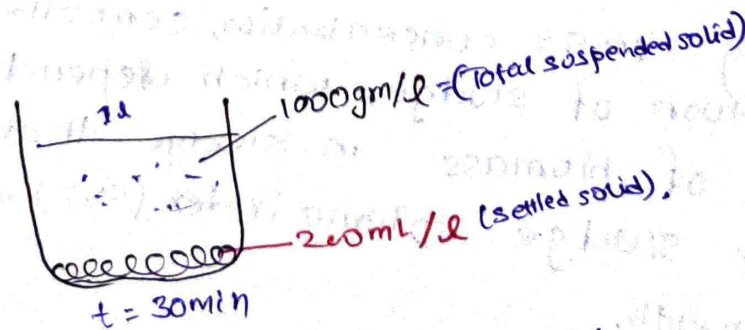
→ The sample after being remixed is further tested for concentration of MLSS in it by performing the standard test. " X_s (mg/lit)".

→ for municipal sewage sludge volume index SVI

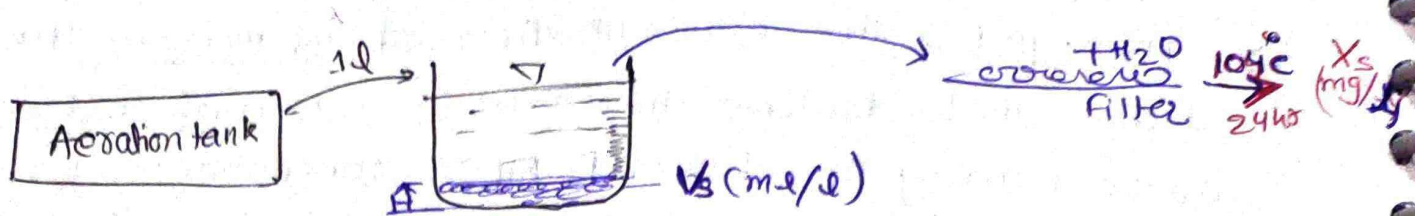
$$\boxed{SVI = 50-150 \text{ ml/gm}}$$

$$SVI \rightarrow X_u \rightarrow Q_R \rightarrow X \rightarrow \frac{F}{V} \rightarrow \eta(\%)$$

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



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



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

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

$$SVI \text{ (ml/gm)} \quad X_u \text{ (mg/l)}$$

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

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

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

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

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

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

$(Q_w = Q_U - Q_R)$

→ 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.

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

Q22-3

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$$\left[\text{Note } SVI \propto \frac{1}{X_U}, \quad SDI \propto X_U \right]$$

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

• waste water related data.
 $Q_0 = 500 \text{ m}^3/\text{hr}$

$BOD_i = 150 \text{ mg/l}$

$BOD_e = 10 \text{ mg/l}$

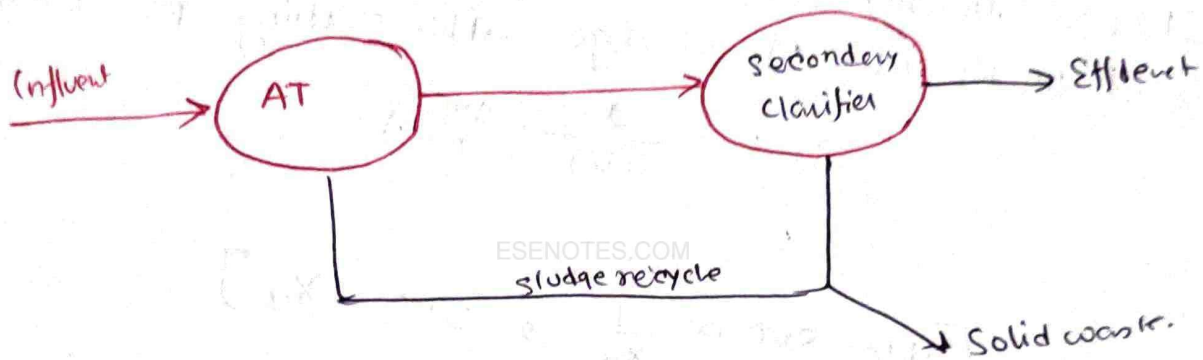
• Aeration tank related data:-

HRT = 8 hr

Mean cell residence time or sludge age θ_c Solid retention time $\theta_c = 240 \text{ hr}$

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)

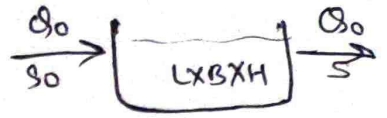
$$\begin{aligned}
 \text{(i) } FM &= \frac{Q_0 S_0}{V X} = \frac{500 \times 150 \times 10^3 \text{ m}^3/\text{hr} \times \text{mg/l}}{4000 \times 2000 \text{ mg/l}} \\
 &= 9.375 \times 10^{-3} (\text{kg/kg/day}) \\
 &= 9.375 \times 10^{-3} \times 24 \\
 &= 0.225 \text{ kg/kg/day}
 \end{aligned}$$

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

$$\theta_c = \frac{V X}{Q_w (S_0 - S_e) + Q_w X_u}$$

$$240 = \frac{4000 \times 2000}{Q_w \times X_u} \Rightarrow Q_w X_u = \frac{4000 \times 2000}{240} = 3333.33 \text{ kg/day}$$

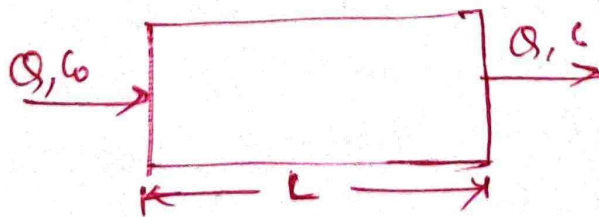
Q A waste water having an organic concentration of 54 mg/l is flowing at a steady state $0.8 \text{ m}^3/\text{day}$ through a detention tank of dimension $2 \text{ m} \times 4 \text{ m} \times 2 \text{ m}$. If the contents of the tank are well mixed and decay constant is 0.1 d^{-1} the outlet concentration in mg/l is _____.



$$\frac{Q}{V} \quad t_d = \frac{V}{Q} = \frac{2 \times 4 \times 2}{0.8} = \frac{16}{0.8} = 20$$

$$S_e = \frac{S_0}{1 + k_d \cdot t} = \frac{54}{1 + 0.1 \times 20} = 18 \text{ mg/l}$$

Q Consider the reactor shown in figure. The flow rate through the reactor is $Q \text{ m}^3/\text{hr}$. The concentration is in mg/lit of a compound in the effluent and effluent are C_0 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 ~~LCMFR~~ $\frac{\text{LCMFR}}{\text{LPFR}}$ while keeping the cross-section of the reactor constant. Assuming steady state & for $C/C_0 = 0.8$ the value of ~~complete~~ $\frac{\text{LCMFR}}{\text{LPFR}}$ is _____.



$$\begin{aligned} \frac{C}{C_0} &= 0.8 \\ C &= 0.8 C_0 \\ \frac{C_0 - 0.8 C_0}{C_0} &= \frac{0.2}{1} \\ \eta &= 20\% \end{aligned}$$

$$\frac{C}{C_0} = 0.8$$

For complete CMFR $\Rightarrow C = \frac{C_0}{1+Kt}$

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

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

$$0.8 + 0.8Kt = 1$$

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

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

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

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

for steady state condition.

$$V = \text{Const}$$

$$L = V \times t$$

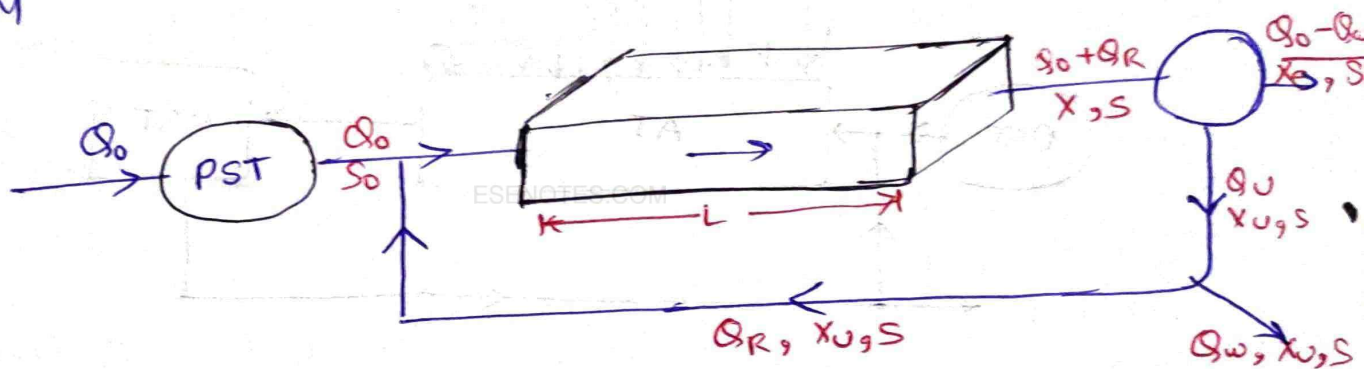
$$\frac{L_{CMFR}}{L_{PFR}} = \frac{V \times t_{CMFR}}{V \times t_{PFR}} = \frac{0.25 K}{K (0.233)} = 1.221$$

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 efficiency 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.

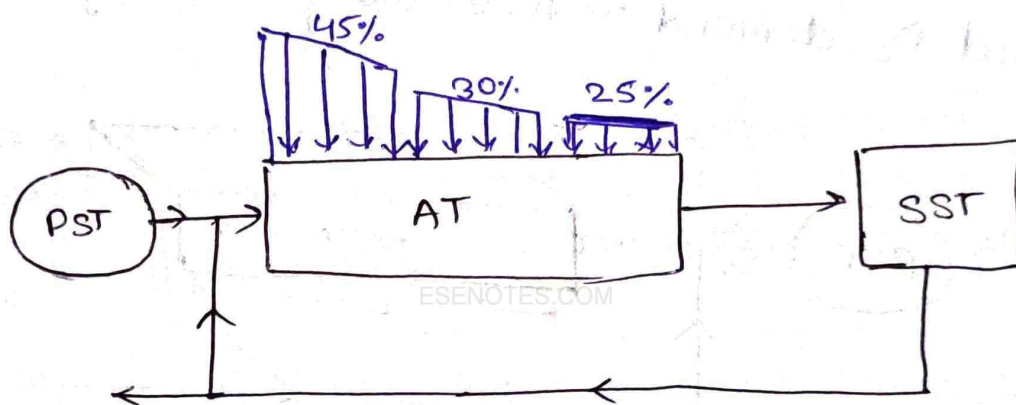


- The main limitation of plug flow process are:-
 - (i) aeration tank volume requirement is very high
 - (ii) The lack of operational stability at times of excessive variation in rate of flow or its BOD concentration.
 - (iii) Uniform distribution of O_2 in this case leads to deficiency of O_2 in initial 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 better utilisation of oxygen along the length of the tank.
- ~~Such a process~~ Ordinarily 45% of air is supplied to first $\frac{1}{3}$ length of tank followed by 30% and 25% for remaining ~~two~~ two, $\frac{1}{3}$ length of tank respectively.



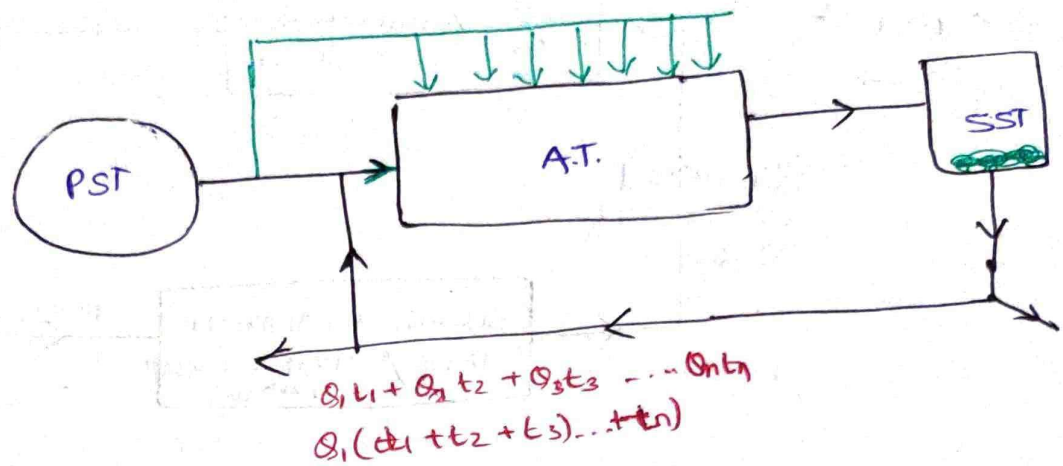
(iii) Step Aeration Process

- In this process the sewage is introduced along the length of the aeration tank in several steps while the return sludge is introduced 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 appreciable reduction in aeration tank volume, without lowering BOD removal efficiency.

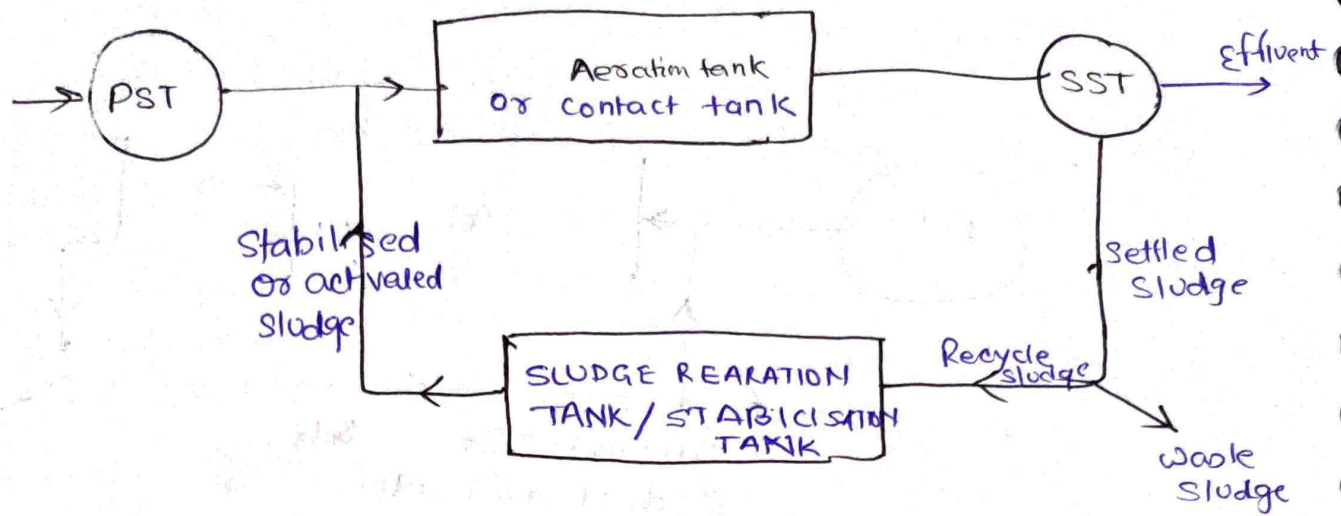
- This method has considerable capacity to absorb shock organic loading.
- It can be used for the plant of capacity upto 1000 MLD



(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 comparatively shorter period of time $0.5-1.5 \text{ hr}$ 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 flocs are retained in SST where the effluent from the contact tank enters.
- The settled sorbed organics & flocs are then transferred to a sludge aeration tank termed as stabilisation tank Aerodigester where the organics are decomposed over a period of about 3 to 5 hours. (6 hr) before it is fed back into the contact aeration tank.

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



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

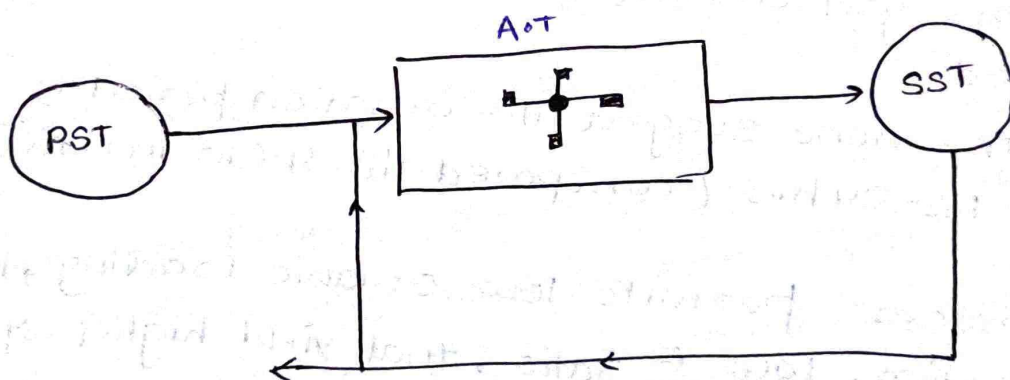
→ This process also prevents greater resistance to toxic substance in sewage as biological mass is exposed to main stream of sewage

→ 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, particularly for ~~the~~ town where municipal & industrial waste water flow together.
- Complete mix signifies that complete mixing of incoming activated sludge is done with waste water, which increases 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 homogenous environment created by mixing.
- Operational stability of this process ~~is~~ is ~~com~~ with respect to shock organic loading is comparatively 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 substantial saving in construction and aeration cost.

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

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

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

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 comminators are often provided.

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

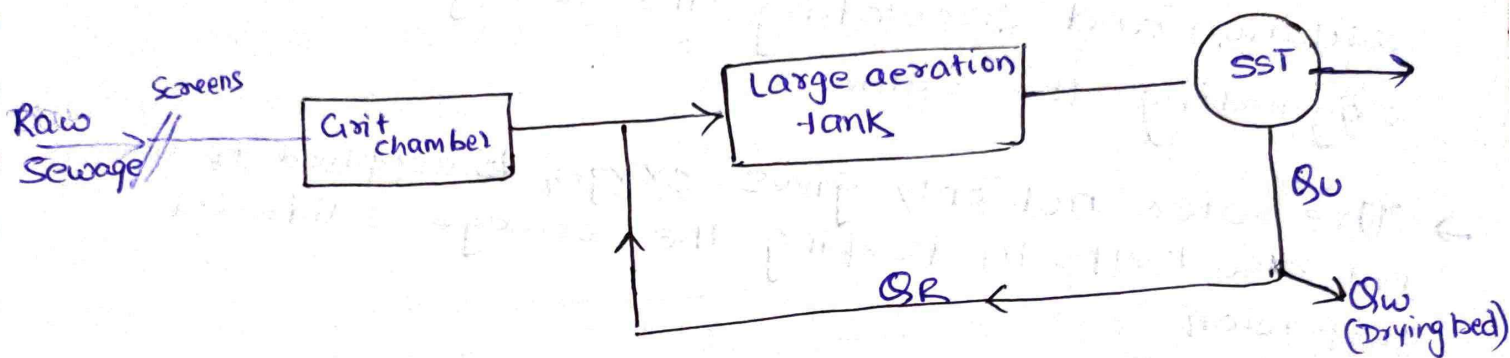
→ This process permits low organic loading, high MLSS concentration, low $\frac{F}{M}$ ratio, that yield higher efficiency of 95-98%.

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

→ This also offers of no separate 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:-

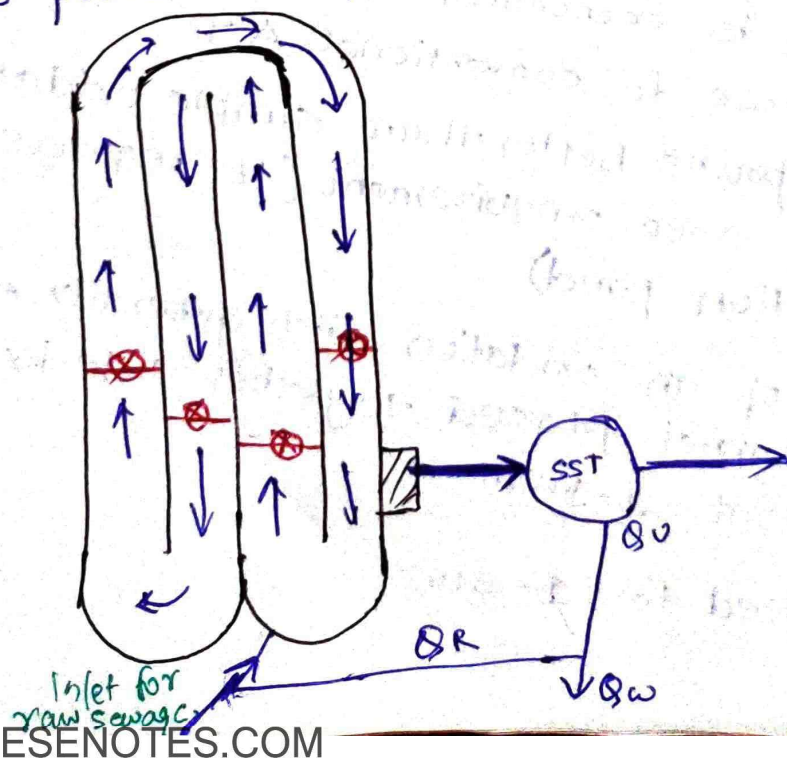
Oxidation Ditch.

- It is also termed as (Pasveer type plant) or extended 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 involves no. of ditches channel placed together side by side having depth of about 1-1.5m
- The width is limited to 1-5m

- 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 150-1000m.
- 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 thereby oxygenating the same.
- The rotor not only gives oxygen to supplied to sewage but also helps in keeping the sewage solids in suspension.

