12-th JULY
SATURDAY

ENVIRONMENTAL ENGINEERING

[10-12 marks]

Objective:

(i) To protect human against environmental factors

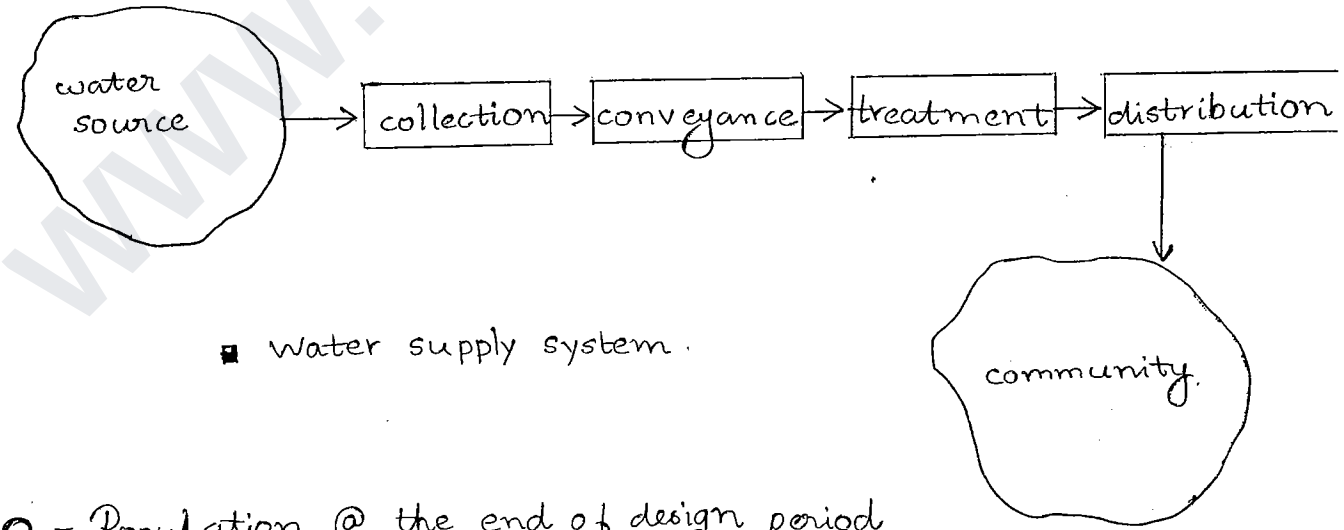
(ii) To protect environments against human actions.

1. Water Supply Engg. – 5 to 6 marks
2. Waste Water Engg. – 4 to 5 marks
3. Solid Waste management
4. Air pollution Control.
5. Noise pollution Control.

} 1 to 2 marks.

WATER SUPPLY ENGINEERING:

Objective: To supply right quality water as per needs of the public (quantity)



■ Water supply system.

Q = Population @ the end of design period
X per capita water demand.

13th July, SUNDAY

Population Forecasting Methods:

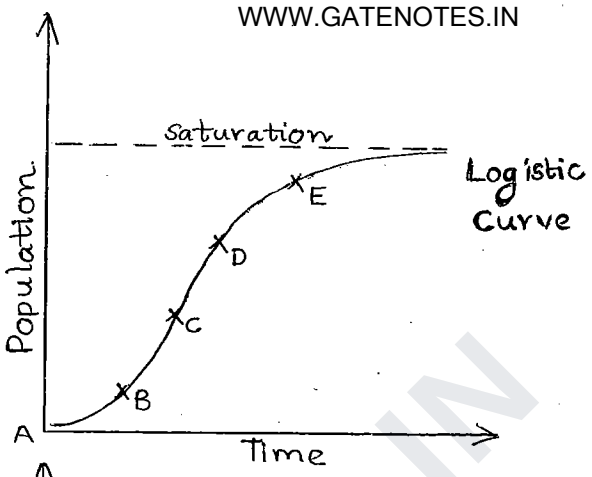
AB → early growth.

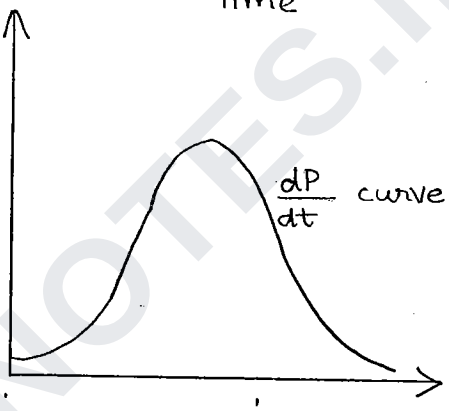
BC → exponential or logarithmic growth.

CD → constant growth

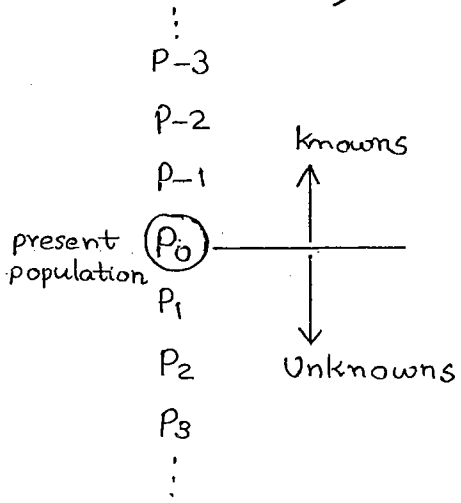
DE → late growth.

Slope of the curve = $\frac{dP}{dt}$





1. Arithmetic Increase method.
2. Geometric Increase method.
3. Incremental Increase method.
4. Decrease in growth rate method.
5. Logistic Curve method.
6. Simple Graphic method.
7. Comparative graphical method.
8. Master plan method.
9. Ratio method.



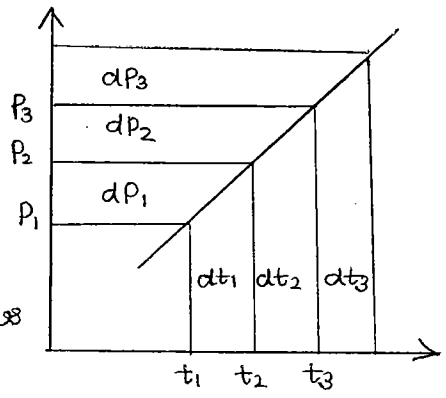
Arithmetic Increase Method:

$dP_1 \simeq dP_2 \simeq dP_3$ [Const. Growth Method]

$dt_1 : dt_2 : dt_3$

$\frac{dP}{dt}$ is constant.

Old and settled communities follows this trend.



Per decade increase in population is assumed to be constant in population forecasting using Arithmetic Increase method.

Let 'P_0' be the latest known last decade population data

'$\bar{x}$' be the arithmetic avg. of per decade increase in population

Population after 1 decade, $P_1 = P_0 + 1\bar{x}$

Population after 2 decade, $P_2 = P_1 + \bar{x}$

$= P_0 + 2\bar{x}$

Population after 'n' decades, $P_n = P_0 + n\bar{x}$

P-6

Level 2 : Q-01

Ans : (c)

Year	Population	Per decade ↑ in popln (dP)
1970	40,000	
		6000 (dP_1)
1980	46,000	
		7000 (dP_2)
1990	53,000	
		5000 (dP_3)
2000	58,000.	

$[dP_1 \simeq dP_2 \simeq dP_3]$

$P_{2010} = P_{2000} + \bar{x}$

$= 58000 + \frac{18000}{3} = \underline{\underline{64000}}$

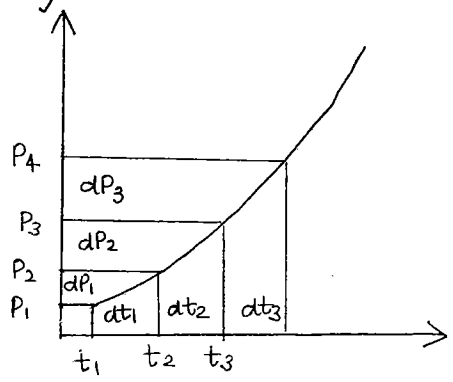
$n = \frac{2010 - 2000}{10} = \underline{\underline{1}}$

Geometric Increase Method : [Exponential / Logarithmic Growth Method]

$dP_1 < dP_2 < dP_3$ (fast growing communities)

$\frac{dP}{dt} \propto P_t$ (population at a time 't')

In Geometric increase method, per decade '% increase' in population is assumed to be constant.

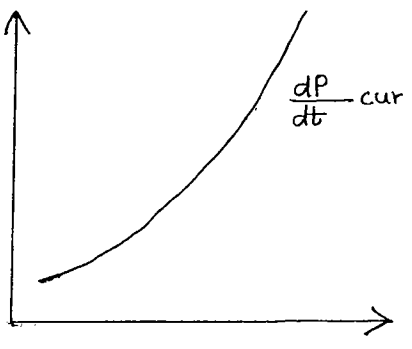


$\frac{dP}{dt} = k P_t$

$\frac{dP}{P_t} = k dt$

Integrating,

$\int_{P_1}^{P_2} \frac{dP}{P_t} = \int_{t_1}^{t_2} k\,dt$



$$[\ln P_t]_{P_1}^{P_2} = k[t]_{t_1}^{t_2}$$

$$\ln\left(\frac{P_2}{P_1}\right) = k(t_2 - t_1)$$

$$\Rightarrow \frac{P_2}{P_1} = e^{k(t_2-t_1)}$$

$\Rightarrow$ exponential growth.

$$P_2 = P_1 e^{k(t_2-t_1)}.$$

Similarly,

$$\boxed{P_{tn} = P_{to}\, e^{k(t_n - to)}.}$$

$k \rightarrow$ percentage increase per unit time (r)

Level 2 : Q-03.

$$P_{2020} = P_{2000}\, e^{\frac{1.6}{100}(2020-2000)}.$$

$$= 10^9 \times e^{\frac{1.6}{100}(20)}.$$

$$= 1.377 \text{ billion.}$$

$$P_{2050} = P_{2020}\, e^{\frac{1.2}{100} \times 30}$$

$$= 1.974 \text{ billion}$$

Q-08

Ans : (a).

$$P_2 = P_1 e^{\frac{20}{100} \times n}.$$

$$2 = e^{\frac{20}{100} n}. \qquad P_2 = 2P_1$$

$$n = \frac{\ln 2}{0.2} = 3.46 \text{ years}$$

Method 2 : If k (or r) not given, then it can be worked out using the past population data.

Let $\bar{r}$ (or k) be the geometric average of per decade percentage increase in population (assume to be constant)

To find future population, $\bar{r}$ is compounded with existing population.

Population after 1 decade $= P_0 + \frac{\bar{r}}{100} P_0$

$= P_0\left(1 + \frac{\bar{r}}{100}\right)$

Population after 2 decade $= P_1 + \frac{\bar{r}}{100} P_1 = P_1\left(1 + \frac{\bar{r}}{100}\right)$

$= P_0\left(1 + \frac{\bar{r}}{100}\right)^2$.

$$\boxed{\text{Population after 'n' decades} = P_0\left(1 + \frac{\bar{r}}{100}\right)^n}$$

$$\bar{r} = (r_1 \times r_2 \times r_3 \times \ldots r_n)^{1/n}$$

$$r_n = \left(\frac{P_n - P_{n-1}}{P_{n-1}}\right) \times 100$$

Q-02 Ans : (c)

$r_1 = 40$, $r_2 = 20$

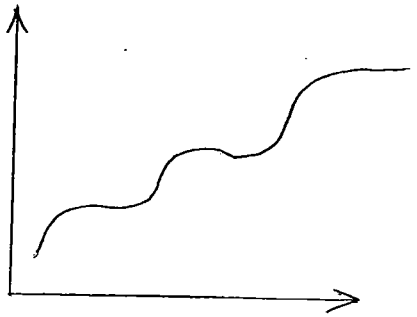
$\bar{r} = (40 \times 20)^{1/2} = \underline{20\sqrt{2}}$

$P_4 = P_3\left(1 + \frac{r}{100}\right) = 1.68\left(1 + \frac{20\sqrt{2}}{100}\right)$

$= \underline{2.155}$ lakhs

4th JULY,
SUNDAY.

Incremental Increase Method



$dP_1 < dP_2 > dP_3$

$dP_1 > dP_2 < dP_3$

This is an extension of Arithmetic increase method. This method forecast population based on past growth without making any assumptions. To find future populati average of increments over increases are worked out

Let '$\bar{y}$' be the avg of the increments over increasing population. This $\bar{y}$ is to be added to arithmetic increase once for 1st decade, twice for 2nd decade and n times for nth decade. to arrive at future population

Population after 1 decade, $P_1 = P_0 + 1\bar{x} + 1\bar{y}$

$= P_0 + 1\bar{x} + \frac{1(1+1)}{2}\bar{y}$

Population after 2 decade, $P_2 = P_1 + 1\bar{x} + 2\bar{y}$

$= P_0 + 2\bar{x} + 3\bar{y}$

$= P_0 + 2\bar{x} + \frac{2(2+1)}{2}\bar{y}$

Population after 'n' decades, $P_n = P_0 + n\bar{x} + \frac{n(n+1)}{2}\bar{y}$ $\quad \bar{y} = \pm ve$

P-6

Level 2: Q-04.

$P_0 = 50000$, $\bar{x} = 5000$, $\bar{y} = 500$, $n = \frac{2020-1990}{10} = 3$

$P_{2020} = 50000 + 3 \times 5000 + \frac{3(4)}{2} \times 500.$

$= 68000$

Q-05

Year	Population	dP	Increment over Increase in popln.
1940	250000		
		230500	
1950	480500		−160700
		69800	
1960	550300		+18500
		88300	
1970	638600		−31700.
		56600.	
1980	695200		

$\bar{x} = 111300$ $\quad \bar{y} = -57966.66.$

Ans: (a)

$dP_1 > dP_2 < dP_3 > dP_4$ (confusing trend, ∴ incremental increase method)

$P_{2000} = 695200 + \frac{2}{3} \times 111300 + \frac{2 \times 3}{2} \times -57966.66.$

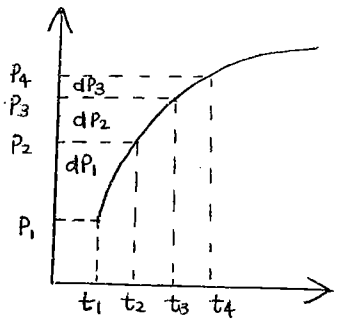
$= 743900$

Total water requirement $= 743900 \times 200$ lpcd

$= 148.78$ MLD $\quad (1\ ML = 10^6\ L)$

$= 149$ MLD

Decreasing Growth Rate Method.



$dP_1 > dP_2 > dP_3$

This trend shown by communities fast moving towards saturation..

Eg: Mumbai, Chennai etc.

$$\frac{dP}{dt} \propto (P_s - P_t)$$

'Rate of increase' decreases. ~~Later~~

Let $\bar{D}$ be the average decrease in percentage increases.

r_0 be the latest known last decade % increase.

This method is an extension of geometric increase method.

Population after 1 decade, $P_1 = P_0 + \left(\frac{r_0 - \bar{D}}{100}\right) \times P_0$

$$= P_0\left(1 + \frac{r_0 - \bar{D}}{100}\right)$$

Population after 2 decade, $P_2 = P_1 + \left(\frac{r_0 - 2\bar{D}}{100}\right) P_1$

$$= P_1\left(1 + \frac{r_0 - 2\bar{D}}{100}\right)$$

Population after n decades, $P_n = P_{n-1}\left(1 + \frac{r_0 - n\bar{D}}{100}\right)$

$$P_n = P_0\left(1 + \frac{r_0 - \bar{D}}{100}\right)\left(1 + \frac{r_0 - 2\bar{D}}{100}\right)\left(1 + \frac{r_0 - 3\bar{D}}{100}\right)\ldots\left(1 + \frac{r_0 - n\bar{D}}{100}\right)$$

P-7 : Q-14

14.

Year	Population	% increase.	decrease in % increase
1970	25000		
		12	
1980	28000		−9.43 ~~−5.65~~
		21.43 ~~17.65~~	
1990	34000		−2.01 ~~−1.39~~
		23.53 ~~19.05~~	
2000	42000		11.63 ~~8.4~~
		11.9 ~~10.64~~	
2010	47000		$\bar{D} = 0.033$

$r_0 = 11.92$

$\bar{D} = 0.033$

$n = 2$

$$P_{2030} = P_{2010}\left(1+\frac{r_0-\bar{D}}{100}\right)\left(1+\frac{r_0-2\bar{D}}{100}\right)$$

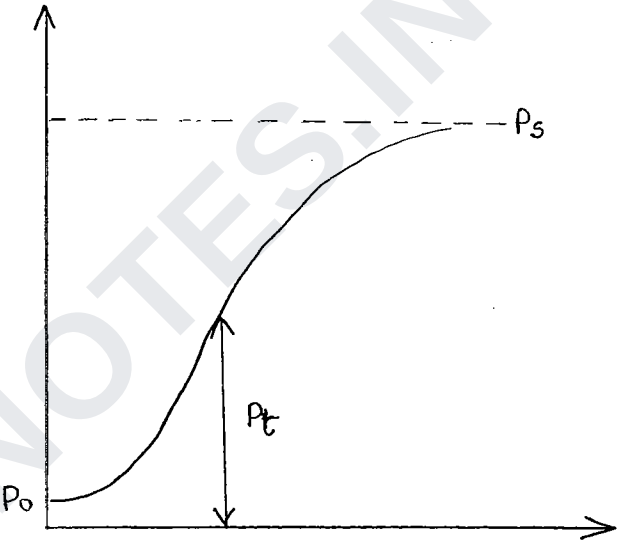
$$= 47000\left(1+\frac{11.62-0.033}{100}\right)\left(1+\frac{11.62-2\times0.033}{100}\right)$$

$$= \underline{\underline{58800}}$$

Logistic Curve Method (Logistic Growth Method):

$$\ln\left(\frac{P_s-P_t}{P_t}\right)-\ln\left(\frac{P_s-P_0}{P_0}\right)=-KP_st.$$

$$\boxed{P_t=\frac{P_s}{1+me^{nt}}}$$



If three population data spreaded at equal intervals are known, ie

At $t_0=0$, P_0

$t_1=t_1$, P_1

$t_2=2t_1$, P_2.

Saturation population, $P_s = \dfrac{2P_0P_1P_2-P_1^2(P_0+P_2)}{P_0P_2-P_1^2}$

$$m = \frac{P_s-P_0}{P_0}$$

$$n = \frac{1}{t_1}\ln\left[\frac{P_0(P_s-P_1)}{P_1(P_s-P_0)}\right]$$

Q-13

$t_1=20$, $P_0=30000$, $P_1=170000$, $P_2=300000$

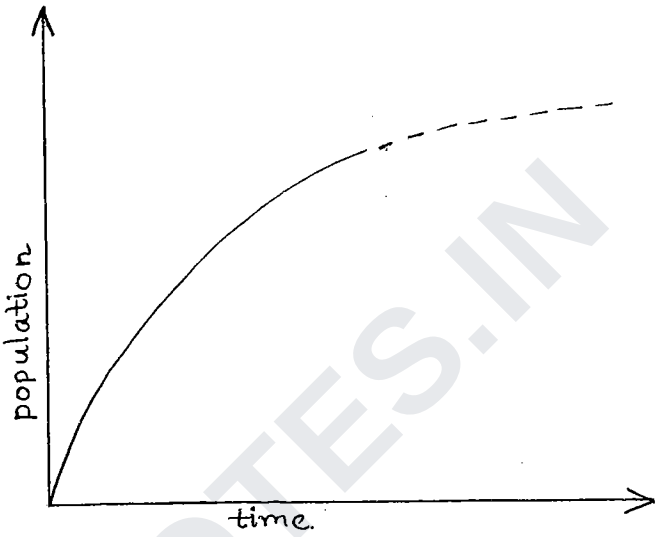
$$P_s = \cancel{463869.346}\ \underline{325478}$$

$$m = \frac{P_s-P_0}{P_0} = \overset{9.85}{\cancel{14.4623}}\ , \quad n = \frac{1}{20}\times\overset{-0.118}{-2.124} = \underline{\underline{\cancel{-0.106}}}$$

$$P_t = \frac{P_s}{1+me^{nt}} = \frac{325\,478}{1+9.85e^{-0.118\times60}} = 322931$$

Simple Graphical Method:

Based on data from past, Population vs Time plot is made.
It is then extrapolated. based on experience to obtain the future population

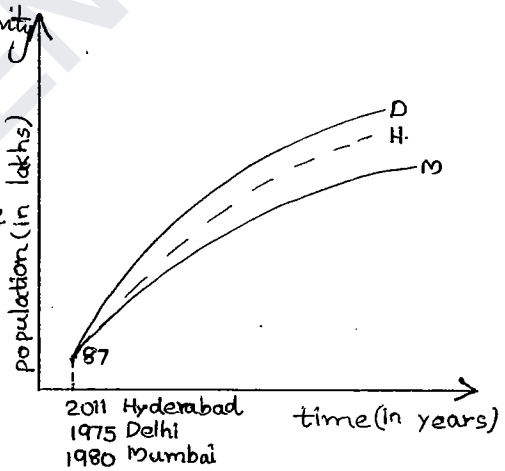


15 JULY
TUESDAY

Comparative Graphical Method:

Population of present community is forecasted based on the population growth trend of some other communities which've developed in a similar manner.

Enhancement of simple graphical method.



Master Plan Method (Zoning Method):

P-6: Q-06.

400 000			
5 58 500	158500 dP_1	39.62	$dP_1 < dP_2 < dP_3$
776 000	217500. dP_2	38.94	(geometrical increase)
1098 500.	322500. dP_3	41.56.	
	$\bar{x}$ =	$\bar{r} = (39.62\times38.94\times41.56)^{1/3}$	

$= 40.07$

$$P_1 = P_0\left(1+\frac{\bar{r}}{100}\right)^1$$

$$= 1098500\left(1+\frac{40.07}{100}\right) = 1537944 \approx \underline{1540,000}$$

07. At $t=0$, $P_0 = 28000$, $Q_0 = 4200$ m^3/day.

$P_D = ?$ $Q_D = 6000$ m^3/day.

At $t=20$ $P_{20} = 44000$

Q = Population x per capita demand.

* For a given community, per capita demand is always constant

$$\frac{Q_0}{P_0} = \frac{Q_D}{P_D} = \frac{Q_D}{P_0 + t\bar{x}}$$

$$P_{20} = P_0 + 20\bar{x}$$

$$\bar{x} = \frac{44000-28000}{20} = 800/yr$$

$$\frac{4200}{28000} = \frac{6000}{28000+t\times800}$$

$$t = \underline{15 \text{ years}}$$

Level 1 : Q-26.

$$P_{20} = P_0 + 20\bar{x}$$

$$\bar{x} = \frac{48000-28000}{20} = 1000/yr$$

$$Q_0 = 28000 \times 150$$

$$= 28\times150 \ m^3/day$$

$$\frac{28\times150}{28000} = \frac{6000}{28000+t\times1000}$$

$$t = \underline{12 \text{ years}}$$

Per capita Water Demand:

Per capita water demand (litre/capita/day) = $\frac{\text{total annual average water consumption of community.}}{\text{Population} \times 365}$

Water Demands of a Community: (IS 1172:1993)

1. Domestic water demands. (50-60%)
2. Industrial water demands.
3. Institutional / Commercial water demands.
4. Demand for public use.
5. Fire demands
6. Demands to compensate losses & theft (15-20%)

* Min 135 lpcd is required to meet the domestic demands of a community. It can vary upto 200 lpcd.
* Min. 50 lpcd required to meet industrial demands
* 20 lpcd required for institutional and commercial demands like railway stations, airports, temples, mosques, restaurants, hospitals, schools
* 10 lpcd needed to address the demands for public use.
* 55 lpcd required to compensate losses and thefts.

Empirical Formulae to Find Fire Demands:

1. Kuichling's formula.

$$Q = 3182\sqrt{P}$$

P → population in thousands.

Q → fire demand in litres/minute (lpm).

2. Freemen's Formula

$$Q = 1136\left(\frac{P}{5} + 10\right)$$; Q in lpm & P in thousands.

3. National Board of Fire underwriter's Formula.

Insurance company in USA. Equation is used for populations less than 2 lakhs. Widely accepted formula.

$$\boxed{Q = 4637\sqrt{P}\,(1-0.01\sqrt{P})}$$

4. Buston's Formula.

$$\boxed{Q = 5663\sqrt{P}}$$

P-7 : Q-12

$$P = \frac{100000}{1000} = 100.$$

$$Q = 4637\sqrt{100}\,(1-0.01\sqrt{100}).$$

$$= 41733 \text{ L/min.}$$

$$= \frac{41733 \times 24 \times 60}{1,00,000} = \underline{\underline{600 \text{ lpcd}}}$$

$$\text{lpm} = \frac{1\,l}{\frac{1}{24\times60}\,\text{day}}$$

Volume of water used to put off fire = Q × duration of fire fighting

= 41733 × (4) × 60. ← duration of fire fighting

Fire demand (in lpcd) = $\frac{\text{vol. of water consumed per month duri... fire accide...}}{\text{Population} \times 30}$

$$= \frac{41733 \times 4 \times 60}{10^5 \times 30} = \underline{\underline{3.33 \text{ lpcd}}}$$

* As per IS code, total demand of a community vary from 270 to 335 lpcd.

Factors Affecting Water Demand :

(i) Size of the community (iv) Pressure (vii) Cost of water.
(ii) Climatic conditions (v) Method of supply (viii) Quality of water
(iii) Socio-economic status of people (vi) Type of sewerage

Variation's in Water Demand :

Average daily water demand, Q = Population * per capita water demand

Masc. daily water demand = 1.8 times the avg. daily demand.

ie $\boxed{Q_{max\ daily} = 1.8\,Q.}$

Masc. hourly water demand = 1.5 times masc daily water demar

$\boxed{Q_{max\ hourly} = 2.7\,Q.}$

* Coincident draft (draft = depth of water = demand)
  = masc. daily water demand + fire demand.

* Total demand (or total draft) = masc. hourly water demand / OR / Coincident draft } which is mas

Component of water supply scheme	Design rate.	Design Period.
1. Dams	masc. daily water demand.	50 years
2. Intakes	masc daily water demand.	30 years
3. Conveying main (pipe)	masc. daily water demand.	30 years.
4. Pumps.	2 x Qmax daily	15 years.
5. Treatment units (except rapid sand).	1.8 x Qmax daily.	30 years.
6. Rapid sand filters	2 x avg daily demand	15 years.
7. Water distribution system	Total demand.	30 years.
8. Distribution reservoir	Masc. hourly demand	15 years.

P-06 : Q-09.

$Q = 300 \times 4 \times 10^5 = 120$ ~~m^3/day~~ MLD.

$Q_{max\ hourly} = 2.7 \times Q = 324$ MLD

$Q_{max\ daily} = 1.8 \times Q = 216$ MLD

Q-10

$Q = 250 \times 10^{65} = 25$ MLD.

Filters and lift pumps are designed for $= 2 \times Q$

$= 2 \times 25 = 50$ MLD

Q-11

$Q = 25$ MLD.

Fire demand = 61 MLD.

Total demand = Q_{max} hourly or Coincident draft.

$= 2.7\,Q$ or $(1.8\,Q + 61)$

$= 67.5$ or $(61 + 45)$

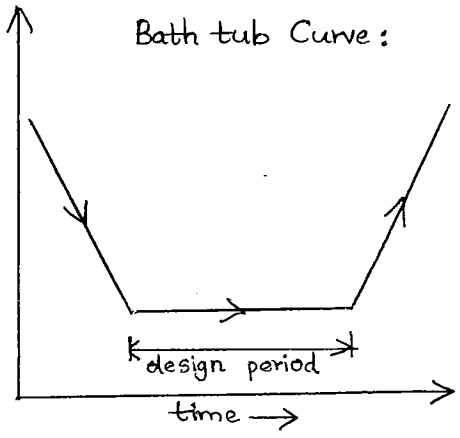
∴ Capacity of distribution system = 106 MLD

Design Period :

Design period is the utility period or useful life period.

By knowing the design period, we can replace the components

Water supply scheme, on an average, has a period of 30-40 years

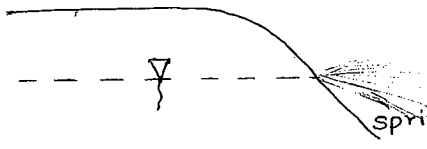


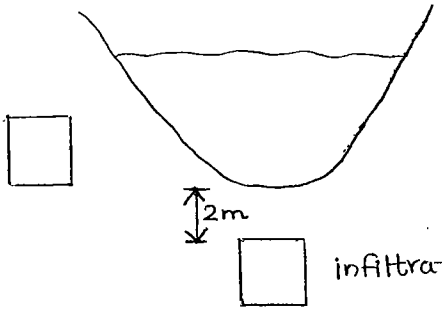
17th JULY, THURSDAY

2. SOURCES & CONVEYANCE OF WATER

Surface Sources – Quality bad but large quantity (Ponds, streams, reservoirs)

Subsurface Sources – high quality but less quantity (ground water) springs, wells, infiltration gall



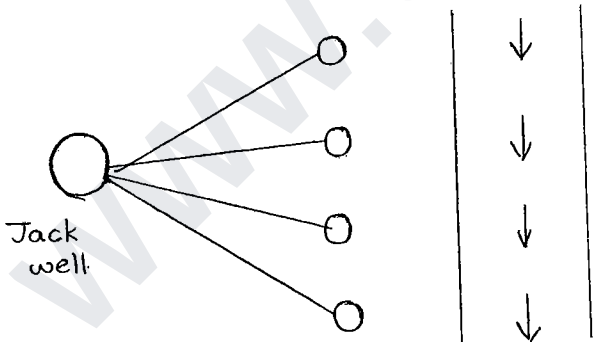


Good quality water due to natural filtration by sand.

infiltration galleries – hollow tunnels made at bed or base of river.

$Q = \frac{KL(H^2-h^2)}{2R}$; if infilt. gallery collects water from only one side (bank)

$Q = \frac{KL(H^2-h^2)}{R}$; if IG collects water from both sides (bed



Vertical holes made to extract ground water – Wells.

Horizontal holes made to extract seepage water – infiltration galleries

Intakes:

Structure constructed within the surface water body (bed to comfortably collect water from source, stores them temporari and conveys it to distribution pipe.

Collects + store + delivers → Wet Intakes (River intake)

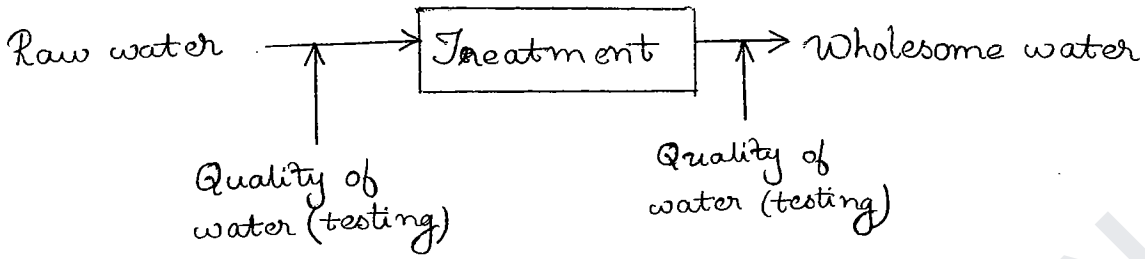
Collect + deliver → Dry intakes (reservoir intake)

WATER CONVEYANCE :

1. Gravity Conveyance
2. Pressure Conveyance - using pipe and pump.

17th JULY
THURSDAY

3. QUALITY OF WATER



Quality of Water (Testing of water):—

1. To know the extent of impurities present in water, based on this method and degree of treatment is decided.
2. To ensure the quality of treated water before supply.

Impurities in Water

1. Based on size of impurities.

macro ← (i) Suspended impurities... 10^{-1} mm to 10^{-3} mm [100 μm – 1 μm]

micro { (ii) Colloidal impurities... 10^{-3} mm to 10^{-6} mm [1 μm – 10^{-3} μm]
micro { (iii) Dissolved impurities... 10^{-6} mm to 10^{-8} mm [10^{-3} – 10^{-5} μm]

2. Based on nature of impurities

(i) Organic impurities – if present in water, they promote growth of miro-organisms. Hence objection

(ii) Inorganic impurities – of natural origin are harmless, but that induced by humans are hazardo

Eg: Cadmium, chromium, nickel, lead, zinc, mercury etc are toxic.

3. State of matter.

(i) Physical impurities.

(ii) Chemical impurities

(iii) Biological impurities.

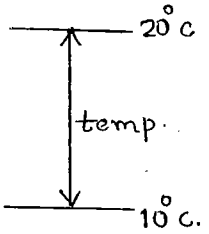
Physical Water Quality:

Assessed by knowing the extend of physical impurities in water. Physical water quality parameters are those which respond to human senses

1. Temperature. ($10^\circ c$ to $20^\circ c$)

2. Colour (i) Warm temperature is ideal for microbial growth

(ii) Solubility of gases dependent of temperature. Higher the temperature, lower will be solubility.