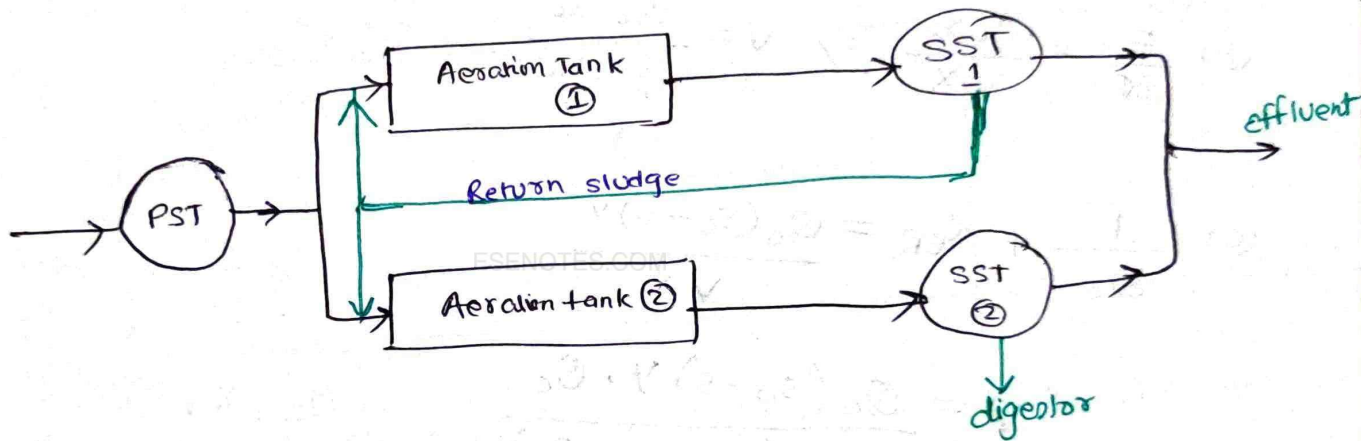
Design data:-

- The quality of effluent obtained is comparatively very good with SS removal of 95% & BOD removal of 98%.
- The power consumption is comparatively more than ordinary ASP. (which is compensated by elimination of PST and anaerobic digester)
- This process requires skilled supervision.



Activated Aeration Process.

- In the activated aeration process, a pair of conventional activated sludge plant operating in parallel 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)	θ_c (days)
(i) Conventional Process ($\eta \uparrow$)	PLUG FLOW	1500-3000	0.4-0.3	4-6	5-8
(ii) Complete Mix (rate $\uparrow$)	Complete mix	3000-4000	0.5-0.3	4-5	5-8
(iii) Extended Aeration ($\eta \uparrow$, rate $\uparrow$)	Complete mix	3000-4000	0.1-0.18	12-24	10-25

Types of process	Q_R/Q_0	η_{BOD}	$\frac{MLVSS}{MLSS}$	Volumetric loading $Kg BOD/m^3$
① Conventional	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

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

$$a) HRT = \frac{V}{Q_0} \Rightarrow V = Q_0 \cdot HRT \quad \{HRT\}$$

$$b) \frac{F}{M} = \frac{Q_0 S_0}{V X} \Rightarrow V = \frac{Q_0 \cdot S_0}{\frac{F}{M} \cdot MLSS} \quad \left\{ \frac{F}{M}, X \right\}$$

$$c) \frac{1}{\theta_c} + K_{ER} = \frac{Q_0 (S_0 - S) Y}{V X}$$

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

→ For conventional plant depth should be range of
 $H = 3-4.5$ (It control aeration efficiency)

$B = 5-10$ m (It control mixing efficiency)

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

$$L = 30-100 \text{ m}$$

$$\text{Free board} = 0.3-0.5 \text{ m}$$

→ Oxygen requirement of the aeration tank

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

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

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

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 @ $20^\circ C$, 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 transferred under field condition ($\text{kg O}_2/\text{kwh}$)

N_s = oxygen transfered capacity under standard conditions. ($\text{kg O}_2/\text{kwh}$)

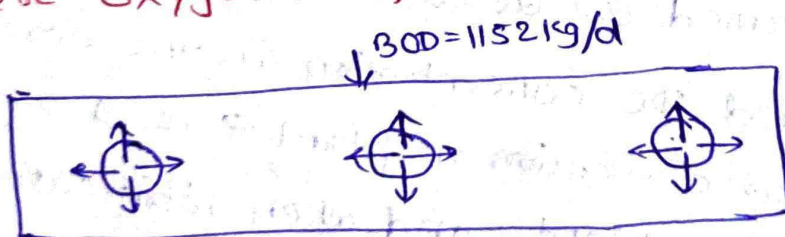
D_s = Dissolved oxygen + saturation value for sewage @ operating temperature.

D = operation DO level in aeration tank.

T = Temp ($^{\circ}\text{C}$)

α = correction factor (0.8 - 0.85)

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



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

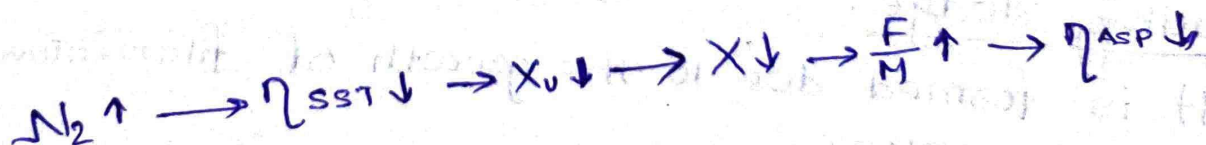
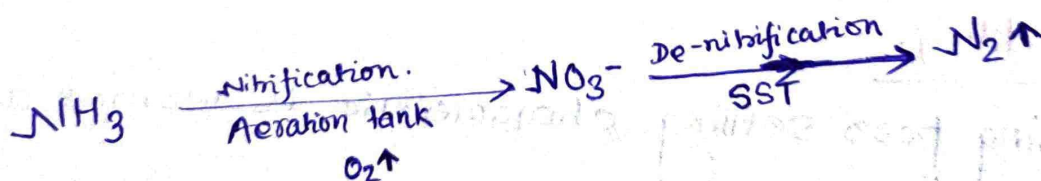
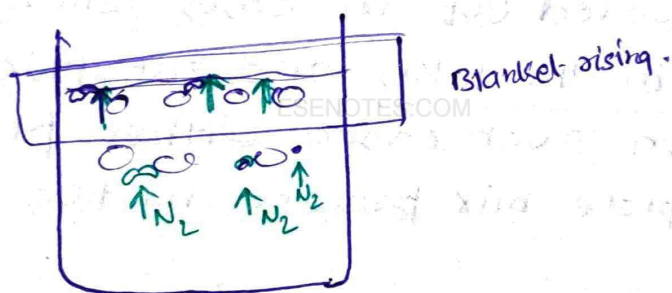
Power requirement by each aerator = $\frac{16}{0.8} = 20 \text{ HP}$

Nitrification in ASP

→ ASP is designed for satisfaction of 1st 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 efficiency of ASP.

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



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

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

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

$$\boxed{\downarrow \theta_c = \frac{VX \downarrow}{Q_w X_w \uparrow}}$$

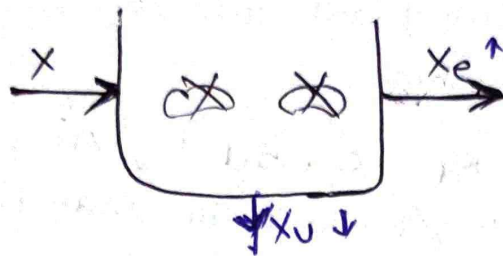
→ In some cases nitrification followed by denitrification is required to be carried out in cases when effluent is to be disposed in the lakes and eutrophication is to be avoided. In such cases either plug flow process or complete mix process in two stages is adopted.

Sludge Bulking.

- Sludge having poor settling characteristic is termed as "bulking sludge".
- It is formed due to the growth of filamentous micro-organisms in the system.
- These are found due to low nutrient content high sludge age, & low $\frac{F}{M}$ ratio. ($< 0.3d^{-1}$)
- Sludge Bulking result in excessive loss of MLSS out from the system thereby reduces its efficiency.

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

$$\text{BOD}_5 : \text{N} : \text{P} = 100 : 5 : 1.$$



Following are the Properties of ASP.

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 to 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.

~~③ Oxidation Pond/Slab/Anaerobic/Symbiotic Pond~~

Q An activated sludge system is to be used for secondary treatment of $10000 \text{ m}^3/\text{day}$ 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.

$Y = 0.5 \text{ kg/kg}$, $k_d = 0.05 \text{ d}^{-1}$, 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.

$$\begin{aligned} \text{(i) Volume} &= \frac{Q_0(s_0 - s) Y \theta_c}{(1 + K_{ER} \theta_c) X} \\ &= \frac{10000 (150 - 5) 0.5 \times 10}{3000 (1 + 0.05 \times 10)} \\ &= 1611.11 \text{ m}^3 \end{aligned}$$

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

$$\begin{aligned} Q_w X_u &= \frac{1611.11 \times 3000 \times 10^{-6} \times 10^3}{10} \\ &= 483.3 \text{ kg/day.} \end{aligned}$$

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

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

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

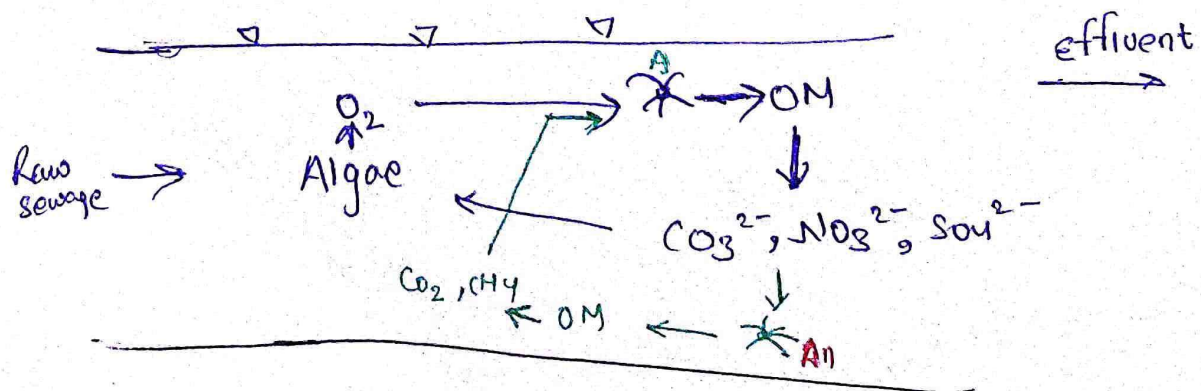
(iv) sludge recirculation pump capacity $Q_R = R Q_0 = 0.428 \times 10^4 = 4280 \text{ m}^3/\text{day}$

(v) O_2 required = $\frac{Q_0 (S_0 - S)}{0.68} - 1.42 Q_w X_u$
 $= \frac{10000 (150 - 5) \times 10^3 \times 10^{-6}}{0.68} - 1.42 \times 483.3$
 $= 1446.02 \text{ kg/day}$

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(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 anaerobic micro organism, anaerobic 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 anaerobic MO.
- Such mutual beneficial relationship is termed as "symbiotic" relationship.



→ ~~stabilisation~~ pond may be classified as aerobic, facultative or anaerobic 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 anaerobic conditions of settled sludge.

(ii) Anaerobic Pond

Here stabilisation of waste is mainly brought about by anaerobic condition leading to formation of unstable end product (acids, CO_2 , CH_4)
→ these depth varies in the range of 2.5-4m

(iii) Facultative Pond

→ In upper layers, here work is under aerobic condition while anaerobic 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, whereas the pond ~~at~~ 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 ~~is~~ = 0.5 - 1 hectare.

$$\Rightarrow \frac{L}{B} = 2$$

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

Latitude ($^{\circ}$ 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 community or villages or having ~~less~~ no or less power supply.
- Removal of sludge from the pond is done approximately once in 6 year for 1.2m deep ponds. where rate of accumulation of sludge is 2.5cm/~~year~~ depth/year.
- Oxidation pond are quite suitable for communities which are hot and dry. having 200 or more sunny days.
- The maintenance cost is also comparatively low. No skilled man power is required, they are quite flexible and do not get upset by ~~the~~ BOD fluctuation.
- Their major drawback is nuisance 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.

iii) 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 Kalkaji, contributing sewage @ 120 l/c/d. BOD of this waste water is 300 mg/l.
Kerala, latitude - 8°

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

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

$$SA = \frac{Q_0 \cdot S_0}{OLR}$$

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

$$= 0.6 \text{ hectare (0.5 - 1 hectare)}$$

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

$$SA = L \times B$$

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

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

$$B = 54.77 \text{ m} \sim 55 \text{ m}$$

$$L = 2 \times 54.77 = 109.54 \text{ m} \sim 110 \text{ m}$$

Assume, $H = 1.5 \text{ m}$

Size of tank = $110 \text{ m} \times 55 \times 1.5 \text{ m}$.

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

$$= \frac{110 \times 55 \times 1.5}{600} = 15.125 \text{ days.}$$
$$= 2.16 \text{ weeks}$$
$$(2-6) \text{ weeks.}$$

Note: Design of inlet pipe.

Assume avg. velocity of sewage $(0.8 - 1 \text{ m/s}) = 0.9 \text{ m/s}$.

operating time be 10 hr .

$$Q = AV$$

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

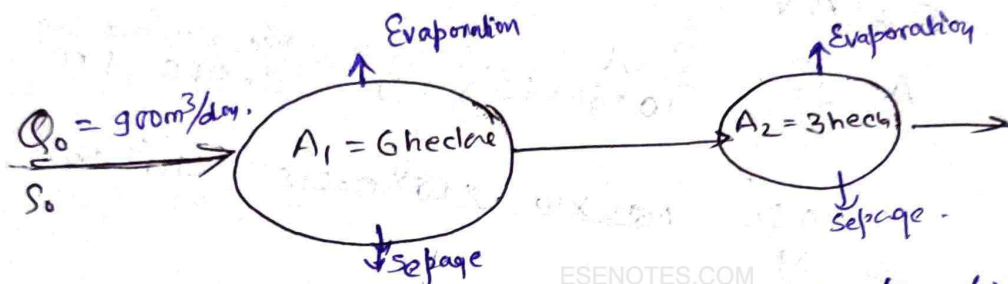
$$\frac{\pi D^2}{4} = 0.01851 \times 10^4 \text{ cm}^2$$

$$D = 15.35 \text{ cm}$$

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

(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 an evaporation and seepage loss of 2.5 mm of water per day.



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

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

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

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

$$\begin{aligned} \text{No. of days of winter storage} &= \frac{8.1 \times 10^4}{675} = 12 \text{ days} \\ &= 4 \text{ month} \end{aligned}$$

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 area required = 5 to 10% of that required in conventional oxidation pond.
- $\eta_{BOD} = 65-90\%$
- It is used for treating industrial waste water as well as city sewage.



VI Septic tank (Anaerobic suspended growth system).

- A septic tank may be defined as primary sedimentation tank with longer detention period of 24 to 48 hr. in comparison 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 anaerobically.
- Digestion of sludge leads to development of foul gases hence it is completely 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 & grease to the surface leading to the formation of scum layer at the top, which act as an insulator, thereby avoid the release of odorous gases & heat out from the system.
- Septic tanks are generally provided in areas where sewers 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 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 of the tank and hanging baffle wall is provided at the outlet of the tank in front of the inlet.

→ $t_d = 24-48 \text{ hr}$

→ cleaning period = 6 to 12 month. ($\neq$ 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

→ $\frac{L}{B} = 2-3$

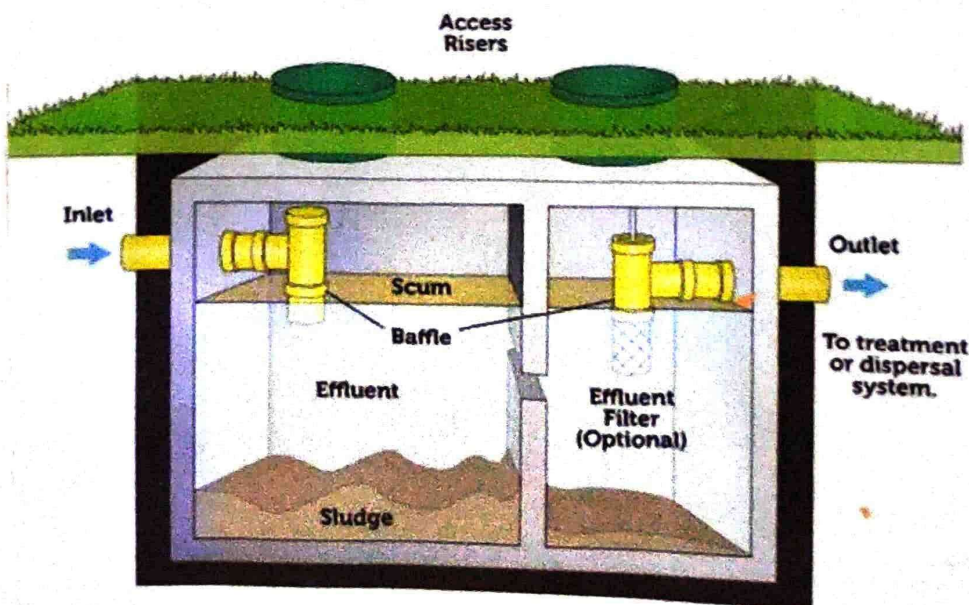
⇒ width, B $\neq$ 0.75 m.

→ H, 1.2 - 1.8 m.

→ $f_B = 0.3 - 0.5 \text{ m}$

⇒ Volume of tank = $V_1 + V_2$

$V_1 = Q_0 t_d$; $V_2 = \text{RAS} \times \text{CP}$



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

$$\text{Design discharge, } Q_0 = 0.8 \times 150 \times 120 \times 10^{-3} \\ = 14.4 \text{ m}^3/\text{day}$$

$$\text{Assume } t_d = 24 \text{ hr.}$$

$$\text{clearing period} = 1 \text{ yr.}$$

$$\text{rate of accumulation of sludge} = 30 \text{ l/c/yr.}$$

$$V = V_1 + V_2$$

$$V_1 = Q \times t_d$$

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

$$V_2 = RAS \times CP$$

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

$$V = 14.4 + 4.5 = 18.9 \text{ m}^3$$

$$\text{Assume } H = 1.5 \text{ m}$$

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

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

$$L = 2B$$

$$SA = L \times B$$

$$12.6 = 2B^2$$

$$B = 2.5 \text{ m} \approx 0.7 \text{ m.}$$

$$L = 5 \text{ m.}$$

$$F_B = 0.3 \text{ m.}$$

→ Disposal of effluent from the septic tank.

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

→ It contains large amount of organic matter ($BOD = 100-200 \text{ mg/l}$)

→ 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 methods:-

(i) Biological filter.

(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 prior to its disposal in Biological filters or in upflow Anaerobic filters.

(iv) 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

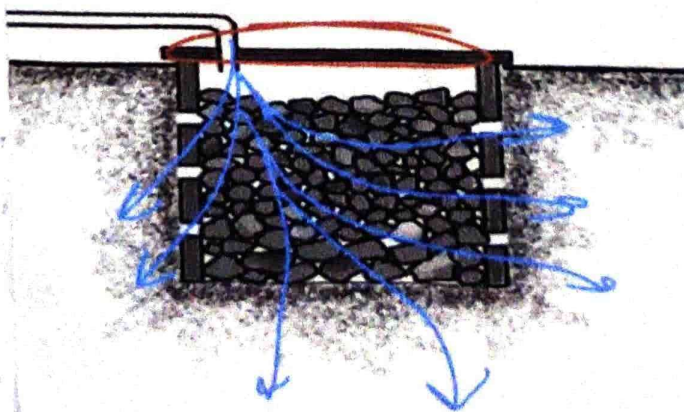
- In this effluent is allowed to enter into a masonry chamber called distribution box from where it is uniformly distributed through an underground network of open jointed pipes into absorption trenches, called Dispersion trench.

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

- Maximum rate of disposal of effluent is given by

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

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.

$$Q, \text{ Design discharge} = 135 \times 100 = 1.35 \times 10^4 \text{ l/c/d.}$$

Max^m rate of disposal,

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

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

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

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

$$D = 12 \text{ m.}$$

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 in weight is about 30%.

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

→ These are best suited for isolated rural areas and isolated institutes.

^{drawback:-} → It requires proper maintenance.

→ They require too large size for serving many people.

→ Its working is unpredictable and non-uniform.

VII Inhoff tank

→ Inhoff 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 this case effluent is not able to carry large undigested solid out from the tank, thereby 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) Retention time = 2-4 hr.

(ii) $V_f = 0.3 \text{ m/min}$

(iii) $SLR = 30 \text{ m}^3/\text{m}^2/\text{d}$

(iv) $L \approx 30 \text{ m}$

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

(vi) Depth of tank ~~3-4 m~~ $= 3-3.5 \text{ m}$

(vii) Freeboard = 0.45 m.

→ for digestion chamber

(i) It is generally designed for a minimum capacity of 57 litres/capita.

(ii) Cleaning period = 1 - 1.5 month.

(iii) Sides slopes of hopper = 1:1

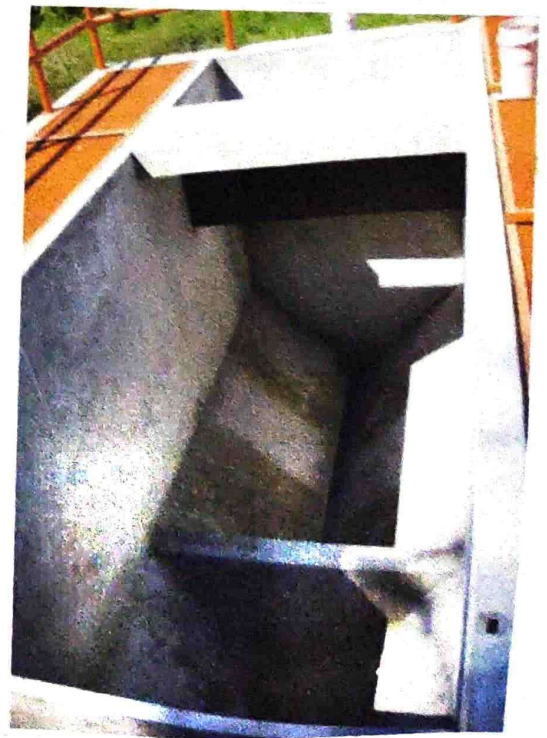
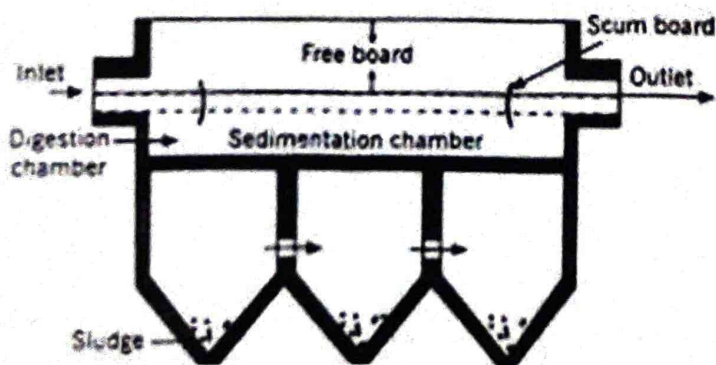
Note: Slot slope (1.25V : 1H)

→ In today's time it is practically obsolete

Note:- Claridigester

→ These are small patented circular inhoff tank type double storey tank. without bottom hoppers and fitted with mechanically sludge ~~removal~~ & scum removal unit which helps in reducing its overall height. to 6m.

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VIII High Rate Anaerobic System

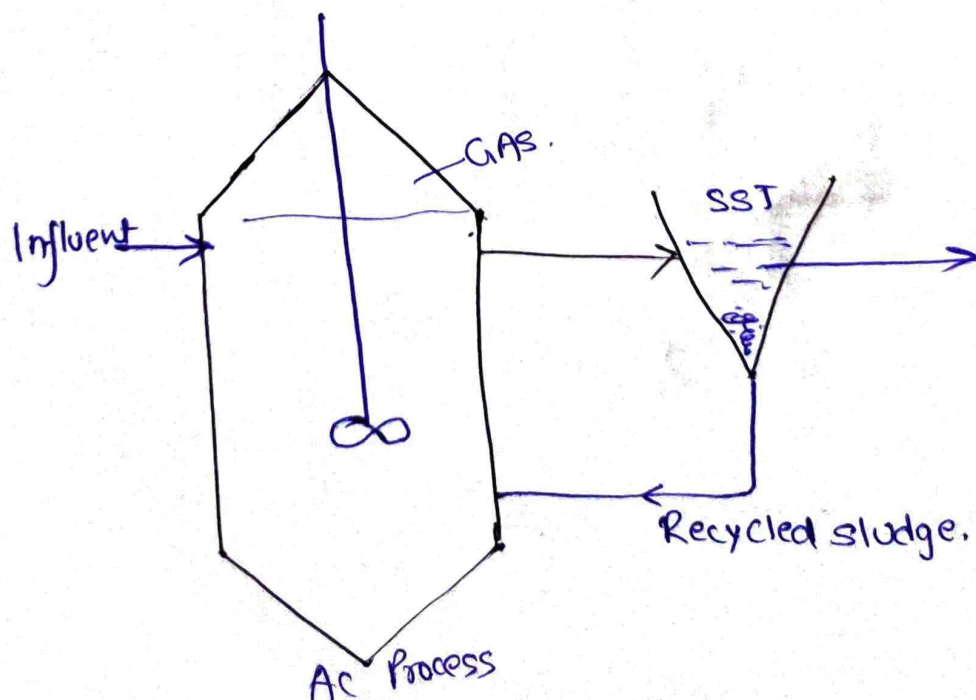
→ In anaerobic process rate of decomposition of organic matter is comparatively less requiring higher detention time, bigger treatment unit, thereby more cost, hence in order to ~~increase the rate~~ reduce the time and area required for treatment high rate anaerobic system are being designed.

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

→ The various high rate anaerobic system:-

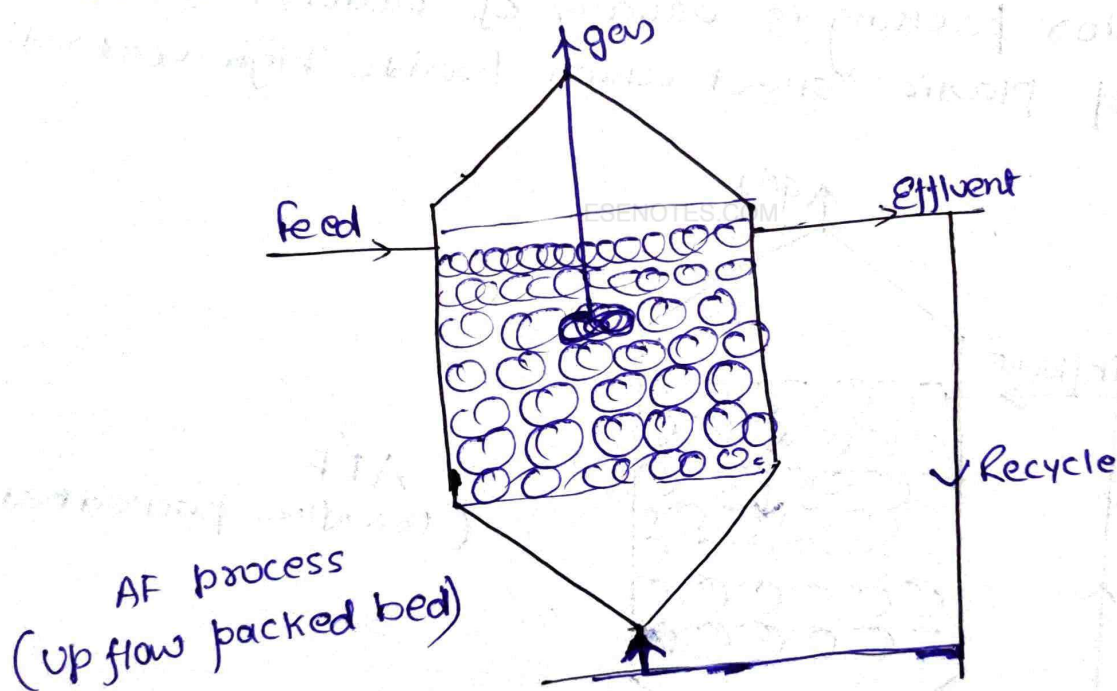
(i) Anaerobic Contact Process.

→ 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)



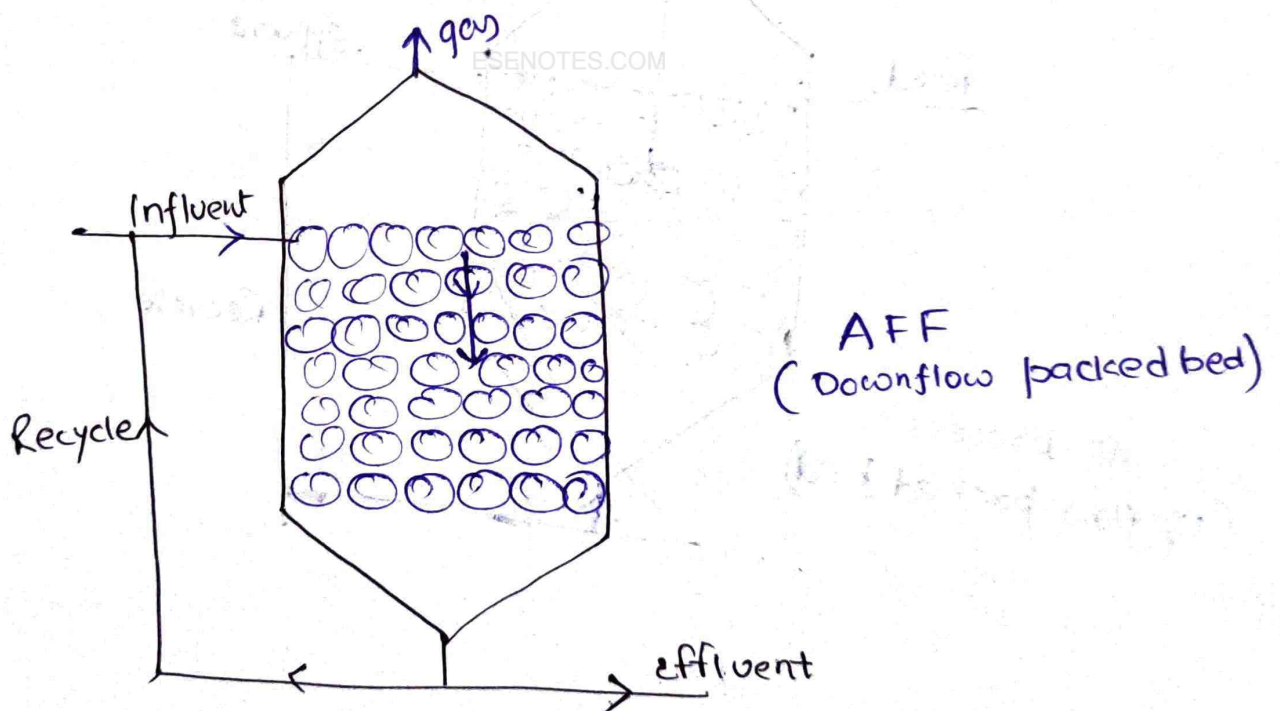
(ii) Anaerobic filter

- In an anaerobic 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 medium.
- 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.



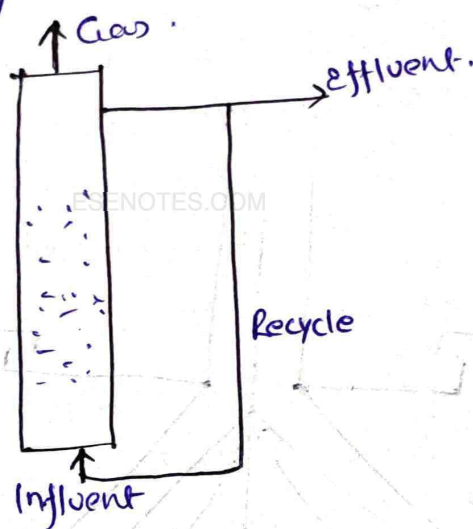
(iii) Anaerobic fixed film Reactor (AFF)

- like an anaerobic filter this process is also characterised by the presence of stationary packing material in the reactor.
- But here the aim is to ~~provide~~ avoid entrapment of suspended solid in the void of packing material
- In order to prevent accumulation of solid in the reactor, the AFF reactor is worked with downflow mode.
- The reactor may be operated either in submerged or in unsubmerged condition.
- The reactor packing is usually of modular construction, consisting of plastic sheet which provide high void ratio.



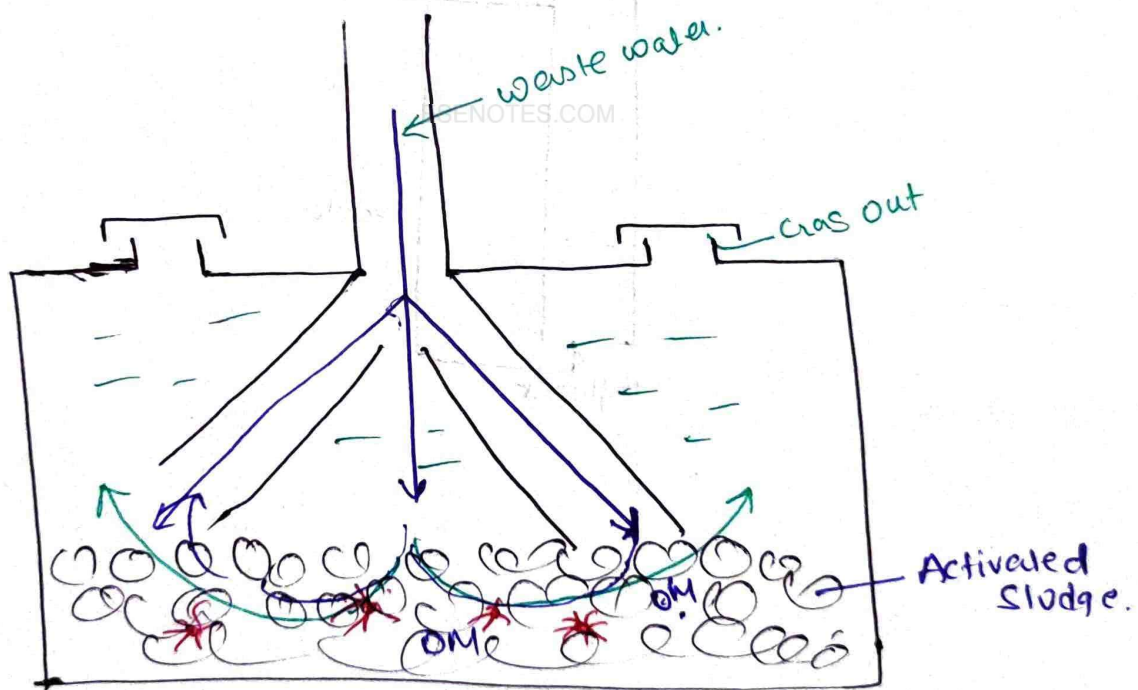
(iv) Fluidized and Expanded Bed Reactor

- A fluidized 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 anaerobic Sludge blanket reactor).

- It maintains a high concentration of bio-mass through the formation of highly settleable microbial sludge aggregates.
- The waste water is then allow to flow upward through a layer of very activated ~~sludge~~ sludge to cause anaerobic decomposition of OM present in the waste water.
- At the top of the reactor, three phase separation of the comitvents takes place as mixing is not induced in it.
- This process is suitable for both soluble waste water as well as containing ~~sett~~ particulate matter.



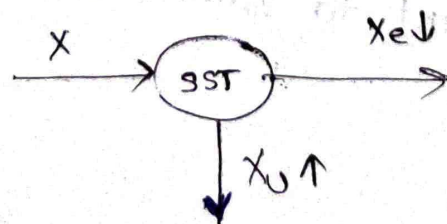
IX Moving Bed Biofilm Reactor (MBBR)

- The MBBR system consist of an aeration tank (similar to an activated sludge tank) with special plastic carriers 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 ($\rho = 0.959 \text{ g/cm}^3$)
- The carrier will be mixed in the tank by the aeration system, thus will have good contact between the substrate in the effluent waste ~~water~~ water & biomass on the carriers.
- To prevent the plastic carriers 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). ~~Both are~~
- Surface area of these tank is computed by ~~using~~ both avg. value of over flow rate & SLR and max^m area computed is adopted.
- This max^m area is further used to ^{check the} find the peak value of both OFR and SLR

Types of SST	Over flow Rate ($m^3/m^2/day$)		Solid Loading Rate ($kg/m^2/d$)		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 aera.	8-15	25-35	25-120	170	3.5-4.5	1.5-2



$$\textcircled{1} \quad SA = \frac{Q_0 + Q_R}{OFR_{avg}} \quad \text{or} \quad \frac{Q_0}{OFR_{avg.}} \quad \left. \begin{array}{l} \text{max.} \end{array} \right\}$$

$$SA = \frac{(Q_0 + Q_R)X}{SLR_{avg}} \quad \text{or} \quad \frac{Q_0 X}{SLR_{avg}}$$

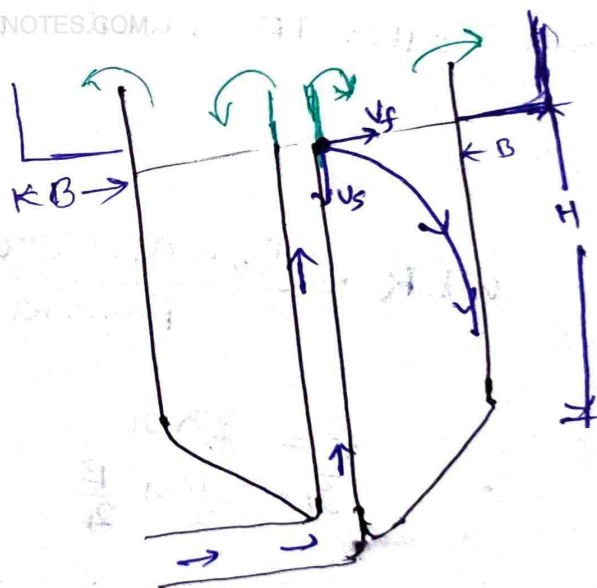
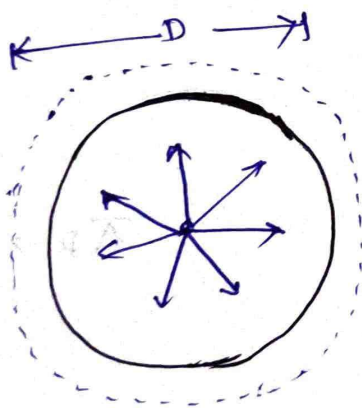
$$\textcircled{2} \quad OFR_{PEAK} = \frac{Q_{PEAK} + Q_R}{SA_{max}} \quad , \quad SLR_{PEAK} = \frac{(Q_{PEAK} + Q_R)X}{SA_{max}}$$

$$\frac{\pi D^2}{4} = S_{max} \Rightarrow \left(\frac{4}{\pi} \times S_{max} \right)^{1/2} = D$$

$$\textcircled{3} \quad V = Q_0 \cdot t_d$$

$$= D^2 (0.011D + 0.785H)$$

$$H = ?$$



- 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".

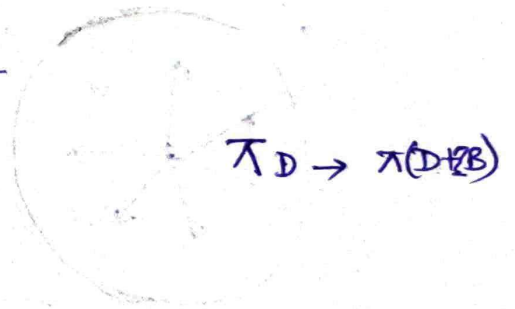
$$\text{Weir Loading rate} = \frac{\text{Rate of flow of effluent}}{\text{length of over flow weir.}}$$

- for SST after ASP $\text{WLR} = 185 \text{ m}^3/\text{m}/\text{day}$ &
- for SST after TF, $\text{WLR} = 125 \text{ m}^3/\text{m}/\text{day}$.

$$\text{WLR} = \frac{Q_o - Q_w \text{ or } Q_o}{\text{Perimeter.}}$$

$$\bar{P} = 2\pi \bar{r}$$

$$\bar{r} = \frac{D}{2} + \frac{B}{2}$$



Q Design SST to treat effluent from ASP from following data.

Average sewage inflow = 20 MLD

MLSS in influent = 3000 mg/l

Peak flow factor = 2.25

Assume $OFR_{avg} = 20 \text{ m}^3/\text{m}^2/\text{d}$
 $SLR_{avg} = 80 \text{ kg}/\text{m}^2/\text{day}$

$$SA = \frac{Q_0}{OFR_{avg}} = \frac{20 \times 10^6 \times 10^{-3}}{20} = 1000 \text{ m}^2$$

$$SA = \frac{Q_0}{SLR_{avg}} = \frac{Q_0 \cdot X}{SLR_{avg}} = \frac{20 \times 10^6 \times 3000 \times 10^{-6}}{80} = 750 \text{ m}^2$$

Check $OFR_{PEAK} = \frac{Q_{PEAK}}{SA_{MAX}} = \frac{2.25 \times 20 \times 10^6 \times 10^{-3}}{1000} = 45 \text{ m}^3/\text{day}$
(40-50 m³/day)

$$SLR_{PEAK} = \frac{Q_{PEAK} \cdot X}{SA_{MAX}} = \frac{2.25 \times 20 \times 10^6 \times 3000 \times 10^{-6}}{1000} = 135 \text{ kg}/\text{m}^2/\text{day}$$

< 210 kg/m²/d

$$SA_{MAX} = 1000 \text{ m}^2$$

$$\frac{\pi D^2}{4} = 1000$$

$$D = 35.68$$

$$D = 36 \text{ m}$$

Assume, $t_d = 2 \text{ hrs}$.

$$V = Q_0 \cdot t_d = 20 \times 10^6 \times 10^{-3} \times \frac{2}{24} = 1666.7 \text{ m}^3$$

$$V = D^2 (0.011 D + 0.785 H)$$

$$1666.7 = 36^2 (0.011 \times 36 + 0.785 H)$$

$$H = 1.17 \text{ m}$$

$$\sim 2 \text{ m } 1.2 \text{ m}$$

Check $WLR = \frac{Q_0}{P} = \frac{20 \times 10^6 \times 10^{-3}}{\pi(136)} = 176.8 \text{ m}^3/\text{m/d} < 185 \text{ m}^3/\text{m/day}$.

No. requirement of weir.

Solid - Waste Disposal

- Mixture of solid and semi-solid waste generated due to different activities in the community is termed as solid waste.
- If it is generated due to domestic activity it is termed as domestic solid waste.
- If it is generated due to industrial activity it is termed as industrial waste.
- If it is generated from hospital and nursing home it is termed as bio-medical waste [however it is to be disposed off separately.]
- Domestic solid waste is further being classified as follows:-

(i) Garbage
It includes all the food waste [bio-degradable] organic waste obtained from kitchens, hotels, restaurant that consist of all food waste, vegetable, peeling, fruit peeling etc.

→ Its density varies in the range of 450-900 kg/m³.

(ii) Rubbish

→ It includes variety of materials ^(non-biodegradable) which may either be compostible, eg. paper, plastic or textile

or incombustible . eg. broken glass , crockery , metal , masonry etc.

→ Its density varies between ~~5~~ 50-400 kg/m³.

(iii) Ashes

→ residue obtained from house hold earth incombustible in nature is termed as ashes.

→ Its density is in range of 700-850 kg/m³.

~~Notes~~

(iv) Street Sweeping.

→ Domestic solid waste may also increase the find dust, silt & sand obtained from street.

Note: Domestic solid waste is also termed as Municipal solid waste (MSW) or refuse.

→ Several other component may also find its way into MSW.

→ Although the amount of municipal solid waste is usually very small (~5%) of total solid waste, but due to its adverse effect its significance is more.

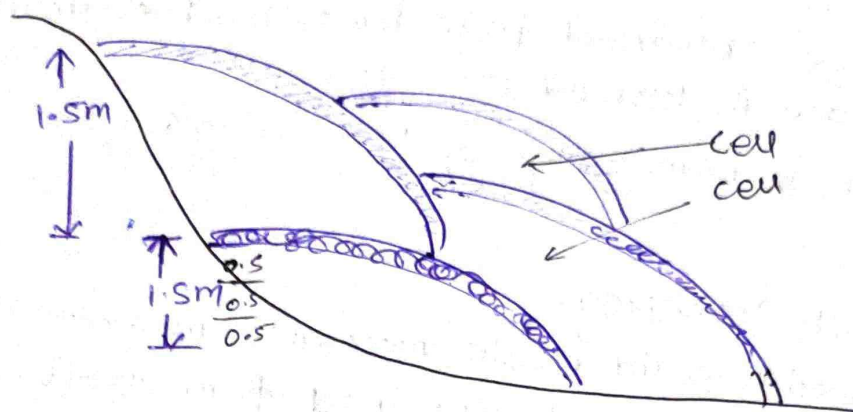
→ Refuse may also be defined as house, street and trade refuse.

Constituent of refuse	Approximate avg. by wt. at disposal site.
Garbage	45%
Rubbish	15%
Ashes	15%
Street Sweepings	25%
<u>Note</u> Density	400-600 kg/m ³
Calorific value	5000-6000 kJ/kg.

Disposal of MSW

Disposal of MSW can be done by any of the following method

(i) Sanitary Land filling Method.



leachate =
mix of (alcohol +
acid +
rubbish +
water)

→ In this method solid waste is disposed over low lying areas (earmarked as land fill site) under an engineered operation, designed and operated in an environmentally sound manner as not to cause any public hazard.