(iii). Chemical reactions are temperature controlled.

(iv) $\mu_{water} \propto \frac{1}{\text{temperature}}$ ∴ temp should not be less than $10^\circ c$.

2. Colour. (5 TCU to 20 TCU)

Apparent colour → suspended solids.

True colour → dissolved substances.

Colour in water measured on Burgess scale (Platinum scale) and expressed in terms of True colour Units (TCU) by using device known as "Tintometer".

1 TCU = 1 mg of Platinum as a chloroplatinate ion mixed in 1 litre of distilled water, then the colour produce is taken as 1 standard TCU.

Eg: if 4 mg mixed in 8 L, TCU = 0.5 TCU (mass ÷ volume)

Drinking water standard unit colour = 5 to 20 TCU (mg/L). Less than 5 TCU is not easily achievable. Colour > 20 TCU is perceptable, so anything above 20 TCU is rejected.

3. Turbidity (5 TU to 10 TU)

Opaqueness in water is known as turbidity. It is the resistance to passage of light through water by the particles

Turbidity in water is caused by suspended & colloidal substances.
Smaller particles – high turbidity (more will be the surface area)
Larger particles – large turbidity
Surface area governs turbidity. Turbidity is a surface phenomenon

* Dissolved particles do not possess any surface area. They go into molecular level. So turbidity is offered by suspended and colloidal particles.

Turbidity is the measure of resistance offered by particles in water to the passage of light through water.

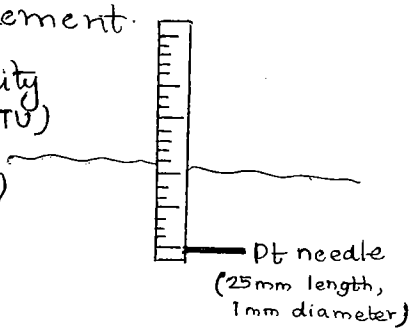
* Turbidity is measured on 'Silica Scale' and expressed in terms of 'Turbidity Units' (TU)

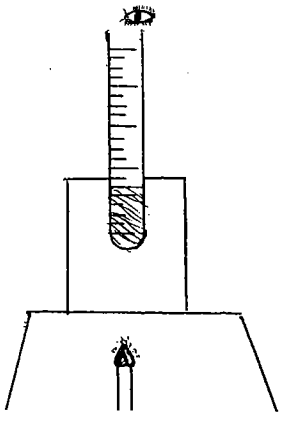
∴ 1 TU = 1 mg of finely divided silica mixed in 1L of distilled water, then the turbidity produced is taken as 1 standard turbidi unit.

It is also known as 'Jackson Turbidi Unit' (JTU

* Turbidity is measured by:
  (i) Devices working on the principle of light absorption.
  (ii) Device working on the principle of light scattering.

* Turbidimeters (Light Absorption)
  (i) Jackson Turbidi rod – field measurement.
  (ii) Jackson Turbidimeter – for high turbidity (> 25 JTU)
  (iii) Baylis meter – low turbidity (0–2 JTU upto 5 JTU)
  (iv) Hillege meter (upto 50 JTU)



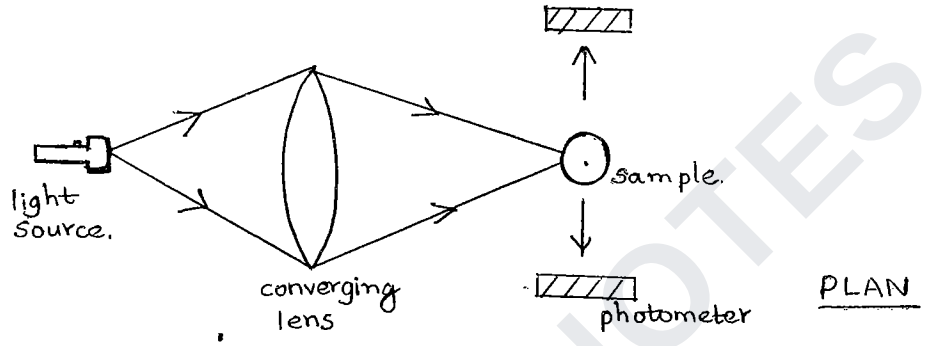


* Nephlometer (light scattering)

Nephlometer measure turbidity in terms of NTU (Nephlometric Turbidity Unit)

1 NTU ≈ 1 mg of formezyn polymer mixed in 1L of distilled water.

It's also expressed in FTU as formezyn is used as a polymer



PLAN

Nephlometer measures amount of light scattered ↑ to incident light and the measure is electronically displayed. Hence its almost free from human errors.

Drinking water std. wnt turbidity = 5 NTU to 10 NTU.

18th JULY FRIDAY

4. Odour & Taste (<3 TON)

Odour in water is measured by a device known as "Osmoscope". It is an inhaler with capacity 200 mL. To ensure the safety of the person who conducts test, diluted samples are always used.

Odour is expressed in terms of 'TON' (Threshold Odour Number). TON is a dilution ratio at which odour is just detectable.

$$TON = \frac{A+B}{A} \quad ; \quad A+B = 200 \text{ mL (capacity of device)}$$

A → vol. of water sample used in dilution (mL).

B → vol. of distilled water (mL)

A	B	TON = 200/15
5mL	195mL	
10 mL	190 mL	
15 mL	185 mL (detected)	

Drinking water standards for Odour = less than 3 TON

TON = 1 can be observed only by trained persons.

TON = 2 can be sensed by avg. individuals if told.

There are no such tests to measure taste. Generally a sample with bad odour is expected to taste bad.

FTN — Flavoured Threshold Number.

P-23

43. $TON = \frac{200}{12.5} = \underline{\underline{16}}$

19

5.8. $FTN = \frac{200}{25} = \underline{\underline{8}}$

Chemical Water Quality:

1. Solids.

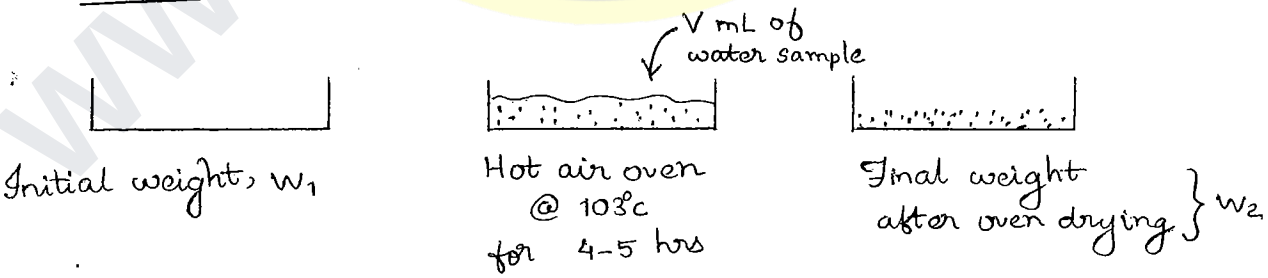
Residue left on evaporation is called Solids in water.

(i) Total Solids
(ii) Suspended Solids
(iii) Total dissolved solids (TDS)

} Gravimetry (weight measurement). Expressed in mg/L or ppm.

NOTE: For water 1 mg/L ≃ 1 ppm.

* Total Solids:-



$$\text{Total solids} = \frac{W_2 - W_1}{V} \text{ mg/L}$$

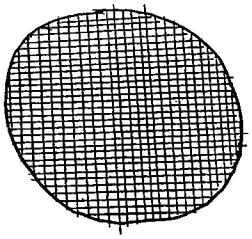
Q A 50 mL of water sample drawn on to an empty dry container whose initial wt. found to be 92.436 g, After oven drying, the final wt. of container found to be 92.468 g. Find total solids in water in mg/L

$$\text{Total solids} = \frac{(92.468 - 92.436)\,10^3}{50 \times 10^{-3}} = 0.064 \times 10^{4}$$

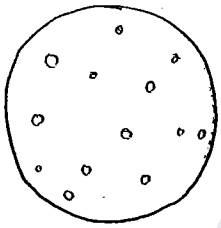
$$= 640 \text{ mg/L}$$

* Suspended Solids :-

Filter papers of pore size smaller than the smallest suspended particle is used for filtration. Weight of solids retained on filter paper are measured.



Initial wt. of empty dry filter paper, W_1



Hot air oven @ 103°C for 4-5 hours

Final wt. of filter paper = W_2

$$\text{Suspended solids (mg/L)} = \frac{W_2 - W_1}{\text{Vol. of water filtered.}}$$

Q. A 50 cc of water sample passed through empty dry filter paper whose initial wt. found to be 1.248 g. After oven drying its final wt. measured to be 1.262 g. Find suspended solids in water in mg/L.

$$\text{Suspended solids} = \frac{(1.262 - 1.248)\,10^3}{50 \times 10^{-3}} = 280 \text{ mg/L}$$

$1\ m^3 = 1000\ L$

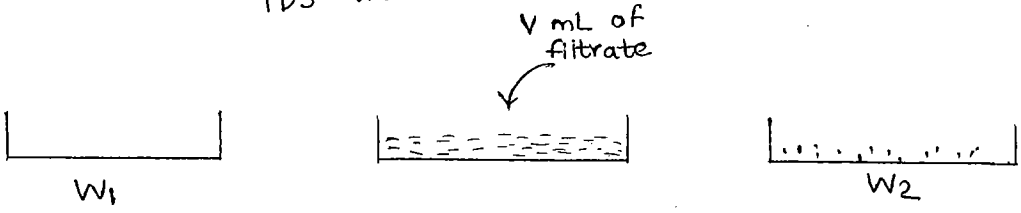
$10^6\ cm^3 = 1000\ L$

$1\ m^3 = 100$

$1\ cm^3 = 1$ n

* Total Dissolved Solids (TDS) [< 500 mg/L]

TDS includes both colloids and dissolved particles.



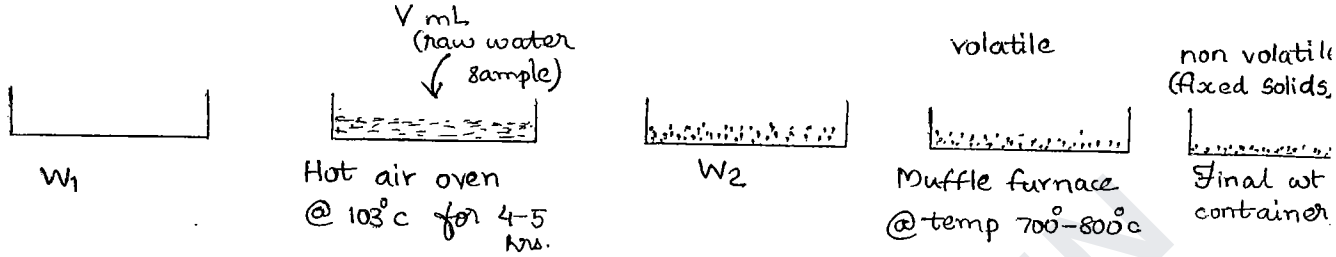
$$\text{TDS (mg/L)} = \frac{W_2 - W_1}{\text{Vol. of filtrate}}$$

; TDS < 500 mg/L (drinking water standards)

* Volatile & Non Volatile (fixed) Solids.

$$TS = \frac{W_2 - W_1}{V} \; ; \; \text{Volatile solids} = \frac{W_2 - W_3}{V} \; ;$$

$$\text{Non volatile solids} = \frac{W_3 - W_1}{V} \quad (mg/L).$$



D-23

47. $V = 100$ mL, $W_1 = 98.42$ g, $W_2 = 98.484$ g, $W_3 = 98.462$ g.

$$TS = \frac{64}{100 \times 10^{-3}} = 640 \text{ mg/L}$$

$$\text{Volatile solids} = \frac{W_2 - W_3}{V} = \frac{22 \times 10^{+3}}{100} = 220 \text{ mg/L}$$

$$\text{Non volatile solids} = \frac{W_3 - W_1}{V} = \frac{42 \times 10^{3}}{100} = 420 \text{ mg/L}$$

2. pH (6.5 to 8)

pH is a measure of potential of hydrogen ion concentration.

$$H_2O \longrightarrow H^+ + OH^- \quad \text{(Ionisation)}$$

$$\text{Ionisation constant, } K = \frac{[H^+][OH^-]}{H_2O}$$

For pure water @ 4°c, $H_2O = 1 \Rightarrow K = 10^{-14}$ mol/L

$$\therefore \boxed{[H^+][OH^-] = 10^{-14} \text{ mol/L}}$$

Applying $\log_{10}$ on either side,

$$\log_{10}[H^+] + \log[OH^-] = \log_{10} 10^{-14} = -14$$

$$\log_{10} \frac{1}{H^+} + \log \frac{1}{OH^-} = 14$$

$$\boxed{pH + pOH = 14.}$$

$$pH = \log_{10}\frac{1}{H^+} = -\log_{10}[H^+]$$

$$pOH = \log_{10}\frac{1}{OH^-} = -\log_{10}[OH^-]$$

P-19
Q-6. Whether its strong acid/base or weak acid/base, product of $[H^+]$ and $[OH^-]$ is always constant (14)

Q. If $[OH^-] = 10^{-8.25}$ mol/L, then find pH of water

$$pOH = -\log_{10}10^{(-8.25)} = 8.25$$

$$pH = 14 - 8.25 = \underline{\underline{5.75 \text{ mol/L}}}$$

P-22
Q.36. $pH = 4.1$

$$-\log[H^+] = 4.1$$

$$[H^+] = 10^{-4.1} = \underline{\underline{7.94 \times 10^{-5}}} \text{ mol/L}$$

P-23
Q.44. $[OH^-] = 17 \text{ mg/L} = \frac{17}{17 \times 1000} = 10^{-3}$ mol/L.

$17 \times 10^{-3} \times 6.024 \times 10^{-24}$ ions/L

mol/L $\rightleftharpoons$ mg/L

$$\boxed{x \text{ (in mg/L)} = x \text{ (in mol/L)} * \text{mol. wt of } x * 1000}$$

$$p(OH^-) = 3$$

$$p[H^+] = 14 - 3 = \underline{\underline{11}}$$

$$\boxed{x_{(mg/L)} = x_{(equ/L)} * \text{Eq. wt. of } x * 1000}$$

$$\text{Eq. wt. of } x = \frac{\text{Mol. wt of } x}{\text{Valency.}}$$

$$\boxed{x_{(mg/L)} = x_{(m.equi/L)} * \text{Eq. wt. of } x.}$$

Q.37. $[OH^-] = 10^{-5.6}$ m mol/L $= 10^{-8.6}$ mol/L

m mol/L = milli mol/L $= 10^{-3}$ mol/L

$pOH = 8.6$

$\therefore pH = 5.4$

P.23

Q.42. $pH = 9.25$; $pOH = 4.75$.

$[OH^-] = 10^{-4.75}$ mol/L

$= 10^{-4.75} \times 1000 \times 17 = 0.302$ mg/L

P-20

Q-15. $(pH)_I = 7.2$ $(pH)_O = 8.4$.

$(pH)_{avg} = -\log_{10}[H^+]_{avg} = 7.47$

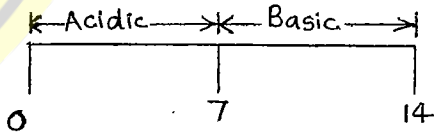
$[H^+]_I = 10^{-7.2}$ mol/L

$[H^+]_O = 10^{-8.4}$ mol/L

$[H^+]_{avg} = \frac{10^{-7.2} + 10^{-8.4}}{2} = 3.35 \times 10^{-8}$ mol/L

Q.16. $[pH]_A = 4.4$ $[pH]_B = 6.4$.

Sample A more acidic than Sample B.



$\frac{[H^+]_A}{[H^+]_B} = \frac{10^{-4.4}}{10^{-6.4}} = 10^2$

$\Rightarrow [H^+]_A = 100[H^+]_B$.

Q.17. A: Vol is 300 mL, $pH = 7$, $[H^+] = 10^{-7}$.

B: Vol is 700 mL, $pH = 5$, $[H^+] = 10^{-5}$.

$[H^+]_{(mixture)} = \frac{10^{-7} \times 300 + 10^{-5} \times 700}{1000} = 7.03 \times 10^{-6}$.

$pH_{(mixture)} = -\log[7.03 \times 10^{-6}] = 5.15$

$$C_{mix} = \frac{V_A C_A + V_B C_B + V_C C_C + \cdots}{V_A + V_B + V_C + \cdots}$$

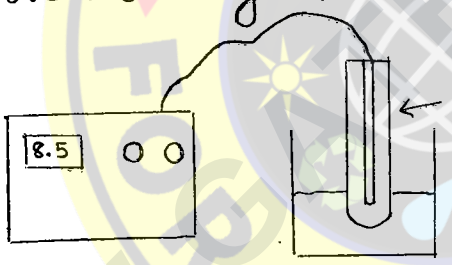
* Chemicals required to treat water depends on pH.
* Extreme high pH and low pH affects the property.
  Eg:- acids cause corrosion
  alkali causes scaling.
* Water treatment is pH sensitive.

Measurement of pH:-

(i) Colorometry (pH paper)
Field determination of pH.

(ii) Titrometry

(iii) Potentiometry (pH meter)



Drinking water standards wrt pH = 6.5 – 8.

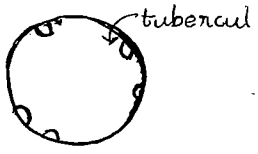
3. Acidity

Acidity is the ability of water to neutralise the base

(i) Mineral Acidity. (strong acid)
caused by minerals; pH in the range 0 to 4.2

(ii) Carbonic acidity (weak acid)
caused by CO_2.; pH in the range 4.2 to 8.3.

Acidity causes :



(i) Corrosion.
(ii) Tuberculation – projections on interior of pipes which results in energy loss.

* Water used in construction should have acidity less than or equal to 50 mg/L as $CaCO_3$

Acidity $\leq$ 50 mg/L as $CaCO_3$

x interms of mg/L of y	$= x_{(mg/L)} * \frac{Eq.wt \text{ of } y}{Eq.wt \text{ of } x}$

* Acidity is estimated by Titrometry

Titrant = NaOH.

Indicators : 1. Methyl Orange (Pink – yellow)
2. Phenolphthalein (Yellow – pink)

Q. 20 mL of 0.02 N NaOH soln is consumed in titrating 50 mL water sample. Find acidity in water sample as mg/L of $CaCO_3$.

$$\text{Concentration of 'x'}_{(mg/L)} = \frac{V * N * Eq.wt. \text{ of } x * 1000}{mL \text{ of water sample tested}}$$

Conc. of 'x' in terms of 'y' $= \frac{V * N * Eq.wt. \text{ of } y * 1000}{mL \text{ of water sample tested.}}$

$$\text{Acidity} = \frac{20 \times 0.02 \times \left(\frac{40+12+3\times16}{2}\right) \times 1000}{50} = \underline{400} \text{ mg/L}$$

4. Alkalinity (TA < 200 mg/L as $CaCO_3$).

Ability of water to neutralise ~~base~~ acids is called alkalinity.

* Alkalinity cause bitter taste to water.
* High alkalinity causes scaling (reduces carrying capacity
* Incrustation.

* Alkalinity in water is caused by:

(i) OH^- (pH: 14-10.2) (ii) CO_3^{2-} (pH: 8.3-10.2) (iii) HCO_3^- (pH: 4.2 to 8.3)

Both CO_3^{2-} & HCO_3^- cause alkalinity.
Alkalinity caused by OH^- negligible

* Total alkalinity

$$\underset{(mg/L\ of\ CaCO_3)}{TA} = \underset{(mg/L)}{OH^-} * \frac{Eq.wt.\ of\ CaCO_3}{Eq.wt.\ of\ OH^-}$$

$$\underset{(mg/L\ of\ CaCO_3)}{TA} = \underset{(mg/L)}{CO_3^{2-}} * \frac{Eq.wt\ of\ CaCO_3}{Eq.wt\ of\ CO_3^{2-}} + \underset{(mg/L)}{HCO_3^-} * \frac{Eq.wt\ of\ CaCO_3}{Eq.wt.\ of\ HCO_3^-}$$

$$TA = \underset{(mg/L)}{OH^-} * \frac{50}{17}$$

$$TA = CO_3^{2-} * \frac{50}{30} + HCO_3^- * \frac{50}{61}$$

19th JULY, SATURDAY

P-20
18 $TA = CO_3^{2-} * \frac{50}{30} + HCO_3^- * \frac{50}{61}$

$= 90 * \frac{50}{30} + 61 \times \frac{50}{61} = \underline{\underline{200\ mg/L\ as\ CaCO_3}}$

P-21
21. $pH = 9$

$\therefore pOH = 14-9 = 5$

$[OH^-] = 10^{-5} \times 17 \times 1000 = 17 \times 10^{-2}\ mg/L$

$TA = 17 \times 10^{-2} \times \frac{50}{17} = \underline{\underline{0.5}}\ mg/L$ as $CaCO_3$ (this can be neglected as pH < 10.2)

P-22
33. $CO_3^{2-} = 5 \times 30 = 150\ mg/L$ [$x_{(mg/L)} = x_{m.equi/L} * Eq.wt.\ of\ x^-$]

$HCO_3^- = 30 \times 61 = 1830\ mg/L$.

$TA = 5 \times 30 \times \frac{50}{30} + 30 \times 61 \times \frac{50}{61} = 1500 + 250 = \underline{1750\ mg/L}$ as $CaCO_3$

pH = 8.5, pOH = 5.5

$OH^- = \frac{-5.5}{10} \times 17 \times 1000 = 0.0537$

$TA = 0.0537 \times \frac{50}{17} = 0.158$ mg/L as $CaCO_3$ (neglected).

* Alkalinity is estimated by Titrometry.

Titrant used – H_2SO_4

Indicators used: 1. Phenolphthalein (Pink – Colourless)
2. Methyl Orange (Yellow – Pink)

First end point is called P-Alkalinity
Second end point is called Methyl Orange Alkanity. It's also called as Total Alkalinity (M-Alkalinity or T-alkalinity)

Relation b/w P & T	OH^-	CO_3^{2-}	HCO_3^-	
$P = 0$	0	0	T	(Sum will be T)
$P < \frac{1}{2}T$	0	2P	T−2P	
$P = \frac{1}{2}T$	0	2P	0	($2P = 2 \times \frac{1}{2}T =$
$P > \frac{1}{2}T$	2P−T	2T−2P	0	
$P = T$	T	0	0	

Q. If TA of water 300 mg/L as $CaCO_3$ and P-Alkalinity is 200 mg/L as $CaCO_3$, then find OH^-, CO_3^{2-}, HCO_3^- in mg/L

P = 200 mg/L, T = 300 mg/L

$\Rightarrow P > \frac{1}{2}T$.

$OH^- = 2P - T = 400 - 300 = 100$ mg/L as $CaCO_3$

$CO_3^{2-} = 2T - 2P = 600 - 400 = 200$ mg/L as $CaCO_3$

$HCO_3^- = 0$ mg/L

* Drinking water standard wrt Alkalinity:-

Total alkalinity $\leq$ 200 mg/L as $CaCO_3$.

5. Hardness (TH < 300 mg/L)

Hardness is caused by the reaction of divalent metallic cations present in water with anions.

Eg:- Ca^{2+}, Mg^{2+}, Sr^{2+}, Fe^{2+}, Mn^{2+}.

Hardness caused by Sr^{2+}, Fe^{2+}, Mn^{2+} are too small to consider.

CO_3^{2-}, HCO_3^-, Cl^-, SO_4^{2-} are the anions causing hardness

$$\begin{bmatrix} Ca^{2+} \\ Mg^{2+} \\ Sr^{2+} \\ Fe^{2+} \\ Mn^{2+} \end{bmatrix} + \begin{bmatrix} CO_3^{2-} \\ HCO_3^- \\ Cl^- \\ SO_4^{2-} \end{bmatrix} \longrightarrow \text{Hardness.}$$

cations

anions

* Total Hardness (TH) = Carbonaceous Hardness (CH) + Non carbonaceous Hardness (NCH)

$$\boxed{TH = CH + NCH}$$

$$TH \text{ in water} = Ca^{2+}_{(mg/L)} * \frac{\text{Eq. wt of } CaCO_3}{\text{Eq. wt of } Ca^{2+}} + Mg^{2+} * \frac{\text{Eq. wt. of } CaCO_3}{\text{Eq. wt of } Mg^{2+}}$$

$$\boxed{TH = Ca^{2+}_{(mg/L)} * \frac{50}{20} + Mg^{2+} * \frac{50}{12}}$$

if TH > TA	very important relation.	if TH $\leq$ TA
CH = TA		CH = TH
NCH = TH − CH		NCH = 0

34 $Ca^{2+} = 100$ mg/L $\quad CO_3^{2-} = 0$

$Mg^{2+} = 6$ mg/L $\quad HCO_3^- = 250$.

$$TH = 100 \times \frac{50}{20} + 6 \times \frac{50}{12} = \underline{\underline{275}} \text{ mg/L as } CaCO_3$$

$$TA = 0 + 250 \times \frac{50}{61} = 205 \text{ mg/L as } CaCO_3$$

$TH > TA \Rightarrow (CO_3^{2-} + HCO_3^-)$ hardness, $CH = TA$

$= 205$ mg/L as $CaCO_3$.

32. $Ca^{2+} = 12$ m.eq/L $= 12 \times 20 = 240$ mg/L

$Mg^{2+} = 18$ m.eq/L $= 18 \times 12 = 216$ mg/L.

$$TH = 240 \times \frac{50}{20} + 216 \times \frac{50}{12}$$

$$= \underline{\underline{1500}} \text{ mg/L as } CaCO_3$$

31 $Ca^{2+} = 55$ mg/L, $Mg^{2+} = 10$ mg/L.

Hardness $= 55 \times \frac{50}{20} + 10 \times \frac{50}{12.2} = \underline{\underline{179}}$ mg/L as $CaCO_3$

P-21

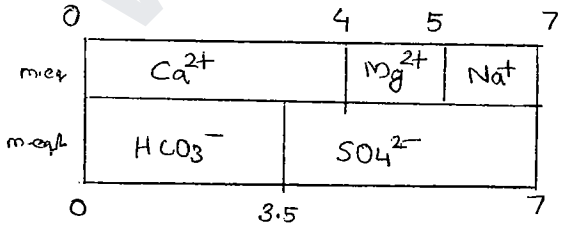
30 $TA = 250$, $TH = 350 \Rightarrow TH > TA$. Ans: (d)

Carbonate hardness $= TA = 250$ mg/L as $CaCO_3$

$NCH = TH - CH = 100$ mg/L as $CaCO_3$.

P-23

45.



$Ca^{2+} = 4 \times 20 = 80$ mg/L $\quad HCO_3^- = 3.5 \times 61 = 213.5$ mg/L.

$Mg^{2+} = 1 \times 12 = 12$ mg/L

$$TH = 4 \times 20 \times \frac{50}{20} + 1 \times 12 \times \frac{50}{12} = 250 \text{ mg/L}.$$

$$TA = 3.5 \times 61 \times \frac{50}{61} = 175 \text{ mg/L}. \Rightarrow TH > TA$$

CH = TA = 175 mg/L

NCH = 250 − 175 = 75 mg/L

$$TH = Ca^{2+}_{(m.eq/L)} * 50 + Mg^{2+}_{(m.eq/L)} * 50.$$

P-19

1. $TH = 160 \times \frac{50}{20} + 40 \times \frac{50}{12} = 567$ mg/L as $CaCO_3$

12. TH = 200 mg/L, TA = 250 mg/L. ⇒ TH < TA

CH = TH. = 200 mg/L

13. NCH = 0.

19. $TH = 65 \times \frac{50}{20} + 51 \times \frac{50}{12} = 375$ mg/L

$TA = 248 \times \frac{50}{61} = 203$ mg/L

TH > TA

CH = TA = 203 mg/L

NCH = 172 mg/L

* Hardness in water estimated by 'Versanate method' (EDTA method).

Titrant : EDTA (Ethylene Diamene Tetra Acetic acid).

Indicator : Erichrome Black Tea (EBT).

Colour change : Wine red to Blue.

NOTE: When titrating with EDTA, colour changes to blue on addition of EBT, there is no hardness in water.

* Hardness in water causes :

a) Bitter taste
b) Corrosion
c) Scaling
d) Increased soap consumption.

Hardness (mg/L of $CaCO_3$).	Nature of water.
< 75	Soft.
75–150	moderately hard. { TH upto 200 mg/L remains unnoticed
150–300	Hard.
>300	Very hard.

* Drinking water standards, TH < 300 mg/L

6. Chlorides (≤ 250 mg/L).

* If chlorides are present in water along with sodium, it cause kidney and cardiac problems

* Seawater intrusion into groundwater is verified with chlorides estimation.

* Chlorides in water estimated by Mohr's Method.

Titrant : silver nitrate.
Indicator: Potassium chromate.
Colour : yellow to brick red.

* Drinking water standard ≤ 250 mg/L

7. Sulphates (< 250 mg/L)

* "Lascative" problems (loose motion → diarrhea)

* Sulphates estimated by 'Turbidimetry' using Barium.

* Drinking water standards < 250 mg/L

8. Dissolved Oxygen (3 – 4 mg/L)

* dissolved oxygen keeps water fresh for long period

* High DO conc. cause corrosion.

* DO of 2 to 4 mg/l should be maintained in water

* It improves taste of water

* DO in water estimated by 'Winkler's Method'

Reagents : 1. Manganous Sulphate } Brown ppt (DO present)
+
Alkali Iodide Azide } White ppt (DO absent)

Titrant : Sodium Thiosulphate.

Indicator : starch

H_2SO_4 + brown ppt = Yellow colour.

If V is the volume of titrant consumed,

Amount of DO = $\frac{V * 0.025 * 8 * 1000}{200}$ = V mL

$$DO \propto \frac{1}{temp}$$

9. Fluorides (1 to 1.5 mg/L)

* Fluorides are only found in groundwater

* Fluorides < 1 mg/L → Dental caries (dental cavities)

Fluorides + Enamel → strong teeth
(in water) (in teeth)

Fluorides > 1.5 mg/L → (i) mottling of teeth (teeth stain yellow patch)

(ii) Fluorosis (for very high Fl conc of 2,2.5 mg/L)

a) Dental fluorosis

b) Skeletal fluorosis (Bone fluorosis)

* Drinking water standard = 1 to 1.5 mg/L

* Fluorides in water estimated by 'Colourometry' using Zirconium.

10. Nitrogen Compounds

* Indicate organic contamination of water. ∴ no amou of nitrogen compounds are allowed in drinking water.

Kjedahal Nitrogen
- (i) Ammonia nitrogen — indicates recent pollution.
- (ii) Organic nitrogen (Albuminoid nitrogen) — indicates old pollution (vegetative and animal waste).

(iii) Nitrites — intermediate stage of pollution.

(iv) Nitrates — oxidized organic matter.

* Nitrogen compounds are estimated by Colourometry

* Drinking water standards :-

Ammonia nitrogen	< 0.15 mg/L
Organic nitrogen	< 0.3 mg/L
Nitrites	= nil.
Nitrates	< 45 mg/L.

If nitrates > 45 mg/L, it leads to 'Blue Baby Disease' known scientifically as 'Methemoglobinoemea'. Haemoglobin in blood has more affinity towards 'nitrates' than oxygen. So nitrates in water are carried to different body parts instead of oxygen. When this happens, body turns blue for infants as a first symptom.

11. Iron & Manganese. (Fe < 0.3 mg/L ; Mn < 0.05 mg/L).

* Iron — reddish colour to water.
* Manganese — purple to brown colour

} when present in high conc.

* Estimated by 'Colorometry'

* Drinking water standards :

Iron < 0.3 mg/L

Manganese < 0.05 mg/L

12. Toxic Chemicals (Heavy Metals)

(i) Damage central nervous system.
(ii) Damage blood cells.
(iii) Damage blood vessels
(iv) cancerogenic (carcinogenic)

Toxic Chemical Substances	Drinking water Standards.
Lead	< 0.1 mg/L
Arsenic	< 0.05 mg/L
Barium	< 1 mg/L
Boron	< 0.01 mg/L
Cadmium	< 0.01 mg/L
Chromium	< 0.05 mg/L
Phenols	< 0.001 mg/L
Mercury	< 0.001 mg/L.
Cyanide.	< 0.05 mg/L

P-21
Q.22 NaCl (mg/L) $= 2\times10^{-3}$ mol/L * molecular wt.

$= 2\times10^{-3}(23+35.5)\times1000 = 117$ mg/L

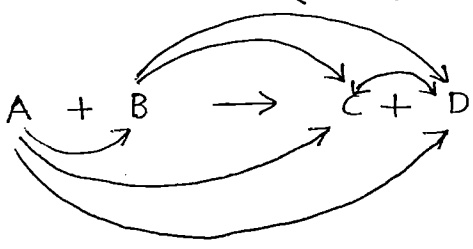
NaCl in mg/L as $CaCO_3 = 117 \times \frac{\text{Eq. wt of } CaCO_3}{\text{Eq. wt of NaCl}}$

$= 117 * \frac{50}{\left(\frac{23+35.5}{1}\right)} = 100$ mg/L as $CaCO_3$

P-23
Q.40 $CaCl_2$ as mg/L of $CaCO_3 = 110 \times \frac{50}{\left(\frac{40+71}{2}\right)} = 100$ mg/L

$Ca + CO_3 \rightarrow$

Q.41.



$$Ca + CO_3 \longrightarrow CaCO_3$$

Mol. wt. of Ca = 40

Mol. wt of CO_3 = 12 + 3×16 = 60

60 parts of CO_3 react with 40 part of Ca.