→ In this method the refuse is dumped and compacted in layers of about 0.5m thickness and after the days work when depth of filling becomes about 1.5m it is covered by good earth of about 15cm thickness termed as daily cover.

→ Since refuse is well compacted mechanically and is well covered daily, it does not cause any public nuisance.

→ After filling all the cell (small area unit) with first lift, the second lift is laid in about 1.5m height and is covered with good earth of about ~~1.5m~~ 15cm depth is termed as intermediate cover.

→ After all the cells have been filled with second, third & more lift the process continues till the top most lift is filled up & over which final cover of 0.6m depth is applied.

→ A "CAP cover system" is also installed over the top of final cover.

→ Over the period of time decomposition of OM in refuse takes place biologically in following stages:-

- (i) Action by Aerobic M/O.
- (ii) Action by acid former.
- (iii) Action by methane former.

→ For optimum decomposition moisture content must be greater than 60%

→ The refuse in land fill is stabilised within a period of 2-4 months (upto 12 month) & settle down by 20-40%.

→ This method of refuse disposal is very suitable to heavier type of refuse, (about 90% of Indian refuse is disposed by this method).

→ It is most simple economical and biological method.

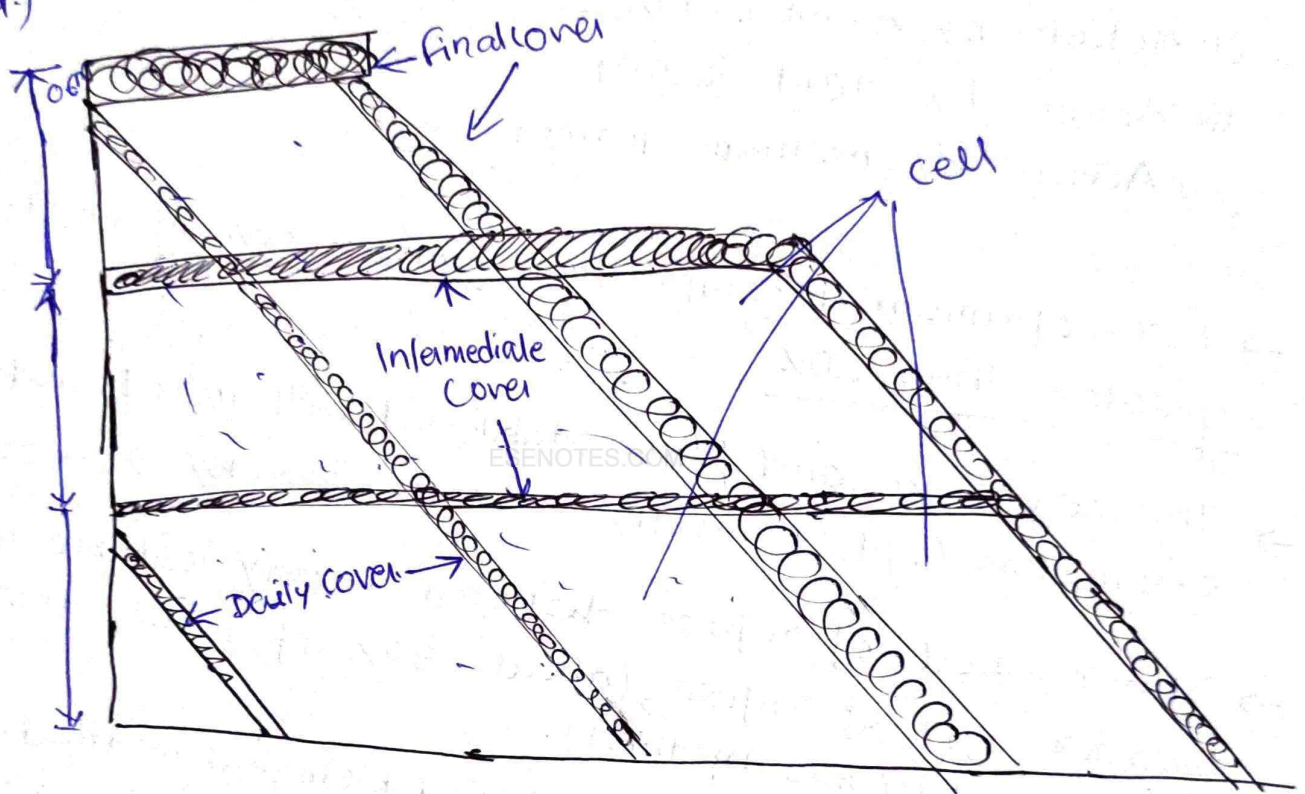
→ Separation of different kinds of refuse as required in other method is not needed here.

→ There is no residue or by product left out in this method and no further disposal is required.

→ Low lying water logged area can be easily used in this method.

Drawback

- Low lying or depression may not ~~be~~ always be available.
- There is continuous evolution of foul gases, near the fill site (these are termed as land fill gases).
- Since the dumped garbage may contain harmful, carcinogenic substance, it mix with acid alcohol and water thereby leads to formation of LEACHATE (this is highly poisonous coloured cancer causing liquid.)



Acids + Alcohol + water + Rubbish = Leachate.

Note - The cap system or cap which is usually provided at the top of landfill after filling it to full height to avoid formation of leachate, consist of following layers

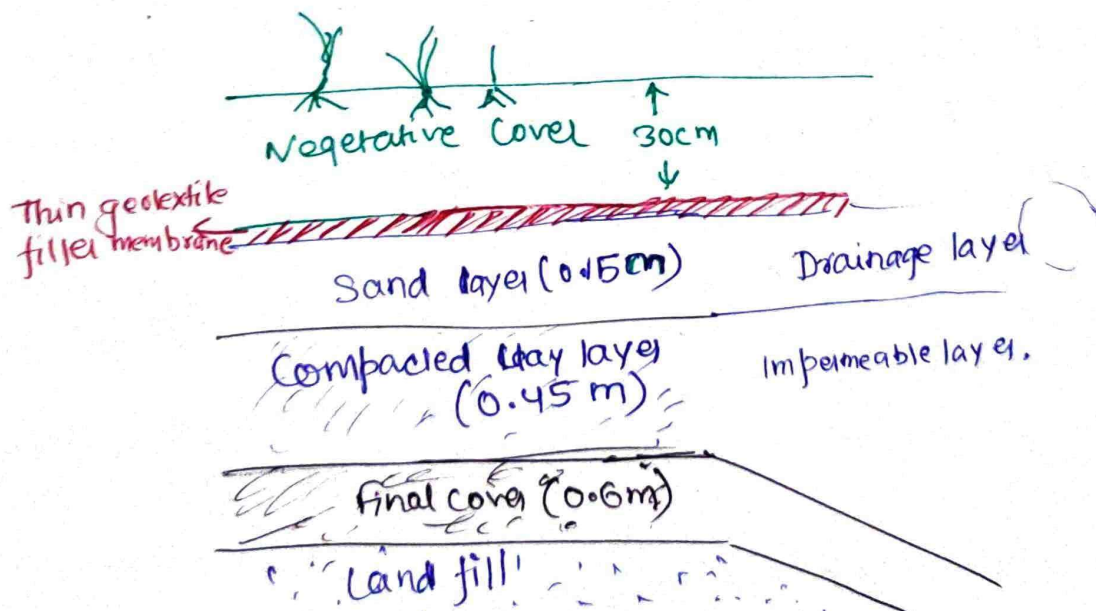
(i) 45cm thick compacted clay layer. is provided at the bottom which is overlain

(ii) By a 15cm thick drainage layer consisting of sand

(iii) Above the second layer of sand is laid a geotextile fibre membrane to protect the drainage layer from being clogged.

(iv) finally 30cm thick soil ~~cover~~ layer is laid at the top to promote the vegetative cover and gramin. This topmost layer is called vegetative layer.

(v) This entire arrangement is termed as CAP system and is provided over the final cover of 0.6m thickness.



Q A city with the population of 2 lakh produces 400gm per capita per day of refuse, 20% of which is recycled, and 80% goes on land-fill site. Compute the area of land fill for disposal of solid waste of 25 years of avg. density 250 kg/m³. Assume compaction ratio to be 4, and height of each lift to be 1.5m. Site condition indicate max^m of 3 life can be provided.

$$V = \frac{400 \times 2 \times 10^5 \times 10^{-3} \times 365 \times 25}{250} \quad (\text{kg/25 year})$$

$$\text{Compacted} = \frac{292 \times 10^4}{4}$$

$$\text{Volume compacted} = 73 \times 10^4 \text{ m}^3$$

$$80\% \text{ goes} = \frac{73 \times 10^4}{10} \times 0.8 = 584000$$

$$\text{Area} = \frac{584000}{(1.5 \times 3)} = 129777.77 \text{ m}^2 = 12.97 \text{ hectare.}$$

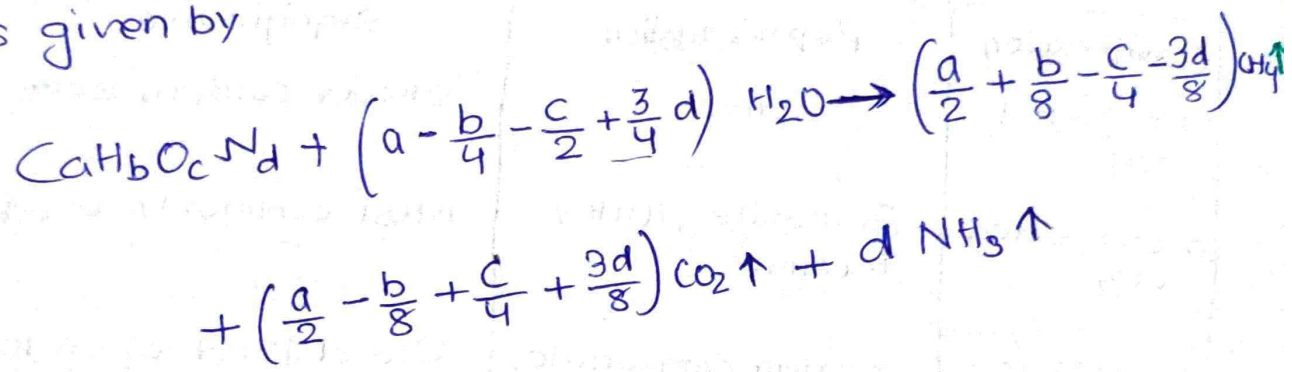
$$640000 = \frac{10}{V} \times 8$$

Gas Production and its Control in land fill method.

→ The rate of decomposition of solid waste in land fill is measured by gas production, which reaches at its peak within first two year and then tapers off continuing in many year (upto 25 year) or more.

→ The total volume of gases produced during anaerobic decomposition can be estimated as follow:-

① If all the organic consistuent in the refuse is generalised in form of $\text{CaH}_b\text{O}_c\text{Nd}$ then total volume of gases produced by complete decomposition of solid waste ~~the~~ is given by



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→ Over 90% of gas volume produced from decomposition of solid waste consist of methane and carbon dioxide

→ Although most of these gases escape into atmosphere but about 40% of these gases is found at the lateral distance of upto 120m from edges of the land fill.

→ Methane being a very light gas rises around the land fill area & increases the chances of explosion in nearby area.

→ CO_2 being heavier (1.5 times than air & approximately 1.8 times than CH_4). remains in high concentration in lower region of land fill area, seeps into the ground along with the water meets the ground water table level and increases its acidity over the period of time.

→ The movement of these gases is controlled by provision of different types of vent and barriers. (generally cell vent, trench vent and well vent are provided around the cell).

→ The movement of landfill gases through adjacent soil formation can be controlled by providing vertical well barriers of landfill sealants which is generally an impermeable material.

→ Commonly used sealants includes.

Classification	Representation	Significance.
→ Compacted soil	soil	Should contain some silt and clay
→ Compacted clay	Bentonite, Illite, Kaolinite,	Most commonly used Sealent.
→ Inorganic material	Sodium carbonate, silicate	Use depend upon local characteristic.
→ Synthetic material	Polymer, rubber, latex	Use for experimental work.
→ Synthetic membrane liner	Polyvinile chloride, Nylone, Butyle rubber	expensive for use.
→ Asphalt	Asphalt concrete	Used in case of differential settlement.

Q 150gm of municipal solid waste after being disposed over the land fill produces 60gm of CO_2 , 130gm of methane and 10gm of ammonia & 2gm of H_2S during its decomposition over a period of 25 years. Compute the rate of green house gas production in (gm/c/day) from the total solid waste of 200T obtained from the community of population of 25 lakh.

green house gas = CO_2 + methane,

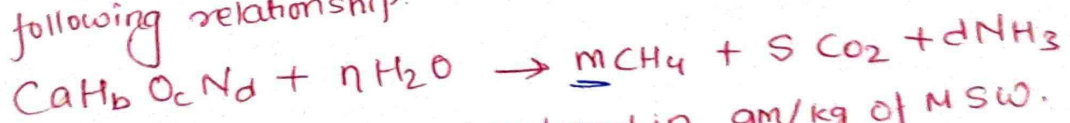
150gm of MSW produces = $(60 + 130)$ gm = 190gm of ~~gas~~

$$\begin{aligned} 200\text{T} & \xrightarrow{\frac{190}{150} \times 200 \times 10^6 \text{ (gm)}} \\ & \xrightarrow{\frac{25 \times 10^5}{\text{population}} \times (25 \times 365)} \\ & = 0.0111 \text{ gm/c/day.} \end{aligned}$$

Q Raw municipal solid waste collected from a city contains 70% decomposable material that can be converted to methane the ~~total~~ ^{analysis} ~~content~~ of decomposable material yield the following mass (%)

C : H : O : N : other = 44 : 6 : ~~43~~ : 0.8 : 6.2

The methane production of decomposable material is govern by the following relationship.



Compute mass of methane produced in gm/kg of MSW.

For 1 kg of MSW decomposable material = $0.7 \times 1 = 0.7 \text{ kg.}$

// solid = ~~0.65~~ ~~0.7~~ = 0.455 kg.

Carbon $\xrightarrow{\text{m.w.}}$ $12 \text{ g} = 0.44 \times 0.7 \times 10^3$
 $a = 0.0256 \times 10^3 = 25.6 \text{ moles}$

Hydrogen $\xrightarrow{\text{m.w.}}$ $1 \text{ g} = 0.06 \times 0.7$
 $b = 0.642 \times 10^3 = 42$

Oxygen, $16C = 0.43 \times 0.7 \times 10^3$
 $C = 0.0188 \times 10^3 = 18.8$

Nitrogen $14d = 0.008 \times 0.7 \times 10^3$
 $d = 0.4$

from mass balance eqn,

$$a = m + s \quad \text{---(i)}$$

$$b + 2n = 4m + 3d \quad \text{---(ii)}$$

$$c + n = 2s \quad \text{---(iii)}$$

$$a = d$$

~~methane produced~~ $m = 13.19$

~~CH₄ = 22.6~~
 methane produced $= 13.19 \times 16 = 211.29 \text{ gm.}$

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ii) Disposal of Refuse by Shredding & Pulverisation.

- In real terms it is not method of disposal of solid waste, it is only used to convert the heavier solid into lighter ones so as to carry out their effective disposal by any other method.
- Shredding refers to action of cutting & tearing where as pulverisation refers to action of crushing and grinding.
- These method may help in reducing the overall volume of original MSW by as much as 40%. but also help in changing the physical character of the waste which becomes practically odour less and unattractive to the insects.
- Pulverisation is carried out in the units termed as hammer mill.
- In order to further reduce the waste volume and facilitate handling of the pulverised MSW it is usually compacted to form rectangular blocks or bails (and the process is termed as bailing).
- Pulverisation and bailing together can reduce the volume up to 90%.

(iii) Composting

- Composting of refuse is also biological method of decomposing solid waste. This decomposition can be carried out either aerobically or anaerobically.
- The final end product is termed as manure, compost or humus.
- Composting is considered to be an aerobic process as it involve piling up of refuse and its regular turning either manually or mechanically to ensure sufficient supply of air and oxygen during decomposition.

- Initially the process starts with Mesophilic bacteria which oxidise the organic matter to carbon dioxide and liberate heat that rises the temperature to about 45%. and at this point thermophilic bacteria takes over and continues the decomposition and raises the temp. to about 60°C.
- For optimum decomposition temp. must be in range of 55 to 60°C beyond which decomposition slows down.
- Once decomposition is complete temp. starts falling.
- ~~if compost~~ Aerobic composting is also termed as Indore Process which is completed in 3 to 4 months.
- If composting is done anaerobically it is termed as Bangalore process in which mixing of refuse is not done hence this method is cleaner / sanitary than Indore process. but time required in this case is more. (4-5) months.
- After stabilisation the refuse changes into a brown coloured odourless, powdery mass called as "humus".
- For optimum composting $\frac{C}{N} = 30-50$

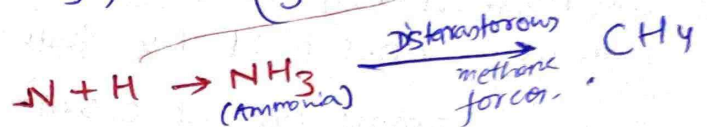
$$\frac{C}{N} = \frac{\text{Carbon/Food/Energy}}{\text{Nitrogen/Nutrient/Growth}}$$
- The above range is on basis of fact that MO utilises Carbon, 30-50 times faster than nitrogen.
- If $\frac{C}{N}$ is more nitrogen is exhausted prior to the carbon leaving growth of M/O incomplete. that in turn carries out incomplete decomposition.

→ If $\frac{C}{N}$ is less, carbon is exhausted prior to that of N & left over nitrogen combine with hydrogen & form ammonia which is disastrous for methane formers.
 → Hence in either of the case composting is incomplete.

Eg $\frac{C}{N} = \frac{30 - 50N}{C_{growth}} [1N, 30.50C]$

(i) $\frac{C}{N} \uparrow = 80 [1N, 80C]$, left $(50-30)C$, N X
 growth of microorganism X

(ii) $\frac{C}{N} \downarrow = 10 [1N, 10C]$, left $(\frac{1}{3} - \frac{1}{5})N$, C X



* Nitrogen govern

A waste Activated sludge is to be blended with green waste the carbon and Nitrogen content per kg of WAS and green waste on dry basis are given in the table.

Parameter	WAS	GW green waste
Carbon(g)	54	360
Nitrogen(g)	10	6

ratio 5.4:1 60:1

$$5.4 \times \frac{a}{N} + \frac{360}{N} = 35$$

The ratio of waste activated sludge to green waste required achieve a blended $\frac{C}{N}$ ratio of 35. $a+b=1$

$$\frac{WAS}{GW} = \frac{a}{b}$$

$$\frac{C}{N} = \frac{54a + 360b}{10a + 6b} = 35$$

$$a = 0.506$$

$$\frac{a}{b} = 0.506 \quad [WAS = 0.5 \times GW]$$

$$\frac{54a + 360b}{10a + 6b} = 35$$

$$54a + 360b = \frac{350a}{+210b}$$

$$150b = 296a$$

$$= \frac{a}{b} = 0.506$$

Note: VERMI-COMPOSTING

→ Vermicomposting uses the natural composting process of decomposition of bio-degradable organic matter by soil bacteria as in ordinary composting technique but takes the assistance of cultured [artificially produced] earth worms like red wiggler or tiger worm, European night crawler, Blue worms, etc.

→ These worms do help in quicker decomposition of organic matter

→ This method helps in adopting the composting technique in individual household by following process:-

(i) Dig a small pit of 0.5m^2 and 1m deep.

(ii) Line the pit with straw or dried leaves.

(iii) Organise the disposal of domestic waste into the pit as when generated.

(iv) Introduce culture of worms.

(v) Cover the pit content daily, by spreading dried leaves and soil

(vi) Water the pit once or twice a week

(vii) Turn over the content, every 15 days.

(viii) In about 45 days the waste will be decomposed by action of MO.

(ix) The produced humus in the pit can be used as manure.

(iv) Disposal by Incineration and thermal Pyrolysis.

→ In this method disposal of the refuse is carried out by burning it at high temperature in furnaces termed as incinerators.

→ Only combustible matter such as garbage, rubbish and dead animal are burnt and incombustible matter like broken glass, china ware, metal etc are either unburnt or are separated out for recycling and reuse before burning the solid waste.

→ The heat produced during the burning of refuse is used in the form of steam power to run the turbine to generate electricity.

→ In Incineration burning of ~~refuse~~ refuse is carried out in the presence of oxygen.

→ In pyrolysis principle is same for disposal but mechanism is different in the sense that heating is carried out in closed container. in O_2 free atmosphere.

→ Most of the organic matter of solid waste can be split through a combination of thermal cracking and condensation reaction. into gaseous, liquids and solid fraction.

→ In contrast to the combustion process which is highly exothermic (releasing heat on burning in presence of O_2) the pyrolysis is highly endothermic (consuming heat). that is why it is also known as DESTRUCTIVE DISTILLATION.

→ When the organic solid waste is pyrolysed following three types of products are obtained.

(i) A gas stream (consisting of H_2 , CH_4 , CO , CO_2) which can be used again as ~~use~~ fuel.

(ii) A liquid fraction (consisting of tar or oil) which is liquid at room temperature.

(iii) A solid fraction (consist of char coal) that can also be used as fuel.

- Pyrolysis may be used for reducing quantity of sludge produce in waste water plant.
- This is the most sanitary method of refuse disposal.
- There is no odour nuisance.
- some cost can be recovered by used of end product.
- It requires very less space.
- It is very costly method and requires technical supervision.
- Solid waste of only high calorific value can be burned
- Pollution created by this method is more.

Q A waste to energy plant burnt dry solid waste of composition

C = 35%

Oxygen = 26%

Hydrogen = 10%

Sulphur = 6%

Nitrogen = 3%

(heats = 20% (does not react))

Burning rate is 1000 ton/day.

Oxygen in air by weight = 23%.

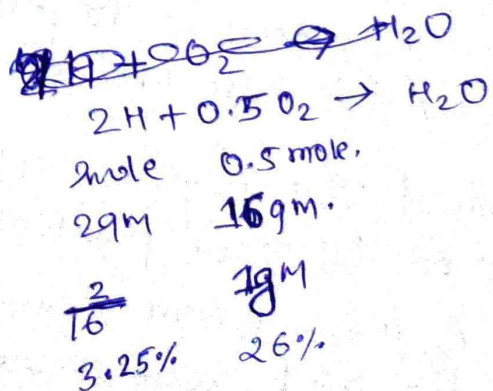
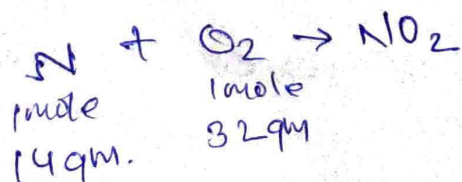
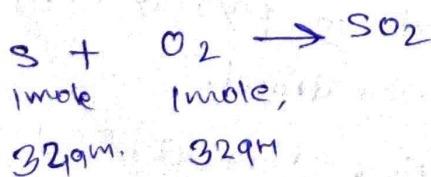
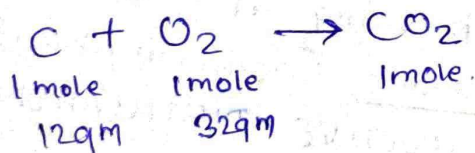
assume complete conversion of carbon $\rightarrow \text{CO}_2$

Hydrogen $\rightarrow \text{H}_2\text{O}$

Sulphur $\rightarrow \text{SO}_2$

Nitrogen $\rightarrow \text{NO}_2$

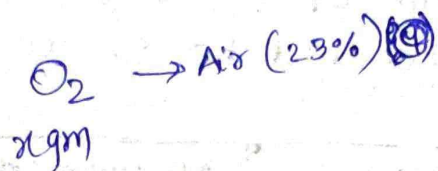
The amount of air (tonnes/day) required for complete burning of this waste would be.



(Remaining hydrogen = $10 - 3.25 = 6.75\%$)

$$\text{Amount of air required} = \frac{32}{12} \times 0.35 + \frac{32}{32} \times 0.06 + \frac{32}{14} \times 0.03 + \frac{16}{2} \times \frac{10-26 \times \frac{1}{16}}{15}$$

$$= \frac{6964.80}{0.23} \text{ tonnes/day}$$



$$y \times 0.23 = x$$

$$y = \frac{x}{0.23}$$

V Bagging/Dumping into the Sea.

- In this method disposal of solid waste is carried out by throwing it away into the sea.
- Disposal of refuse in the sea must be done far away from the shore/coast (15-20km), below the water surface level (30-50m) and at the time of low tide so as to avoid flow back of solid waste on the shore.
- This method proves to be quite cheap and simple but poses several drawbacks.
- The bulky and lighter part of refuse do not settle down and remains floating and return to the shore.
- This method requires ship for taking the refuse into the sea, movement of which is difficult during monsoon or stormy weather.
- Even inspite best care is taken, some refuse in this method come back to the shore.
- The dumping of refuse causes sea pollution and affects the marine life.

VI Autoclaving

→ It is a low heat thermal process which is used for disposal of bio-medical waste in which steam under controlled temperature, and pressure is passed over the waste to carry out its disinfection.

Q The composition of MSW is given, compute the difference between energy content of waste sample calculated on dry basis & as discarded basis. in KJ/kg.

Component	% BY Mass	Moisture Content (%)	Energy Content (KJ/Kg) on discarded basis
Food waste	20	70	2500
Paper	10	4	10000
Cardboard	10	4	8000
Plastic	10	1	14000
Plants	40	60	3500
Woods	5	20	14000
Tin Can	5	2	100

100 - moisture %	dry content % X % mass	% mass X energy
Dry content (%)	total dry mass (%)	Energy on discarded basis
30 %	6	500
96 %	9.6	1000
96	9.6	800
99	9.9	1400
40	16	1400
80	4	700
98	4.9 %	5
	60 %	5805 KJ/Kg.

Difference =
 $(9675 - 5805)$
 $= 3870 \text{ KJ/Kg.}$

Energy on discarded basis when (60% dry) = 5805 KJ/1kg.
 Energy on dry basis = $\frac{5805}{0.6} = 9675 \text{ KJ/kg}$
 (when it is 100% dry)

AIR POLLUTION.

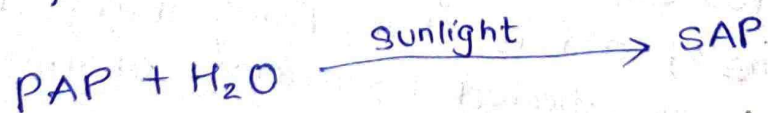
- Presence of one or more air pollutant, in such concentration and for such duration that the life in the biosphere gets effected is termed as air pollution.
- All these pollutants which are emitted directly from the identifiable sources either from natural process like dust storm, volcanic eruption etc. or from human activities like burning of coal, oil in industries, houses, Automobile etc are termed as "primary air pollutant".

eg. oxides of Sulphur (SO_2, SO_3)
oxides of carbon (CO, CO_2)
oxides of nitrogen (NO, NO_2)
Hydrocarbons (HC)
Suspended Particulate matter (SPM)
 $\text{H}_2\text{S}, \text{H}_2\text{F}$
Lead (Pb)

} These contribute 90% of global air pollution.

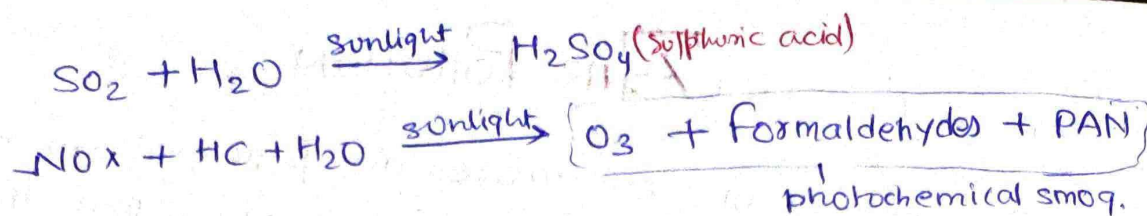
Ethyle Mercaptan $(\text{C}_2\text{H}_6\text{S})$ & Methyl Mercaptan (CH_4S)

- These primary ~~air~~ pollutant often combine with each other or with water vapours in the presence of sunlight and leads to formation of "secondary air pollutants".

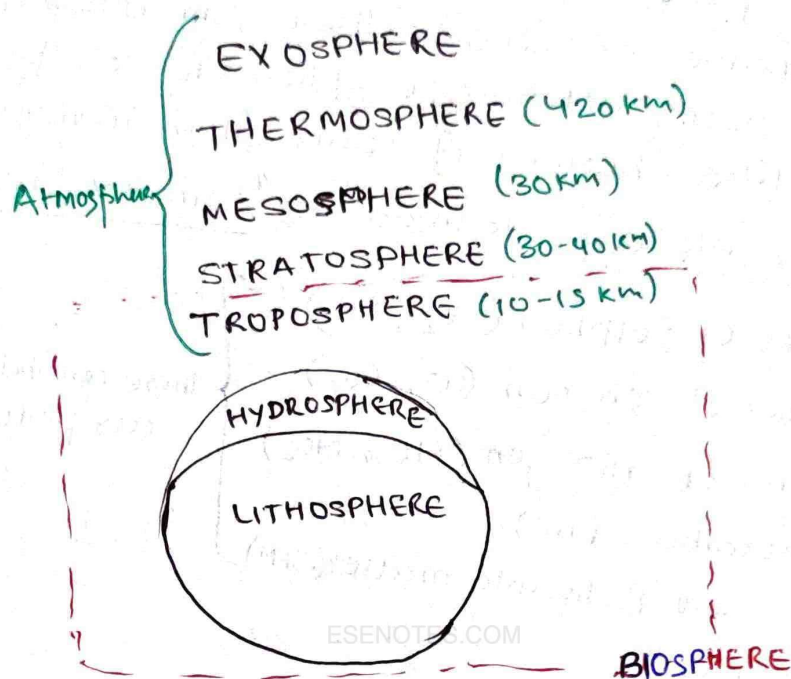


Note: → The above reaction is termed as photochemical reaction.

- The secondary air pollutant can be more dangerous than original basic primary air pollutant.
Like, Sulphuric acid, Ozone, formaldehydes, Peroxy Acetyl Nitrate (PAN)



SMOG: It is mixture of smoke and Fog.



Note:-

The concentration of suspended particle and gaseous pollutants are generally expressed as weight/volume at a given pressure and temperature.

→ The reason for expressing the pollutants as mass per unit volume of air is that, living things breathe air by volume but are affected by mass of the pollutant absorbed by them.

$$1 \text{ ppm} = \frac{1 \text{ part of any gas/pollutant}}{10^6 \text{ parts of air}} = \frac{1 \text{ m}^3 \text{ of any gas}}{10^6 \text{ m}^3 \text{ of air}}$$

1 mole of any gas at STP occupies volume of 22.4 l (1 atm, 0°)

1 mole of any gas at different temp. pressure has volume of V_2 l is given by.

$$PV = nRT$$

$$n = 1 \text{ moles.}$$

$$\frac{P_1 V_1}{T_1} = \text{constant}$$

$$\frac{P_1 V_1}{T_1} = \frac{P_2 V_2}{T_2}$$

$$V_2 = \frac{P_1 V_1}{T_1} \left(\frac{T_2}{P_2} \right)$$

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Q Express the concentration of 10 ppm of sulphur dioxide at 20°C & 2 atm is Mg/m^3

$$\text{Sol}^n = 1 \text{ ppm} = \frac{1 \text{ part of SO}_2}{10^6 \text{ part of air}} = \frac{1 \text{ m}^3 \text{ of SO}_2}{10^6 \text{ m}^3 \text{ of air.}}$$

1 mole of SO_2 at STP has vol of 22.4 lt.

$$\frac{P_1 V_1}{T_1} = \frac{P_2 V_2}{T_2}$$

$$\frac{1 \times 22.4}{273} = \frac{2 V_2}{293}$$

$$V_2 = 12.02 \text{ lt.}$$

1 mole of SO_2 at 20°C, 2 atm has volume of 12.02 lt

64 gm of SO_2 ————— 12.02 l

12.02 lt of SO_2 ————— 64 gm.

12.02
12.02

$$1 \text{ m}^3 \text{ of } \text{SO}_2 \text{ at } 20^\circ\text{C and } 2 \text{ atm} = \frac{64}{12} \times 10^3 = 5324.5 \text{ gm}$$

$$1 \text{ ppm of } \text{SO}_2 = \frac{1 \text{ m}^3 \text{ of } \text{SO}_2}{10^6 \text{ m}^3 \text{ of air}} = \frac{5324.5 \times 10^6 \text{ } \mu\text{g}}{10^6 \text{ m}^3}$$

$$1 \text{ ppm of } \text{SO}_2 = 5324.5 \frac{\mu\text{g}}{\text{m}^3} \Big|_{\text{at } 20^\circ\text{C}, 2 \text{ atm.}}$$

$$10 \text{ ppm of } \text{SO}_2 = 5324.5 \times 10 \frac{\mu\text{g}}{\text{m}^3} \Big|_{\text{at } 20^\circ\text{C}, 2 \text{ atm}}$$

$$= 5.324 \times 10^4 \frac{\mu\text{g}}{\text{m}^3} \Big|_{\text{at } 20^\circ\text{C}, 2 \text{ atm.}}$$

Q The concentration of sulphur dioxide (SO_2) in ambient atmosphere was measured as $30 \mu\text{g}/\text{m}^3$. Under the same condition the above SO_2 concentration expressed in ppm is _____

Given, $\frac{P}{RT} = 41.6 \text{ mol}/\text{m}^3$

where P = Pressure,

T = Temp.

R = universal gas constant

M. weight of $\text{SO}_2 = 64$.

$$30 \mu\text{g}/\text{m}^3 \rightarrow \text{ppm}$$

$$1 \text{ ppm} = \frac{1 \text{ m}^3 \text{ of } \text{SO}_2}{10^6 \text{ m}^3 \text{ of air}}$$

1 mole of SO_2 at STP has vol. of 22.4 L .

$$\frac{P}{RT} = 41.6 \text{ mol}/\text{m}^3$$

$$PV = nRT$$

$$V = n \cdot \frac{RT}{P} = 1 \cdot \frac{RT}{P} = 1 \times \frac{10^3}{41.6}$$

$$V = 24.03 \text{ Lts.}$$

1 mole of SO_2 at ~~at STP~~ STP has volume of 24.03 l
64 gm of SO_2 ——— 24.03 l

~~1 m³ of SO_2 at STP~~
24.03 l of SO_2 ——— 64 gm.

$$1 \text{ m}^3 \text{ of } \text{SO}_2 \text{ ——— } \frac{64 \times 10^3}{24.02} = 2664.44 \text{ gm.}$$

$$1 \text{ ppm of } \text{SO}_2 = \frac{1 \text{ m}^3 \text{ of } \text{SO}_2}{1 \text{ m}^3 \text{ of air}} = \frac{2664.44 \times 10^6 \text{ Mg}}{10^6} \frac{\text{Mg}}{\text{m}^3}$$

$$1 \text{ ppm of } \text{SO}_2 = 2664.44 \frac{\text{Mg}}{\text{m}^3}$$

$$= \frac{1 \text{ Mg}}{\text{m}^3} = \frac{1}{2664.44} \text{ ppm}$$

$$\frac{30 \text{ Mg}}{\text{m}^3} = \frac{30}{2664.44} \text{ ppm} \\ = 0.0113 \text{ ppm}$$

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* Harmful Effects of Air pollutants on Human Health.

(i) Suspended Particulate Matter (SPM)

- These are solid particles like dust, smoke, fumes and liquid particles like mist & fog.
- These are released during dust storm, smoke, burning of garbage & fossil fuels.
- It affects breathing & respiration, causes cardiovascular disease. lungs problem effects still births etc. (dead born).

(ii) Oxide of Sulphur (SO_2)

- It is colourless gas having taste threshold at about 0.3 ppm.
- It is released due to combustion of oil & coal in power station or automobiles.
- It affects breathing, respiratory illness than breakdown of lung defence cause cardio-vascular disease, bronchial spasms (Bronchial spasm^{clot} is sudden blockage of muscles in walls of bronchioles).

(iii) Oxides of Carbon (CO and CO_2)

- It is colourless, tasteless, odourless gas at atmospheric concentration.
- It is evolved due to incomplete combustion of coal & oil & fuels.
- It leads to cardio vascular diseases, combines with the hemoglobin ~~to~~ 200 times faster than oxygen. forms Carboxy hemoglobin (pink in colour) and may also lead to death.
- It results in headache, dizziness, nausea, heart palpitation, difficulty in breathing.

(iv) Oxides of Nitrogen.

- It is reddish brown highly reactive gas. odour threshold is at 0.2 ppm.
- It is emitted from high temperature combustion in automobile and from thermal power station.
- Nitrogen oxide plays a major role in tropospheric ozone formation. ~~it combines~~
- It combines 3 lakh times faster with haemoglobin than oxygen hence is also the cause of "ASPHYXIATION". (Choking/Suffocation).
- NO_2 is responsible for eyes & nasal, irritation, respiratory discomfort and may also lead to emphysema (swelling due to air in body).

(v) Hydrocarbon (HC)

- These are being released from the exhaust of automobile
- they lead to irritation of eyes and respiratory tract.

Note: In order to identify and quantify the quality of air in a particular area, a parameter termed as AQI (Air Quality Index used).

Dispersion of Air pollutant into the atmosphere

→ Dispersion of air pollutants into the atmosphere depends upon prevailing wind, temperature and pressure condition

(i) Wind

→ ~~wind speed is inversely~~

→ Dispersion of pollutant into the atmosphere is directly dependent upon wind speed. (hence concentration of pollutant is inversely proportional to wind speed)

→ Wind speed is measured by ANEMOMETER and velocity of wind at any height can be found using $\frac{1}{7}$ powerth law.

$$U = U_0 \left(\frac{Z}{Z_0} \right)^k$$

$$k = \frac{1}{7}$$

U_0 is wind velocity is measured by anemometer at height Z_0 .

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(ii) Temperature

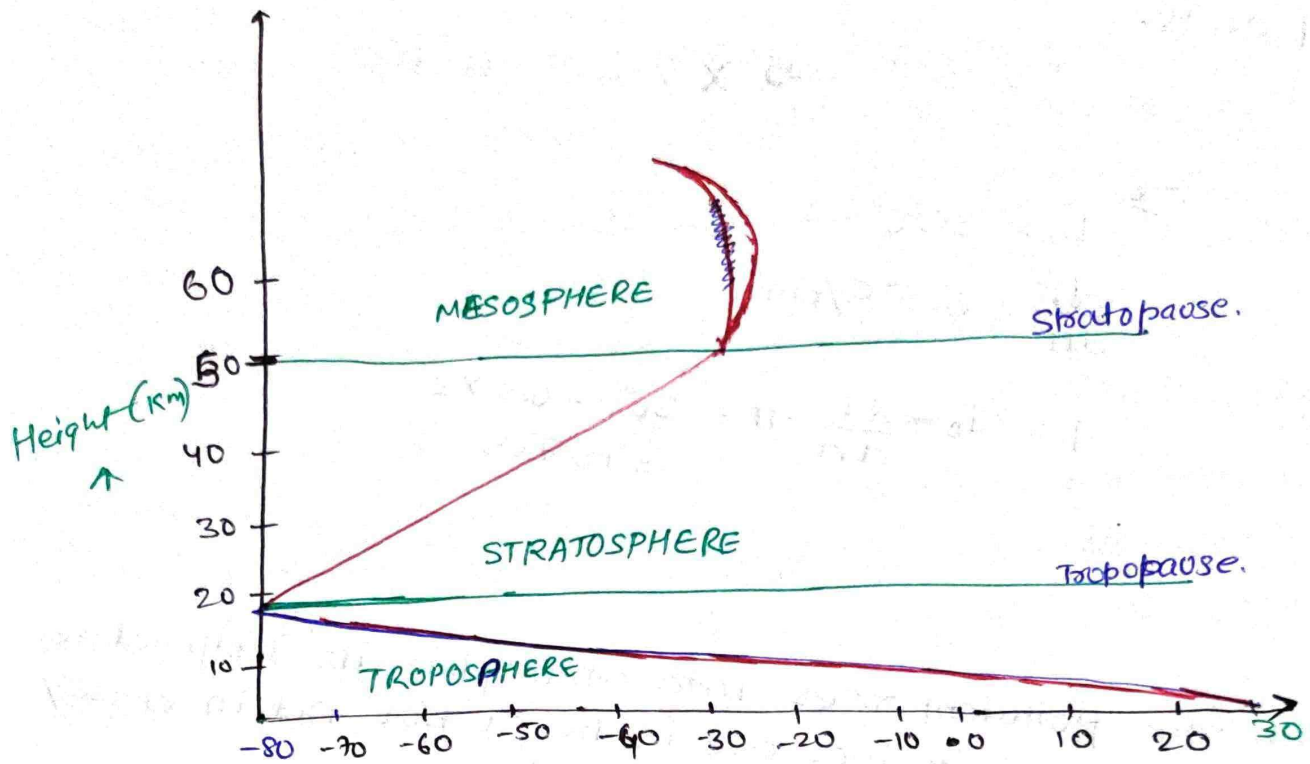
→ Temperature of the air decreases with increase in height above the surface of earth in troposphere. and the rate at which it decreases with increase in height is termed as 'Environmental Lapse Rate'. (ELR)

→ ELR varies from height to height and can be computed separately for different height but if not given it may be assumed to approximately $6.5^\circ\text{C}/\text{km}$.

Note:-

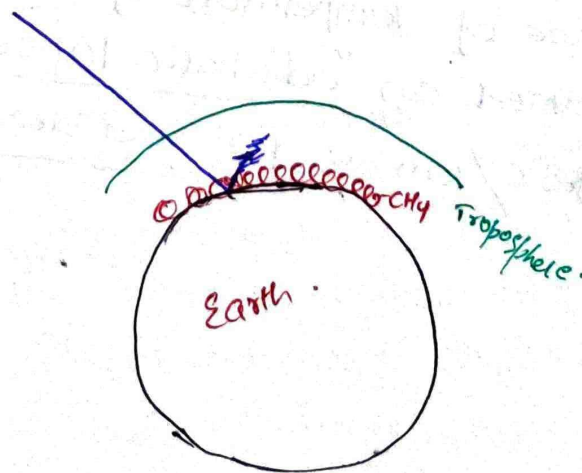
→ Temperature decreases with height in troposphere since the main heat source is solar radiation which is absorbed majorly at ground level.

(ii) Variation of temperature in different layers of atmosphere is as follows:-



Note:- Temp of air increases in stratosphere, and is almost same as that of temperature on the earth surface at stratopause.

→ Its warmth is due to absorption of UV rays from sun by oxygen and by O_2 & ozone of this layer.



Q The temperature of ambient air over the surface of earth is 30°C . find its temp at height of 2.2km from the surface of earth.

$$T = 30^{\circ}\text{C} - 6.5 \times 2.2 = 15.7^{\circ}\text{C}$$

Q6

$$T_0 = 30^{\circ}\text{C}$$

$$\frac{dT}{dH} = 6.5^{\circ}\text{C/km}$$

$$T = T_0 - \frac{dT}{dH} \cdot H = 30^{\circ}\text{C} - 6.5 \times 2.2 = 15.7^{\circ}\text{C}$$

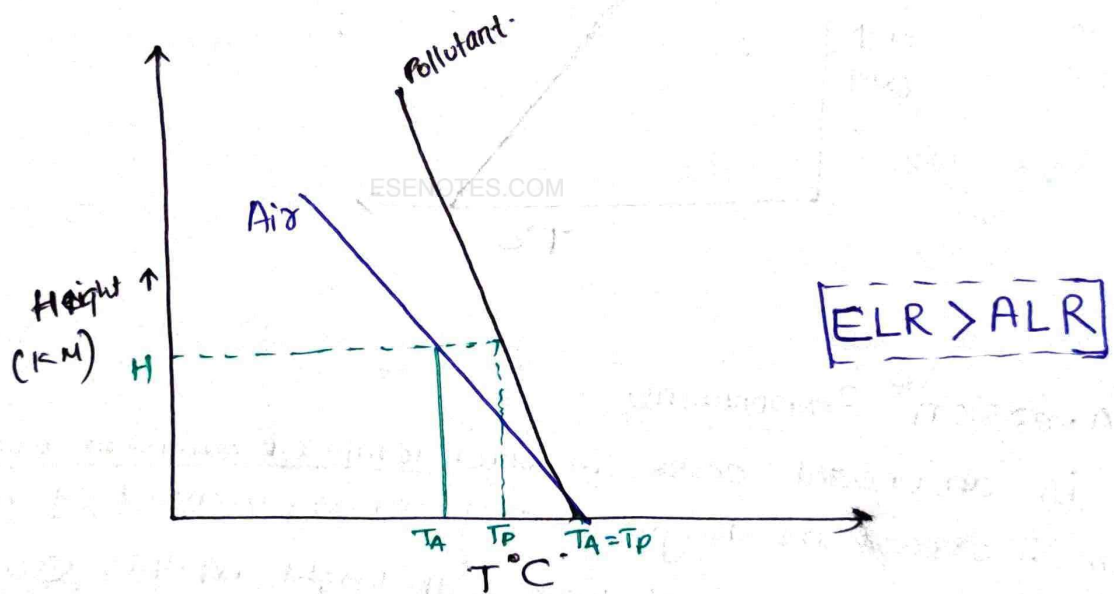
- When the pollutant rises into atmosphere its temperature also decreases with increase in height but not in a ~~any~~ pattern observed in case of air.
- Hence its analysis is carried out by assuming this variation to be "adiabatic." (The process which takes place at constant ~~heat~~ is termed as adiabatic process), $PV^{\gamma} = \text{const}$]
- Hence the temperature of pollutant decrease due to expense of its own internal energy to carry out expansion (when it rises into the atmosphere).
- The rate of decrease of temperature of air pollutant with increase in height is termed as "adiabatic lapse Rate" which for dry air is 9.8°C/km & for wet air is 6°C .