1 part CO_3 $\longrightarrow$ $\frac{40}{60}$ part Ca.

90 g CO_3 $\longrightarrow$ $\frac{40}{60} \times 90 = \underline{\underline{60 \text{ gm}}}$ Ca.

29th JULY, TUESDAY

Biological Water Quality. (Microbial Examination of Water)

(MPN ≤ 1 no./100 mL)

Testing Procedures:

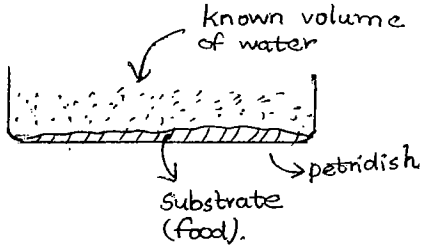
(i) Qualitative Tests → tests whether organisms are present or not.

(ii) Quantitative Tests → quantify the amount of organisms present.

(i) Qualitative Tests –

a) Plate Count Method.

Substrate is provided to trigger growth in petridish. Petridish is kept in incubator at 37°c to maintain warm conditions for growth. Min. time period of 24 hours is provided for growth.



b) Membrane filter Technique.

Filter papers which can trap even viruses is precoated with food. After passing known volume of water sample (sucked), the membranes (filter papers) are placed in incubator @ 37°c for a minimum duration of 24 hours. Presence of purple/black dots ensures the presence of micro-organisms.

(ii) Quantitative Tests.

a) E-coli Test (Escheiricha coli) ← German Biologist

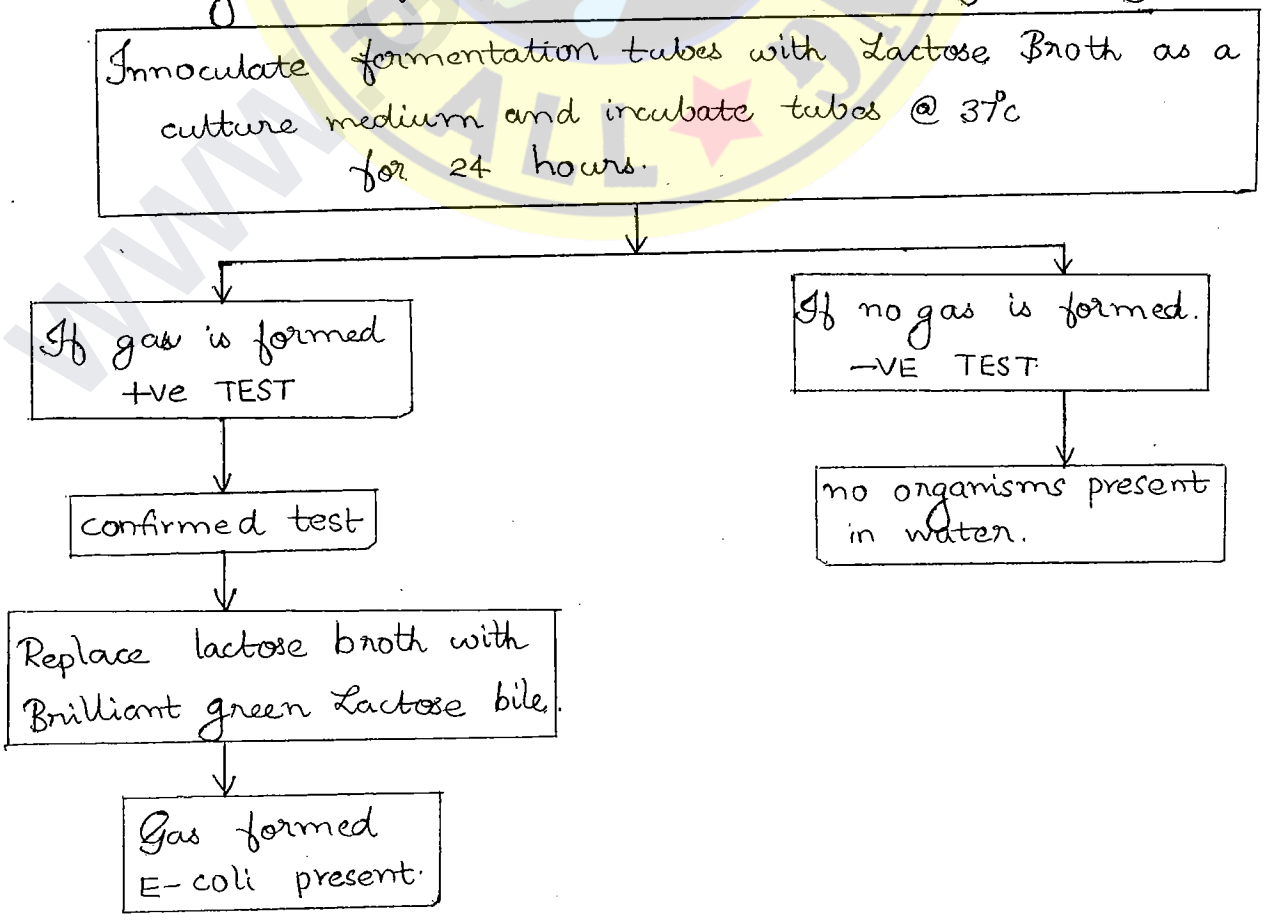
E-coli organism belongs to Coliform group of Bacteria. It is non pathogen. E-coli organism is presents in intestines of all warm blooded animals including humans. They are responsible for converting food into energy and waste products. But E-coli organism is treated as indicator organism. Presence of E-coli indicates presence of pathogens.

E-coli test is conducted in 3 stages :

E-Coli test is also known as Multiple Tube Fermentation Tes

1. Presumptive Test.
2. Confirmed Test.
3. Completed Test

E-coli organisms ferment lactose and in doing so they release ga



E-coli test results are finally expressed interms of :

1. E-coli Index
2. MPN (Most Probable Number).

1. E-coli Index.

Its defined as the reciprocal of the smallest qty of sample that would give +ve results.

Eg:- ① 100 mL | 10 mL | 1 mL | 0.1 mL | 0.01 mL | 0.001 ml
+ | + | + | + | − | −

$$\text{E-coli Index} = \frac{1}{0.1} = 10 \text{ no.s}/100 \text{ mL}$$

② 100 mL | 10 mL | 1 mL | 0.1 mL | 0.01 mL | 0.001 mL
+ | + | + | − | − | −

$$\text{E-coli Index} = \frac{1}{1} = 1 \text{ no.}/100 \text{ mL}$$

2. MPN. – Most Probable Number.

Laws of Statistical Probability are applied for E-coli test results and results are expressed in terms of MPN. M.P.N represents the bacterial density which is most likely to be present in the given water sample for the given test results.

MPN Tables

	10 — 1 — 0.1
5-5-5	25
5-5-4	24
⋮	⋮
3-2-1	10.

	+ve	-ve
100 mL – 5 tubes	4	1
10 mL – 5 tubes	3	2
1 mL – 5 tubes	2	3
0.1 mL – 5 tubes	1	4
0.001 mL – 5 tubes	0	5

P-21

20. 1 — 0.1 — 0.01 (test dilution)

4 — 3 — 1.

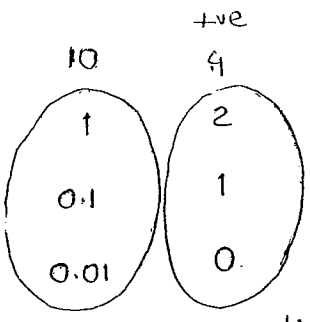
10 — 1 — 0.1 (table dilution

4 — 3 — 1 ⇒ MPN = 33.

∴ For test dilution,

MPN = 33 × 10 = 330 no./100 mL

P-23
Q.39



⇒ critical sample.

∴ 1 — 0.1 — 0.01 (test dilution.

For table dilution of 10 — 1 — 0.1

MPN = 7. (for 2 — 1 — 0 +ve).

∴ For test dilution,

MPN = 7 X 10 = 70 no./100mL

* Drinking water standard wrt Microbial water quality:

E-coli Index = 0 $\left(\frac{1}{100} = 0.01\right)$

MPN = 1 no./100 mL

MPN ≯ 1 no./100 mL

6th Aug, WEDNESDAY

Water Borne Diseases:

Name of the disease	Organism causing the disease	Category
1. Cholera	Vibrio cholerae or Vibrio comma.	Bacteria.
2. Typhoid.	Salmonella typhi	Bacteria.
3. Paratyphoid.	Salmonella paratyphi	Bacteria.
4. Dysentry	Shigella dysentena.	Bacteria.
5. Amebiasis } intestine	Entamoeba & Hystolytica.	Protozoa.
6. Giadiasys } intestine	Giardia Lamblia.	Protozoa.
7. Jaundice (liver)	Infectious Hepatitis	Virus
8. Polio.	Polio myelitis	Virus.

Entozal Diseases – diseases caused due to physical contact with wrong water.

Eg:- Yellow fever, Hook worm, Guniea. worm, tape worm infecti
Hay fever etc.

People who work in sewage, paddy fields, drawing water from open wells are exposed to this disease.

6th Aug,
WEDNESDAY

TREATMENT OF WATER

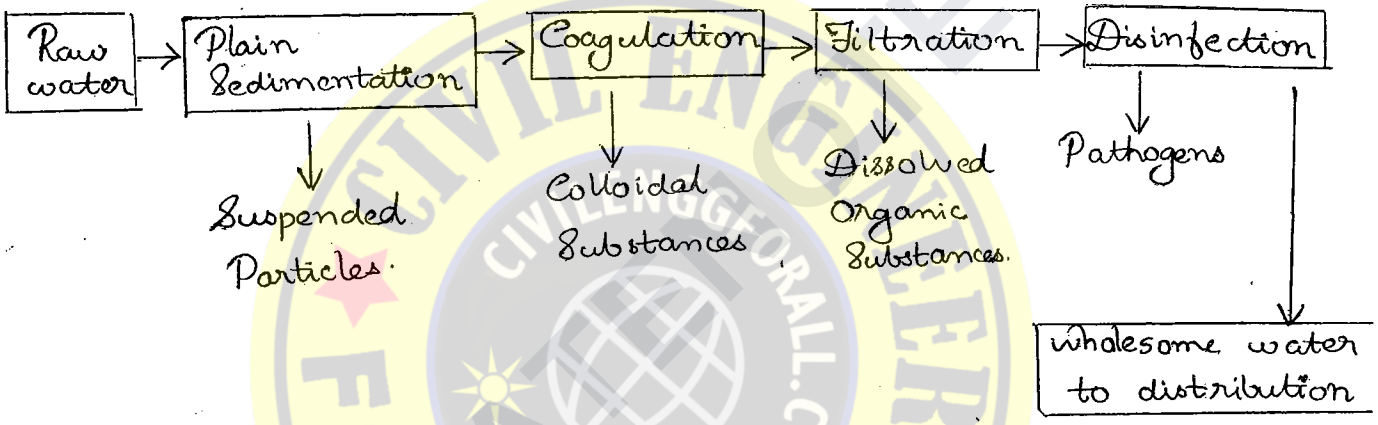
Water Treatment.

It is essentially a removal process. Two types :-

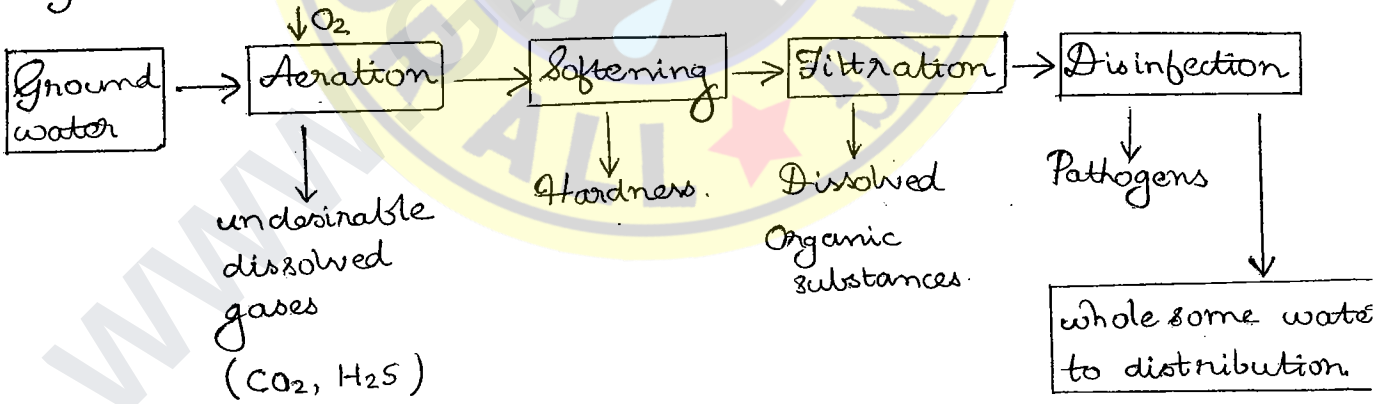
(i) Unit Operation — contaminant removal by physical force
   → Eg :- gravity, floatation.

(ii) Unit Processes — contaminant removal by chemical proces
   → Eg :- coagulation, chlorination

Surface water Treatment:



Ground water Treatment:



PLAIN SEDIMENTATION

Plain Sedimentation is the first unit operation employed to capture suspended particles from water.

Suspended particles in water remain in suspension due to turbelence and flow velocity. All the particles whose sp. gravity is more than that of water have a tendency to settle down. But turbulence and flow velocity offer resistance.

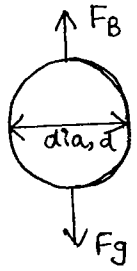
Plain sedimentation is the method of removing suspended particles present in water by controlling turbulence and flow velocity, thereby giving rise to particle settlement by virtue of their own mass (forces).

Plain sedimentation follows TYPE 1 settling.

Type-1 settling deals with discrete particle settlement in dilute suspension.

Discrete particle is the one which do not alter its size, shape, specific gravity etc with respect to time and space.

Settling Velocity of Discrete Particles: "Vs"
(Suspended Particles)



$$F_g = \gamma_p V_p = \rho_p g V_p \ (\downarrow)$$

$$F_B = \gamma_w V_p = \rho_w g V_p \ (\uparrow)$$

$$\rho_p > \rho_w$$

$$\Rightarrow F_g > F_b \ (\downarrow)$$

if $F_g < F_b$ then $\uparrow$

$$F_{net} = F_g - F_b \quad (\downarrow)$$

$$= g(\rho_p - \rho_w) V_p \quad (\downarrow)$$

Once motion of particle initiate in water, a third force act on the particle known as Drag Force, F_D. $\left(F_D = \frac{C_D A_p \rho_w V_s^2}{2}\right)$

$$F_{net} \geq F_D$$

$$g(\rho_p - \rho_w) V_p = \frac{C_D A_p \rho_w V_s^2}{2}$$

$$V_s^2 = \frac{2g(\rho_p - \rho_w) V_p}{C_D \rho_w \times A_p}$$; $\frac{V_p}{A_p} = \frac{\pi d^3/6}{\pi d^2/4} = \frac{2}{3} d$

$$\boxed{V_s^2 = \frac{\frac{4}{3} g(\rho_p - \rho_w) d}{C_D \rho_w}}$$ → Stoke's law for settling velocity.

$C_D = \frac{24}{Re}$ → for laminar flow conditions

$= \frac{24}{Re} + \frac{3}{\sqrt{Re}} + 0.34$ → for transitional flow.

$= 0.4$ → for turbulent flow

$$Re = \frac{\rho_w V d}{\mu} = \frac{V d}{\gamma}$$

$$\gamma = \frac{\mu}{\rho_w}$$

For open channel flow,

$Re < 500$

$500 < Re < 1500$

$Re > 1500$

For pipe flow,

$Re < 2000$

$2000 < Re < 4000$

$Re > 4000$.

For a spherical particle in medium conditions,

$Re < 1$

$1 < Re < 10^4$

$Re > 10^4$

For laminar flow condition,

$$C_D = \frac{24}{Re} = \frac{24}{\rho_w V_s d/\mu} = \frac{24\mu}{\rho_w V_s d}$$

$$V_s^2 = \frac{\frac{4}{3} g (\rho_p - \rho_w) d}{\frac{24\mu}{\rho_w V_s \cdot d} \times \rho_w}$$

$$\therefore \boxed{V_s = \frac{g(\rho_p - \rho_w) d^2}{18\mu}}$$

$$V_s = \frac{g \rho_w \left(\frac{\rho_p}{\rho_w} - 1\right) d^2}{18\mu} = \frac{g(s-1) d^2}{18 \mu/\rho_w}$$

$$\boxed{V_s = \frac{g(s-1) d^2}{18 \nu}}$$

Hazen's Equation for settling velocity, V_s

$$\boxed{V_s = 418 (s-1) d^2 \left(\frac{3T+70}{100}\right)}$$

$V_s \longrightarrow$ mm/s

$d \longrightarrow$ mm

$T \longrightarrow {}^\circ C$

These 3 equations are used only when $d < 0.1$ mm

For particles whose diameter, $d > 0.1$ mm, it is very difficult to maintain laminar flow conditions.

$$\boxed{V_s = 418 (s-1) d \left(\frac{3T+70}{100}\right)} \qquad 0.1 < d < 1 \text{ mm}.$$

$$\boxed{V_s = 1.8 \sqrt{g(s-1) d}} \qquad d > 1 \text{mm}$$

$$V_s^2 = \frac{\frac{4}{3} g (\rho_p - \rho_w) d}{C_d \times \rho_w}$$

For turbulent flow, $C_d = 0.4$,

$$\Rightarrow V_s = 1.8\sqrt{g(s-1)d}$$

Q. Find settling velocity of 0.02 mm dia particle with $S = 2.65$ settling from water, coeffecient of dynamic viscosity of water (μ) $= 10^{-3}$ Pa/s.

$$V_s = \frac{g(\rho_p - \rho_w)d}{18\mu} = \frac{9.81\,(2.65-1) \times 0.02^2 \times 10^{-6}}{18 \times 10^{-3}/1000}$$

$$= 3.597 \times 10^{-4} \text{ m/s}$$

$$= 0.359 \text{ mm/s}$$

1 Pa.s = 1 kg/msec

$= 1 \frac{Ns}{m^2}$

= 10 poise

Q. Find settling velocity of 0.018mm size discrete particle with $G = 2.7$, settling in water whose coeffecient of kinematic viscosity, $\gamma = 10^{-2}$ stokes.

1 stoke = 1 cm^2/s

$$V_s = \frac{9.81\,(2.7-1) \times 0.018^2 \times 10^{-6}}{18 \times 10^{-4} \times 10^{-2}}$$

$$= 0.3 \text{ mm/s}$$

Q. Find settling velocity of particle of diameter 0.018 mm with $S = 2.7$ settling in water whose temp is 20° C.

$$V_s = 418\,(S-1)d^2 \left(\frac{3T+70}{100}\right) \qquad (d < 0.1 \text{ mm})$$

$$= 418 \times (2.7-1)\, 0.018^2 \left(\frac{3\times 20 + 70}{100}\right)$$

$$= 0.299 \text{ mm/s}$$

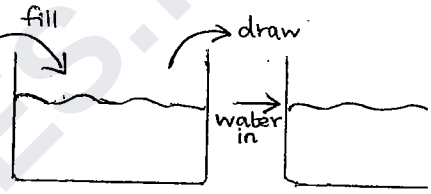
8. $$V_s = \frac{9.81 \times (2.65-1) \times 0.025^2 \times 10^{-6}}{18 \times 10^{-6}} = 5.62 \times 10^{-4} \text{ m/s}$$

$$= 0.0562 \text{ cm/s}$$

Sedimentation Tanks :

The place where settling operation is carried out is known as Sedimentation Tank or Settling Tank or Clarifier

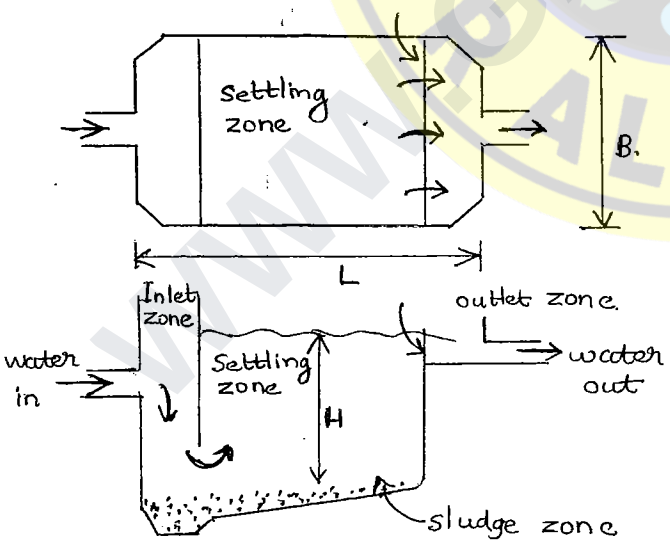
1. Draw and fill type Settling tanks.
2. Continuous flow Settling tanks.



* Continuous flow Settling Tank :-

1. Horizontal flow rectangular Settling Tanks.
2. Radial flow Circular Settling Tanks.

Rectangular Settling Tank.



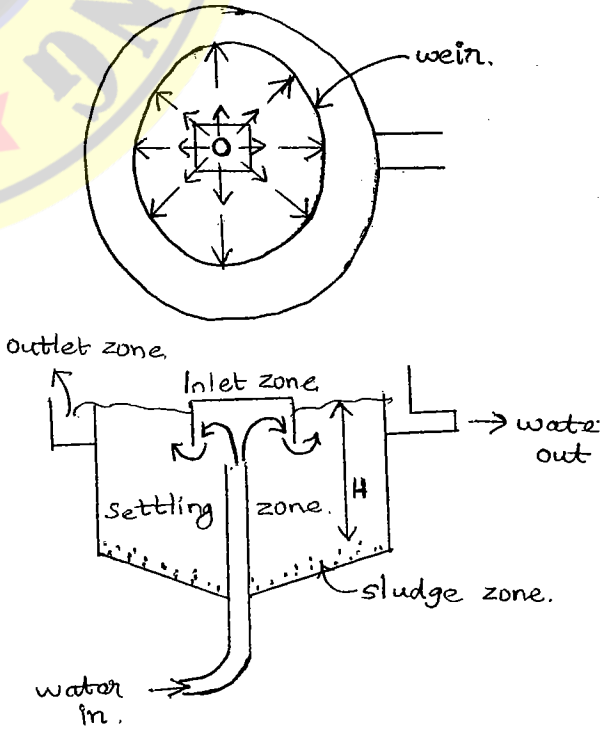
Inlet zone

Outlet zone.

Settling zone

Sludge zone.

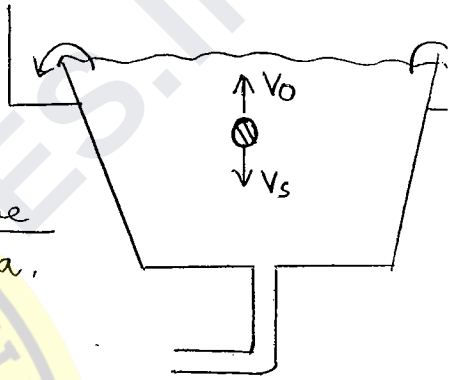
Circular Settling Tank.



Design Parameters:

1. Surface Loading rate (or) Surface Overflow rate (or) Hydraulic Loading rate (or) Overflow velocity "V_o"

It is the volume of water applied in unit time per unit surface area (plan area) of settling tank is known as surface loading rate.



$V_s > V_o$
$\Rightarrow$ particles captu

Surface loading rate (SLR)

Surface overflow rate (SOR), $V_o = \frac{\text{Volume / time}}{\text{Surface area}}$.

$$V_o = \frac{Q}{\text{Surface Area}}$$

$$\text{Surface area} = \frac{Q}{V_o}$$

Particles are captured from water, if $V_s > V_o$

If $V_s > V_o \longrightarrow$ particles captured.

If $V_s < V_o \longrightarrow$ particles escape.

17th Aug, TUESDAY

- SLR or SOR, V_o = 500 to 750 lit/hr/m^2
  = 12 to 18 m^3/day/m^2 (10–20 m^3/day/m^2)

2. Detention Time (Hydraulic Detention Time) or Retention Time.

It is the average theoretical time taken by water to trave from inlet to outlet of settling basin.

- DT = 4 to 8 hours

Volume of settling tank = Q * DT

3. Flow through Velocity (Horizontal flow velocity), V_H

Speed with which water travel from inlet to outlet of settling tank is known as 'flow through velocity' or 'velocity of flow'.

For rectangular settling,

$$V_H = \frac{Q}{\text{c/s area}} = \frac{Q}{B*H}$$

$$\text{c/s} = \frac{Q}{V_H}$$

V_H = 0.2 - 0.9 m/min

= 0.3 m/min.

Length of settling tank = V_H * DT

$$DT = \frac{L}{V_H}$$

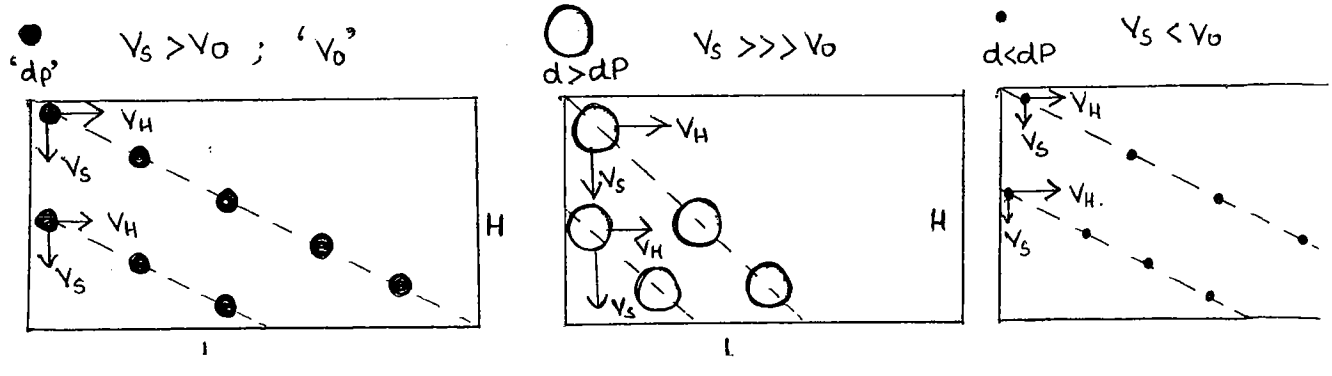
4. Weir Loading Rate., WLR

Amount of water overflown per unit length of tank.

$$WLR = \frac{Q}{\text{length of weir}}$$

$$= \frac{Q}{B} \text{ ; rectangular settling tank}$$

$$= \frac{Q}{\pi d} \text{ ; circular settling tank.}$$



↓ Efficiency of particle removal, $\eta \propto \frac{1}{V_o \uparrow}$

↓ Surface area $\propto \frac{1}{V_o \uparrow}$

$\Rightarrow \quad \eta \propto SA$

∴ Particle removal efficiency, $\eta = \frac{V_s}{V_o} * 100$

⊙ Particle removal efficiency depends only on Surface area of settling tank. It is independent of depth of tank.

If D ↓, then $V_H \uparrow$, $\left(c/s = \frac{Q}{V_H}\right)$

∴ particle escapes without hitting the bottom as V_H is increased

⊙ Overall efficiency removal of all particles, $\eta = \Sigma P_i \eta_i$; $P_i \rightarrow$ % of partic

$= \Sigma P_i \left(\frac{V_{si}}{V_o} \times 100\right)$

⊙ Overall η = 60 to 70%

* For 100% removal of particles from water in Settling tar (Effective removal)

1. $V_{si} > V_o$

2. $DT = \frac{L}{V_H}$; Particle settling time ≤ Detention time

$$\frac{H}{V_{si}} \leq \frac{L}{V_H}$$

P 27

Q.07 $Q = 10^5$ m³/day.

$V_s = 20$ m/day

($V_s = V_o$ for 100% removal)

Surface area $= \frac{Q}{V_o} = \frac{Q}{V_s} = \frac{10^5}{20} = \underline{\underline{5000}}$ m²

09.

a) $S = 2.65$, $\gamma = 0.897\ mm^2/s = 0.897 \times 10^{-6}\ m^2/s$, $d = 0.03\ mm = 0.03 \times 10^{-3}\ m$.

$$V_s = \frac{g(S-1)d^2}{18\gamma} = \frac{9.81(2.65-1)\times(0.03\times10^{-3})^2}{18\times0.897\times10^{-6}}$$

$$= 9.02 \times 10^{-4}\ m/s$$

$$V_o = 12000\ L/hr/m^2 = 12\ m/hr = 3.33\times10^{-3}\ m/s$$

$$\eta = \frac{V_{si}}{V_o} \times 100 = \frac{9.02\times10^{-4}}{3.33\times10^{-3}} \times 100 = 27.06\%$$

b) $d = 12\ mm$

$$V_s = 144.36\ d^{1.8}\sqrt{g(s-1)d} \qquad (d > 1\ mm)$$

$$= 1.8\sqrt{9.81\times1.65\times12\times10^{-3}} = 0.793\ m/s$$

$$\eta = \frac{V_{si}}{V_o} = \frac{0.783}{3.33\times10^{-3}} \times 100 > 100\% \simeq 100\%$$

13. $\gamma = 1.01\times10^{-6}$, $G = 2.3$, $d = 50\times10^{-6}\ m$.

$$V_s = \frac{9.81\times(2.3-1)\times50^2\times(10^{-6})^2}{18\times1.01\times10^{-6}} = 1.75\times10^{-3}\ m/s.$$

$$Q = 100\ MLD = 100\times10^6\ L/day.$$

$$= \frac{100\times10^3}{24\times60\times60} = 1.15\ m^3/s.$$

$$\text{Surface area} = L\times B = \frac{Q}{V_s} = \frac{1.15}{1.75\times10^{-3}} = 661.376.$$

$$\frac{L}{B} = \frac{3}{1} \quad (\text{Given})$$

$$3B^2 = 661.376$$

$$B = 14.84 \approx 15\ m.$$

$$L = 45\ m.$$

$$DT = \frac{\text{Volume}}{Q} = \frac{45\times15\times3}{1.15} = 1760.87\ s = 29.34\ min$$

$$\approx 30\ min$$

2. $d = 0.04$ mm, $S = 2.65$, $T = 20^0$ c

$$V_S = 418\,(S-1)\,d^2\left(\frac{3T+70}{100}\right)$$

$$= 418\,(2.65-1) \times 0.04^2 \left(\frac{3\times 20+70}{100}\right) = 1.65 \text{ mm/s}.$$

$$= 0.00143 \text{ m/s}.$$

$$Q = 10 \text{ MLD} = \frac{10\times10^3}{24\times60\times60} = 0.116 \text{ m}^3/\text{s}.$$

$$\text{Surface area} = L \times B = \frac{0.115}{0.00143} = 80.94 \text{ m}^2.$$

$$\frac{B}{L} = \frac{1}{3} \quad \text{(given)}$$

$$B = 5.19 \text{ m} \quad \& \quad L = 15.58 \text{ m}.$$

$$\text{Detention time} = \frac{15.58\times5.19\times3.5}{0.115} = 2440.13 \text{ s}$$

$$= 40.67 \text{ min} = 0.68 \text{ hour}$$

11th Aug,
FRIDAY

4. $Q = 10^5 \text{ m}^3/\text{day} = \frac{10^5}{86400} = 1.157 \text{ m}^3/\text{s}$ $\quad S = 2.65$

$L \times B \times H = 100 \times 50 \times 3$

$\gamma = 1.02\times10^{-2}\times10^{-4} \text{ m}^2/\text{s}$

$$\text{SOR} = \frac{Q}{S.A} = \frac{Q}{L\times B} = \frac{1.157}{100\times50} = 2.314\times10^{-4} \text{ m}^3/\text{s}/\text{m}^2$$

$$= 20 \text{ m}^3/\text{day}/\text{m}^2$$

$$V_S = V_O = 2.314\times10^{-4} \text{ m}^3/\text{s}/\text{m}^2$$

$$V_S = \frac{g\,(S-1)d^2}{18\,\gamma}$$

$$2.314\times10^{-4} = \frac{9.81\,(2.65-1)d^2}{18\times1.02\times10^{-4}}$$

$$d = 1.62\times10^{-4} \text{ m}$$

$$= 0.162 \text{ mm}$$

15. $V_S = \dfrac{9.81 \times 1.65 \times 0.01^2 \times 10^{-6}}{18 \times \frac{10^{-3}}{1000}} = 8.9925 \times 10^{-5}$ m/s

$V_O = \dfrac{8000/86400}{900} = 1.028 \times 10^{-4}$ $m^3/s/m^2$

Percentage efficiency $= \dfrac{V_S}{V_O} \times 100 = \dfrac{8.9925 \times 10^{-5}}{1.028 \times 10^{-4}} \times 100$

$= \underline{87.28\%}$

16. $L = 7.5$ m, $V_H = 0.3$ m/s, $H = 0.9$ m.