→ Depending upon the relative magnitude of ~~AL~~ ELR and ALR different types of environment are being defined.

(i) Unstable Environment / Super Adiabatic Environment

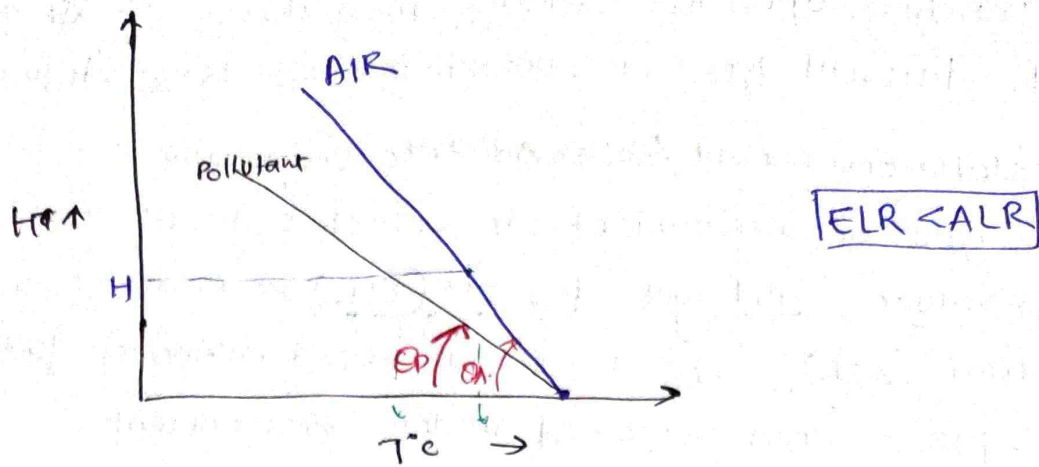
→ It is type of environment in which rate of decrease of temperature with height for air (ELR) is more than that of pollutant (ALR), due to which rapid mixing of pollutants takes place when released in this environment.

→ In such a case the rising parcel of air will always remain warmer than the surrounding air hence it keep on rising into the atmosphere, and a descending parcel of heavier air will keep on falling on to the earth surface.



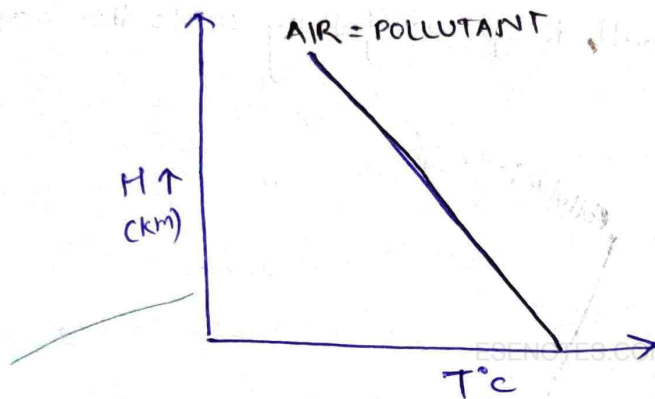
(ii) Stable or Subadiabatic Environment

→ It is the type of environment in which rate of decrease of temperature for air (ELR) is less than that of pollutant (ALR).
→ This prevailing environmental lapse rate in such case is termed as sub-adiabatic lapse rate. & environment is termed as sub-adiabatic environment.



(iii) Neutral Environment.

→ It is type of environment in which $ELR = ALR$

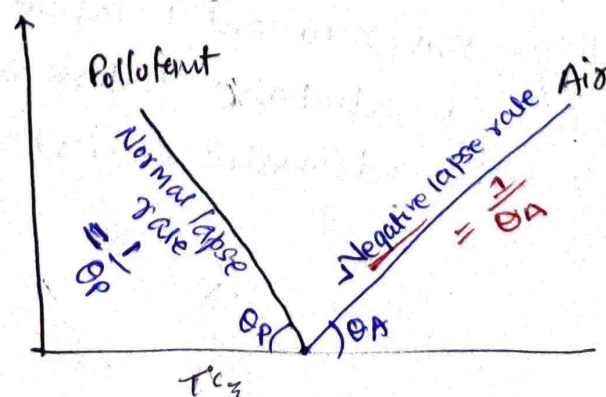


(iv) Inversion Environment.

→ It is an unusual case in which temp. of environment increases with increase in height in troposphere instead of decreasing

→ The rate of increase of temp with height in this case is termed as negative lapse rate.

→ In this case warmer air lies over the colder air ~~below~~ parcel ~~below~~ (pollutant) below, leading to represent a highly stable environment.



→ These inversion condition are of two types:-

(i) Radiation Inversion.

(ii) Subsidence Inversion.

Radiation Inversion

→ It is a phenomenon occurring from the unequal cooling rates for earth and air above the surface of the earth.

→ Earth cools rapidly or more quickly than the air above it (as may happen at ~~the~~ night when the earth may lose heat by radiation there by cooling the surrounding air) the naturally the temperature in environment will be less at the earth and will increase above it, causing the negative lapse rate.

→ This type of inversion is prominent in nights in winters and may extend to few hundred meters above the surface of the earth.

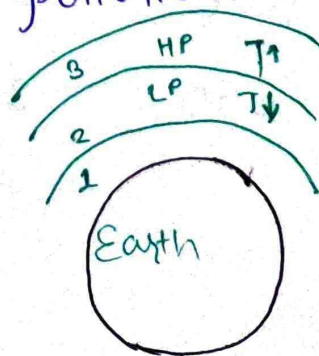
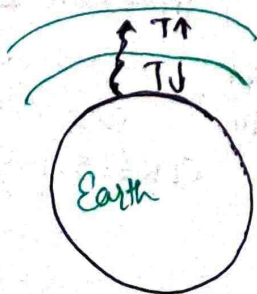
→ This inversion may also be found in valley regions.

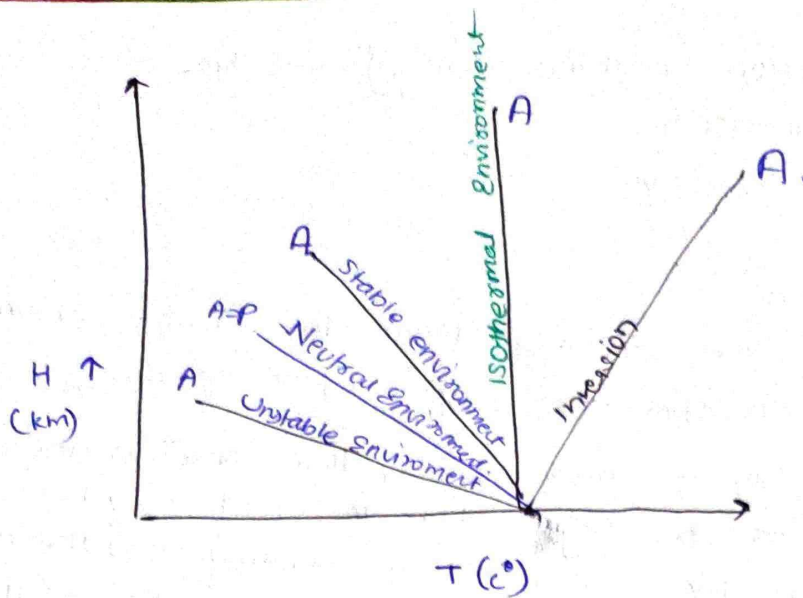
Subsidence Inversion.

→ It is associated with the high pressure region system and is caused by sinking or subsidence of air in high pressure area surrounded by low pressure area. (ie anti-cyclone).

→ The air around high pressure region gets compressed and descend thereby gets heated to form a warm dense layer over the ground surface. upto the height of 1.5-2 km.

→ Such inversion layer may by stopping upward movement of pollution causes the air pollution.





Q. For the following information identify the type of environment for location A, B, C, D above the surface of the earth when the pollutant is released in dry air.

Location	Height (m)	Temp °C
A	20	19
A →	320	30
B →	650	25
C →	800	20.95
D →	900	22.97

Solⁿ

	HT (m)	T °C
A	20	19
A	320	30
B	650	25

$$\frac{dt}{dh} = \frac{-(30-19)}{320-20 \times 10^{-3}}$$

$$= -36.66 \frac{^{\circ}\text{C}}{\text{km}}$$

Inversion ~~env~~ environment.

$$\frac{dt}{dh} = \frac{-(25-30)}{(800-320)} = 15.15 \frac{^{\circ}\text{C}}{\text{km}}$$

$$\text{ELR} \approx 15.15 \frac{^{\circ}\text{C}}{\text{km}} > \text{ALR}_{\text{dry}} (9.8 \frac{^{\circ}\text{C}}{\text{km}})$$

→ Unstable.

→ Superadiabatic.

	650	23.95
C	800	23.95
D		
⊙	900	22.97

$$\frac{dT}{dh} = - \frac{(23.95 - 22.97)}{(800 - 650) \times 10^3} = 7^\circ\text{C/km}$$

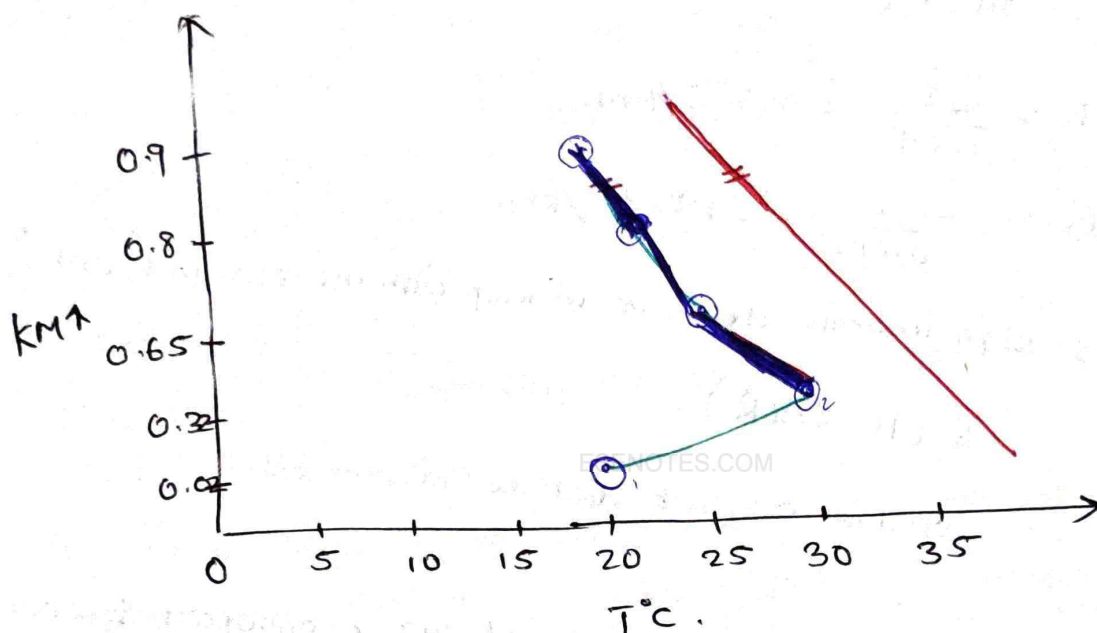
$$\text{ELR}(7^\circ\text{C/km}) < \text{ALR}_{\text{Dry}}(9.8^\circ\text{C/km})$$

Stable or sub adiabatic.

$$\frac{dT}{dh} = - \frac{(22.97 - 23.95)}{(900 - 800) \times 10^3} = 9.8^\circ\text{C/km}$$

$$\text{ELR}(9.8^\circ\text{C}) = \text{ALR}(9.8^\circ\text{C/km})$$

- Neutral



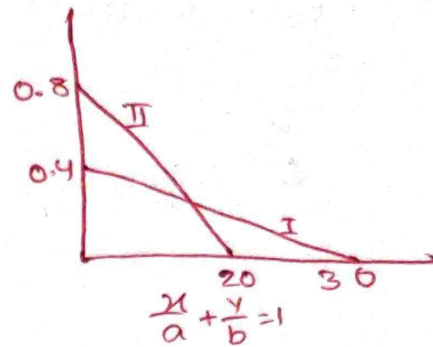
Q IF the state of variation of temp with height in air & pollutant is given by following relationship.

$$H_{P_{\text{pollut}}} = -0.133 T_p + 0.4$$

$$H_{A_{\text{air}}} = -0.04 T_p + 0.8$$

Here, HT is in km & temp in °C.

(i) Identify the type of environment.



$$y = mx + c$$

$$ELR = \frac{-1}{0.04} = -25^\circ\text{C/km}$$

$$ALR = \frac{-1}{0.0133} = -75.2^\circ\text{C/km}$$

(minus sign indicate decrease in temp. with increase in height)

$$(ELR < ALR)$$

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stable or subadiabatic environment

(ii) Find the height at which temp. of air & pollutant become same.

Let at height H , $T_A = T_P$

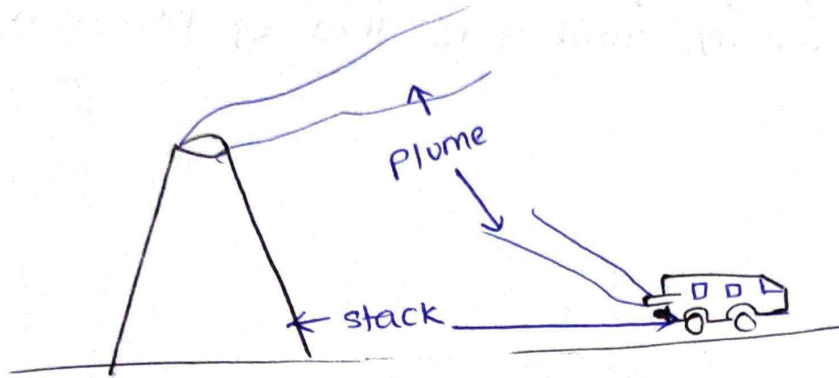
$$\frac{0.4 - H}{0.0133} = \frac{0.8 - H}{0.04}$$

$$H = 0.2 \text{ km}, T = 14.98^\circ\text{C}$$

$T_A > T_P$
 $\uparrow$
 0.2 km
 $\downarrow$
 $T_A < T_P$

→ Gases being released into the atmosphere is termed as "PLUME" and the source of their origin is termed as "STACK".

→ Depending upon the prevailing environmental condition following types of plume may be observed:

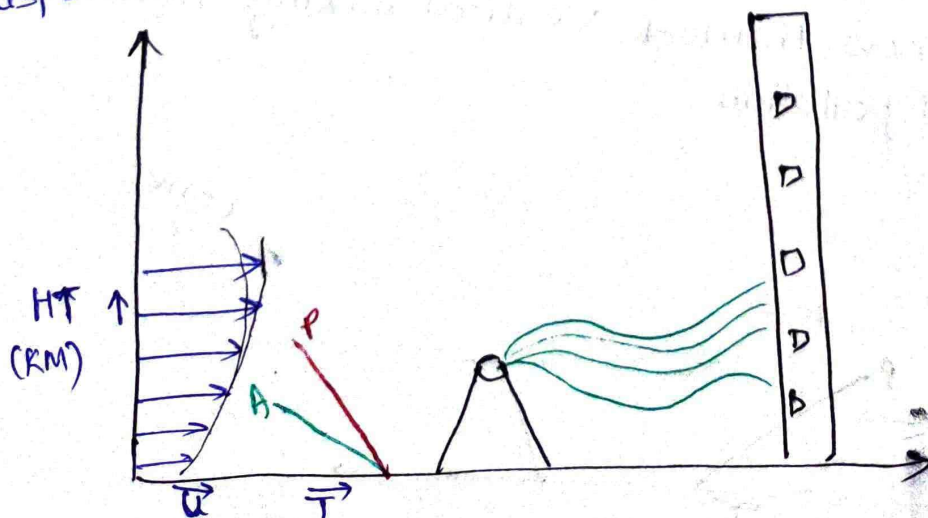


(i) Looping Plume

→ This plume is formed in unstable environment where $ELR > ALR$.

→ Because of rapid mixing of the pollutant in this case loops are formed wave character is observed hence it is termed as looping plume.

→ During the high degree of turbulence the dispersion of plume would be rapid, yet higher concentration near the ground may occur due to the turbulence, before the dispersion is finally completed.

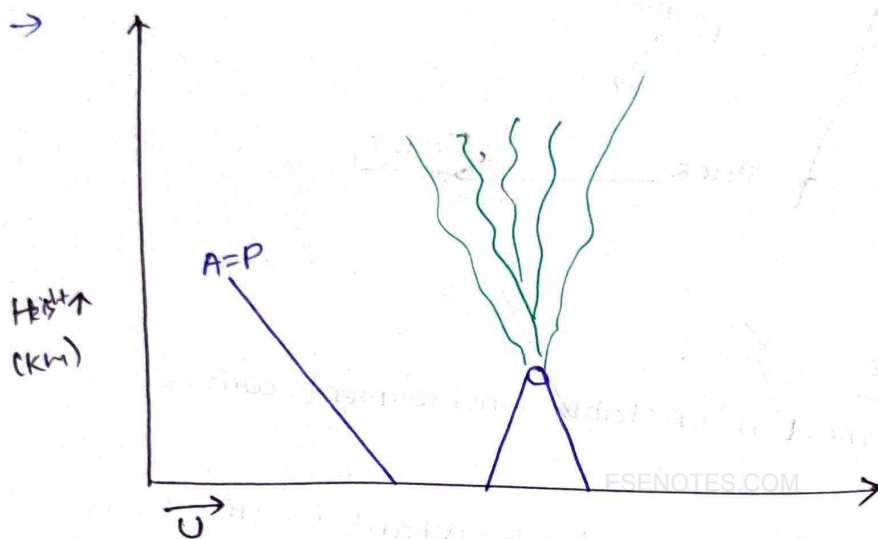


(ii) Neutral Plume

→ It is type of plume which is formed in neutral environment where $ELR = ALR$

→ It is characterised with the vertical rise of the pollutant from the stack

→ The upward lifting of the plume will continue till it reaches an air of density similar to that of plume itself.

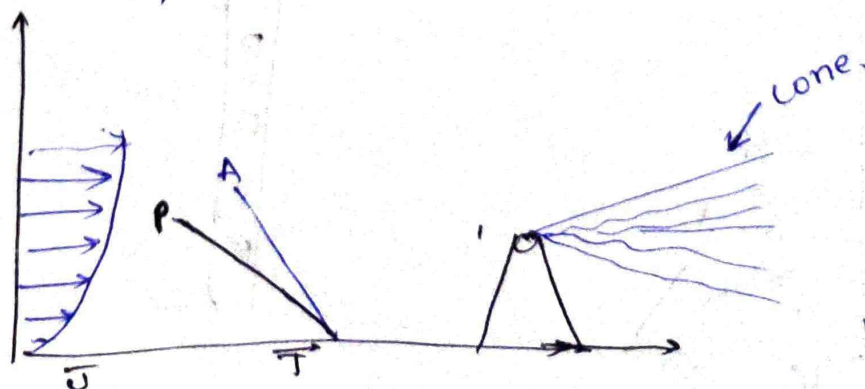


(iii) Coning Plume

→ This plume is observed in stable environment where environment $ELR < ALR$ and wind velocity is greater than 32 km/hr

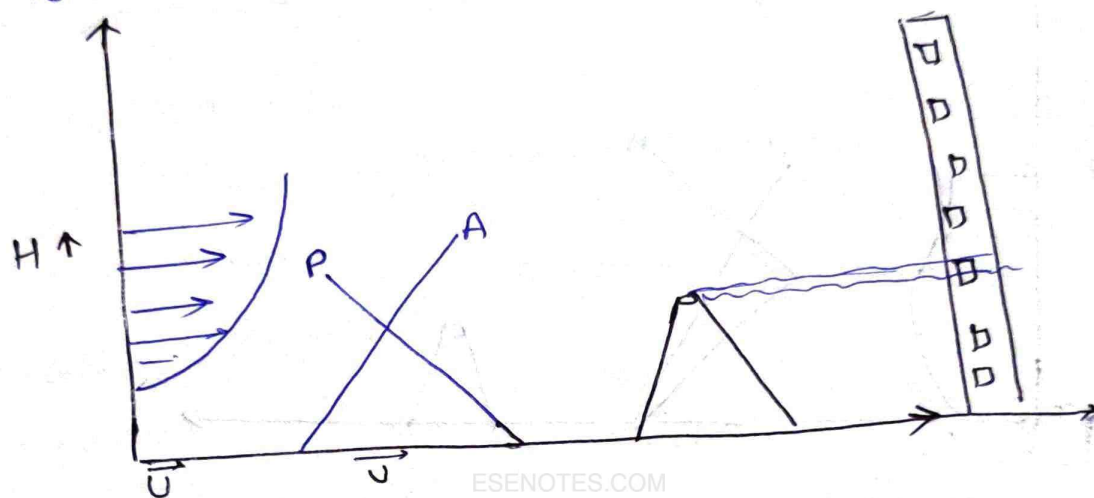
→ It is observed when cloud cover blocks the solar radiation by day and by night ~~radiation~~ are ~~are~~ terrestrial radiation are blocked.

→ It appears like cone in shape hence is termed as coning plume. (ie it has limited vertical mixing, hence increases the chances of pollution.



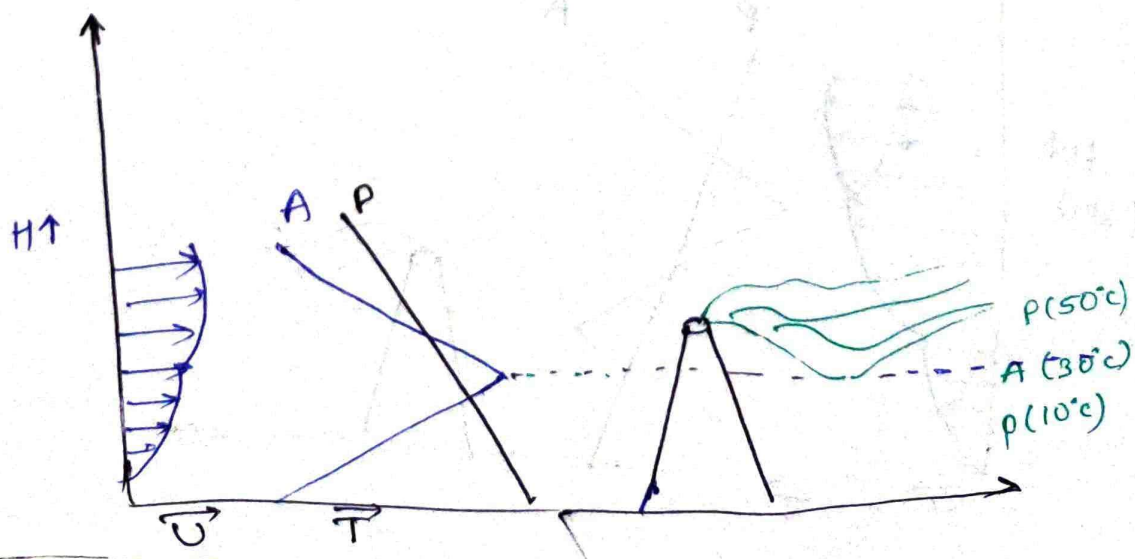
(iv) Fanning Plume

- This plume is observed in extreme inversion condition in which negative lapse rate is observed
- This plume is characterised with horizontal spread into the atmosphere due to limited tendency to rise vertically
- In order to avoid interference with life in this case height of the stack is properly designed.



(v) Lifting Plume

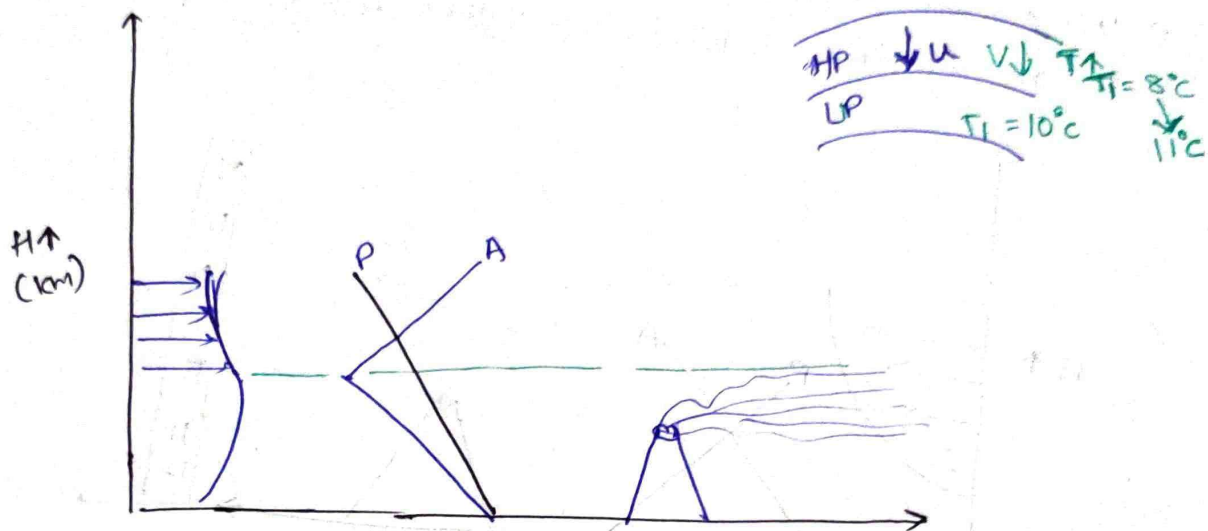
- This plume is found where there exist a strong super adiabatic lapse rate over a surface inversion.
- This plume has minimum downward mixing due to the presence of inversion below it.
- This plume is regarded as most ideal plume as it would have very less concentration of pollutants over the ground



(vi) Fumigating Plume

→ When inversion layer occurs at a short distance above the top of stack & super adiabatic condition prevails below it, leads to the formation of fumigating plume.

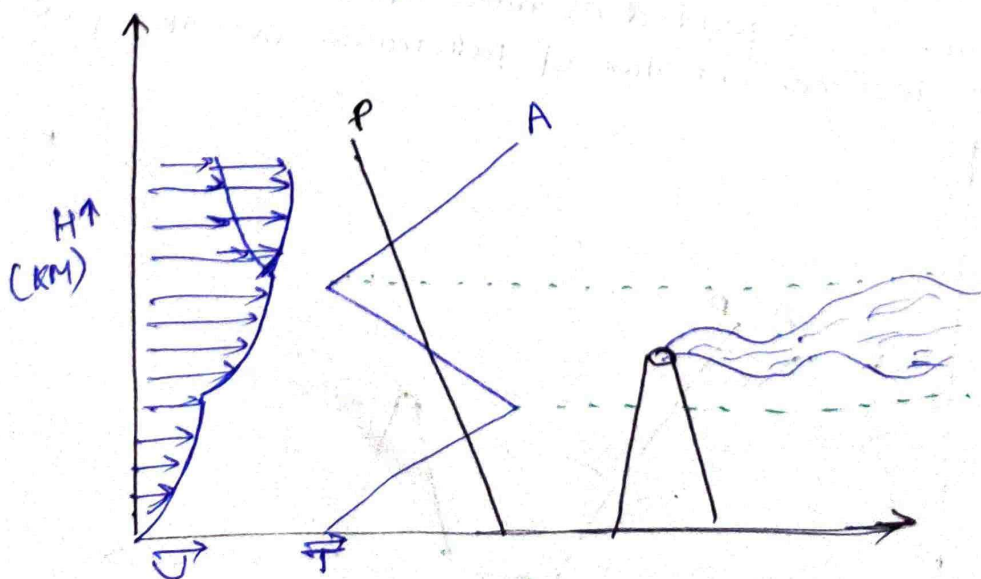
→ In such case the pollutant cannot escape above the top of the stack because of inversion layer over it, hence continues to interface with the life in biosphere.



(vii) Trapping Plume

→ When inversion layer exist between above & below the super adiabatic environment it leads to formation of trapping plume.

→ It is considered a bad condition for dispersion, as the dispersion cannot go above a certain height.



Air Pollution Control

- (i) Natural
- (ii) Artificial

Nature self cleansing properties of Environment:-

(i) Dispersion

Dispersion of pollutants by winds reduces the concentration of air pollutants at one place but doesn't remove them as a whole.

(ii) Gravitational Settling

Large heavy particles from the ambient air settle down on building.

(iii) Absorption / Washout

Gaseous as well as particulate pollutants gets collected in rain and settle out with that moisture.

(iv) Rainout

Process involving precipitation above the cloud level.

(v) Adsorption

Gaseous, liquid or solid pollutants are kept attached electrostatically by a surface.

(i) Gravitational settling chamber.

- The emitted pollutants, when made to pass through a settling chamber, drop some of their large size particles in the chamber (under Stokes law)
- The largest size of particle with 100% removal efficiency is given, by,

$$d = C \sqrt{\frac{18 \cdot \mu \cdot V_h \cdot H}{g \cdot L \cdot \rho_p}}$$

- V_h = horizontal velocity of gas.
- μ = viscosity of air.
- H = Height of chamber.
- L = length of chamber.
- C = correction factor
- ρ_p = density of particle removed.

- It removes particle of size greater than 50 μ m.
- Its $\eta < 50\%$.
- It is simple to design maintain and has low pressure loss.
- It requires large space and collection efficiency is also very low.
- Only large sized particles are removed.

(ii) Cen

(a) Cycl

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(ii) Centrifugal collectors

(a) Cyclone collector

(b) Dynamic collector.

(a) Cyclone Collector.

→ A cyclone collector is designed in such a way the velocity of inlet gas is transformed into spinning motion (vortex) and the particles get removed from the gas under the action of centrifugal force, which are further collected from the bottom.

→ The efficiency of this unit depends upon the velocity, mass of the particle, radius of cyclone as centrifugal force depends on all these parameters.

$$F_c = \frac{M_p v^2}{R}$$

M_p = Mass of particle.

v = Inlet gas velocity

R = Radius of cyclone.

→ It removes particle in range of 5-25 μm

→ Its efficiency is in range of 50-90%.

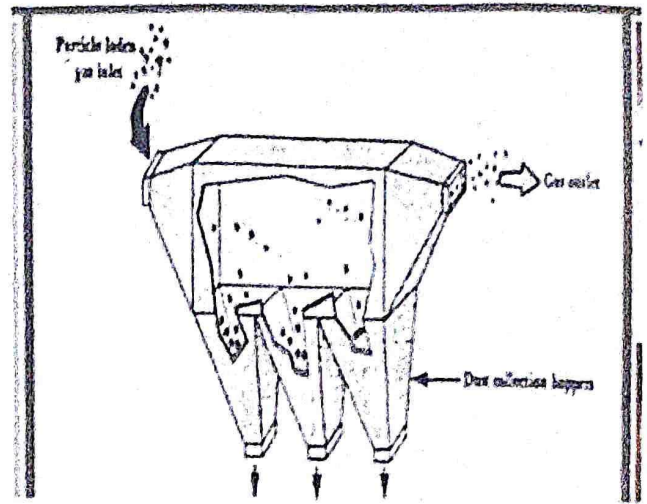
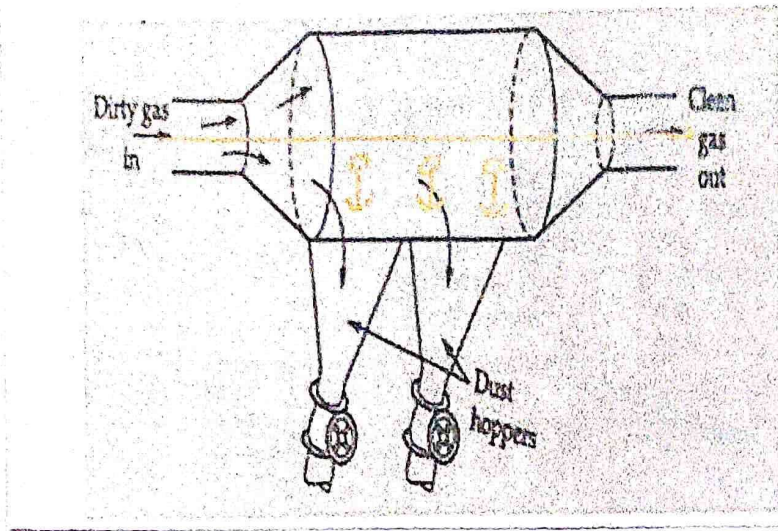
→ It is relatively inexpensive, simple to design require less floor area.

→ It insures ~~dust~~ dust & continuous of collected dust moreover it can handle large vol. of gases upto temp of 90°C.

→ It requires much head room, collector efficiency is low for smaller particle.

→ It is sensitive to variable dust loading & flow rate.

Note: In order to increase its effectiveness, dynamic precipitator is used which imparts centrifugal force to the entering gases with the help of rotating vane. (It is 7 times more effective than Cyclone precipitator).



(iii) Wet Scrubbers / collectors.

(a) spray tower

$$(d > 10 \mu m)$$

$$\eta < 80\%$$

(b) wet cyclonic Scrubbers.

$$(d > 2.5 \mu m)$$

$$\eta < 80\%$$

(c) Venturi Scrubber.

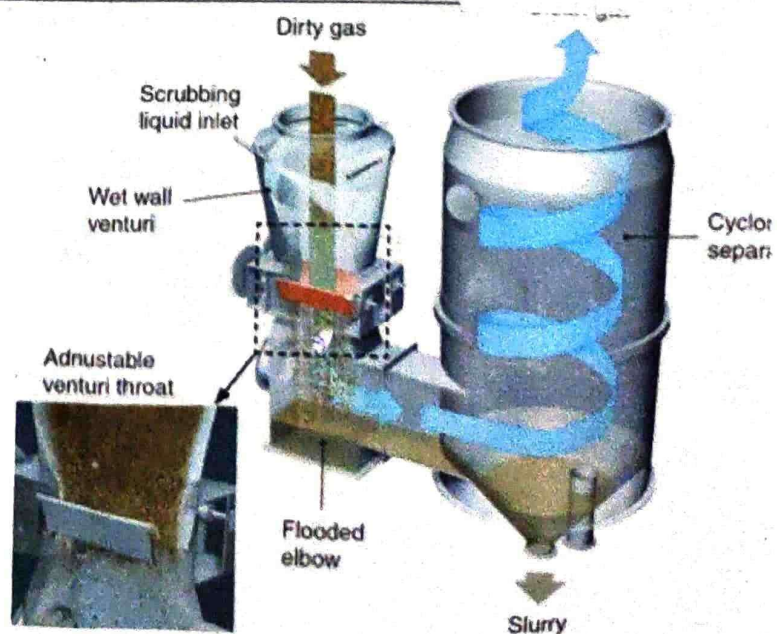
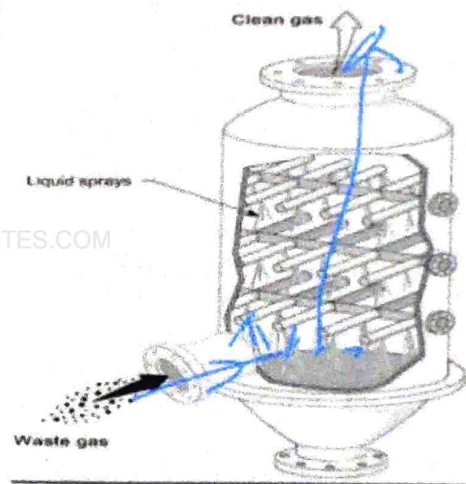
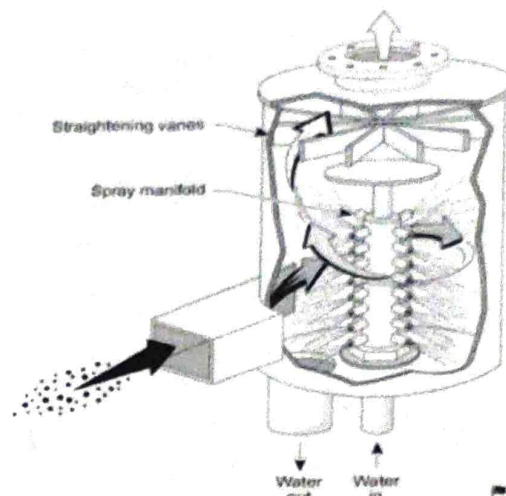
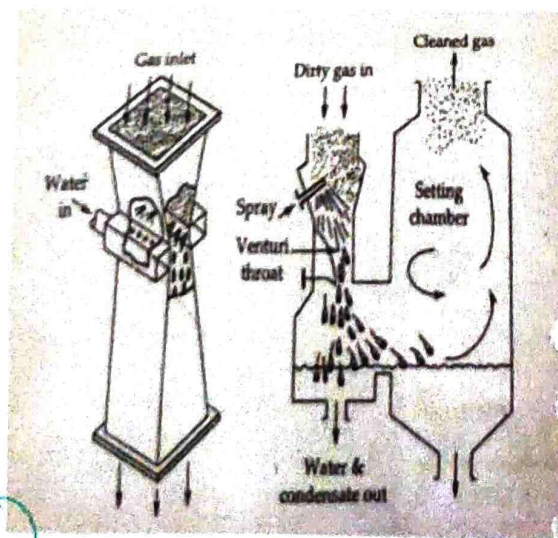
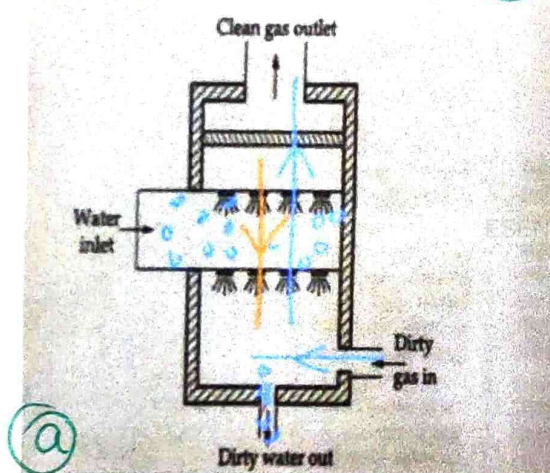
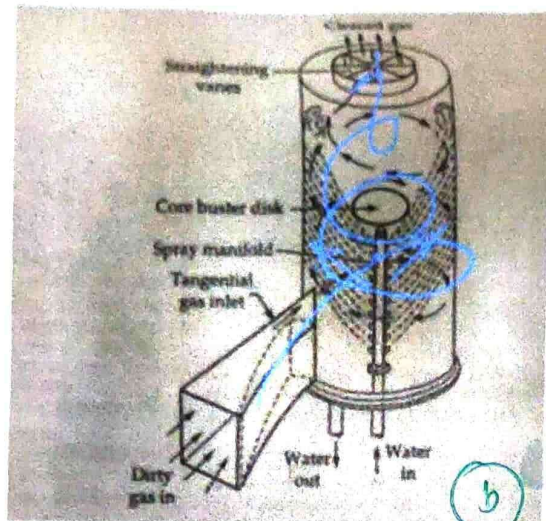
$$(d > 0.5 \mu m)$$

$$\eta < 90\%$$

→ In these devices the flow gas is made to push up against a downfalling water, in which the particulate matter mixes up with water droplets, & thus falls down and get removed.

→ Water solution, when replaced with other chemical solution like lime, K_2CrO_4 , slurry of magnesium oxide and MgO etc helps in removing gaseous pollutant along with particulate matter.

- In this process corrosive gases can also be removed.
- A lot of waste water is generated in this which requires disposal.
- Maintenance cost is high.



(iv) Electrostatic Precipitator.

→ In these unit flue gas is made to pass through a highly ionized zone where the particles get electrically charged and are separated from the gas, with the help of electrostatic forces in powerful electric field.

→ They are widely used in thermal power plant, pulp and paper industry.

→ It removes particles of size greater than 1 μ m.

→ Its efficiency is 95-99% and is given as follows.

$$\eta = 100 \left[1 - e^{-\frac{VA}{Q_a}} \right]$$

Q_a = flow rate of air

A = collector plate area.

V = velocity of air

→ It can remove particles in wet or dry state

→ It offers very high efficiency.

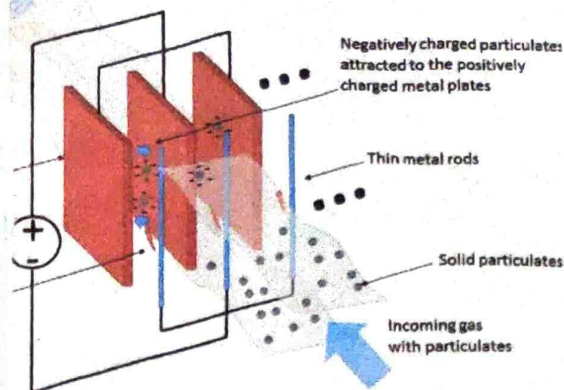
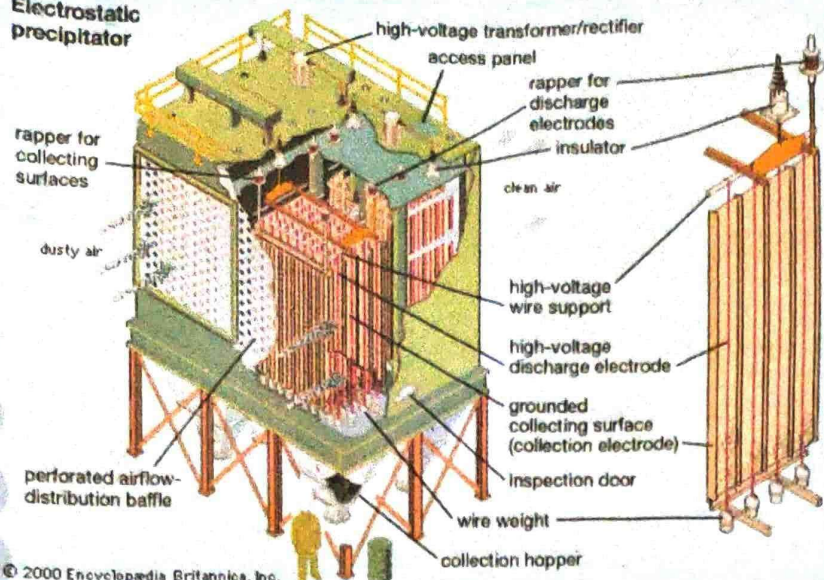
→ Its maintenance is nominal. & even small ^{dust} particles can also be removed.

→ It offers high initial cost.

→ It is sensitive to variable dust loading and flow rates.

→ It uses high voltage. hence power requirement is more.

Electrostatic precipitator



(V) Fabric / Bag filter

- In such unit the flue gases are allowed to pass through a bag bag filter which filters out the particulate matter and allows the gas to pass.

→ Small particles are retained on the fabric initially through interception and electrostatic attraction and later on when dust mat is formed it start getting retained over it more efficiently.

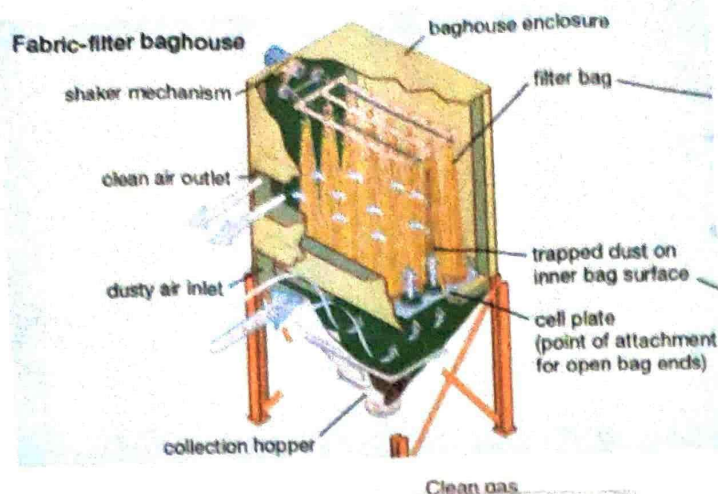
→ It removes the particle of size $< 1\mu m$.