$V_O = \dfrac{Q}{L \times B}$

$V_H = \dfrac{Q}{BH} \Rightarrow Q = BH V_H$.

∴ Surface overflow rate, $V_O = \dfrac{BH V_H}{L \times B} = \dfrac{H V_H}{L}$.

$V_O = \dfrac{0.9 \times 0.3}{7.5} = 0.036$ m/s.

For 100% efficiency of removal of a particle of diameter d,

$V_S = V_O$.

$\dfrac{g(S-1)\, d^2}{18\, \nu} = V_O$.

$\dfrac{9.81\,(2.5-1)\, d^2}{18 \times 1.002 \times 10^{-3} / 1000} = 0.036$.

$\Rightarrow \quad d = 2.1 \times 10^{-4}$ m.

$= \underline{0.21 \text{ mm}}$

$L = 20m,\ B = 10m,\ H = 3m.$

$Q = 4\ MLD,$

$T = 20^{\circ}C,\ \mu = 1.002 \times 10^{-3}\ Pa.s.$

$\rho_w = 998.2\ kg/m^3\ ;\ S = 2.65$

$$SOR = \frac{Q}{SA} = \frac{4 \times 10^3\ m^3/86400}{20 \times 10} = 2.315 \times 10^{-4}\ m^3/s/m^2$$

$$= 20\ m^3/day/m^2.$$

$$V_s = \frac{g(S-1)d^2}{18\gamma}$$

$$2.315 \times 10^{-4} = \frac{9.81(2.65-1)d^2}{18 \times \frac{1.002 \times 10^{-3}}{998.2}}$$

$$d = 1.607 \times 10^{-5}\ m = 0.016\ mm = 16 \times 10^{-3}\ mm$$

20 $SOR = V_o = 43.2\ m^3/d/m^2 = 5 \times 10^{-4}\ m^3/s/m^2$

$= 0.5\ mm/s$

$\eta = \frac{V_{si}}{V_o} \times 100$	Particle	%	V_s (mm/s)
20	1	10	0.1
40	2	60	0.2
200 ≈ 400	3	30	1

Overall particle removal,

$$\eta_{overall} = \sum P_i \eta_i$$

$$= \frac{10}{100} \times 20 + \frac{60}{100} \times 40 + \frac{30}{100} \times 100$$

$$= 56\%$$

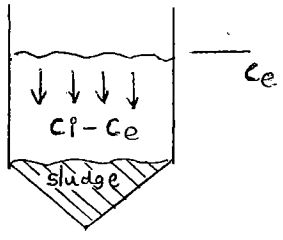
$SOR = 3\ m^3/m^2/hr$ *

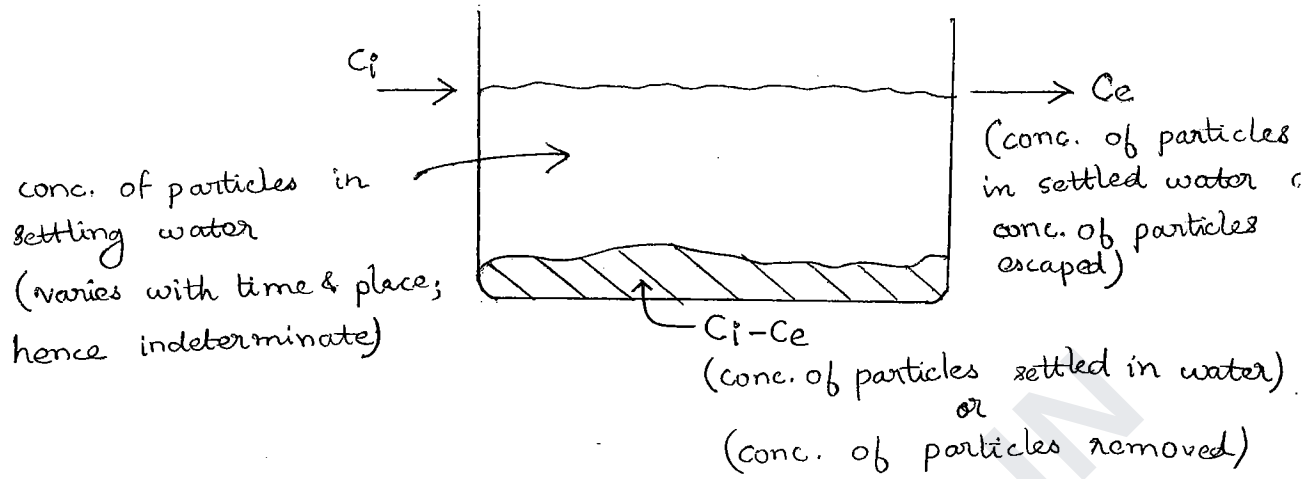
Particle type	Settling velocity (m/hr)	SOR (m/hr)	Concentration C_i	η_i	Conc. of particles removed	Conc.
I	3	3	200 mg/L	100%	200	0
II	1	3	300 mg/L	33.3%	100	20

Concentration of particle removed $= C_i \eta_i$

Conc. of particle in settled water

= conc. of particle escaped = 200 mg/L





22. Sedimentation laws of Newton & Stokes → Type I settling
Discrete settling in dilute suspension.

23. $SOR = 30\ m^3/day/m^2$, $G = 2.65$, $\rho_w = 1000\ kg/m^3$, $\mu = 0.001\ Ns/m^2$

$$V_o = 30\ m^3/day/m^2 = 3.47 \times 10^{-4}\ m^3/s/m^2$$

$$V_s = \frac{g(s-1)d^2}{18\nu}$$

$$3.47\times10^{-4} = \frac{9.81 \times 1.65 \times d^2}{18 \times \frac{0.001}{1000}} \Rightarrow d = 1.96\times10^{-5}\ m.$$

$$= 0.0196\ mm \approx \underline{0.02\ mm}$$

24. $d_1 = 0.1\ mm$, $Q = 0.01\ m^3/s$, $g = 9.81\ m/s^2$, $G = 2.65$, $\nu = 1.0105\times10^{-6}\ m^2/s$
$\underline{d_2 = 0.06\ mm}$ (100% removal)

$$V_s = \frac{9.81(2.65-1) \times 0.06^2 \times 10^{-6}}{18 \times 1.0105 \times 10^{-6}} = 3.2036 \times 10^{-3}\ m/s.$$

$$V_o = V_s = 3.2036\times10^{-3}\ m/s \quad (100\%\ removal)$$

$$V_o = \frac{Q}{SA} \Rightarrow SA = \frac{0.01}{3.203\times10^{-3}} = \underline{3.1214\ m^2}$$

04. $Q = 1.8 \times 10^6$ L/D DT = 4 hours.

$= \frac{1.8 \times 10^3}{24} = 75$ m³/h

Volume = Q. × DT.

$= 75 \times 4 = \underline{300\ m^3}$

05. SOR = 0.5 m³/hr/m²

$SA = L \cdot B = \frac{Q}{SOR} = \frac{75}{0.5} = 150\ m^2.$

$\frac{L}{B} = \frac{4}{1}$

$\Rightarrow \frac{L^2}{4} = 150$ or $L = \underline{24.5\ m}$

01. L = 15m, B = 6m, H = 3m. (consider only liquid depth)

Q = 2 MLD = 83.33 m³/hr.

$SOR = \frac{Q}{LB} = \frac{83.33}{15 \times 6} = 0.925\ m^3/hr/m^2$

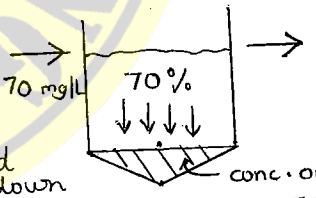
$= 926\ L/hr/m^2$

02. Detention time $= \frac{Volume}{Q} = \frac{15 \times 6 \times 3}{83.33} = \underline{3.24\ hours}$

03. Tank efficiency = 70%

Conc. of dry solids deposited = conc. of solids settled down

$= 70 \times \frac{70}{100} = 49$ mg/L



Conc. of dry solids deposited in 2 ML $= 49 \times 10^{-3} \times 2 \times 10^6$

$= \underline{98\ kg}$

10. Tank diameter = 26 m, h = 2.1 m.

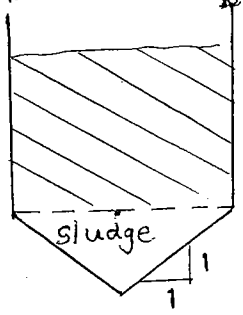
Q = 13000 m³/day.

$DT = \frac{Volume}{Q} = \frac{\pi r^2 h}{Q} = \frac{\pi \times 13^2 \times 2.1}{13000/24}$

$= \underline{2.06\ hours} \approx 2.41\ hours.$

For 1:1 slope hopper bott

Assuming entire tank used for settlir



$V = d^2(0.011d + 0.785H$

11. Weir loading $= \frac{Q}{\pi D} = \frac{13000}{\pi \times 26} = 159.155\ m^3/day/m.$

Total amount of any substance (kg/day) = Q (MLD) * conc. of substance (mg/L)

15th Aug,
FRIDAY

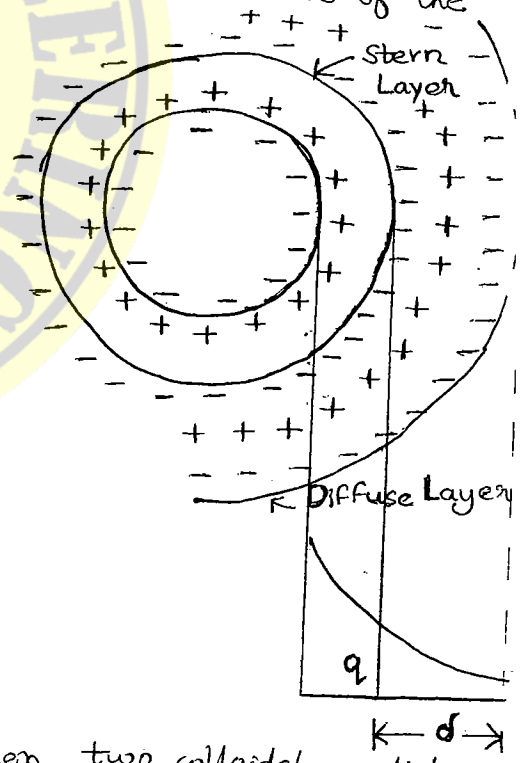
COAGULATION

(Sedimentation aided with Coagulatic

Sedimentation aided with coagulation is employed to capture suspended particles which escape plain sedimentation along with colloidal particles. Colloidals can't be captured by plain sedimentati ∴ to remove colloids accelerated sedimentation process is employed

1. Colloidal particles are charged particles. Generally they carry negative charges (Colloids acquire -ve charges while travelling in wat due to abrasion, wear & tear, Brownian motion etc).
2. Colloids carry charges on their surface.
3. Most of the colloids are stable in water because of the charges they carry.
4. Colloidal stability is measured in "zeta-potential" (ζ)

$$\zeta = \frac{4\pi q \delta}{D}$$

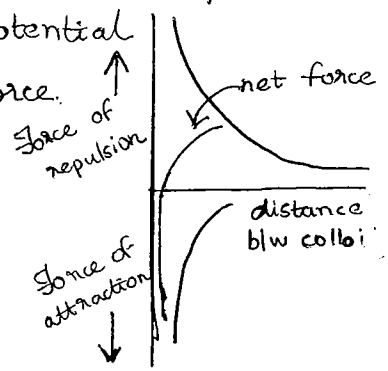


$q \rightarrow$ charge on shear surface.

$\delta \rightarrow$ thickness of diffuse layer

$D \rightarrow$ dielectric constant.

5. There are two forces acting between two colloidal particles
   a) Force of repulsion due to electrostatic potential
   b) Force of attraction due to VanderWaal's force.



$$\text{Force of attraction} \propto \frac{1}{\text{distance blw particles}}$$

26th Aug, TUESDAY

The chemical substances that are added to water to carry out coagulation is known as **coagulants**.

After adding the coagulants, the following mechanisms occur:

1. Compression of electronic double layer.
2. Adsorption and charge neutralization
3. Entrapment in precipitation (Sweep coagulation)

These three mechanisms are collectively called Colloidal stabilisation. This concept was named after two Russian and two Deutsch scientists who discovered this as **DLVO** Theory. (Derzagin, Landav, Verwey, Overbeak)

4. Interparticle bridging.

'coagulation' → 'flocculation' → sedimentation

'coagulation': Chemical conditioning of water (addition of chemicals to water).

'flocculation': Physical conditioning (growth of particles). } → Settling of flocculents

→ Common Coagulants.

Trivalent aqua metallic cationic substances thought to have very high coagulating powers. They are used as coagula

1. Aluminium Sulphate.
2. Ferrous Sulphate.
3. Ferric Sulphate and Ferric chloride.
4. Sodium Aluminate.

* Aluminium Sulphate (Alum)

- universal coagulant
- cheap, easy to handle and store
- produce effective floc.
- it also remove colour, odour and improve taste.

– requirements.

a) pH 6.5 to 8.5.

b) alkalinity in water.

– drawbacks.

a) imparts hardness (NCH).

b) water become corrosive.

$$Al_2(SO_4)_3.18H_2O + 3Ca(HCO_3)_2 \longrightarrow 3CaSO_4 + 2Al(OH)_3\downarrow + 6CO_2\uparrow + 18H_2O$$

(alkalinity) (NCH) (floc)

Molecular wt. of $Al_2(SO_4)_3 = 2\times27 + 3(32+4\times16) + 18(2+16)$

$= \underline{666}$

$$3Ca(HCO_3)_2 \longrightarrow 3CaCO_3 + 3CO_2 + 3H_2O$$

Molecular wt. of $3CaCO_3 = 3(40+12+3\times16)$.

$= \underline{3\times100}$

"666 parts of alum require 3×100 parts of alkalinity"

$$CaCO_3 \longrightarrow \underset{\text{(quick lime)}}{CaO} + CO_2$$

Mol. wt of $CaCO_3 = 100$.

Mol. wt of $CaO = 56$.

"100 parts of alkalinity require 56 parts of lime."

Q. 02. $Q = 10$ MLD.

Dose of alum = 20 mg/L

Alkalinity present in water = 6 mg/L

Mol. wt of alum = 666

Mol. wt of $3CaCO_3 = 3\times100$.

Mol. wt. of $CaO = 56$.

666 parts of alum combine with 3*100 parts of alkalinity as $CaCO_3$.

1 part alum : $\frac{300}{666}$ parts of alkalinity.

20 mg/L of alum : $\frac{3*100}{666} \times 20$ mg/L of alkalinity.

Alkalinity required in water = 9 mg/L as $CaCO_3$.

Total alkalinity required per day (kg/day) = Q * conc. of alkali (MLD) (mg/L). reqd

$= 10 \times 9 = 90$ kg/day

$= 90 \times 10^6$ mg/day

3. Deficiency of alkalinity in water = 9 − 6 = 3 mg/L as $CaCO_3$

100 parts of alkalinity as $CaCO_3$ require : 56 parts of lime as CaO.

1 part alkalinity = $\frac{56}{100}$ parts of lime

3 mg/L alkalinity as $CaCO_3$ = $\frac{56}{100} \times 3$ mg/L of lime as $CaCO_3$.

CaO dose (lime required) = $\frac{56}{100} \times 3 = 1.68$ mg/L

Total lime required = Q x dose of lime

$= 10 \times 1.68$ kg/day.

Annual requirement of lime as $CaCO_3$

$= 16.8 \times 365 = 6132$ kg/year

$= 6132 \times 10^6$ mg/yr.

4. $Q = 12$ MLD

Dose of alum = 14 mg/L = 14 ppm.

Total alum required per day = Q x dose

= 14 x 12 = 168 kg/day.

Mol. wt. of alum = 666

Mol. wt. of $6CO_2$ = $6(12 + 2 \times 16) = 6 \times 44$.

1 part of alum release = $\frac{6 \times 44}{666}$ parts of CO_2.

14 mg/L of alum release = $\frac{6 \times 44}{666} \times 14$ mg/L of CO_2

CO_2 released = 5.55 mg/L

* Ferrous Sulphate (copperas)

- effective in water if pH > 8.5
- copperas react with hydrated lime and produce floc.
- cheaper than alum, but does not react with coloured wate

$$CaO + H_2O \longrightarrow Ca(OH)_2$$

(quick lime) (hydrated lime)

$$FeSO_4.7H_2O + Ca(OH)_2 \longrightarrow CaSO_4 + Fe(OH)_2\downarrow + 7H_2O$$

(floc)

05. Total ferrous sulphate required = Q * dose of ferrous sulphate

= 12 x 10 = 120 kg/day.

Mol. wt. of $FeSO_4.7H_2O$ = 56 + 32 + 4 x 16 + 7(2+16)

= 278

$$Ca(OH)_2 \longrightarrow CaO + H_2O.$$

Mol. wt. of CaO = 56

278 parts of ferrous sulphate required = 56 parts of li as CaO.

1 part of ferrous sulphate = $\frac{56}{278}$ part of lime as Ca

10 mg/L of $FeSO_4$ = $\frac{56}{278} \times 10$ mg/L of CaO.

CaO dose required = $\frac{56}{278} \times 10 = 2015$ mg/L

Total lime required = Q * dose of lime

$= 12 \times 2.015 = 24.18$ kg/day.

* Chlorinated Copperas (Ferric Sulphate & Ferric Chloride

- Copperas + chlorine → chlorinated copperas.

$$6FeSO_4 \cdot 7H_2O + 3Cl_2 \longrightarrow \underset{\text{(ferric sulphate)}}{2Fe_2(SO_4)_3} + \underset{\text{(ferric chloride}}{2FeCl_3} + 42H_2O$$

- High pH (>8.5) and low pH (<6.5) can easily be coagulated by chlorinated copper.

$$Fe_2(SO_4)_3 + 3Ca(OH)_2 \longrightarrow 3CaSO_4 + \underset{\text{(floc)}}{2Fe(OH)_3}\downarrow$$

$$2FeCl_3 + 3Ca(OH)_2 \longrightarrow 3CaCl_2 + 2Fe(OH)_3 \downarrow$$

- Coloured water is not a constraint to use them.
- Slightly costlier than alum & copperas.

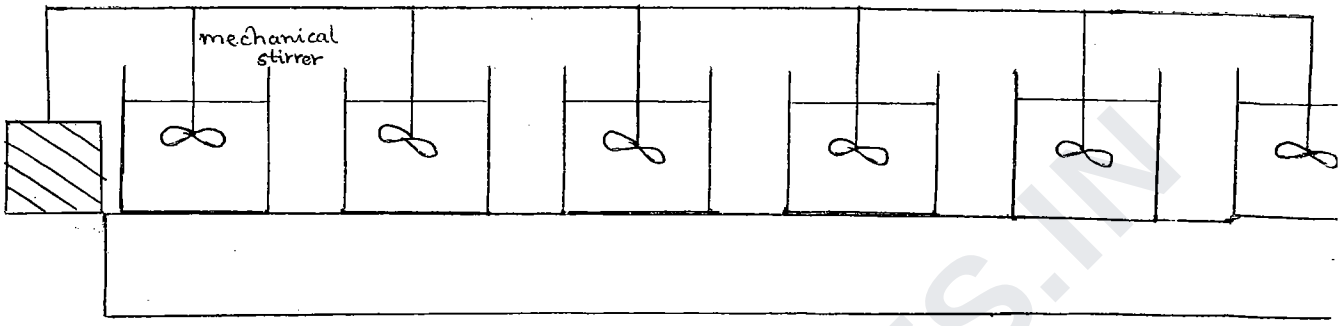
* Sodium Aluminate ($Na_2Al_2O_4$)

- independent of pH.
- very expensive of all coagulants.

$$Na_2Al_2O_4 + Ca(HCO_3)_2 \longrightarrow \underset{\text{(floc)}}{CaAl_2O_4} + Na_2CO_3 + CO_2 + H_2O$$

10th Sept,
WEDNESDAY → Jar Test

Jar Test is performed to find optimum dose of coagulant. (ie minimum coagulant dose and max removal



* Chemical conditioning (coagulation)

Addition of coagulant and mixing the contents

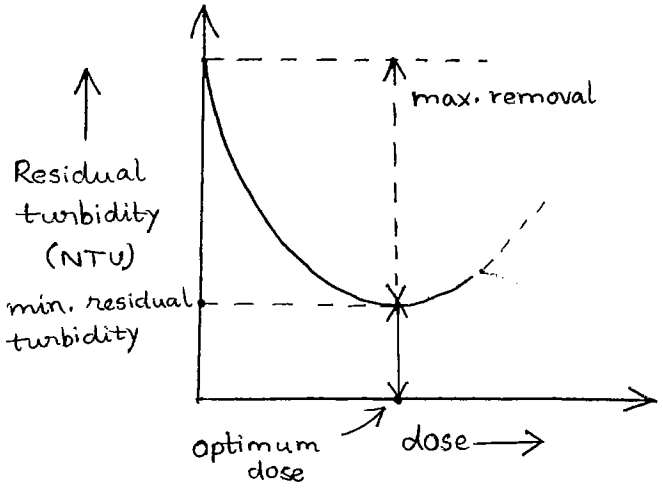
Rapid mixing – 1 to 3 min at 100 to 120 rpm (uniform bler

* Physical Conditioning (orthokinetic flocculation)

Perikinetic flocculation :– particles move and attract on their own.

Slow mixing – 20 to 40 min at 1 to 3 rpm

* Allow the floc so formed to settle down by keeping the contents of jar quite and quiscent, for a period of 30 min to 1 hour.



→ Design of Clariflocculator :

Clariflocculator consists of –

1. Rapid Mixing Unit (Flash mixer)
2. Flocculator (Slow mixing)
3. Sedimentation tank.

* Design of Rapid Mixing Unit :-

Mechanical mixing unit is designed to blend coagulant with water.

Design of rapid mixer is based on:-

a) Detention time.

b) Velocity gradient.

– Detention time : 1 to 3 min.

Volume of mixing basin = Q × DT

Depth : 1 m to 1.5

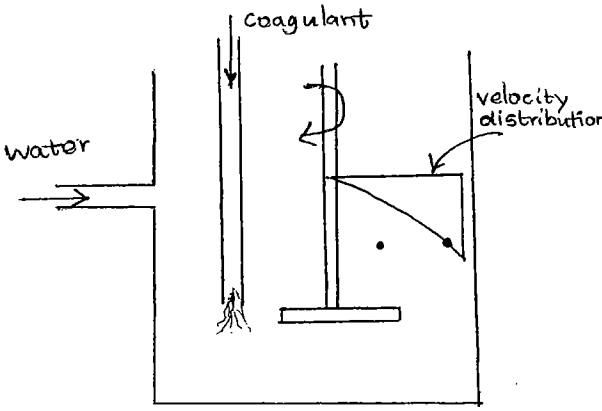
$$\text{Surface area} = \frac{\text{Volume of mixer}}{\text{Depth}}$$

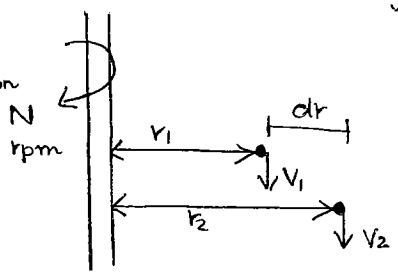
– Mixing units are circular or square in plan.

– Velocity gradient, 'G'

$$G = 700 \text{ to } 1000 \ s^{-1}$$

Degree of mixing ∝ degree of turbulent ∝ 'G'.





For one rotation, 2π rad

For N rotation, $\omega = 2\pi N$

$V = r\omega$

Slope of velocity distribution curve is G.

$$G = \sqrt{\frac{P}{V\mu}}$$

where P = power in watts.

V = vol. of mixing basin in m^3

μ = coefficient of dynamic viscosity.

$$P = F_D \cdot V.$$

$$= \frac{C_D \rho_w A_p V^2 \cdot V}{2} \Rightarrow P = \frac{C_D \rho_w A_p V^3}{2}$$

~~01.~~ ~~$Q = 28800$ m^3/day, $\mu =$~~

08. $G = 600\ s^{-1}$, $\mu = 10^{-3}$ Ns/m^2, $V = 2\ m^3$

$$G^2 = \frac{P^2}{V\mu}$$

$$P^2 = 600^2 \times 2 \times 10^{-3}$$

$$P = \underline{720\ W}$$

01 $Q = 28800$ m^3/day, $\gamma = 10^{-6}$, $G = 900\ s^{-1}$, $DT = 2$ min.

Volume of mixing tank = $Q \cdot DT$.

$$= \frac{28800}{24 \times 60} \times 2 = 40\ m^3$$

$$G = \sqrt{\frac{P}{\mu V}}$$

$$\therefore P = 900^2 \times 10^{-3} \times 20 = \underline{32400}\ W$$

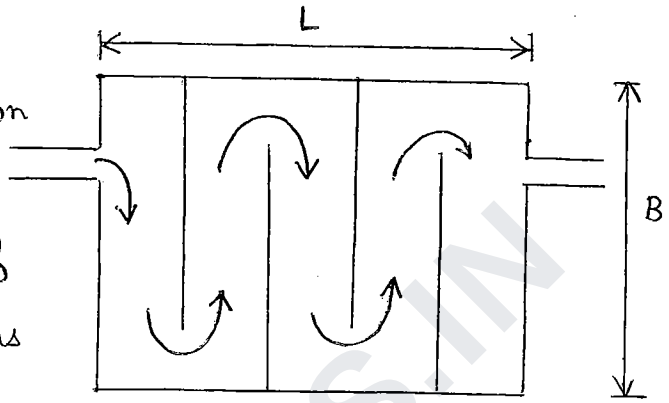
* Design of Flocculator :-

(i) Hydraulic Flocculators

These are outdated.

Process of floc formulation is not under our control.

So, to get required type of floc, mechanical flocculators are used.



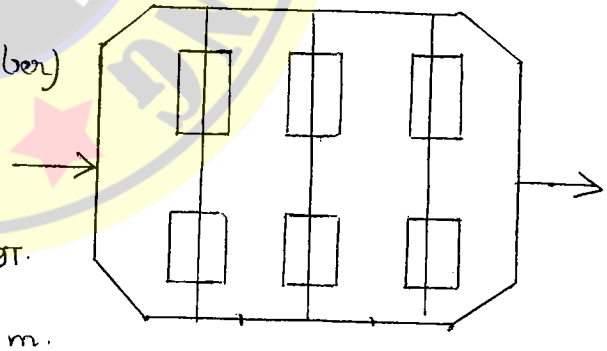
(ii) Mechanical Flocculator.

Mechanical flocculator consists rectangular tank with slow mixing device (drum rotates or paddle rotates)

– Design parameters:

(i) DT (detension time)

(ii) Gt (non dimensional number)

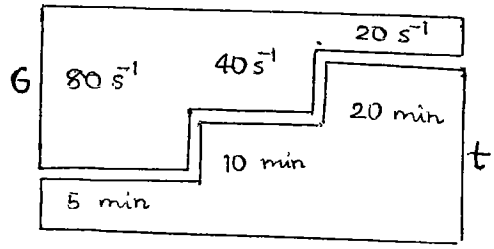


– DT : 20 min to 40 min.

Volume of flocculator = Q * DT.

Assume depth : 2.5 m to 4.5 m.

$$\text{Surface area} = \frac{\text{Volume}}{\text{Depth}}$$



– Gt : 20 to 80 s^{-1}

t = DT = 20 to 40 min.

Gt = 10^4 to 10^5 (effective floc)

– G↑ t↓ → smaller & denser floc

G↓ t↑ → larger and lighter floc.

G↑ t↑ → breaks

G↓ t↓ → only size increases

6. $Q = 3000\ m^3/hr$, $DT = 20\ min$, $G = 40\ s^{-1}$, $\mu = 1.0087 \times 10^{-3}$ Pas.

$L:B = 2$, depth $= 0.4\ B$.

$$\text{Volume} = \frac{3000}{60} \times 20 = 1000\ m^3.$$

$$L \times B = \frac{1000}{0.4\ B}$$

$$\frac{L}{B} = 2. \Rightarrow L = 2B.$$

$$2B \times B \times 0.4B = 1000$$

$$B = 10.772\ m.$$

$$L = 21.5443$$

$$P = G^2 \mu V = 40^2 \times 1.0087 \times 10^{-3} \times 1000$$

$$= 1613.92\ W$$

11th Sept, THURSDAY

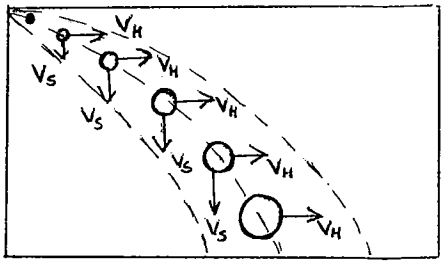
* Design of Sedimentation Tank (Settling tank)

Settling tank follows TYPE II settling. Type II settling deals with flocculant particles settlement in dilute suspensions. Flocculant particles are those which change their size, shape, mass, specific gravity with time and space.

To design settling tank:

SLR or SOR, V_o : 20 to 30 $m^3/day/m^2$

DT : 2 to 4 hours



Non linear path followed by flocculant particle.

9. $Q = 4.2\ m^3/min.$

$V_o = 0.2\ mm/s$

Depth $= 3.5\ m.$

$$V_o = \frac{Q}{S.A} \Rightarrow SA = \frac{Q}{V_o} = \frac{4.2/60}{0.2 \times 10^{-3}} = 350\ m^2$$

12. $Q = 3.5\ m^3/min. = 3.5 \times 10^3 \times 24 \times 60 = 5.04$ MLD.

Alum dosage $= 25$ mg/L

$GT = 4 \times 10^4$

Total alum required /day (kg/day) $= 5.04 \times 25 = 126$ kg/day.

Alum required for 30 days $= 126 \times 30 = \underline{\underline{3780}}$ kg

→ Relation among turbidity (concentration of colloids), Alkalinity and Coagulant Dose:

Turbidity	Alkalinity	Dose of coagulant	Predominant Mechanism
1. High	Low (pH ~ 7)	Small.	Adsorption & cha neutralisation.
2. High	High.	High	Sweep Coagulatio
3. Low	High	Moderate	Adsorption & charg neutralisation and Sweep Coagulatio
4. Low	Low	particles can't be destabilised for any amount coagulant dose.	

Push 4th type of water to 1st type of water by adding Bentonite clay (increases turbidity)

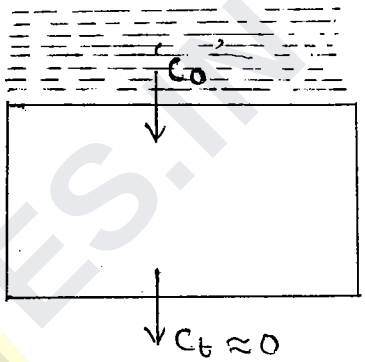
Push 4th type of water to 3rd type of water by adding lime (increases alkalinity)

- Deltas are formed because of coagulation.

11th Sept,
THURSDAY

6. FILTRATION

Filtration is the method of passing water through a stc
bed of granular media. Media retain particles and
yield particle free water.



Commonly used media are:

(i) Silica sand
(ii) Anthracite coal.
(iii) Garnet (green sand).

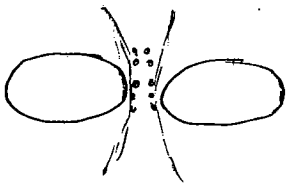
In India, we use silica sand.

→ Mechanisms of Filtration

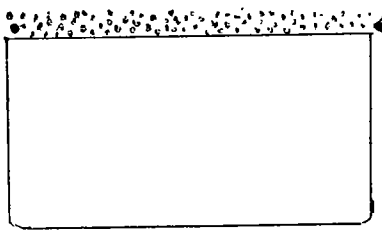
(i) Mechanical Straining
(ii) Sedimentation
(iii) Impaction
(iv) Interception
(v) Biological metabolism.

Sedimentation

impaction



interception



The place where media is packed and water is passed through media is known as Filter.

→ Classification of Filters

1. Based on the force by which filtration is carried ou
   (i) Gravity Filters
   (ii) Pressure Filters.

2. Based on rate of filtration (volume of water filtered in unit time per unit surface area)
   (i) Slow Filters.
   (ii) Rapid Filters.

3. Based on No. of media employed.
   (i) Single media filters.
   (ii) Dual media filters.
   (iii) Multimedia or mixed media filters.

→ Slow Sand Filter

- It is a single media, gravity type, slow filter.
- Rate of Filtration, ROF = 100 to 200 L/hr/m²
- Particle removal efficiency, η = 98 to 99 %
- No pretreatment of water is required, post treatment is optional. Coagulated water is not allowed to pass through slow sand filter. If its allowed to pass, smaller particles in coagulated water grows inside the filter media, blocks it permanently and cause operational problems.