→ Its $\eta > 99\%$.

→ It can remove particle in dry state

→ High temperature gases needed to be cooled down upto the range of $100-450^\circ C$ before processing.

→ The flue gas must be dry. or otherwise chances of condensation inside the filter increases.



Q A gas contain two types of suspended particle having average size of 2 μ and 50 μ amongst the option given, the most suitable pollution control strategy for removal of these particle is.

- (i) Electrostatic precipitator followed by Cyclonic Separator.
- (ii) ^{Settling} chamber followed by bag filter.
- (iii) Bag filter followed by ESP.
- (iv) ESP followed by venturi scrubber.

Q It was decided to construct a fabric filter of 0.45m diameter & 7.5 long for removal of industrial stack gas containing particulates. The expected rate of inflow into the filter is 10m³/s. If filtering velocity is 2m/s compute the minimum no. of bags required for continuous of cleaning operation.

$$N = \frac{SA}{SA_I} = \frac{QA}{SA \cdot V_A} = \frac{10}{2} = 5m^2$$

$$SA_I = \pi DH = 3.14 \times 0.45 \times 7.5 = 10.59m^2$$

$$N = \frac{5}{10.59} = 0.47 \approx 1 \text{ filter}$$

Q An electrostatic precipitator with 5600 m^2 of collector plate area is 96% efficient in treating $185 \text{ m}^3/\text{sec}$ of flow gas from 200 MW thermal power plant. It was found that in order to achieve 97% efficiency the collector plate area should be 6100 m^2 . In order to increase the efficiency to 99%. How much area of ESP collector plate is required.

$$96\% \rightarrow 5600 \text{ m}^2$$

$$\eta = 100 \left[1 - e^{-\frac{AV}{Q}} \right]$$

$$96 = 100 \left[1 - e^{-\frac{5600 \times V_1}{185}} \right]$$

$$97 = 100 \left[1 - e^{-\frac{6100 V_2}{185}} \right]$$

$$V_1 = 0.1061 \text{ m/s} \quad \left| \quad V_2 = 0.1061 \text{ m/s} \right.$$

$$V_{\text{avg}} = \frac{V_1 + V_2}{2} = 0.106 \text{ m/s.}$$

$$99 = 100 \left[1 - e^{-\frac{A \times 0.1061}{185}} \right]$$

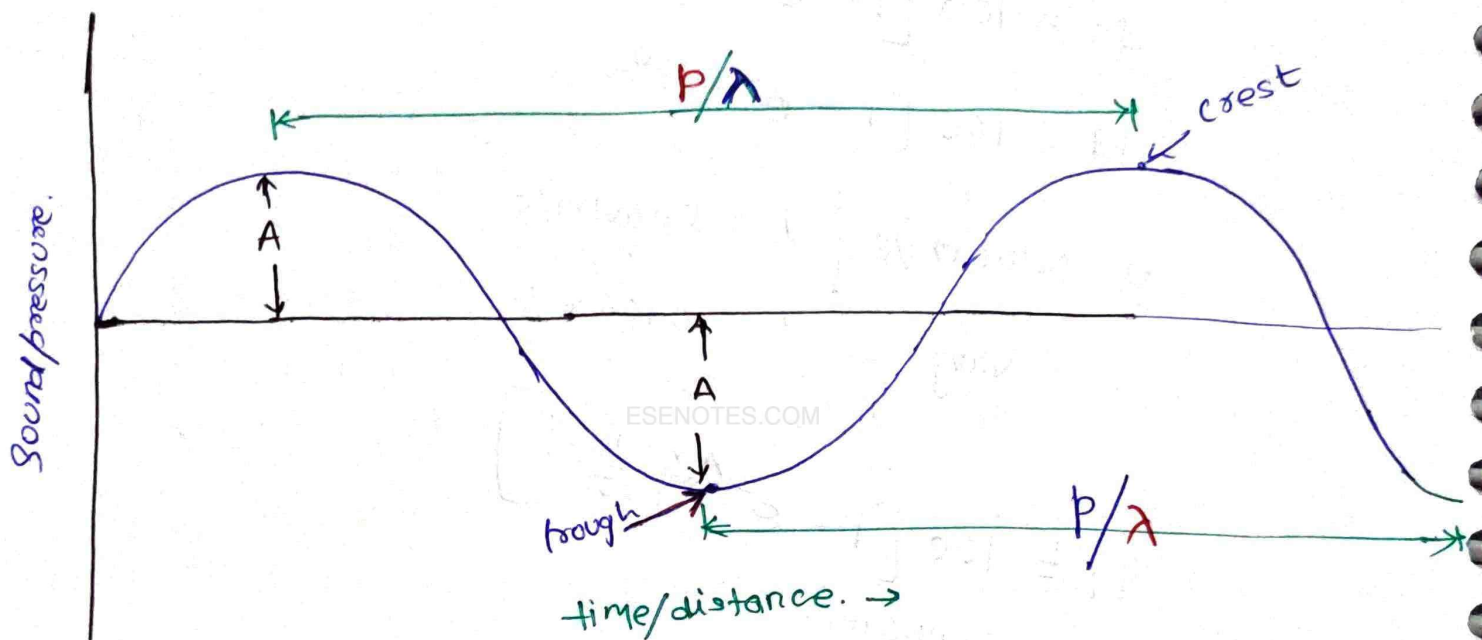
$$A = 8014.64 \text{ m}^2.$$

Noise Pollution

→ Presence of one or more sound in such duration at such pressure level and at such frequency that the life in biosphere gets effected is termed as noise pollution.

Properties of sound and its measurement.

→ Sound in any medium propagates/travels in the form of pressure waves undergoing compression and rarefaction/expansion.



(i) Time period.

Time between two successive crest or trough is termed as time period,

$$p = \frac{1}{f}$$

(ii) Wave length (λ)

Distance between two successive crest or trough is termed as wave length.

$$\lambda = cp$$

$$\lambda = \frac{c}{f}$$

$$c = \lambda f$$

c = velocity of sound wave.

(iii) Amplitude.

It is maximum or minimum pressure over or below the mean pressure. ($\bar{p}=0$) is termed as amplitude.

Note: All the sound measuring devices are designed to give root mean square pressure.

$$p_{rms} = \sqrt{p(t)^2}$$
$$= \sqrt{\frac{1}{T} \int_0^T p(t)^2 dt}$$

$p(t)$ = pressure at any time t .

$$\left[\sigma = \sqrt{\frac{\sum (x_i - \bar{x})^2}{N}} = \sqrt{\frac{\int_0^T (p(t) - 0)^2}{T}} = \sqrt{\frac{1}{T} \int_0^T p(t)^2 dt} \right]$$

Q If $p(t) = A \sin \omega t$, find p_{rms} ?

$$p_{rms} = \sqrt{\frac{1}{2\pi} \int_0^{2\pi} A^2 \sin^2 \omega t dt} = \frac{A}{\sqrt{2}}$$

(iv) Power of Sound (w)

It is defined as the rate of work done by travelling sound wave in the direction of propagation of wave ~~or~~ it is the energy transmitted by a sound wave in direction of its propagation.

(v) Sound Intensity (I)

It is defined as the sound power averaged over the time per unit area normal to the direction of propagation of sound wave.

$$I = \frac{W}{A} \text{ (watt/m}^2\text{)}$$

w = power of sound (watt)

A = area $\perp$ to direction of propagation of sound wave (m^2)

Note: Intensity of sound, $I = \frac{p_{rms}^2}{\rho c}$

ρ = density of air (of given medium)
 c = velocity of sound wave

$$[I \propto p_{rms}^2]$$

(ii) Smallest level of noise that can be heard by human ear is 20 μ Pa and highest may extend upto 200 Pa.

Level of Noise.

Level of noise of any given sound pressure is always measured w.r.t standard reference sound pressure.

$$L = \log_{10} \frac{Q}{Q_0} \quad (\text{Bels})$$

Q = Measured quantity of sound pressure (P) or sound intensity I .

Q_0 = Reference standard quantity of sound pressure, (P_0) or sound intensity.

$$L = 10 \log_{10} \left(\frac{Q}{Q_0} \right) \quad (\text{dB})$$

$$[1 \text{ Bel} = 10 \text{ dB}]$$

Case-I Sound Intensity level (L_i)

$$L_i = 10 \log_{10} \left(\frac{I}{I_0} \right) \text{ dB}$$

here, $I_0 = 10^{-12} \text{ W/m}^2$.

Case-II Sound pressure level (L_p)

$$L_p = 10 \log_{10} \left(\frac{p_{rms}}{p_{rms_0}} \right)^2$$

$$L_p = 20 \log_{10} \left(\frac{p_{rms}}{p_{rms_0}} \right) \text{ (dB)}$$

Here, $p_{rms_0} = 20 \mu\text{Pa}$.

$$[1 \text{ Bar} = 10^5 \text{ N/m}^2]$$

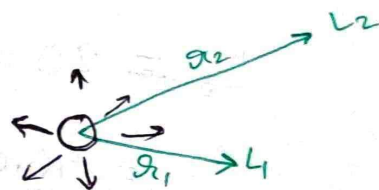
Note: An increase in 20 dB in sound pressure level will correspond to sound pressure or loudness of sound increasing by 10 times.

→ 1 dB is the faintest sound which can be heard by human ear and maximum can be tolerated upto 180 dB.

Note: In real medium, level of varies as follow: -

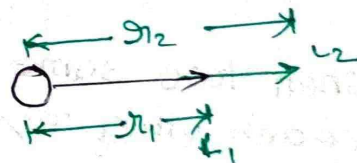
(i) For point source.

$$L_2 = L_1 - 20 \log_{10} \left(\frac{r_2}{r_1} \right)$$



(ii) For linear source

$$L_2 = L_1 - 10 \log_{10} \left(\frac{r_2}{r_1} \right)$$



(iii) Addition of two different level of noise is not simple arithmetic addition due to the involvement of log scale.

ex. $20 \text{ dB} + 40 \text{ dB} \neq 60 \text{ dB}$.

$$E \propto p_{rms}^2$$

$$E_0 = E_1 + E_2$$

$$K p_{rms_0}^2 = K p_{rms_1}^2 + K p_{rms_2}^2$$

$$p_{rms_0} = \sqrt{p_{rms_1}^2 + p_{rms_2}^2} \Rightarrow L_0 = 20 \log_{10} \left(\frac{p_{rms_0}}{20 \mu\text{Pa}} \right) \text{ dB}$$

$$\underline{Q} \quad \cancel{20 \text{ dB}} \quad L_1 = 20 \text{ dB} = 20 \log_{10} \frac{p_{rms1}}{20 \mu \text{Pa}}$$

$$p_{rms1} = 200 \mu \text{Pa}$$

$$L_2 = 40 \text{ dB} = 20 \log_{10} \frac{p_{rms2}}{20 \mu \text{Pa}}$$

$$p_{rms2} = 2000 \mu \text{Pa}$$

$$\begin{aligned} p_{rmsr} &= \sqrt{p_{rms1}^2 + p_{rms2}^2} \\ &= \sqrt{200^2 + 2000^2} \\ &= 2009.9 \mu \text{Pa} \end{aligned}$$

$$\begin{aligned} L_r &= 20 \log_{10} \left(\frac{2009.9}{20} \right) \\ &= 40.04 \text{ dB} \end{aligned}$$

Note ③ When two same sound pressure level are added with each other resultant is increased by "3dB"

$$L_r = L + L$$

$$L_r = L + 3$$

$$L = 20 \log_{10} \frac{p_{rms}}{20 \mu \text{Pa}} \Rightarrow p_{rms} = x \mu \text{Pa}$$

$$p_{rmsr} = \sqrt{p_{rms}^2 + p_{rms}^2} = \sqrt{2} p_{rms}$$

$$L_r = 20 \log_{10} \frac{\sqrt{2} p_{rms}}{20 \mu \text{Pa}}$$

$$= 20 \log_{10} \sqrt{2} + 20 \log_{10} \frac{p_{rms}}{20 \mu \text{Pa}}$$

$$L_r = 3 + L$$

$$\text{eq. } 20\text{dB} + 20\text{dB} = 23\text{dB}$$

$$\text{eq. } 100\text{dB} + 100\text{dB} = 103\text{dB}$$

$$\text{eq. } 50\text{dB} + 50\text{dB} + 50\text{dB} + 50\text{dB} = 56\text{dB}$$

$$\text{eq. } 20\text{dB} + 40\text{dB} = 40\text{dB} \text{ — } 43\text{dB}$$

$$\text{eq. } \underbrace{10\text{dB} + 20\text{dB}}_{20-23\text{dB}} + \underbrace{50\text{dB} + 80\text{dB}}_{50-53\text{dB}} + \underbrace{100\text{dB}}_{80-83\text{dB}} = 100-103\text{dB}$$

Note 4 Average of different sound level is not the simple arithmetic average due to the involvement of log scale.

$$L_{\text{avg}} = 20 \log_{10} \frac{1}{N} \sum (10)^{L_i/20}$$

$$\text{eq. } 20\text{dB}, 40\text{dB}, 80\text{dB}, 100\text{dB}.$$

$$L_{\text{avg}} = 20 \log_{10} \times \frac{1}{4} \left[10^{20/20} + 10^{40/20} + 10^{80/20} + 10^{100/20} \right]$$

$$= 88.8\text{dB}.$$

$$\text{Q8} \quad L_{\text{avg}} = 20 \log_{10} \frac{1}{4} \left[10^1 + 10^2 + 10^4 + 10^5 \right]$$

$$x < 10^5$$

$$L_{\text{avg}} = 20 \log_{10} \frac{1}{4} \left[10^{100/20} \right]$$

$$= 20 \log_{10} \left[10^{5/4} \right]$$

$$= 20 \left[\log_{10} 10^5 + \log_{10} \frac{1}{4} \right]$$

$$= 20 \left[5 - 2 \log_{10} 2 \right]$$

$$= 20 \left[5 - 2 \times 0.3010 \right] = 87.96$$

Note 5. Ambient air quality standard w.r.t. to noise as per EPA.

Area Code	Category of Area zone	Limits in dB (Leq)	
		Day time	Night time
A	Industrial area	75	70
B	Commercial Area	65	55
C	Residential area	55	45
D	Silence.	50	40

Note 6: In order to reduce the noise pollution obstruction and barrier between the noise source and point of impact can be constructed which reduces the noise level given as follows:-

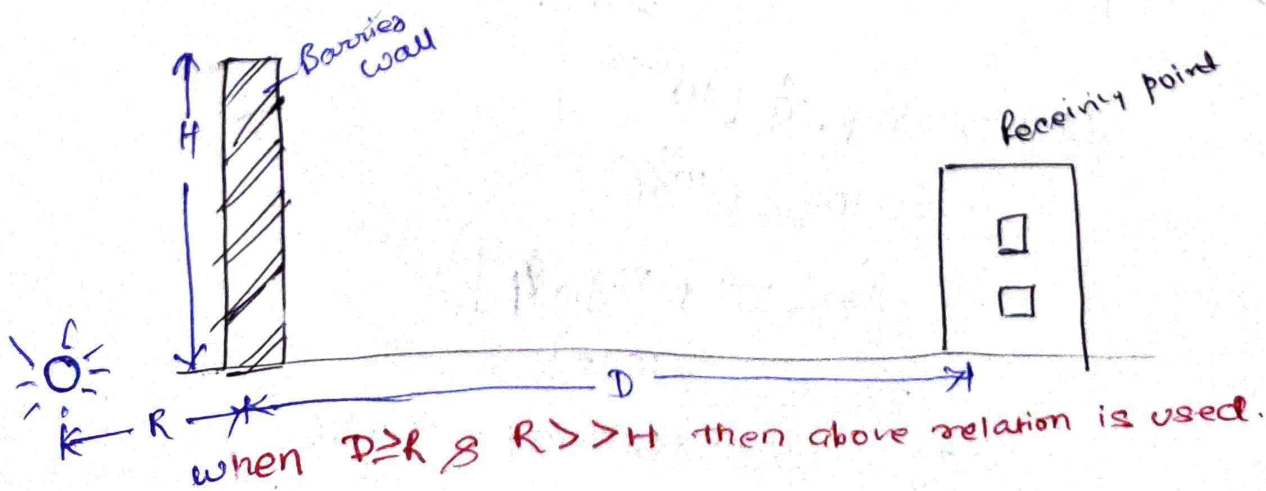
$$\text{Noise reduction (dB)} = 10 \log_{10} \left(\frac{20H^2}{\lambda R} \right)$$

R = distance of barrier wall from the source in (m)

H = Height of barrier wall.

λ = wave length of sound.

D = distance between barrier and receiving point.



#Rating of Noise.

As different types of noise exist in the system at different pressure level at different frequency and for different duration in order to find their resultant effect. Rating of noise is done by any of the following method.

(i) Equivalent Concept (L_{eq})

It is the constant sound pressure level which over a given time expand the same amount of energy as is being expanded by fluctuating sound pressure level in same time

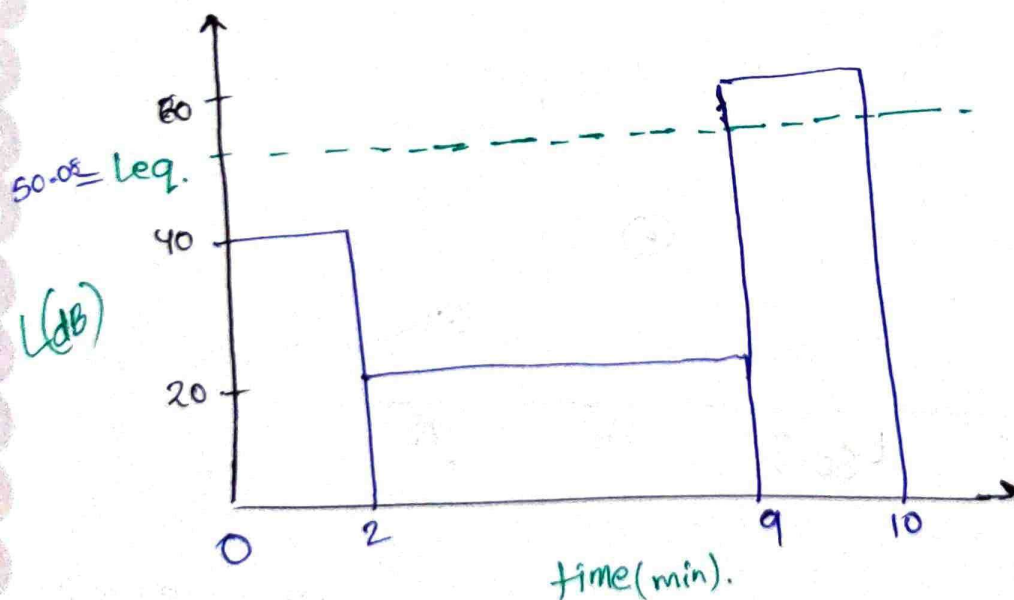
$$L_{eq} = 10 \log_{10} \sum_{i=1}^n 10^{(L_i/10)} \times t_i$$

n = total no. of sound sample.

L_i = given sound level sample.

t_i = time duration of i th sample of sound, expressed in terms of total time.

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$$L_{eq} = 10 \log_{10} \left(10^{40/10} \times \frac{2}{10} + 10^{20/10} \times \frac{7}{10} + 10^{60/10} \times \frac{1}{10} \right)$$
$$L_{eq} = 50.088 \text{ dB}$$

(ii) L_N Concept

L_N represents the sound pressure level that is exceeded for N% of gauging time.

eg

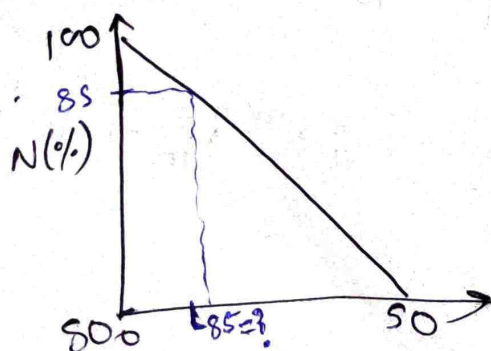
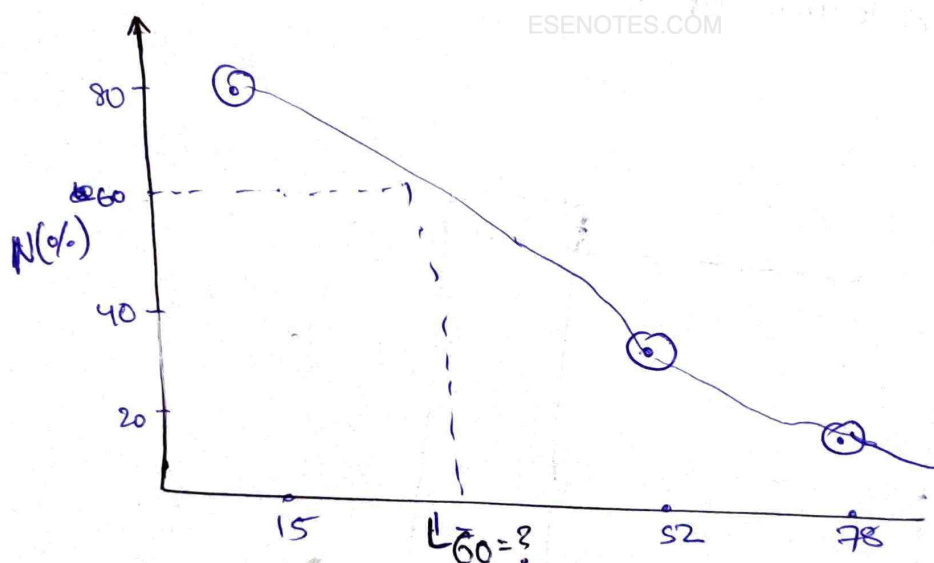
	L (dB)	<u>rank in descending</u>
1.	80	100
2.	78	80
3.	100	78
4.	5	65
5.	15	52
6.	24	48
7.	36	36
8.	48	24
9.	52	15
10.	65	5

$$L_N = x \text{ dB}$$

eg $L_{20} = 70 \text{ dB}$

$$L_{40} = 52 \text{ dB}$$

$$L_{80} = 15 \text{ dB}$$



$$L_{85} = ? \quad \frac{100-85}{L_{85}-0} = \frac{100-80}{50-0}$$

$$L_{85} = 12.5 \text{ dB}$$

$$L_{85} = 37.5 \text{ dB}$$