* Components of Slow Sand Filter :

(i) Filter Box. — rectangular, open to atmosphere, water tight tank
   Size : 100 – 2000 m²
   Bed slope : 1 in 100

(ii) Filter media — silica sand used as media.
   Thickness : 90 cm to 110 cm.
   Effective size of sand, D_{10} : 0.2 to 0.35mm

Uniform coeffecient, Cu : $\frac{D_{60}}{D_{10}}$ = 1.8 to 3

Smaller the particles → smaller the pore size → higher area of contact → low rate of filtration → better quality water.

(iii) Gravel bed – gives support to sand media

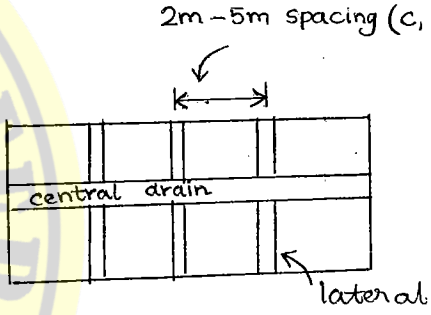
Thickness : 30 cm to 75 cm.

Size : 3 mm to 60 mm (well graded gravel).

It o

It doesn't permit sand particles to escape from filter media.

(iv) Under drainage System – open drainage system provided with laterals and central drains.

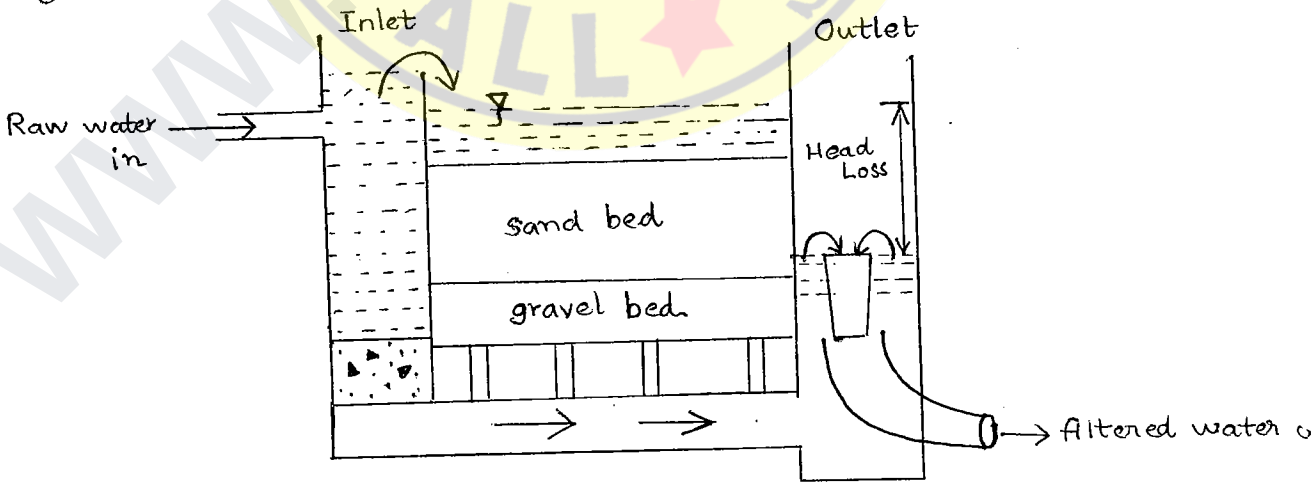


(v) Inlet & Outlet

Inlet → to apply water uniformly over sand bed

Outlet → to collect filtered water at constant rate.

Piezometer → to know the water level on the top of sand bed.



Initial head loss (head loss before starting of filtration): 10 cm to 15 cm

When head loss reaches to a value of 70 cm – 90 cm (90% sand bed thickness), it indicates that filter is not working at rated capacity. It badly need cleaning or washing.

- Slow sand filters are cleaned once in every one month to three months. Top 1.5 – 3 cm of sand bed is scrapped off every until depth of sand bed gets reduced to 50 cm. Beyond this depth coarser sand (low filtration) is found. So the scrapped off fine sand washed with 0.2 – 0.6% of filtered water.

24th sept, WEDNESDAY

→ Design of Slow Sand Filters:

- Filters are designed based on Rate of filtration (ROF = Superficial velocity)
- Total surface area of slow sand filter required = $\frac{Q}{ROF}$
- Fix size of each filter : L & B
- Find no: of filters : n

(OR)

- Fix no: of filters (n) and find size of each filter (L & B) based on length & breadth relations
- Provide one extra filter (n+1) for reliability condition

P 39

01 ROF = 180 lit/hr/m²

Population = 50,000 persons.

Percapita demand = 150 lit/head/day.

Max demand = 1.8 x avg demand.

1 unit is kept as stand by.

No: of filters in operation = 6 – 1 = 5

$Q = 50000 \times 150$ lit/day

Design demand = max. demand = $1.8Q = 1.8 \times 50000 \times 150$

Total area of slow sand filter required = $\frac{Q_{design}}{ROF}$

$$= \frac{1.8 \times 50000 \times 150/24}{180} = 3125 \text{ m}^2$$

$n = 5$ · (no: of filters in operation).

Area of each filter $= \frac{3125}{5} = 625$.

$$L \times B = 625.$$

$$2B^2 = 625 \Rightarrow B = 17.678 \text{ m}$$

$$L = 35.35 \text{ m}$$

Q2. Find how many no: of 20×10 size slow sand filters required to treat 10 MLD water with ROF = 200 L/hr/m².

$$n \times \text{Area } (L \times B) = 2 \cdot \frac{Q}{ROF}$$

$$= \frac{10 \times 10^6/24}{200}$$

$$n = \frac{10 \times 10^6/24}{200 \times (20 \times 10)} = 10.41 \approx 11 \text{ filters}$$

→ Rapid Sand Filters

- Single media gravity type rapid filters.
- ROF : 3000 to 6000 Lit/hr/m². (×30 slow sand)
- Particle removal efficiency, η = 80 to 90% (98-99% slow sand)
- Pretreatment and post treatment of water is compulsory (not required in slow sand)
- consumes little space and meets the demands of large communities.

* Components of Rapid Sand Filter :

(i) Filter box. – open, water tight, rectangular tank.

(ii) Size of filter – 10 m² to 80 m²

(iii) Bed slope = 1/100

(iv) Depth of filter = 2.5 m to 3 m

(v) Filter media:

- Sand is filter media.
- Thickness of sand bed = 60 to 90 cm
- Effective size of sand = D_{10} = 0.35 to 0.55 mm
- Uniform coeffecient, $Cu = \frac{D_{60}}{D_{10}}$ = 1.2 to 1.8

Here pore space increases and hence rate of filtration increases but contact with soil decreases. Due to less contact with sand, removal effeciency decreases.

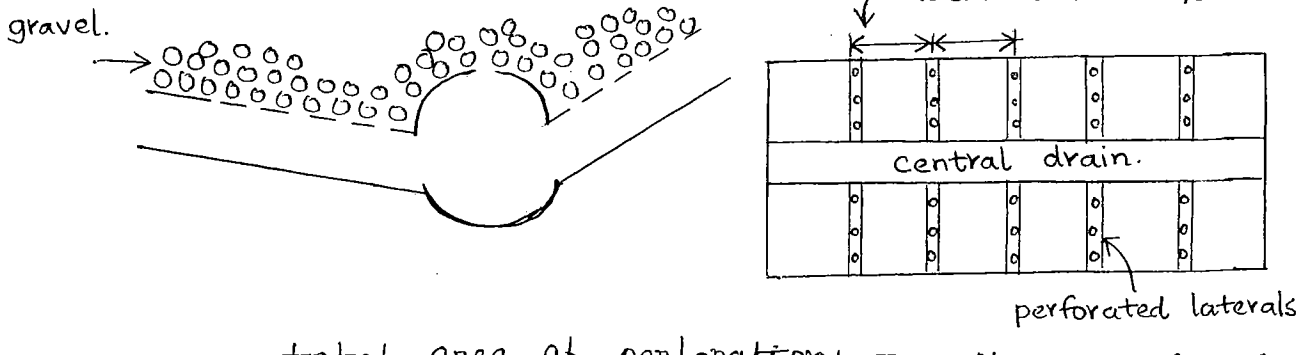
$Cu \approx 1 \Rightarrow$ uniformly graded. Every sand grain is effectively involved in particle capture. This accounts for 80-90% efficiency though pore spaces are opened.

(vi) Gravel bed:

- it gives support to sand bed.
- it doesn't allow sand grains to escape along with water
- Thickness of gravel bed = 60cm to 90cm
- well graded gravel is packed (3mm to 40mm)

(vii) Under drainage System:

- closed drains are used to collect water



- total area of perforations = 0.2% area of each filter

- total area of laterals = 2 times total area of perforations.

- total area of central drain = 2 times x area of laterals.

- $\frac{\text{Length of lateral}}{\text{diameter of lateral}} \not> 60$; OK

(viii) Inlet & Outlet Arrangements :

- Inlet applies water uniformly over sand bed. without disturbing sand grains.

- Outlet :- central drain acts as outlet.

(ix) Other Components :

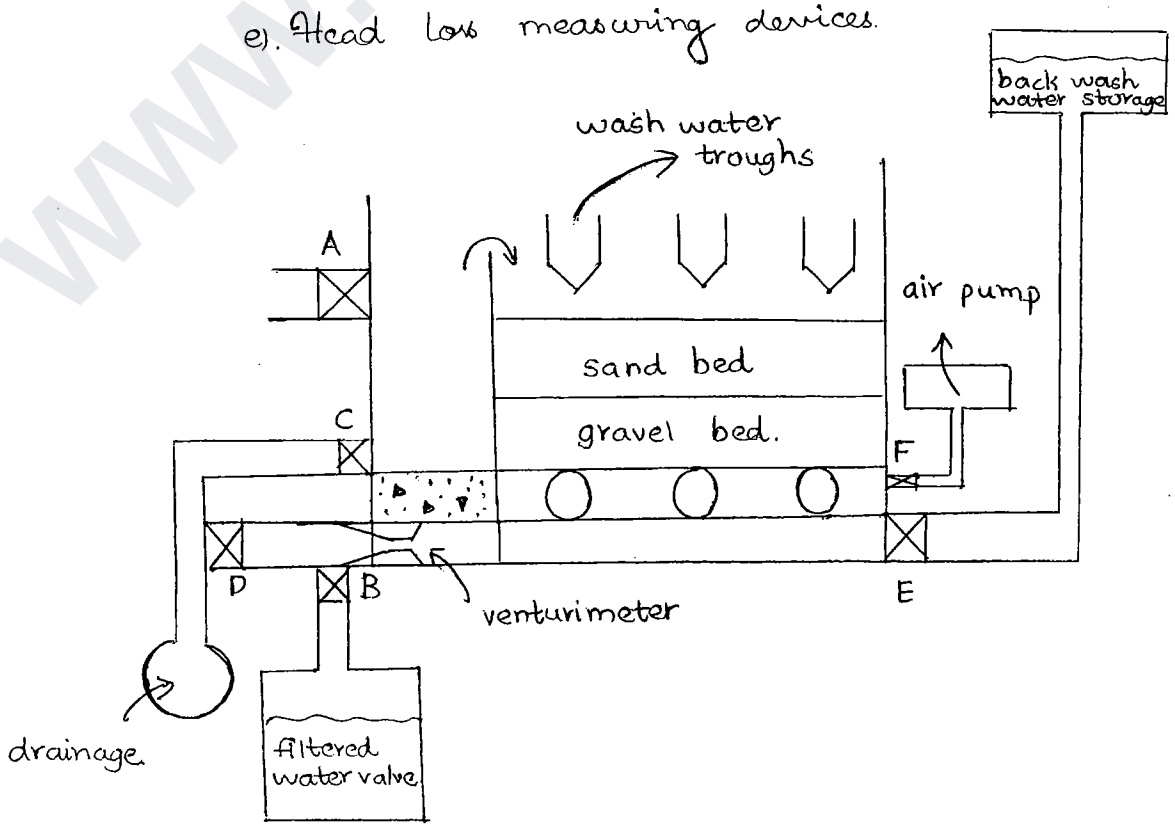
a) Wash water troughs.

b) Air compresor (air pump)

c) Valves.

d) Rate control device (venturimeter).

e) Head loss measuring devices.



A → influent valve

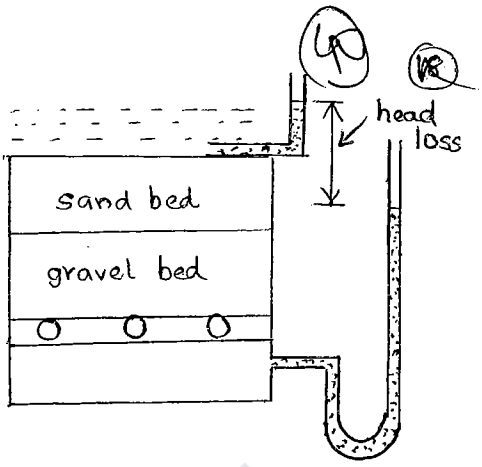
B → Filtered water valve.

C → Wash water valve.

D → Filtered water waste valve.

E → Backwash water valve.

F → Air valve.



Initial head loss = 10 to 15 cm

Terminal head loss = 3m

Turbidity brake through = 2.5 NTU

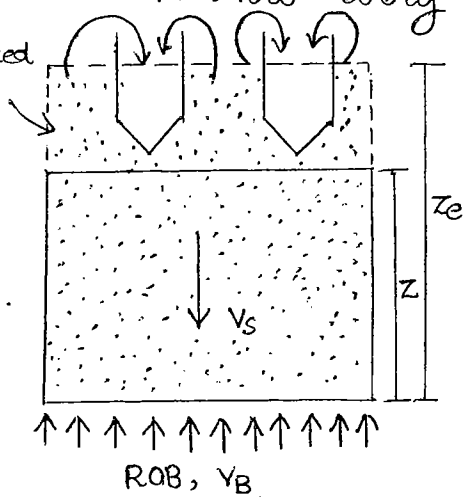
} Based on either of these conditions and filter is stopped and washed once in every 24 hr to 48 hr.

When filtration is continued for longer periods, pore spaces gets closed and head loss increases. As pore space area is reduced, velocity of water filtering through sand bed shoots up and it shears some of the particles retained on the filter bed and carries it along with the filtered water. This increases the turbidity of filtered water known as 'turbidity break through'.

– Rapid sand filters are cleaned (or) washed by 'backwashing'.

Amount of filtered water required for backwashing = 2 to 6% of filtered water

Airwashing is the first step in backwashing where compressed air is pumped through the filter bed for 4–5 min, which agitates the particles, and by opening valve F. Now every valve is closed except E & C. This increases the pore space and sand bed get expanded upto a height Ze. (less than lip of wash water trough). Water gets collected in the drains as valve C is opened.



* Design of Rapid Sand Filter :

- Total area of RSF required $= \frac{Q}{ROF}$

- Fix the size of each filter (L & B) and find no: of filters (n)

(OR)

Fix the no: of filters (n) and find size of each filter

- % of filtered water used in backwashing $\Big\}$ $= \frac{\text{vol. of water used in backwash}}{\text{vol. of water filtered blw backwash}}$

- Volume of filtered water used in backwash $\Big\}$ = Rate of backwashing * duration of backwash * area of each filter.

= ROB * DOB * area of each filt

- Volume of water filtered blw backwashings $\Big\}$ = Rate of filtration * duration of filtration * area of each filter

= ROF * DOF * area of each f

2. $Q = 24$ MLD.

$ROF = 5\ m^3/hr/m^2$.

Area of filter bed, $(L \times B) = \frac{24 \times 10^3/24}{5} = 200\ m^2$

$\therefore \quad L \times 2L = 200$

$\Rightarrow L = 10\ m.$ & $B = 20\ m$

% filtered water required for backwashing $\Big\}$ $= \frac{ROB * DOB * \text{area of each filter}}{ROF * DOF * \text{area of each filter}}$

$= \frac{6 \times ROF * 10/60}{ROF * (24 - \frac{10}{60})} = 4.19\ \%$

2nd Oct,
THURSDAY

Q3. $Q = 0.25\ m^3/s$; $ROF = 5\ m^3/m^2/hr$

$n = 4$

a) $L : B = 2:1$; Back wash rate $= 10\ l/m^2/s.$

$$\text{Total SA} = \frac{Q}{ROF} = \frac{0.25 \times 60 \times 60 \times 1}{5\ m^3/hr/m^2}$$

$$= 180\ m^2.$$

$$\text{SA of single filter} = \frac{180}{4} = 45\ m^2.$$

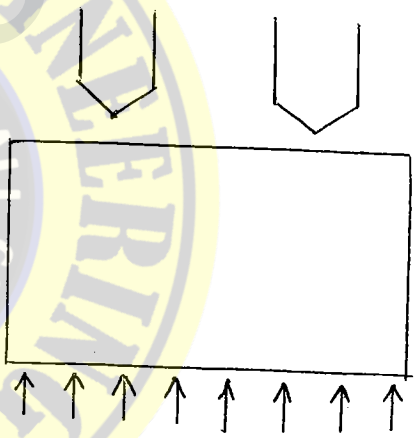
$$\frac{L}{B} = \frac{2}{1}.$$

$$2B^2 = 45$$

$$B = 4.74\ m$$

$$L = 9.49\ m.$$

$$ROB = 10\ l/m^2/s.$$



b) $V_B = ROB = 10\ lit/m^2/s.$

$Q_B = 10 \times 45 = 450\ lit/s.$

$Q_B = V_B \times$ area of filter

Flow rate in each wash water trough $= \frac{Q_B}{2} = 225\ lit/s = \underline{\underline{0.225\ m^3/s}}$

Q5. $Q = 2.5\ MLD = 2.5 \times 10^3 / 24 \times 60 \times 60 = 0.029\ m^3/s.$

$ROF = 125\ l/m^2/min. = 0.125 \times 60 = 2.08 \times 10^{-3}\ m^3/m^2/s.$

$$\therefore \text{Total SA} = \frac{0.029}{2.08 \times 10^{-3}} = \underline{\underline{13.92\ m^2}}$$

$$\% = \frac{\text{volume of water used for backwashing}}{\text{vol. of water filtered b/w backwashing}} = \frac{ROB \times \text{duration} \times \text{area of filter}}{ROF \times \text{duration} \times \text{area of filter}}$$

$$= \frac{7.5 \text{ ROF} \times 5 \times \text{Area}}{\text{ROF} \times (48 \times 60) \times \text{area}} = \underline{\underline{1.302\ \%}} \qquad \frac{7.5 \times 5}{\left(48 - \frac{5}{60}\right) 60}$$

9. $Q = 0.5\ m^3/s$

ROF $= 200\ m^3/m^2/d$.

Total SA required $= \dfrac{0.5}{\frac{200}{86400}} = 216\ m^2$.

10. No: of filter units required $= \dfrac{216}{50} = 4.32 \approx \underline{\underline{5}}$

15. $Q = 1\ m^3/s$; $n = 14$. m^3/s

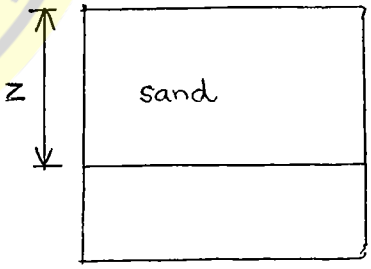
SA $= (14 \times 2)\ 50 = 600\ m^2$.

$$\text{ROF} = \frac{Q}{SA \times \frac{1}{86400}} = 144\ m^3/day/m^2$$

→ Head Loss During Filtration

* Carmen – Kozney Equation.

$$\boxed{H_f = \frac{f z v^2}{g d} \left(\frac{1-n}{n^3}\right)}$$



z → depth of sand bed.

$$f = \frac{150(1-n)}{Re} + 1.75$$

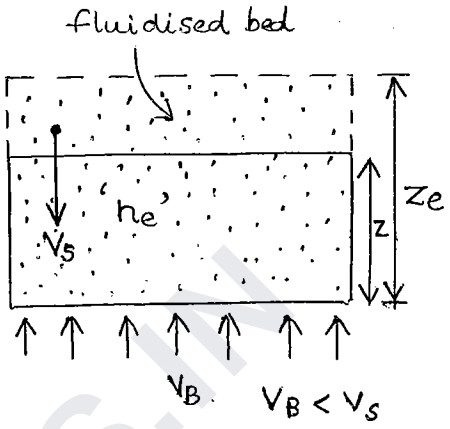
$$Re = \frac{\rho_w v d}{\mu} \phi = \frac{v d \phi}{\gamma} \quad ; \quad \phi \rightarrow \text{shape factor}$$

Particles are assumed to be spherical. ϕ is used to denote the variation from sphericity.

v → superficial velocity = ROF ; n → porosity of sand bed.

d → diameter of sand grains.

This equation is applicable only for a particular porosity. As time increases, porosity and velocity decreases. So, this is a rarely used equation.



* Head Loss during Backwash.

$$H_b = z(1-n)(s-1)$$

* Rate of backwash, ROB (V_B)

$$V_B = V_s (n_e)^{4.5}$$

$$V_s = \frac{g(s-1)d^2}{18\gamma}$$

$$\frac{Z_e}{z} = \frac{1-n}{1-n_e}$$ Also $$H_b = Z_e(1-n_e)(s-1).$$

06. $$Re = \frac{Vd\phi}{\gamma} = \frac{\frac{3}{3600} \times 0.5\times10^{-3} \times 0.8}{10^{-6}} = \frac{0.66}{2} = 0.33$$

$$f = \frac{150(1-n)}{Re} + 1.75$$

$$= \frac{150(1-0.4)}{0.66/2} + 1.75 = \cancel{136.75}\ 271.75$$

$$H_f = \frac{f z v^2}{gd}\left(\frac{1-n}{n^3}\right).$$

$$= \frac{\cancel{136.75}\ 271.75 \times 0.6 \times \left(\frac{3}{3600}\right)^2}{g \times 0.5\times10^{-3}} \left(\frac{1-0.4}{0.4^3}\right) = \cancel{0.1089}\ 0.216\ m$$

7. $S = 2.66$, $n = 0.42$, $Z = 0.65\ m$

$d = 0.65 \times 10^{-3}\ m$

$$\frac{Ze}{Z} = \frac{1-n}{1-ne}$$

$$\frac{1.5Z}{Z} = \frac{1-0.42}{1-ne}$$

$$ne = 0.613$$

$$V_s = \frac{g(S-1)d^2}{18\nu} = \frac{g(2.66-1)\times(0.65\times10^{-3})^2}{18 \times 1.33\times10^{-2} \times 10^{-4}}$$

$$= 0.2873\ m/s$$

Rate of backwash, ROB, $(V_b) = V_s \times ne^{4.5}$

$$= 0.2873 \times (0.613)^{4.5}$$

$$= 0.032\ m/s$$

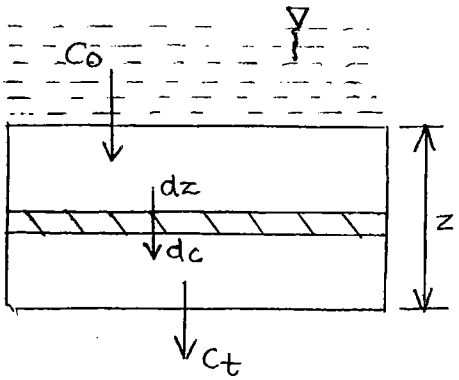
Head loss at this rate, $H_b = Z(1-n)(S-1)$

$$= 0.65(1-0.42)(2.66-1)$$

$$= 0.62582\ m$$

* Particle Removal Efficiency:

$$\eta = \frac{C_o - C_t}{C_o} * 100$$



Iwasaki & Isaku equation for filters:-

$$\frac{dc}{dz} \propto C.$$

$$\frac{dc}{dz} = -\lambda C.$$

$$\int_{C_o}^{C_t} \frac{dc}{C} = \int_0^z -\lambda z.$$

$$\ln \frac{C_t}{C_0} = -\lambda z.$$

$$\boxed{\lambda = \frac{1}{z} \ln \frac{C_0}{C_t}} \quad ; \quad \lambda \rightarrow \text{Filter constant}$$

$$\frac{\eta}{100} = 1 - \frac{C_t}{C_0}.$$

$$\frac{C_0}{C_t} = \frac{100}{100-\eta}$$

$$\Rightarrow \boxed{\lambda = \frac{1}{z} \ln\left(\frac{100}{100-\eta}\right)}$$

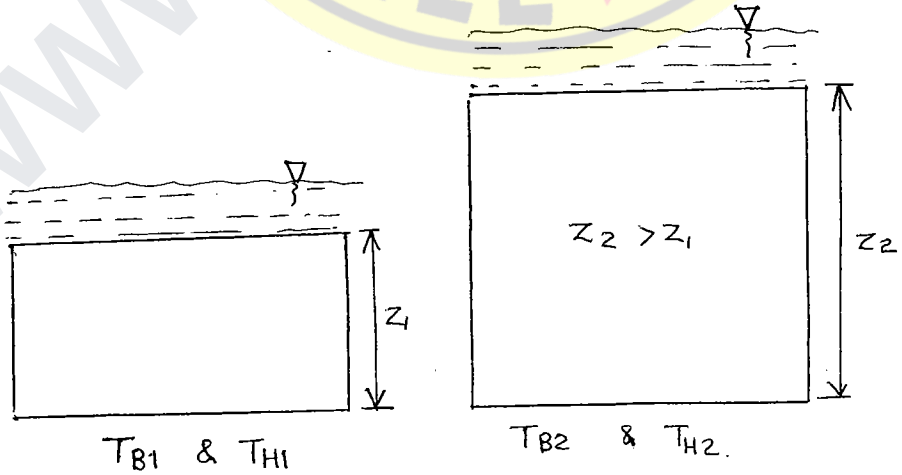
08. $\lambda_1 = \lambda_2$

$$\frac{1}{z_1} \ln\left(\frac{100}{100-\eta}\right)_1 = \frac{1}{z_2} \ln\left(\frac{100}{100-\eta_2}\right)_2$$

$$\frac{1}{0.05} \ln\left(\frac{100}{100-90}\right) = \frac{1}{z_2} \ln\left(\frac{100}{100-99}\right)$$

$$z_2 = 0.1 \text{ m}$$

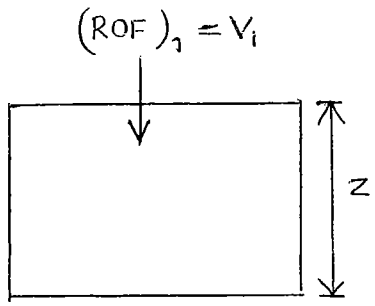
12.

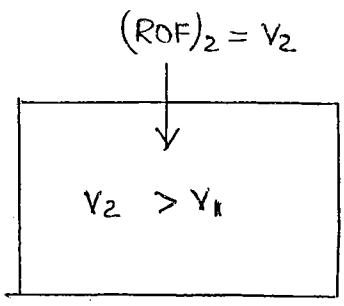


$T_{B2} > T_{B1} \Rightarrow T_B$ increases.

$H_f = \frac{f z v^2}{g d}\left(\frac{1-n}{n^3}\right) \Rightarrow H_f \propto z.$ Also $T_H \propto \frac{1}{H_f}$

$T_{H1} > T_{H2} \Rightarrow T_H$ decreases.





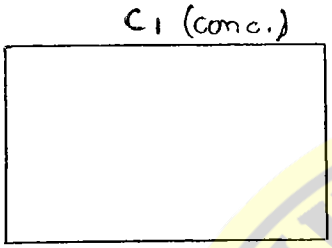
$T_{B2} < T_{B1}$

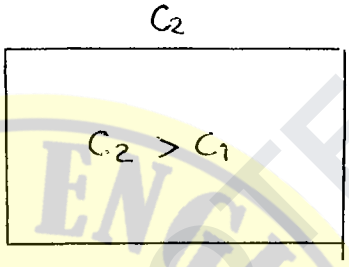
$\Rightarrow T_B$ decreases.

$T_{H2} < T_{H1}$

$\Rightarrow T_H$ decreases.

$$H_f = \frac{fzv^2}{gd}\left(\frac{1-n}{n^3}\right) \Rightarrow H_f \propto v^2 \propto \frac{1}{T_H}$$





$T_{B2} < T_{B1}$

$T_{H2} < T_{H1}$.

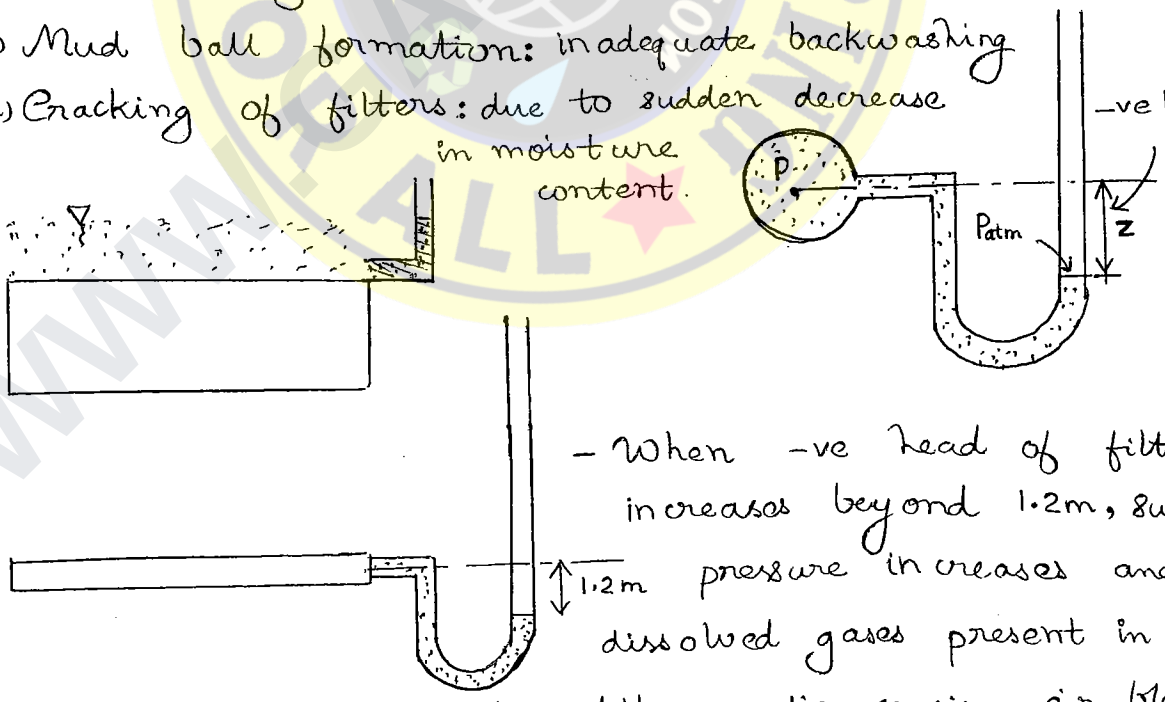
Both T_B & T_H decreases.

* Operational Problems associated with Rapid Sand Filter

(i) Air binding: -ve head of filter > 1.2 m

(ii) Mud ball formation: inadequate backwashing

(iii) Cracking of filters: due to sudden decrease in moisture content.



- When -ve head of filter increases beyond 1.2m, suction pressure increases and dissolved gases present in water are trapped in the filter media causing air blocking.

- To remove the mud balls formed, soak the sand bed in caustic soda.

- Sudden decrease in moisture content in the mud accumulated on top of sand.

8th Oct, WEDNESDAY

→ Pressure Filters

Pressure filter is a rapid sand filter where filtration is carried out under a pressure head of 30m to 70 m. ($P = 300$ to 700 kN/m^2)

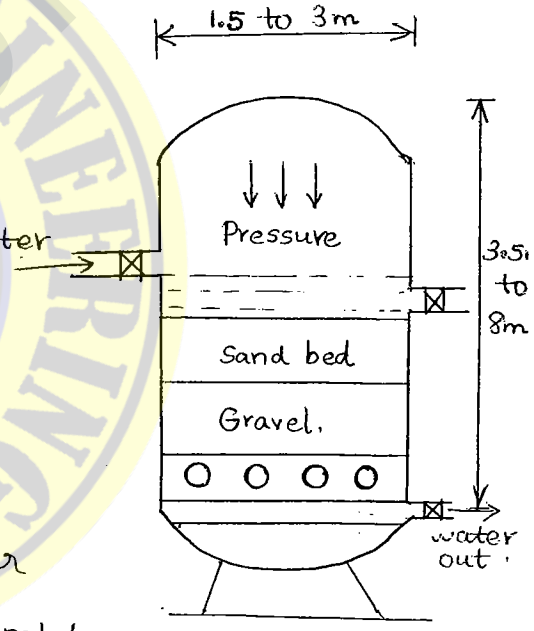
$$ROF = 6000 \text{ to } 15000 \text{ litre/hr/m}^2$$

$$\eta = 60 - 70\%$$

* Pressure filters are widely used at:
  (i) Swimming Pools
  (ii) Industries.
  (iii) Private buildings.

$D_{10} = 0.5$ mm to 0.6 mm.

$C_u = 1.2$



Surface area pressure filter,

$$\frac{\pi}{4} d^2 = \frac{Q}{ROF}$$

Q. Find the diameter of pressure filter of discharge 1 MLD and ROF = 10000 kL/hr/m².

$$\frac{\pi}{4} d^2 = \frac{1 \times 10^6}{10000 \times 24}$$

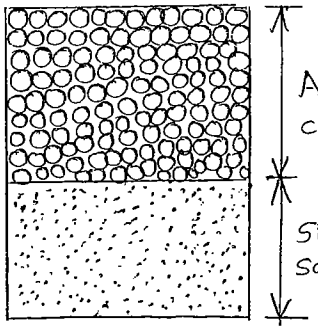
$$d = 2.303 \text{ m}$$

→ Dual Media Filters (Double Media)

– Dual media filters are rapid sand filters where filter is packed with two filter media.

– Anthracite Coal $S = 1.4$ to 1.5 $D_{10} = 0.9$mm to 1 mm

Silica Sand $S = 2.65$ $D_{10} = 0.5$ mm.



- Filtration is from coarse to fine
- ROF = 10000 L/hr/m² to 15000 L/hr/m²
- η = 80 to 90%

Length of filter operation increases (time interval b/w backwash to backwash increases).
Cost of construction & operation is high

4. Q = 72 MLD.

ROF = 15 m³/hr/m²

$$n \times (5 \times 8) = \frac{72 \times 10^6/24}{15000} = 200$$

$$\therefore n = 5$$

No: of filter units required, = n+1 = 6 (one standby).

Assuming backwashing carried out in 24 hours.

Volume of water filtered b/w backwashings (Gross production) = ROF * duration * area of each filter

$$= 15 * \left(24 - \frac{30}{60}\right) * 40$$

$$= 14100 \text{ m}^3/\text{day}$$

{30 min = 20 min (bw) + 10 min (wasted)

Volume of filtered water used in backwashing = ROB * duration * area of each filte

$$= 36 * \frac{20}{60} * 40 = 480 \text{ m}^3/\text{day}.$$

Net product in MLD of each filter unit

$$= 14100 - 480 = 14100 - 480$$

$$= 13620 \text{ m}^3/\text{day}$$

$$= 13.62 \text{ MLD}$$

→ Multimedia (or) Mixed Media Filters

- Rapid sand filter packed with 3 filter media

- Anthracite coal	$S = 1.4$ to 1.5	$D_{10} = 0.9$ to 1 mm
Silica Sand	$S = 2.65$	$D_{10} = 0.5$ mm
Garnet (green sand)	$S = 4.2$	$D_{10} = 0.15$ mm

- ROF = 15000 litre/hr/m^2 to 20,000 litre/hr/m^2.
- η = 80 to 90%
- Length of filter run is more.
- Very costly.

→ Roughening Filter

- Pretreatment done to reduce the load on slow sand filters.

8th Oct,
WEDNESDAY

07. DISINFECTION

– Disinfection is the method of destroying disease causing organisms known as Pathogens present in water.

Objective:

- To kill existing germs in water
- To protect water against future possible potential contamination.

– Substances used to carry out disinfection are known as 'Disinfectants'.

→ The Requirements of an Ideal disinfectant:

– effective and efficient kill.
– fast rate of kill
– should be toxic only to organisms and not to humans.
– should have enough residual power.
– should be of such nature that it can be easily detected from water.

→ Mechanisms of Disinfection:

1. By damaging the cell wall.
2. By alteration of cell wall permeability.
3. By damaging the protoplasm.
4. By precipitating the enzymes, nutrients etc. (Poisoning)

→ METHODS OF DISINFECTION.

1. Physical Methods
   (i) using heat energy (Eg: boiling)
   (ii) using light energy (Eg: UV rays)

2. Chemical Methods

(i) using excess lime
(ii) Surfectants
(iii) Metal ions
(iv) Potassium Permanganate
(v) Ozone
(vi) Halogen group of chemicals.
a) Iodine b) Bromine c) Chlorine

→ Boiling:
- The best method in terms of destroying the organism.
- No residual power.
- uneconomical for large scale purposes.

→ UV treatment:
- UV rays of wavelength 240 to 260 nm act as powerful disinfectants.
- Instant kill, effective & efficient kill (damages the protoplasm) no secondary problems for any amount of UV exposure.

$$\ln \frac{N_t}{N_0} = -K I t$$

where N_t → No: of organisms remain in water after of exposure 't'.

N_0 → initial no: of organisms.

K → disinfection rate constant

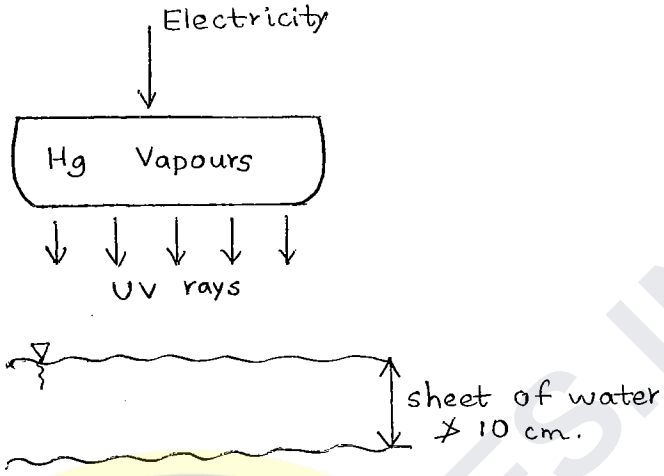
I → intensity of UV radiation in ($\mu W/cm^2$)

t → time of exposure (in s or min)

Q. To reduce microorganisms in water from 10^6 to 100 in 2 min, what intensity of UV radiation is required if $K = 0.006$ cm/μw·sec.

$$\ln \frac{100}{10^6} = -0.006 \times I \times 0.00(60 \times 2).$$

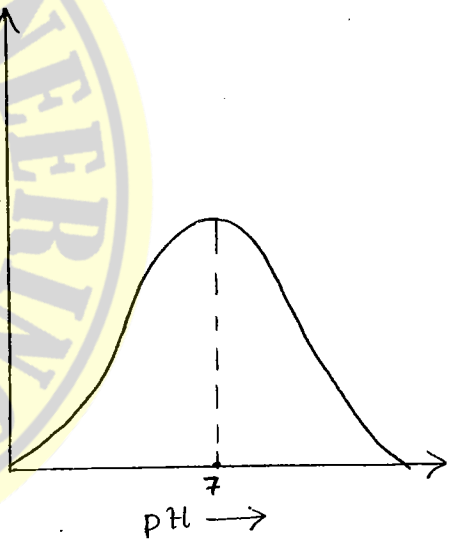
$I = 12.79 \ \underline{\underline{\text{uw/cm}^2}}$



14th Oct, TUESDAY

→ Excess Lime Treatment

- Growth of micro-organisms is influenced by pH.
- pH > 9.4, growth of micro organisms almost zero.
- adding excess lime to water increases pH more than 9.4
- Recarbonation is required to dissolve the precipitates formed as a result of lime treatment; which is expensive.
- growth of micro-organisms optimum at pH = 7. However decreasing the pH increases acidity and leads to corrosion.



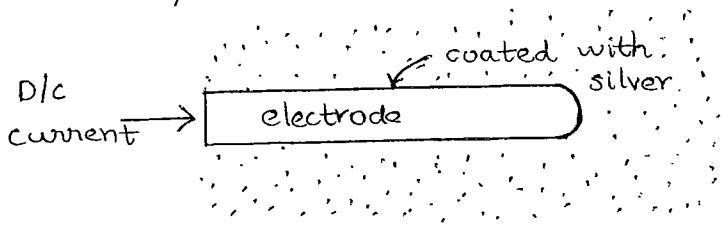
→ Surfectants

- Surface active chemical substances
  Eg: Soaps, detergents, phenols.
- Quality of water is affected.

→ Treatment with Metal ions

– Silver and copper used to carry out disinfection.

– Electrokatadyn Process :- treatment of water with silver



– No residual power

→ Treatment with Potassium Permanganate.

– wells in rural areas are disinfected with $KMnO_4$

– disinfection is slow & takes 24–48 hours.

– when reacting with organism, pink colour is formed which takes 24–48 hours to disappear.

– very cheap.

– No residual power.

→ Ozone.

– Most happening disinfection method.

– Disinfecting water with ozone is known as 'ozonisation'.

– Instant kill, effective & efficient kill.

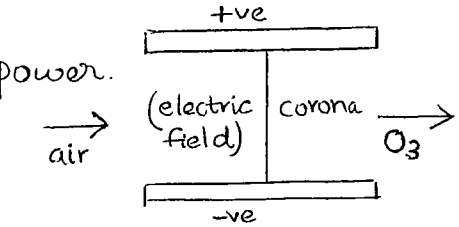
– Little residual power.

– Removes taste, colour, odour, volatile organic compounds, excess amounts of Fe & Mn. etc,

– Ozone exists in unstable form. So it needs to be generated just before ozonisation as it cannot be stored.

– Absolutely safe for any amount of ozone doze; no secondary problems.

– 1 kg ozone requires 0.8~~6~~ 2 kw–hr power.



$$O_2 + 2\bar{e} \longrightarrow 2O$$

$$O + O_2 \longrightarrow O_3$$

$$3O_2 + 2\bar{e} \longrightarrow 2O_3$$

– O_3 is highly unstable isotope of oxygen. It always tries to attain stable molecular form.

$$O_3 + H_2O \longrightarrow 2\underline{HO_2} \quad \text{(powerful oxidants)}$$

$$HO_2 + O_3 \longrightarrow 2O_2 + \underline{HO}$$

– HO_2 & HO (hydroxyl free radicals) released in water oxidise microorganisms present in water and destroy them

→ Halogen Group of Chemicals.

– Iodine & bromine are available in the form of pellets or tablets. They are used in small scale. They are carried by military men when working in remote rural areas.

→ Chlorine

– addition of chlorine or its compounds to the water to carryout disinfection is known as 'Chlorination'.

$$Cl_2 + H_2O \longrightarrow HOCl + HCl \quad \text{(hydrolisation)}$$

$$HOCl \overset{pH}{\rightleftharpoons} H^+ + OCl^- \quad \text{(ionisation)}$$

– $HOCl + OCl^-$ released in water reacts with micro-organisms and destroy them.

$HOCl$ & OCl^- together known as <u>free chlorine</u> or 'freely available chlorine'

– HOCl is chargeless compound and is <u>80 times</u> more powerful than OCl^- (micro-organisms too have –ve & +ve charges and ∴ repulsion occurs b/w them & OCl^-).

- $HOCl >>> OCl^-$ to have effective disinfection.
- HOCl in water depends on pH

$$HOCl \rightleftharpoons H^+ + OCl^-$$

$$K = \frac{[H^+][OCl^-]}{[HOCl]}$$

$$\therefore \frac{K}{[H^+]} = \frac{[OCl^-]}{[HOCl]} \quad ; \; K \rightarrow \text{ionisation const.}$$

$OCl^- \propto \frac{1}{[H^+]}$ & $HOCl \propto [H^+]$.

$[H^+] \propto \frac{1}{pH}$

Lower the pH, higher will be concentration of HOCl.

- $\% HOCl = \frac{HOCl}{HOCl + OCl^-} \times 100$

$$= \left(\frac{1}{1 + \frac{K}{H^+}}\right) \times 100$$

- Concentration of HOCl = dose of Cl_2 * % HOCl.

- Low pH in water is required for efficient chlorination

pH : 0 to 5.5 → only HOCl formed in water

pH : 5.5 to 7.5 → both HOCl & OCl^- in water, but $HOCl >> OCl^-$

pH : 7.5 to 10.5 → both HOCl & OCl^- in water, but $OCl^- >>$ HOCl.

pH > 10.5 → only OCl^- form in water.

– Practically, chlorine remain as elemental Cl_2 in the pH range 0 to 5.5. It does not hydrolyse in water.

– So pH range of 5.5 to 7.5 is more suitable.

P–46.

Q.4. $$\% \text{ HOCl} = \frac{1}{1 + \frac{K}{H^+}}$$

$$0.9 = \frac{1}{1 + \frac{K}{H^+}}$$

$$H^+ = 2.43 \times 10^{-7}$$

$$pH = -\log H^+ = -\log 2.43 \times 10^{-7} = \underline{\underline{6.61}}$$

14. $pH = 7 \Rightarrow H^+ = 10^{-7}$

$$\% \text{ HOCl} = \frac{1}{1 + \frac{K}{H^+}} = \frac{1}{1 + \frac{2.5 \times 10^{-8}}{10^{-7}}} = \underline{\underline{0.8}}$$

09. Concentration of HOCl = dose of Cl_2 * % HOCl.

$$\% \text{ HOCl} \propto \frac{1}{pH}$$

Conc. of HOCl = const. for same removal.

$$\Rightarrow (\text{dose of } Cl_2 * \% \text{ HOCl})_1 = (\text{dose of } Cl_2 * \% \text{ HOCl})_2$$

$$0.7 * \left(\frac{1}{1 + \frac{2.7 \times 10^{-8}}{10^{-7}}}\right) = (\text{dose of } Cl_2)_2 * \left(\frac{1}{1 + \frac{2.7 \times 10^{-8}}{10^{-8}}}\right)$$

$$\Rightarrow (\text{dose of } Cl_2 \text{ at } pH = 8) = \frac{0.7 \times 78.74}{27.02} = \underline{\underline{2.035 \text{ mg/L}}}$$

If pH decreases to 6, dose of Cl_2 = $\frac{0.7 \times 1.27^{\frac{1}{3}}}{(1.027)^{-1}}$

= 0.566 mg/L

As pH ↓, dose of Cl_2 ↓ for same removal.

* Chlorine Demand of Water

– Chlorine has more affinity towards Fe & Mn; then it react with Ammonia. Chlorine, then, it will react with micro-organisms.

– Amount of chlorine consumed in water is known as 'Chlorine demand of water'. After meeting the chlorine demand of water in reacting with above, if any excess chlorine remain in water, it will appear as "Residual Cl_2"

Cl_2 demand = Cl_2 dose – Residual Cl_2

– Advantages of using chlorine are that it is cheap and residual powers for future disinfection.

02. $Q = 20{,}000\ m^3/day$.

Total Cl_2 usage = 8 kg/day.

$$\underset{(kg/day)}{\text{Total } Cl_2} = \underset{(MLD)}{Q} * \underset{(mg/L)}{\text{dose of } Cl_2}$$

$$8 = 20 \times \text{dose of } Cl_2$$

$$\text{Dose of } Cl_2 = \frac{8}{20} = 0.4 \text{ mg/L}.$$

Cl_2 demand = Cl_2 dose – Residual Cl_2

= 0.4 – 0.15 = 0.25 mg/L

* Forms of Chlorine

1. Hypochlorites.
2. Chloramines.
3. Cl_2 gas.
4. Chlorine dioxide.

*Addition of chlorine in the form of hypochlorites is known as hypochlorination

(i) Calcium hypochlorite (Bleaching Powder) → 30-35% available chlorir
(ii) Sodium hypochlorite → 10% chlorine.

Advantages of using Bleaching Powder :-

a) Economical. (cheapest of all forms of chlorine)
b) Free chlorine.

Disadvantages of Bleaching Powder :-

a) It doesn't dissolve quickly.
b) It looses its strength & odour when exposed to light or air.
c) It imparts colour to water. (more secondary problems)
d) High dosages are required (∵ % chlorine is very low).

$$Cl_2 \text{ dose} = \text{dose of bleaching (BP) powder} * \%\ Cl_2 \text{ in BP}$$

Q. 300 kg per month of bleaching powder consumed to treat 10 MLD to water to have Cl_2 residual of 0.2 mg/L. Find chlorine demand of water in mg/L. If BP contains 30% Cl_2.

$\frac{300\ kg}{30}$

$$Cl_2 \text{ dose} = \text{dose of BP} * \%\ Cl_2$$

$$\text{Dose of BP} = \frac{\text{Total BP usage}}{Q} = \frac{300 kg/30\ day}{10}$$

$$= \underline{1\ mg/L}.$$

Cl_2 dose = 1 mg/L * 0.3 = 0.3 mg/L.

Chlorine demand = Cl_2 dose − Residual Cl_2.

= 0.3 − 0.2 = 0.1 mg/L

13. BP.

Cl_2 dose = 2 ppm = 2 mg/L

Available chlorine = 20%

Q = population * per capita demand.

= 10,000 × 50 L/day = 0.5 MLD.

Total BP required = Q * dose of BP

= 0.5 × 2 = 1 kg/day

Q. 7.3 tons of BP is consumed per annum to chlorinate 10 MLD water to meet the Cl_2 demand of 0.45 mg/L in water. If Cl_2 BP contain 80% Cl_2, then residual Cl_2 in water is..

$$\text{Dose of BP} = \frac{7.3 \times 10^3/365}{10} = 2 \text{ mg/L}$$

Cl_2 dose. = 0.3 × 2 = 0.6 mg/L.

Residual chlorine = Cl_2 dose − chlorine demand.

= 0.6 − 0.45 = 0.15 mg/L

→ Chloramines. (Chlorine + Ammonia → Chloramines)

Chlorine in the form of chloramines is called 'combined chlorine'

$Cl_2 + H_2O \longrightarrow HOCl + HCl$.

$HOCl + NH_3 \longrightarrow NH_2Cl$ (monochloramine)

$HOCl + NH_2Cl \longrightarrow NHCl_2$ (dichloramine)

$HOCl + NHCl_2 \longrightarrow NCl_3$ (trichloramine)

pH > 8 → only monochloramine in water

pH : 5 − 8 → both monochloramine & dichloramine

pH : 4.4 − 5 → only dichloramine in water

pH < 4.4 → trichloramine.

- mono & di are more powerful than trichloramine, so pH is maintained blw 4.4 – 8.
- Chloramines produce long and persistant residuals in water.
- Quickly dissolve in water.
- It doesn't lose its strength when exposed to air & light.
- No colour, taste, odour problems in water for any dose. It doesn't even form toxic chemical substances in water.
- It is slower than other forms of chlorine; (only disadvant but not as slow as $KMnO_4$.

→ Chlorine Gas

- most sought after form of chlorine.
- It is 2.5 times heavier than air and quickly dissolve and produce free chlorine in water.

$$Cl_2 \text{ gas} \xrightarrow{P = 1.5 \text{ kgf/cm}^2} \text{liquid } Cl_2$$

- Stored and shipped in the form of liquid Cl_2, added to water in the form of gaseous Cl_2.
- High free chlorine residuals are carcinogenic. Free chlorine react with organic substances in water and produce trihalomethanes (THM) which are carcinogenic.

Eg: of THM are Chloroform, bromoform etc.

- Free chlorine residual ≯ 0.2 mg/L. Free chlorine residual is maintained at a range of 0.1 – 0.2 mg/L.

Free chlorine residual ≮ 0.1 mg/L ⇒ no future protection.

However dosage of chloramine does not influence THM formation.

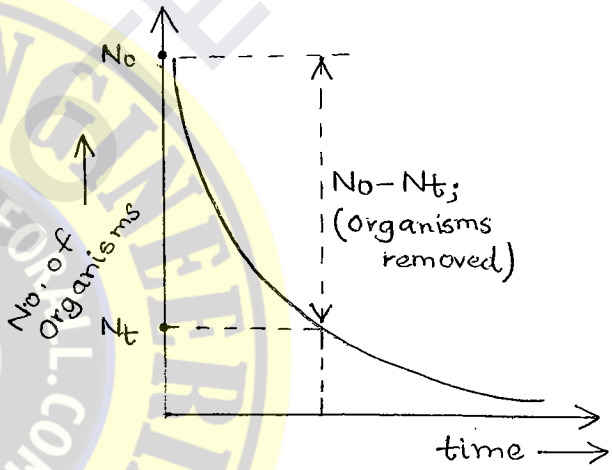
→ Chlorine dioxide.

- Effecient, effective and instant kill.
- No THM problem for any dosage.
- independent of pH.
- unstable, there it can't be stored and generated just before use.
- expensive.

→ Kinetics of Disinfection.

* Chick's Law for disinfection.

$$N_t = N_0 . e^{-kt}$$



N_t → no. of organisms remained after contact time, 't'.

N_0 → initial no. of organisms

t → contact time

k → disinfection rate constant.

⊙ Percentage removal, η = (% of kill) $= \frac{N_0 - N_t}{N_0} * 100.$

$$\eta = \frac{N_0 - N_0 e^{-kt}}{N_0} * 100$$

$$\therefore \eta = (1 - e^{-kt}) * 100$$

⊙ log removal $= \log_{10}(N_0) - \log_{10}(N_t)$

$$C^n * t = \text{const.}$$

C → concentration of HOCl. ≈ Cl_2 dose (pH = const.)

t → contact time. (detention time)

n → dilution factor. (type of organism killed)

: th Oct
N DAY

11. $N_0 = 10^6$ no./100 mL ; $N_t = 10^2$ no./100 mL

$$\% \text{ removal} = \frac{N_0 - N_t}{N_0} \times 100 = \frac{10^6 - 10^2}{10^6} \times 100 = \underline{99.99\%}$$

$$\log \text{ removal} = \log_{10} N_0 - \log_{10} N_t.$$

$$= \log_{10} 10^6 - \log_{10} 10^2 = 6 - 2 = \underline{4}$$

16. $Q = 2670$ m^3/day ; $\eta = 98\%$

$$N_t = N_0 e^{-0.145t}.$$

$$\eta = \left(1 - e^{-kt}\right) 100.$$

$$0.98 = 1 - e^{-0.145t}.$$

$$\Rightarrow t = 26.98 \text{ min.}$$

Volume of disinfection unit, $V = Q \times t$

$$= \frac{2760}{24 \times 60} \times 26.98$$

$$= \underline{51.71} \ m^3$$

8. $\eta = 99.7\%$; $K = 3 \times 10^{-2}$ /s.

$$\eta = 1 - e^{-kt}$$

$$0.997 = 1 - e^{-3 \times 10^{-2} t}$$

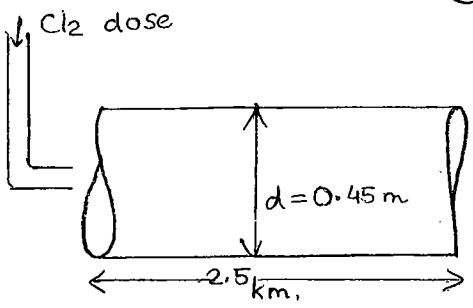
$$\Rightarrow t = 193.64 \text{ s} = \underline{3.227 \text{ min}}$$

15. $Q = 16$ MLD.

Total Cl_2 added = 32 kg/day

Total Cl_2 added = Cl_2 dose $\times$ Q.

Dose of $Cl_2 = \frac{32}{16} = 2$ mg/L.



$$A = \frac{Q_1}{V_1} = \frac{Q_1}{\frac{\text{distance}}{(\text{time})_1}}$$

$$= \frac{Q_2}{\frac{\text{distance}}{(\text{time})_2}}$$

$Q_1 t_1 = Q_2 t_2$.

$\frac{Q_1}{C_1} = \frac{Q_2}{C_2}$ (For same removal).

But $C_1^n t_1 = \text{constant}$.

$t_1 = \frac{\text{const.}}{C_1^n}$; $n=1$

$\therefore t_1 \propto \frac{1}{C_1}$.

$\frac{16}{2} = \frac{22}{C_2}$

$\Rightarrow C_2 = 2.75$ mg/L

Total Cl_2 added = Cl_2 dose * Q.

$= 2.75 * 22 = 60.5$ kg/day

(As Q ↑, velocity ↑ and time of contact ↓.
∴ to achieve same degree of removal, Cl_2 dose needs to be increased.)

Q Find out Cl_2 dose required to treat 10 MLD water having pH = 8 if Cl consumed per day to treat 20 MLD of water is 40 kg/day. having pH = 6. Assuming that same degree of removal ~~and same contact time~~. Take $K = 2.7 \times 10^{-8}$ mol/L and n=1. ~~Find the contact time required. If contact time required to treat 20 MLD is 30 min.~~

$$\% \text{ HOCl} = \frac{1}{1 + \left(\frac{K}{H^+}\right)} \times 100.$$

Concentration of HOCl = % HOCl x Cl_2 dose.

For pH = 6, $H^+ = 10^{-6}$.

$$\% \text{ HOCl} = \frac{1}{1 + \frac{2.7 \times 10^{-8}}{10^{-6}}} \times 100 = 97.37\%$$

$$\text{Conc. HOCl} = \frac{97.37}{100} \times \frac{40}{20} = 1.95 \text{ mg/L}.$$

$$\frac{Q_1}{C_1} = \frac{Q_2}{C_2}$$

At pH = 8, $H^+ = 10^{-8}$.

$$\% \text{ HOCl} = \frac{1}{1 + 2.7} \times 100 = 27.027\%$$

$$\text{Conc. of HOCl} = Cl_2 \text{ dose} \times \frac{27.027}{100}$$

$$1.95 \text{ mg/L} = Cl_2 \text{ dose} \times 0.027$$

$$Cl_2 \text{ dose} = 7.215$$

$$C_1^n t_1 = C_2^n t_2$$

$$1.95\, t_1 = 1.95 \times t_2$$

$$\Rightarrow t_1 = t_2 = 30 \text{ min}$$

Q To treat 20 MLD of water, 40 kg/day of Cl_2 is consumed when pH of water 6 taking 30 min to disinfect water. To have same disinfection while treating 10 MLD of water with a Cl_2 dose of 5 mg/L at pH = 8, what amount of contact time is required.

At pH = 6, % HOCl = 97.37%

[HOCl] = 1.95 mg/L

At pH = 8, % HOCl = 27.00

[HOCl] = 5 × 0.027

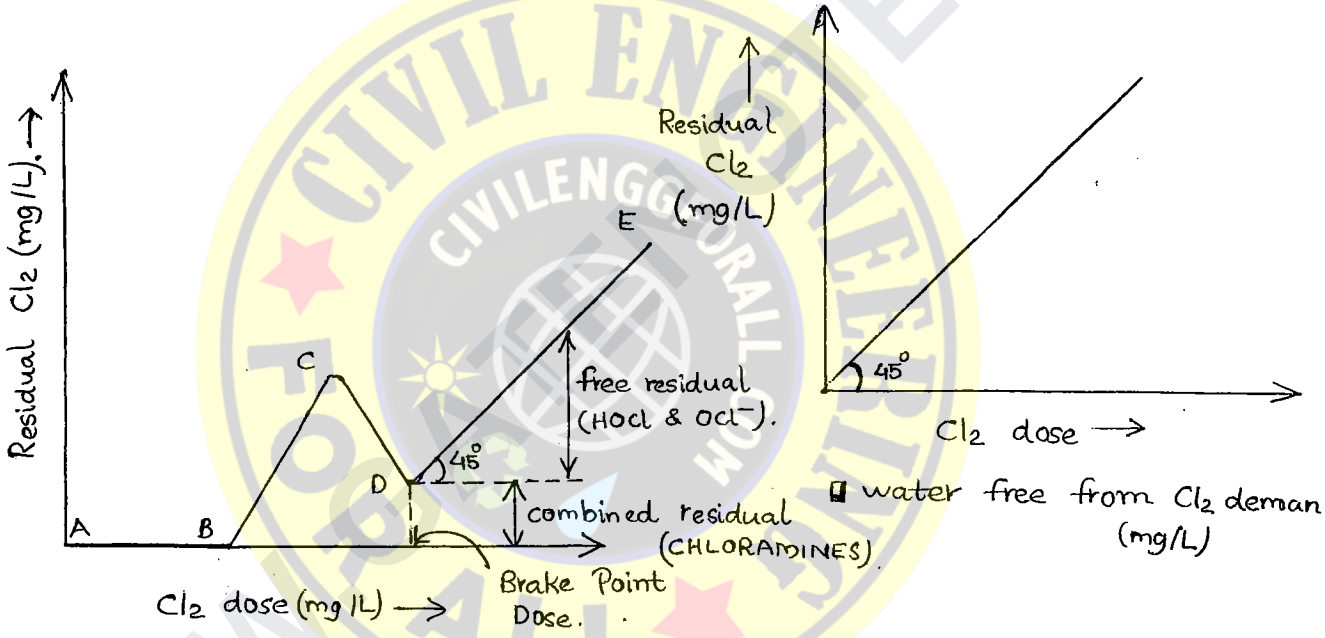
= 0.135 mg/L.

$$1.95 \times 30 = 0.135 \times t$$

$$\Rightarrow t = 43.33 \text{ min}$$

→ Types of Chlorination.

1. Plane Chlorination. — only chlorination is used.
2. Pre Chlorination. — before filtration
3. Post Chlorination. — after filtration
4. Double Chlorination. — before and after filtration.
5. Super Chlorination. — beyond normal doses.
6. Dechlorination. — Sodium thiosulphate.
7. Break Point Chlorination.



Total residual Cl_2 = free residual Cl_2 + combined residual Cl_2

AB → Oxidation of Fe^{2+} & Mn^{2+}

BC → Formation of chloramines.

CD → destruction of micro organisms.

DE → formation of free chlorine.

To obtain residual chlorine and efficient kill of micro organisms, Cl_2 dose is added above break point chlorination.

03. Break point chlorination dose = 1.5 mg/L.

Residual chlorine at Break point = Combined chlorine.

= 0.3 mg/L.

Free Chlrone residual = Total chlorine dose − Break point

= 2 − 1.5 = 0.5 mg/L

Total Cl_2 residual = Free + Combined.

= 0.5 + 0.3 = 0.8 mg/L

12. Dose of Cl_2 gas = 8 mg/L as Cl_2

Free chlorine residual = 2 mg/L. as Cl_2.

pH = 7.5.

$$OCl^- + H^+ \rightleftharpoons HOCl. \quad ; \quad K = 10^{7.5} \text{ mol/L.}$$

Free chlorine residuals = residual HOCl + residual OCl^-

= 2 mg/L.

$$\text{Free residual } Cl_2 \text{ (in mol/L)} = \frac{\text{Free residual (in mg/L)}}{\text{mol. wt. of } Cl_2 \times 1000}$$

$$= \frac{2}{2 \times 35.5 \times 1000} = 2.816 \times 10^{-5} \text{ mol/L.}$$

Residual HOCl + residual OCl^- = 2.816×10^{-5} mol/L.

$$\frac{[HOCl]}{[H^+][OCl^-]} = K.$$

$$\frac{HOCl}{OCl^-} = K H^+ = 10^{7.5} \times 10^{-7.5} = 1$$

$\therefore [HOCl] = [OCl^-]$.

$\therefore$ Residual HOCl = Residual OCl^- = $\frac{2.816 \times 10^{-5}}{2} = 1.408 \times 10^{-5}$ mol/L

→ Tests to find Residual Chlorine

1. Orthotolidine test (O-tolidine test) → total residuals
2. OTA test. (Orthotolidine - Arsenate test) → free residuals
3. Starch - Iodide test.

Arsenate inhibits the reaction of chloramines with Orthotolidine. So O-tolidine reacts with free chlorine.

Starch iodide test is titrometry test. Titrant used is Sodium thiosulphate and indicator is starch. Colour change is from blue to ~~bb~~ colourless.

20th Oct,
MONDAY

08. MISCELLANEOUS WATER TREATMENT

Aeration:

- Only used for treating ground water under normal circumstances.
- Aeration is the method by which water is exposed to free atmosphere.
- Aeration is aimed to remove undesirable dissolved gases such as CO_2 & H_2S (which gets trapped in soil spaces due to decomposition of organic matter) and at the same time water is enriched with O_2. Water desorb (degassify) CO_2 & H_2S and absorb (gasify) O_2.

$$\boxed{\frac{dC}{dt} \propto (C_s - C_t)}$$

where $C_s \rightarrow$ saturation concentration.

$C_t \rightarrow$ actual concentration.

$C_s > C_t \longrightarrow$ absorption

$C_s < C_t \longrightarrow$ desorption.

In addition to absorption & desorbtion of above gases, aeration can also improve taste of water, remove odour & colou from water.

If oxygen levels in water once increases, Fe^{2+} & Mn^{2+} pushed to higher valency states and produces insoluble precipitate in water. Aeration followed by precipitation, precipitates removed and hence colour caused by Fe & Mn are also removed.

Aeration → Sedimentation

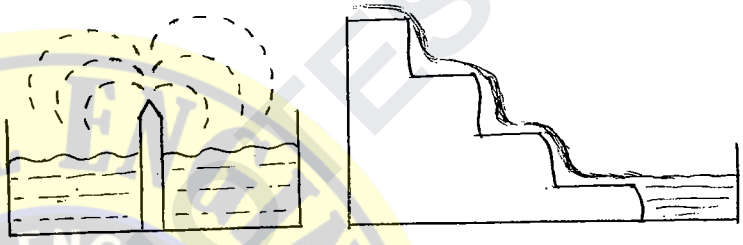
↓ ↓ ↓

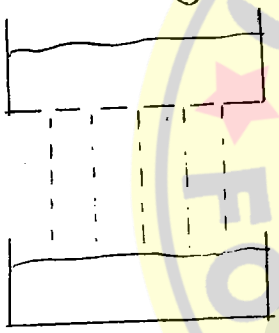
insoluble ppt. settle down.

$\{Fe^{2+}, Mn^{2+}\}$ soluble → $\{Fe^{3+}, Mn^{4+}\}$ insoluble.

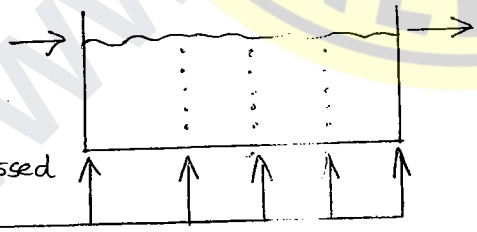
* Aeration is carried out by two methods:
  (i) Dispersion of water into air ⇒ Disorption >> Absorptic
  (ii) Dispersion of air into water ⇒ Absorption >> Desorption

* Dispersion water into air
  (i) Fountains.
  (ii) Cascade.
  (iii) Tray tower

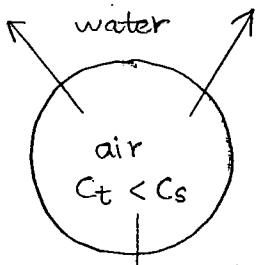




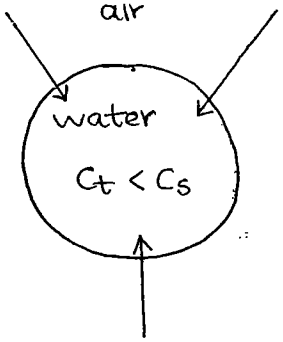
* Dispersion of air into water



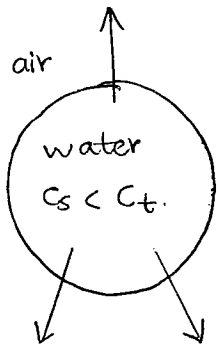
Eg: Fish Aquarium



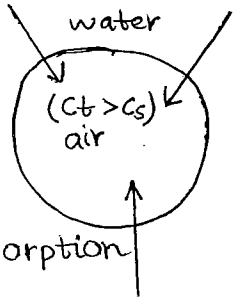
■ Absorption



■ Absorption



■ Desorption



■ Desorption

WATER SOFTENING:

Removal of hardness from water is called water softening.

1. Boiling → removes only CH caused by Ca
2. Lime treatment → removes CH caused by Ca & Mg.
3. Lime–soda Process → NCH caused by Mg
   → NCH caused by Ca (by soda)
4. Ion–exchange method → zero hardness

Types of Hardnesses are:

CH → Ca NCH → Ca

CH → Mg NCH → Mg.

Lime Soda process brings down the hardness to 50 mg/L

26th Oct, SUNDAY

Lime as CaO required $= \frac{56}{100} * TH$
Soda required $(Na_2CO_3) = \frac{106}{100} * NCH$

When conc. given in (m.eq/L)

Hydrated lime $[Ca(OH)_2]$ required $= CO_2 + HCO_3 + Mg$
Soda required $= Ca + Mg - [CO_3 + HCO_3 + OH^-]$

Q. Find lime and soda ash required per day in kg to handle 10 MLD of water with TH 240 mg/L and CH = 150 mg/L as $CaCO_3$.

Total lime required $= \frac{56}{100} \times 240 = 134.4$ mg/L

Lime required per day $= Q \times 134.4 = 1344$ kg/day

Soda ash required $= \frac{106}{100} \times (240 - 150) = 95.4$ mg/L

Soda required per day $= 95.4 \times 10 = 954$ kg/day

Q The following substances are found in water. Find hydrated lime and soda required (in mg/L).

Substance	Concentration (mg/L)	Eq. wt.	Concentration (m.eq/L)
CO_2	10	22	0.45
Ca	70	20	3.5
Mg	40	12	3.33
HCO_3	250	61	4.09
CO_3	2	30	0.06
OH.	0.02.	17.	0.0012.

Lime required (in m.eq/L) = $CO_2 + HCO_3 + Mg$

$= 0.45 + 4.09 + 3.33.$

$= 7.87$ meq/L.

Lime required (in mg/L) $(Ca(OH)_2)$ = 7.87 m.eq/L * eq. wt.

$= 7.87 \left(\frac{40 + 2(16+1)}{2}\right) = \underline{\underline{291.19}}$ mg/L

Soda required (in m.eq/L) =

$Ca + Mg - (CO_3 + HCO_3 + OH^-)$

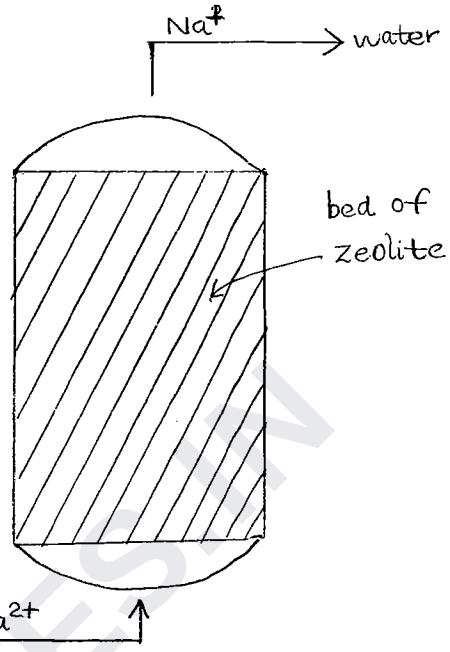
$= 3.5 + 3.33 - (0.06 + 4.09 + 0.0012)$

$= \underline{\underline{2.6788}}$ m.eq/L

Soda required (in mg/L) $= 2.6788 \times \frac{106}{2} = \underline{\underline{141.98}}$ mg/L

* Design of Ion Exchanger

Zeolite – Sodium Aluminium Silicate.



Water with Ca^{2+} & Mg^{2+} ions are passed through a bed of zeolite. (against gravity to have more contact) Zeolite acts as a filter and ion exchange takes place to exchange Ca^{2+} & Mg^{2+} ions in water with Na^+ ions.

Na^+ imparts 'Pseudo Hardness' to water.

This process is also called as Zeolite Process.

$$\text{Volume of ion exchanger} = \frac{Q * TH}{\text{Capacity of exchanger}}$$

Capacity of ion exchanger = 20 to 100 $kg/m^3/day$

Rate of Flow = 0.2 to 0.5 $m^3/min/m^2$

$$\text{C/s area of ion exchanger, } \frac{\pi}{4}d^2 = \frac{Q}{\text{rate of flow}}$$

Q. Find diameter and height of ion exchanger required to handle 1 MLD of water with TH 150 mg/L as $CaCO_3$. Take rate of flow as 0.4 $m^3/min/m^2$ and capacity of ion exchanger as 90 $kg/m^3/day$.

Volume of ion exchanger $= \frac{Q * TH}{\text{Capacity of exchanger.}}$

$= \frac{1 * 150}{90} = 1.667\ m^3$

C/s area of ion exchanger $= \frac{Q}{\text{rate of flow}} = \frac{1 \times 10^3/24 \times 60}{0.4.}$

$= \frac{1.736}{2.5}\ m^2.$

Height of ion exchanger $= \frac{\text{Volume}}{\text{C/s area}} = \frac{1.667}{1.736} = 0.96\ m$

Diameter of ion exchanger $= \sqrt{\frac{1.736 \times 4}{\pi}}$

$= 1.487\ m$

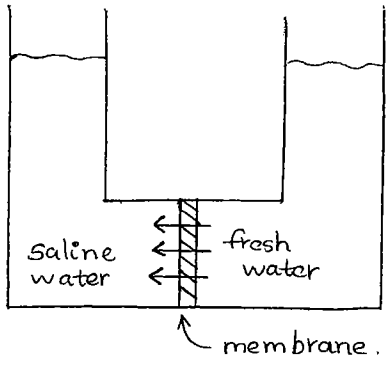
→ Desalination of Water

- convertion of saline water into fresh water

* Desalination done by :-

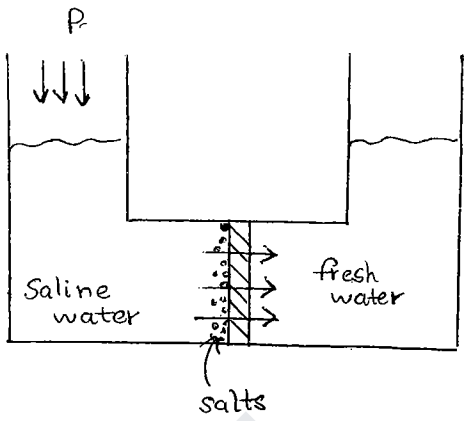
1. Reverse Osmosis. } membrane seperation techniques.
2. Electrodialysis. }
3. Distillation.

In normal osmosis, flow is from fresh water to saline water (low conc. to high conc.). Saline water offers more resistance.



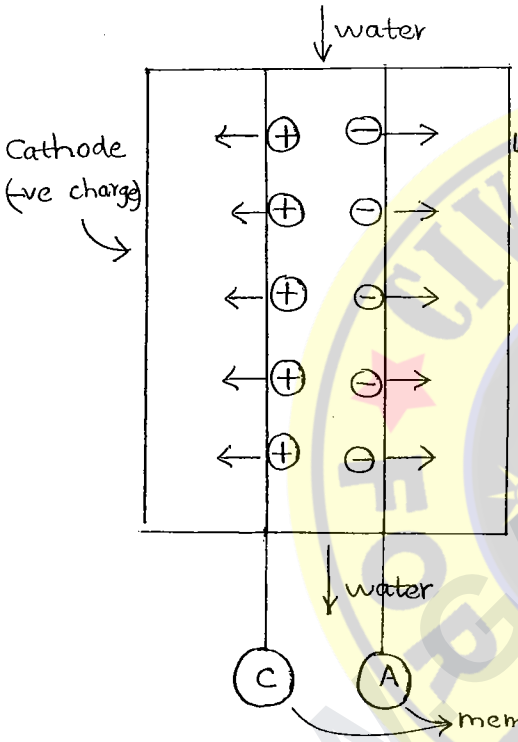
Osmosis

When the applied pressure is greater than osmotic pressure, reverse osmosis takes place and flow is from saline water to fresh water and salts get deposited on the membrane.



■ Reverse Osmosis

* Electrodialysis.



Cation (+ve charge) gets collected in cathode. (-ve charg. and anion (-ve charged ions) gets collected in anode.

* Distillation.

Water $\xrightarrow{\text{vapourized}}$ Vapours.

Vapour $\xrightarrow{\text{condensed.}}$ H_2O (pure water)

All the desalination methods are expensive methods.

→ Fluoridation & Defluoridation.

* Fluoridation.

Fluoridation is addition of NaF to water to increase fluorides > 1 mg/L

* Defluoridation.

Removal of fluorides from water if fluorides greater than 1.5 mg/L.

- Defluoridation is carried out:-
  1. Treatment with activated alumina
  2. Nalgonda Technique. (city in Telangana)
     (Alum + Lime + Bleaching Powder)

→ Activated Carbon (Adsorber)

Colour, odour are removed and taste of water can also be improved by treating with activated carbon.

→ Eutrophication

- excess growth of algae in water bodies which leads to death of water bodies. (unfit for use).
- algae imparts green colour, odour and bad taste to water
- to check the growth of algae (to control algae growth), copper sulphate is added to water.

26th Oct,
SUNDAY

09. DISTRIBUTION SYSTEM

Water distribution – Piped water supply.

→ Water Distribution System.

(i) Network of pipes.

(ii) Pumps.

(iii) Distribution reservoirs.

(iv) Valves.

(v) Water meters.

(vi) Fire hydrants.

→ Objective.

To supply water with a minimum residual pressure of 1.6 m to 2m head of water (16 kN/m² to 20 kN/m²

$$h = \frac{P\ (kN/m^2)}{\gamma_w} = \frac{P\ (kgf/m^2)}{\rho_w}$$

→ Methods of Supply

1. Continuous flow System
2. Intermittent System.

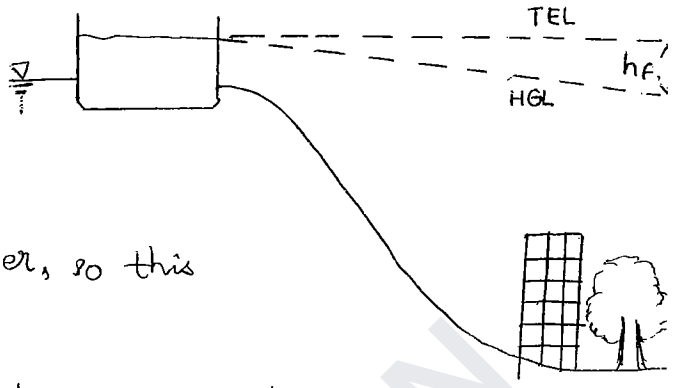
→ Water Distribution Systems.

(i) Gravity System

(ii) Pumping System.

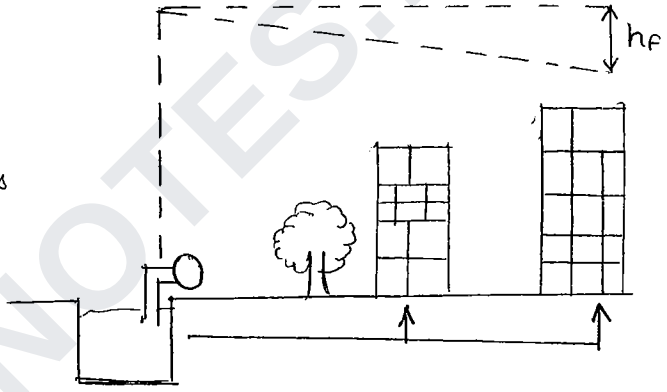
(iii) Gravity & Pumping System.

* Gravity System.



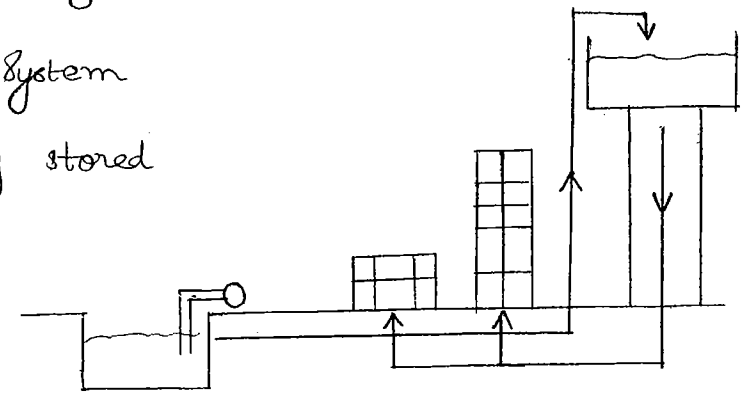
- used when source has some potential energy by virtue of its position.
- does not require any power, so this system is economical.
- supply cannot be increased or decreased based on demands.

* Pumping System



- when source and destinations are at same level.
- head can be varied as per demands.
- depends on power, so its expensive.
- cannot be operated during power failure.

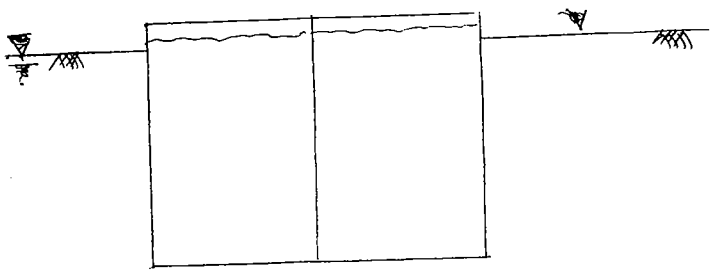
* Gravity & Pumping System



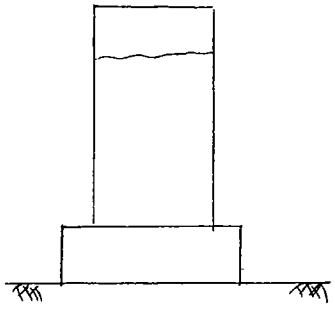
- Excess head is directly stored in a storage reservoir
- continuous supply of water can be ensured even during times of power failure.

→ Reservoirs

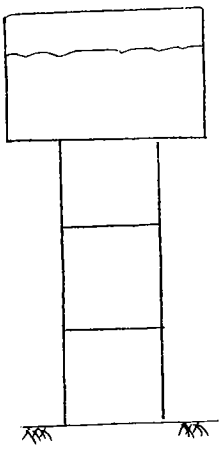
1. Surface Reservoirs

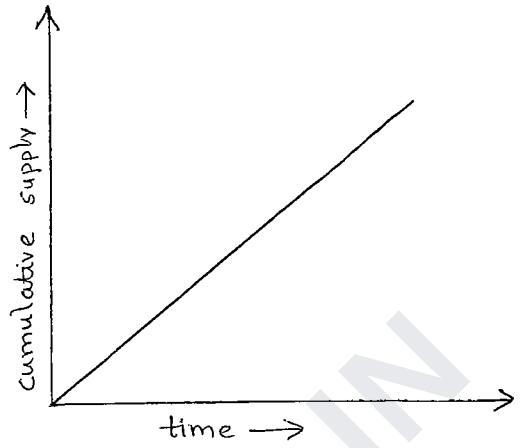


2. Stand Pipes



3. Elevated Reservoirs.





* Fixing the Storage Capacity of Reservoir.

Storage capacity (vol. of space) = Balancing (or) Equalising storage (>91 capacity + Fire storage + Break down (~~or~~) Repair storage.

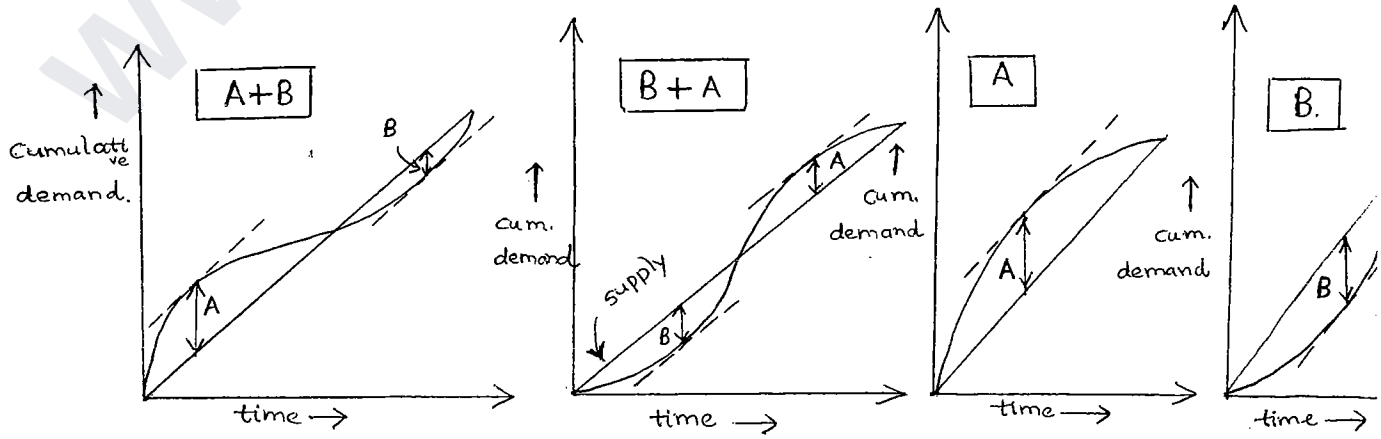
Balancing storage capacity of reservoir is fixed by :-

(i) Mass curve of supply and demand.

(ii) Analytical method.

* Demand Patterns.

(i) Cumulative Demand vs. Time.



6. Q = population per capita demand

$= 10^5 \times 180 = 18$ MLD.

Volume of water to be supplied per day = 18 ML

Rate of water supply = $\frac{\text{vol. of water to be supplied}}{\text{duration of supply}}$

$= \frac{18}{24} = 0.75$ ML/hr ↳ continuous supply (24 h).

Time	0–8	8–16	16–24
Demand %	45	25	30
Demand (in ML)	$\frac{45}{100} \times 18 = 8.1$	4.5	5.4
Supply (in ML)	6 (=0.75×8)	6	6
Cumulative demand	8.1	12.6	18
Cumulative supply (in ML)	6	12	18
Deficit (in ML)	2.1	0.6	0
Surplus (in ML)	0	0	0

∴ Balancing storage capacity = 2.1 ML

7. $Q = \frac{12}{24}$ MLD.

Volume of water to be supplied = 12 × 1 = 12 ML

Supply: 5 AM – 8 AM

4 PM – 7 PM.

Rate of water supply = $\frac{\text{volume}}{\text{duration}} = \frac{12}{6} = 2$ ML/hr.

Time	12 – 6 (am)	6 – 12 (am – pm)	12 – 6 (pm)	6 – 12. (pm – am)
Demand %	15	40	20	25
Demand	1.8	4.8	2.4	3.
Supply	2	4	4	2
Cumulative Demand.	1.8	6.6	89	12.
Cumulative supply	2	6	10	12
Deficit.	0	(0.6)	0	0
Surplus.	0.2	0	(1)	0

Equalising average capacity of reservoir = max. deficit + max. surplus.

= 0.6 + 1 = 1.6 ML

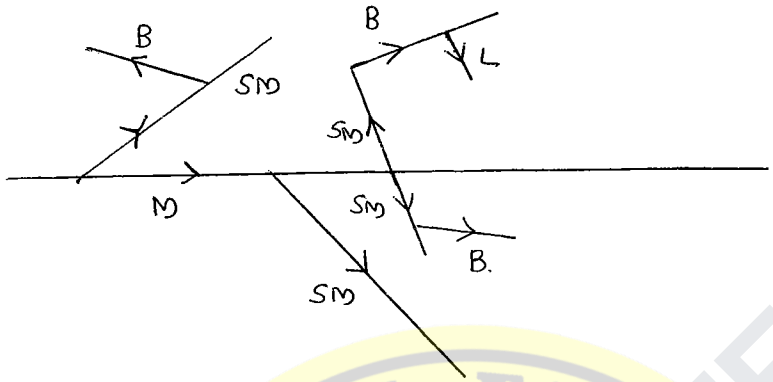
→ Lay out of Water Distribution (Arrangement of Pipes)

Main → Submain → Branch → Lateral → House connection.

1. Dead End system.
2. Grid Iron system.
3. Circular System.
4. Radial System.

* Dead end System. (tree system)

- communities which have grown in haphazard manner, this layout is most suitable.



- advantages :-

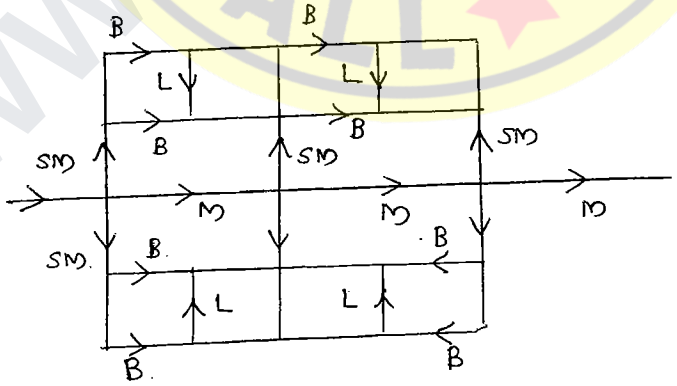
(i) Economical.
(ii) Analysis and design is simple
(iii) Future expansion is easy.

- disadvantages :-

(i) chances of stagnation
(ii) water supply gets interrupted during repairs

* Grid Iron System. (Interlaced (or) Raticulation)

- suitable for made communities like Chandigarh, Bubhaneswar, Gandhinagar etc.



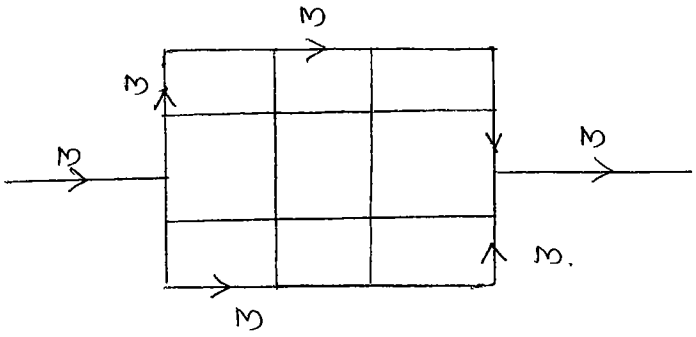
- advantages :-

(i) No stagnation (no dead ends)
(ii) every area supplied by more than one route
(iii) large volume of water is available.

- disadvantages :-

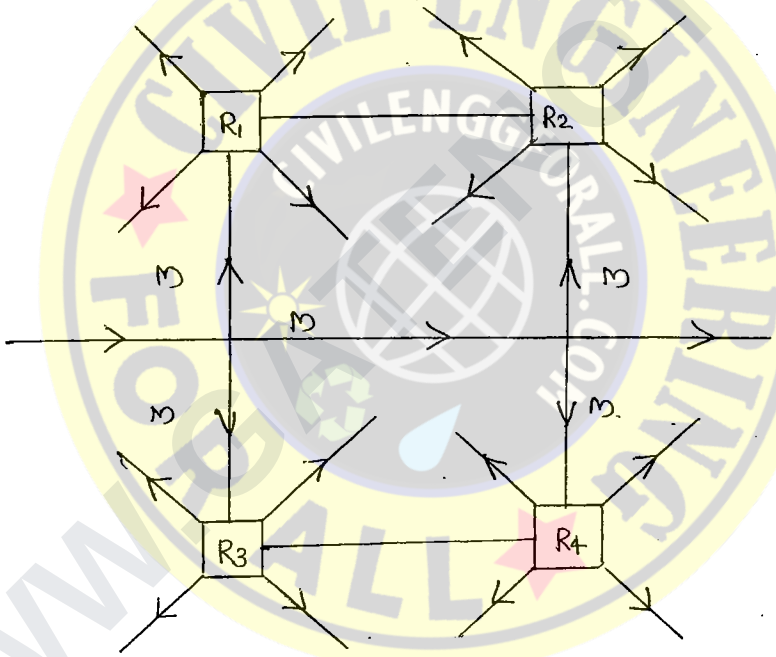
(i) Analysis is complicated.
(ii) Future expansion is difficu
(iii) Expensive

* Circular (or) Ring System.



- similar to grid iron system except than main pipe run along the periphery of the system.

* Radial Distribution (Zonal distribution)



→ Analysis of Distribution Network.

* Analysis

(i) Pressure drop (head loss), 'h_f'

(ii) Flow rate (discharge) in each pipe, 'Q'

⊙ Dead end lay out analysed by (Simple pipe layout).

(i) Darcy Weisbach equation.

(ii) Continuity equation.

⊙ Grid iron, Circular, Radial layouts (complex pipe networks) are best analysed by:

(i) Hardy Cross Method.

→ Hardy Cross Method. (closed complex pipe networks)

- Trial & error procedure.
- Head loss is a function of discharge.

$$\boxed{H \propto Q^x}$$

$$h_f = \frac{fLQ^2}{12.1\, d^5}$$

$$H = kQ^x$$

$$h_f \propto Q^2$$

- Flows are assumed and head losses are worked out for assumed flows. If assumed flows are correct, it has to satisfy the following two conditions:-

(i) At any pipe junction, flow towards junction is equal to flow away from junction.

$$(Q)_{in} = (Q)_{out}$$

(ii) Algebraic sum of head losses around a closed loop must be zero.

$$\Sigma H = 0$$

If above two conditions are satisfied then it is a "Balanced distribution network.". Otherwise assumed flow carry error, δ.

$$\text{Error in assumed flow, } \delta = \frac{-\Sigma H}{x \cdot \Sigma \left|\frac{H}{Q}\right|}$$

Adjust error to the assumed flow & again verify with above

* sign conventions.

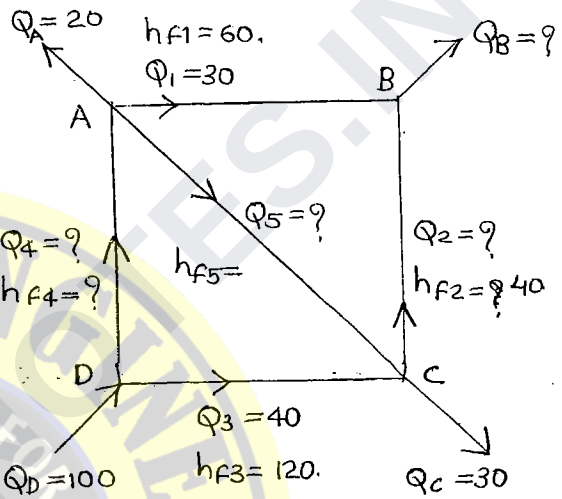
- Clockwise flows and their corresponding head losses are taken as '+ve'.

- Anti clockwise flows and their corresponding head losses are taken as '-ve'.

03.

$Q_3 + Q_4 = Q_D$ (at junction D)

$\therefore Q_4 = 100 - 40 = 60$



$Q_A + Q_1 + Q_5 = Q_4$ (at jn. A)

$20 + 30 + Q_5 = 60$

$Q_5 = 10$

$Q_3 + Q_5 = Q_C + Q_2$ (at jn. C).

$Q_2 = 40 + 10 - 30 = 20$

$Q_1 + Q_2 = Q_B$. (at jn. B).

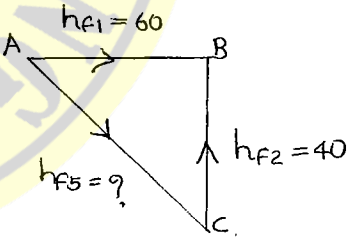
$Q_B = 30 + 20 = 50$

Consider loop ABCA,

$\Sigma H = 0$

$60 - 40 - h_{f5} = 0$

$\therefore h_{f5} = 20$

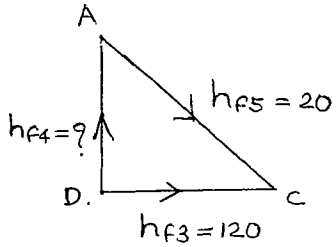


Consider loop ACDA,

$\Sigma H = 0$

$h_{f4} + 20 - 120 = 0$

$\therefore h_{f4} = 100$



$60 - 40 - x = 0.$

$x + 20 - 120 = 0.$

→ Valves

1. Gate valve (or) Sluice valve.

   To regulate and control flow.

2. Check Valve (or) Reflux valve (or) Non return valve.

   Allow flow only in one direction.

3. Pressure Relief Valve.

   To relief high pressure.

4. Air valve.

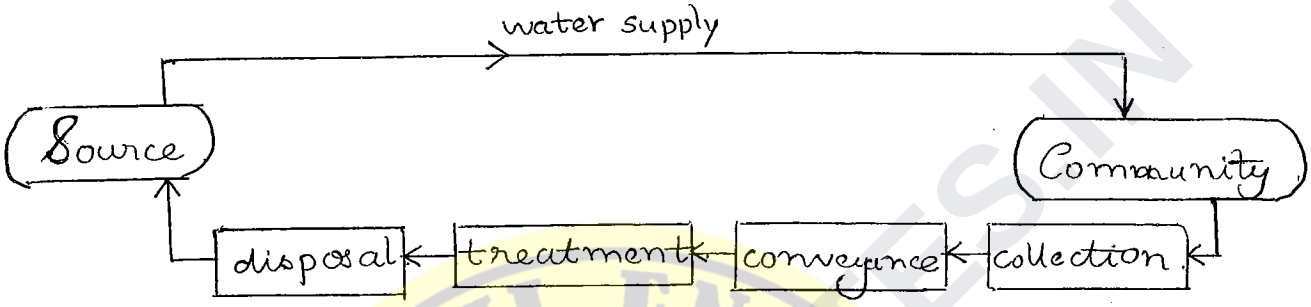
   To expel air from pipe and to admit air into the pipe

5. Scour Valve.

   To empty pipes before cleaning.

27th Oct,
MONDAY

WASTE WATER ENGINEERING



SEWERAGE SYSTEM.
(Sanitary Scheme)

→ Objectives:-

(i) To quickly drain waste water away from community (to prevent break up of water borne diseases).

(ii) To make waste water fit to dispose.

→ Terms:

(i) Sewage – All kinds of liquid waste produced by the community (includes domestic waste, industrial waste, storm water etc)

(ii) Sullage – Less offensive liquid waste produced by the community. (includes waste water from bathrooms, washbasins kitchen sink & storm water)

(iii) Sewer – a conduit which carries sewage

(iv) Sewerage – network of sewers used in collection, conveyence and disposal of sewage.

(v) Dry weather flow – DWF

Domestic and industrial waste water produced by the community.

(vi) Wet Weather Flow :- (WWF)

Storm water resulting from the community.

(vii) Human excreta (or) night soil (or) fecal matter

Human waste.

→ Methods of Sewage Collection & Disposal.

1. Dry and conservancy System.
2. Water Carriage System.

→ Types of Sewerage System.

1. Combined System

- only one sewer is employed which collects both DWF & WWF.
- suitable for communities which receive moderate uniform rains spread for short period.

2. Seperate System.

- two sewers are employed; one sewer carry only DWF and the other one carry WWF alone
- employed in areas which receive high intensity rains and rainfall season spread for long period.

3. Partially Seperated & Partially Combined system

- two sewers are employed; under normal rainfall conditions it act as seperate system. During heavy rainfall period, it act as a combined system.
- suitable for areas which receive moderate intense rainfalls with occasional heavy rains

01. ESTIMATION OF DWF & WWF

→ Dry Weather Flow :

$$Q_{DWF} = \text{population} * \text{per capita sewage flow.}$$

Per capita sewage flow = per capita water supply * factor.

Factor is to account for percent of water supply converted into waste water.

NOTE:

70 to 80% water supplied to community becomes waste water and reach sewer as sewage. (factor = 0.7 to 0.8)

$$Q_{DWF} = \text{population} * \text{per capita water supply} * \text{factor.}$$

→ Wet Weather Flow:

To estimate stormwater from urban areas (urban catchments), rational formula is used.

$$Q_{WWF} \ (m^3/s) = \frac{AIR}{360}$$

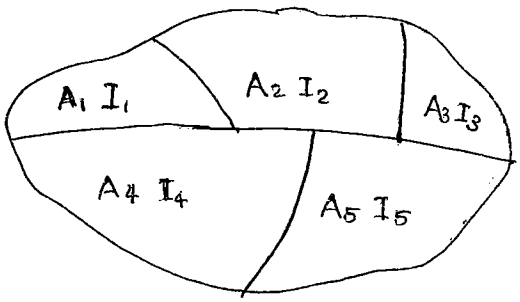
where A → area of catchment. (in ha)

I → impermeability (coeffecient) factor (or) Run off factor.

$$I = \frac{\text{run off}}{\text{rainfall.}}$$

R → rainfall intensity (in mm/hr)

$$I = \frac{\Sigma A_i I_i}{A}$$



$$R = \frac{\text{depth of rainfall}}{\text{duration of rainfall.}}$$

$$\boxed{R = \frac{25.4\,a}{t_c + b}}$$

where $t_c \rightarrow$ time of concentration (min)

a & $b \rightarrow$ constants whose values depend on duration of storm, 't'

If $t = 5$ min to 20 min; $a = 30$ & $b = 10$

$t >$ 20 min; $a = 40$ & $b = 20$

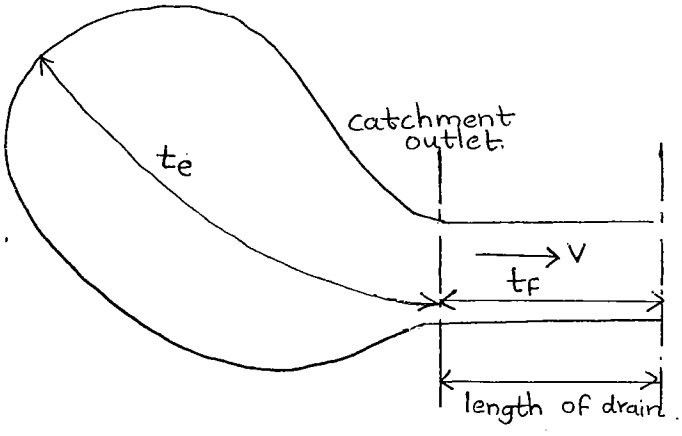
⊙ If duration of storm 't' is not given, then it is taken as time of concentration 't_c' (ie $t = t_c$)

$$\boxed{t_c = t_e + t_f}$$

$t_e \rightarrow$ time of entry (min)

$t_f \rightarrow$ time of flow. (min)

$$t_f = \frac{\text{distance travelled in drain}}{\text{velocity of flow in drain.}}$$



2. $A = 100$ ha, $I = 0.5$, $t_c = 50$ min.

$$R = \frac{25.4\,a}{t_c + b} = \frac{25.4 \times 40}{50 + 20} = 14.51 \text{ mm/hr} \quad (t > 50;\ a = 40,\ b = 20).$$

$$Q_{WWF} = \frac{AIR}{360} = \frac{100 \times 0.5 \times 14.51}{360} = 2.016 \text{ m}^3/\text{s}$$ 2.016

Q_{DWF} = population * per capita water supply * factor

$$= 10^5 \times 200 \times 0.75 \text{ (L/day)} = 0.1736 \text{ m}^3/\text{s}$$

$$Q_{WWF} = 11.61\ Q_{DWF} \quad (Q_{WWF} >> Q_{DWF}).$$

3. $t_e = 10$ min; $t_f = \frac{1}{r} = \frac{2400}{1} = 40$ min

$$t_c = t_e + t_f = 50 \text{ min}.$$

$$R = \frac{25.4 \times 40}{50 + 20} = 14.51$$

$$Q_{WWF} = \frac{50 \times 0.8 \times 14.51}{360} = 1.612 \text{ m}^3/\text{s}$$

4. $Q_{DWF} = \frac{50000 \times 270 \times 10^{-3}}{86400} = 0.15625 \text{ m}^3/\text{s} \times 0.75 = 0.1172 \text{ m}^3/\text{s}$

$t_c = 20 + 5 = 25$ min.

$$R = \frac{25.4 \times 40}{25 + 20} = 22.578 \text{ mm/hr}$$

$$Q_{WWF} = \frac{150 \times 0.45 \times 22.578}{360} = 4.233 \text{ m}^3/\text{s}$$

Total $Q = Q_{DWF} + Q_{WWF} = 4.350$. (combined sewer)

$$A = \frac{Q}{V} = \frac{4.350}{3.2} = 1.359 \text{ m}^2 \quad \{\text{for full flow}\}$$

Diameter of sewer = 1.316 m

5. $Q_{DWF} = \frac{40000 \times 120 \times 0.7}{1000 \times 3600 \times 24} = 0.039 \ m^3/s$

$t_c = 40 \ min.$

$$R = \frac{25.4 \times 40}{40 + 20} = 16.933 \ mm/hr.$$

$$Q_{WWF} = \frac{AIR}{360} = \frac{75 \times 0.7 \times 16.933}{360} = 2.47 \ m^3/s.$$

Discharge for combined sewer $= Q_{DWF} + Q_{WWF}$

$= 0.039 + 2.47.$

$= 2.508 \ m^3/s$

1 ha	$= 10^4 \ m^2$
1 km^2	= 100 ha
1 acre	$= 4046.8 \ m^2$

$10^6 m^2 = 1 \ km^2$

6. $R = \frac{1.2 \times 10}{24} = 0.5 \ mm/hr.$

$A = 1 \ km^2 = 10^6 \ m^2.$

$= 100 \ ha.$

Population $= 1000 \times 100$

$= 10^5$

Q_{DWF} = population * per capita water supply * factor.

$$= \frac{10^5 * 200 \times 10^{-3} \times 0.8}{3600 \times 24} = 0.185 \ m^3/s.$$

$$Q_{WWF} = \frac{AIR}{360} = \frac{100 \times 0.5 \times 1}{360} = 0.1388 \ m^3/s$$

30th Oct,
WEDNESDAY

02. DESIGN OF SEWERS

→ Sewers are designed to :

(i) fix the cross section of sewer.

(ii) finding slope (or) gradient.

⊙ Sewers are designed as open channels though closed conduits are used.

⊙ To design sewers following equations are used :-

a) Continuity equation.

$$Q = AV$$

b) Manning's Formula

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

- Sewers always carry partial flow. ie empty space is provided inside the sewer to accomodate unexpected, unforeseen, incidental sewage flow. That space can also be used to accomodate gases evolved temporarily.

- Circular geometric cross sections are preferred to other geometric c/s because they are hydraulically more efficient.

- While designing the sewers, flow velocity should not be too high and not too low. Velocity of flow in the sewer should be maintained blw non scouring velocity to self cleansing velocity.

Self-Cleansing Velocity:

It is the minimum flow velocity at which no silting or settling of particles occur.

$$V_{self} = \sqrt{\frac{8K}{f}(S-1)gd}$$ (0.6 m/s to 0.9 m/s)

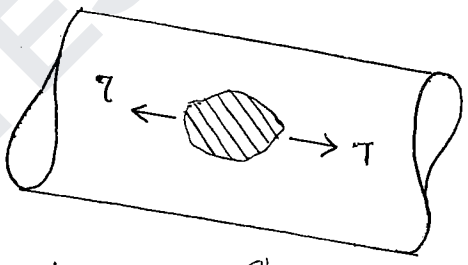
where K → character of sediment (constant)

f → friction factor

S → sp. gravity of particles.

g → acceleration due to gravity.

d → diameter of particles



Inactive force, T ≥ Shear resistance, τ

Non Scouring Velocity:

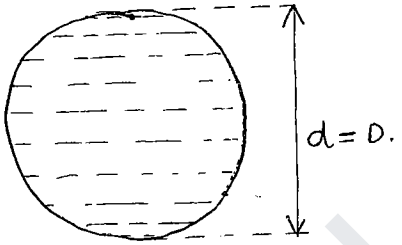
It is the max. flow velocity at which there is no scouring of sewer interiors.

Type of Sewer	Non-scouring velocities (m/s)
Earthen sewers	0.6 – 1.2
Brick lined sewers.	1.5 – 2
Concrete (or) RCC sewers.	2.5 – 3
Stone ware sewers.	3.5 – 4.
CI sewers.	3.5 – 4.5
Vitrified glazed sewers.	4 to 5.0

Case I : Sewers running full. ($d = D$)

$d \rightarrow$ depth of flow.

$D \rightarrow$ diameter of sewer.



$$Q = AV. \quad \rightarrow (1)$$

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

$$R = \frac{A}{P} = \frac{\frac{\pi}{4} D^2}{\pi D.} = \frac{D}{4.}$$

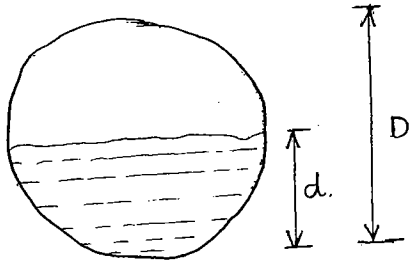
$$Q = \frac{\pi}{4} D^2 . \frac{1}{n} \left(\frac{D}{4}\right)^{2/3} S^{1/2} \rightarrow (2)$$

Solving (1) & (2), D & S are obtained.

Case II : Sewers running <u>half full</u>. ($d = \frac{D}{2}$)

$$A = \frac{\pi}{8} D^2$$

$$R = \frac{\frac{\pi}{8} D^2}{\frac{\pi D}{2}} = \frac{D}{4}$$



$$V = \frac{1}{n} R^{2/3} S^{1/2} = \frac{1}{n} \left(\frac{D}{4}\right)^{2/3} S^{1/2}$$

$$Q = AV.$$

NOTE :

- Sewers with given diameter and slope will generate same flow velocities in half full and full flow conditions. But as area of flow is different, discharge will be different.

Case III : Sewer running less than half full. $(d < \frac{D}{2})$

$$Q = AV.$$

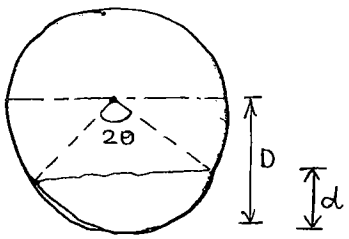
$$A = r^2\left(\theta - \frac{\sin 2\theta}{2}\right)$$

$$P = 2r\theta$$

$$R = \frac{A}{P}$$

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

$$Q = AV.$$



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

Case IV : Sewer running more than half full. $(d > \frac{D}{2})$

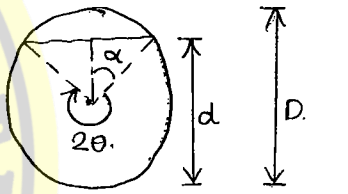
$$\theta = 180 - \alpha$$

$$A = r^2\left(\theta - \frac{\sin 2\theta}{2}\right)$$

$$P = 2r\theta$$

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

$$Q = AV$$



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

P-70

3. $D = 1$ m, $S = \frac{1}{1000}$, $v = 1$ m/s. (full flow).

$\Rightarrow$ half full condition also, $v = 1$ m/s

5. $D = 0.45$ m, $n = 0.012$,

$$Q = \frac{30000 \times 150 \times 0.8 \times 3.5}{1000 \times 86400} = 0.146 \text{ m}^3/\text{s}$$

$$V = \frac{Q}{A} = \frac{0.146}{\frac{\pi}{4} \times 0.45^2} = 0.92 \text{ m/s}$$

$$0.92 = \frac{1}{0.012} \times \left(\frac{0.45}{4}\right)^{2/3} \times S^{1/2}$$

$$\Rightarrow \quad S = 2.23 \times 10^{-3} = \frac{1}{448}$$

06. $D = 1.25\ m$, $S = \frac{1}{360}$, $n = 0.011$.

$$Q = \frac{\pi}{8} D^2 \times \frac{1}{n} \left(\frac{D}{4}\right)^{2/3} \times (S)^{1/2}.$$

$$= \frac{\pi}{8} \times 1.25^2 \times \frac{1}{0.011} \times \left(\frac{1.25}{4}\right)^{2/3} \times \left(\frac{1}{360}\right)^{1/2}$$

$$= 1.354\ m^3/s$$

07. $$V_{self} = \sqrt{\frac{8K}{f}(s-1)gd} = \sqrt{\frac{8 \times 0.1}{0.03}(2.65-1) \times 9.81 \times 0.001}$$

$$= 0.657\ m/s$$

$$V = \frac{1}{n}\left(\frac{D}{4}\right)^{2/3} S^{1/2}.$$

$$0.657 = \frac{1}{0.013} \times \left(\frac{1}{4}\right)^{2/3} \times S^{1/2}$$

$$S = 4.630 \times 10^{-4} = \frac{1}{2159.7}$$

08 $$V = \frac{1}{n}\left(\frac{D}{4}\right)^{2/3} S^{1/2}$$

$$= \frac{1}{0.013} \times \left(\frac{D}{4}\right)^{2/3} \times \left(\frac{1}{1000}\right)^{1/2}$$

$Q = 0.05$ cumecs.

$A = \frac{\pi}{4} d^2$.

$$\Rightarrow \quad 0.05 = \frac{\pi}{4} D^2 \times \frac{1}{0.013} \times \left(\frac{D}{4}\right)^{2/3} \times \left(\frac{1}{1000}\right)^{1/2}.$$

$$D = \underline{0.36 \text{ m.}}$$

Given $\frac{d}{D} = 0.6 \Rightarrow \frac{q}{Q} = 0.54 \; ; \; \frac{v}{V} = 0.88$

$$q = 0.54 \times Q = 0.54 \times 0.05$$

$$= \underline{0.027 \text{ cumecs}}$$

$$v = 0.88 \times \frac{1}{0.013} \times \left(\frac{0.36}{4}\right)^{2/3} \times \left(\frac{1}{1000}\right)^{1/2}$$

$$= \underline{0.429 \text{ m/s}}$$

12 $V \propto D^{2/3} S^{1/2}$

$$0.7 \frac{V_1}{V_2} = \frac{0.0028}{0.05}$$

$$V_2 = \underline{1.562 \text{ m/s}}$$

$$\frac{V_1}{V_2} = \left(\frac{D_1}{D_2}\right)^{2/3} \left(\frac{S_1}{S_2}\right)^{1/2}$$

$$\frac{0.7}{V_2} = \left(\frac{D_1}{2D_1}\right)^{2/3} \times \left(\frac{S_1}{25_1}\right)^{1/2}$$

$$V_2 = \underline{1.571 \text{ m/s}}$$

09. $D = 300 \text{ mm}$, $S = \frac{1}{280}$, $q = 1728 \text{ m}^3/\text{day}$, $n = 0.015$

$$= 0.02 \text{ m}^3/\text{s}$$

$$Q = AV.$$

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

$$= \frac{\pi}{4} \times 0.3^2 \times \frac{1}{0.015} \left(\frac{0.3}{4}\right)^{2/3} \left(\frac{1}{280}\right)^{1/2}$$

$$= 0.05 \ m^3/s$$

$$\frac{q}{Q} = \frac{0.02}{0.05} = 0.4$$

From the graph,

$$\frac{d}{D} = 0.5.$$

$\therefore$ Depth of flow, $d = 0.5 \times 0.3 = 0.15$ m.

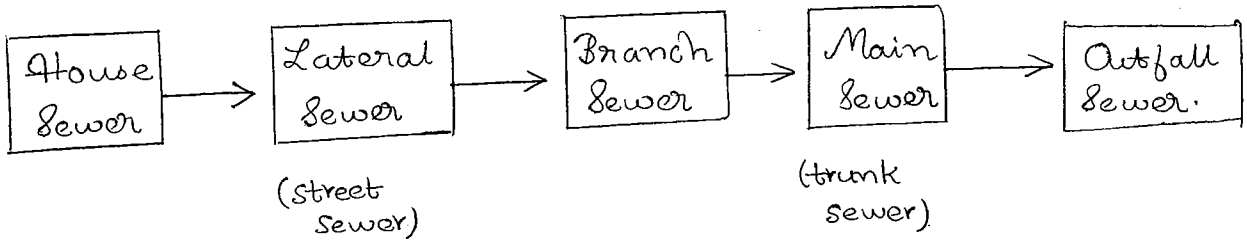
For $\frac{d}{D} = 0.5$,

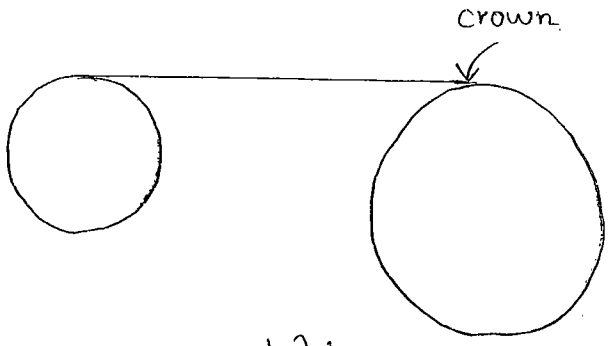
$$\frac{V}{V_{full}} = 0.8.$$

$$V_{full} = \frac{Q}{A} = \frac{0.05}{\frac{\pi}{4} \times 0.3^2} = 0.707 \ m/s.$$

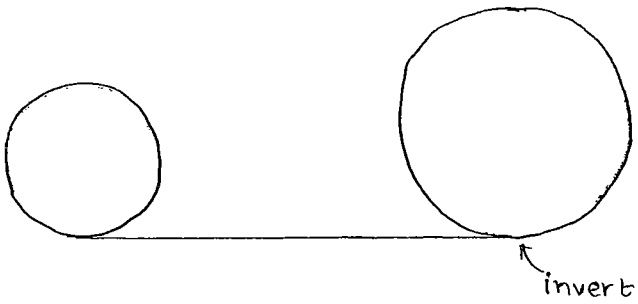
$$\Rightarrow V = 0.8 \times 0.707 = \underline{\underline{0.57}} \ m/s$$

→ Sewerage

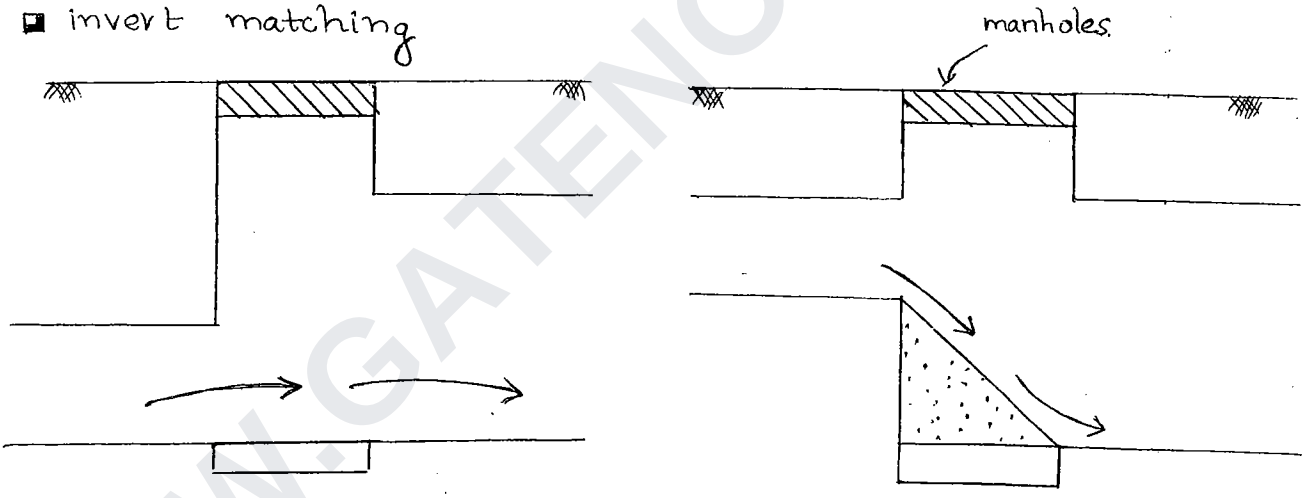




- crown matching



- invert matching



- manholes are provided at regular intervals along the alignment of sewerage system.
- Manholes facilitate maintenance, repair works and service. In addition it is provided wherever sewer change its direction, elevation.

2rd Nov,
MONDAY

03. CHARACTERISTICS OF SEWAGE

Before treating the waste water and disposing the waste, it is necessary to know the nature and character of waste water. Based on this treatment system and disposal systems are designed.

To know the nature and character of waste water it is necessary to examine the sewage.

Sewage consist 99.9% water and 0.1% solids. A small portion of solids present in sewage undergo changes known as decomposition causing nuissance to man and his surroundings. To know the nuissance potential of a sewage, they are thoroughly examined by conducting tests.

Decomposition (Degradation / Stabilisation) → Oxidation + Reduction.

Oxidation → aerobic decomposition. (in presence of air)

Reduction → anaerobic decomposition (in absence of air)

1th nov,
TUESDAY

To know the nuisance potential of sewage, it is examined in the laboratory.

1. Physical Examination.
2. Chemical Examination.

→ Physical Examination of Waste water

To know the state and condition of waste water, physical examination is carried out.

1. Colour

Fresh sewage → Yellow to light brown.

Stale & septic sewage → dark brown to black.

2. Odour

Fresh sewage → Musti odour.

Septic & Stale sewage → Foul odour.

3. Turbidity

Fresh sewage → less turbid.

Septic & Stale sewage → more turbid.

4. Temperature

Fresh sewage → $< 30^{\circ}C$ (room temperature)

Septic & Stale sewage → $> 30^{\circ}C$

→ Chemical Examination of Waste Water

It is done to determine :

(i) Organic matter

(ii) Inorganic substances.

(iii) Dissolved gases.

The organic content present in waste water measured by estimating the oxygen demand of the waste. Organic content cannot be estimated by straight testing procedures because it is composed of carbohydrates, proteins, oils, glucose etc

Following oxygen demand tests are conducted on waste water to know its organic strength:-

(i) BOD — Biochemical Oxygen Demand.

(ii) COD — Chemical Oxygen Demand.

(iii) TOD — Total Oxygen Demand.

(iv) Th.OD — Theoretical Oxygen Demand.

→ BOD – Biochemical Oxygen Demand.

– BOD is defined as the amount of oxygen demanded by the micro organisms (aerobic bacteria) to decompose biodegradable organic matter present in waste water under aerobic conditions.

– BOD is the measure of strength of waste water

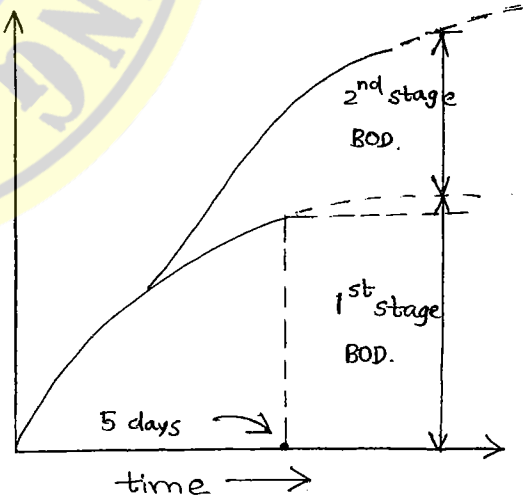
Nuisance potential. ∝ Organic matter ∝ Strength of waste ∝ BOD.

– 1st stage BOD:

carbonaceous organic matter oxidised.

– IInd stage BOD:

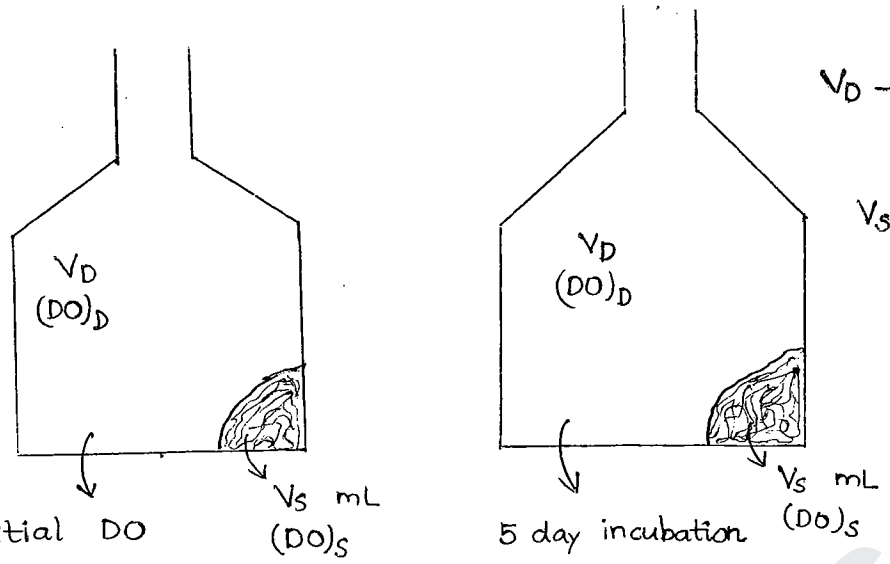
Nitrogeneous organic matter oxidised.



– within 5 days, 60 to 70% of organic content is oxidised. Therefore 5 day BOD is taken as standard reference.

– BOD test is conducted at 20°c for 5 days.

∴ 5 day BOD @ 20°c is taken as a strength parameter of waste water. ⇒ $y_5^{20^\circ c}$



$V_D \rightarrow$ vol. of distilled water

$V_S \rightarrow$ vol. of sewage

5 day BOD @ 20°C $= \left[(DO)_{initial} - (DO)_{final}\right] *$ dilution factor.

$$\text{Dilution factor} = \frac{300}{V_S} = \frac{100}{\%\ \text{dilution}}.$$

$$(DO)_{initial} = (DO)_{mix} = \frac{V_D (DO)_D + V_S (DO)_S}{V_D + V_S}$$

Distilled water = Blank Sample

$$(DO)_D = (DO)_B$$

$$y_5^{20^\circ C} = \left((DO)_I - (DO)_F\right) * DF = \left((DO)_B - (DO)_F\right) * DF - \left((DO)_B - (DO)_S\right)$$

12th nov, WEDNESDAY

P-79

Q.16 5 day BOD @ 20°C $= \left((DO)_I - (DO)_F\right) * DF$

$$DF = \frac{100}{2}$$

$$\therefore \quad y_5^{20^\circ C} = (8.5 - 5.5) * \frac{100}{2} = 150 \text{ mg/L}$$

19. $V_S = 2$ mL

$$y_5^{20^\circ C} = (8-2) * \frac{300}{2} = \underline{900 \text{ mg/L}}$$

3. DF = 100.

$(DO)_I = 7$ mg/L

$(DO)_F$ = zero. $\Rightarrow$ test is failure. (increase the DF).

Failed in estimating the BOD.

9.

V_S (mL)	Initial DO (mg/L).	Final DO (mg/L).	DF	$y_5^{20^\circ C}$.
5	9.2	6.9	60	138
10	9.1	4.4	30	141
50	8.4	0.0	6.	can't be determined. (test failed).

$$\text{Average BOD} = \frac{138 + 141}{2} = \underline{139.5 \text{ mg/L}}$$

2. $(DO)_B = 8.8$ mg/L $\quad (DO)_S = 0.8$ mg/L

$(DO)_F = 3.8$ mg/L $\quad DF = \frac{100}{5}$

$$y_5^{20^\circ C} = [(DO)_B - (DO)_F] * DF - ((DO)_B - (DO)_S)$$

$$= (8.8 - 3.8) \times \frac{100}{5} - (8.8 - 0.8) = \underline{92 \text{ mg/L}}$$

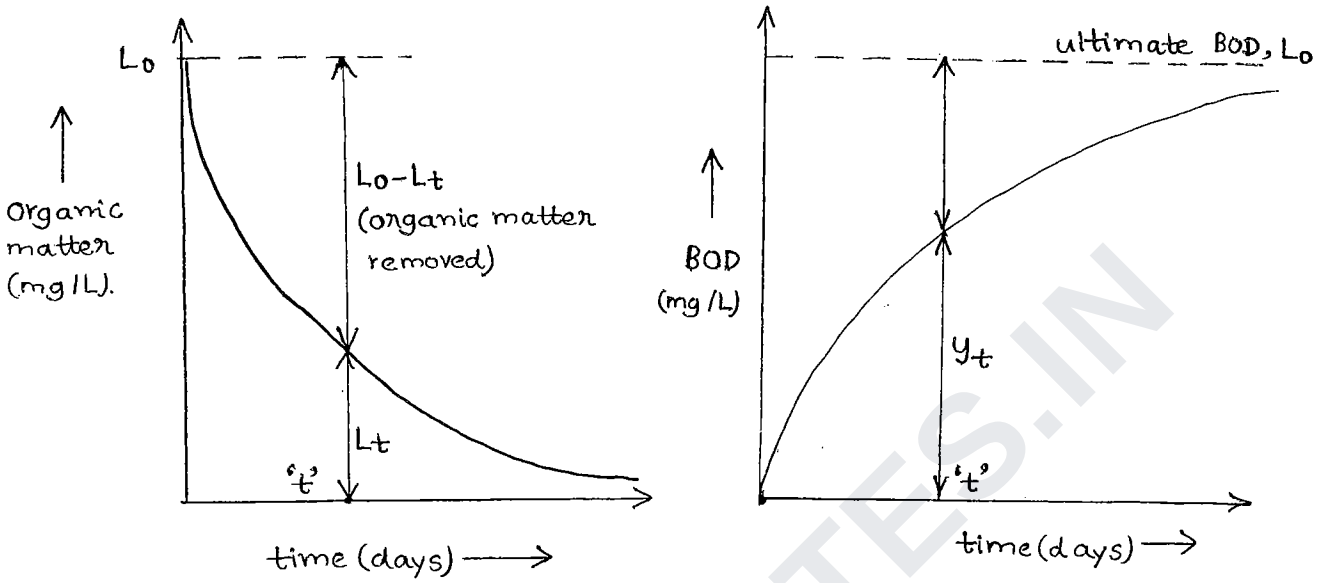
(OR)

$$(DO)_1 = (DO)_{mix} = \frac{285 \times 8.8 + 15 \times 0.8}{285 + 15} = 8.4 \text{ mg/L}$$

$$y_5^{20^\circ C} = [(DO)_I - (DO)_F] * DF$$

$$= (8.4 - 3.8) \times \frac{300}{15} = \underline{92 \text{ mg/L}}$$

→ Mathematical Expression for BOD (or) BOD Model.



Let initial organic matter in the waste waster be 'Lo'. and 'Lt' be the organic matter remaining at any time.

$$\boxed{\frac{dL}{dt} \propto L_t}$$

$$\frac{dL}{dt} = -kL_t.$$

$$\int \frac{dL}{L_t} = \int -k\,dt$$

$$\ln(L_t) = -k(t) + c.$$

Applying boundary conditions:

(i) At $t = 0$, $L_t = L_o$

$$\ln(L_o) = c.$$

$$\therefore \quad \ln(L_t) = -kt + \ln(L_o).$$

$$\Rightarrow \quad \frac{L_t}{L_o} = e^{-kt}$$

Organic matter remaining at any time t,

$$L_t = L_o e^{-kt}$$

$$y_t = L_o - L_t.$$

$$= L_o - L_o e^{-kt}.$$

$$\Rightarrow \boxed{y_t = L_o (1 - e^{-kt})}$$

where K → BOD rate constant.

t → time (in days)

By using two BOD test results, L_o & K_{20} can be evaluated.

$$y_5^{20^\circ c} = L_o (1 - e^{-K_{20} * 5})$$

Using above equation, L_o & K_{20} are obtained.

$$\boxed{K_T = K_{20} (1.047)^{T-20}}$$

K_{20} (base e) $= 2.3 * K_{20}$ (base 10)

BOD @ any time at any temperature, $y_t^{T^\circ c} = L_o (1 - e^{-K_T t})$

01. $y_5^{20^\circ c} = (8.6 - 5.4) \times \frac{300}{6} - (8.6 - 0)$ $\quad DF = \frac{300}{6} = 50$

$= 151.4$ mg/L

$K_{20} = 0.25$/day

$$y_5^{20} = L_o (1 - e^{-K_{20} \times 5}).$$

$$151.4 = L_o (1 - e^{-0.25 \times 5}).$$

Ultimate BOD, $L_o = 212.19$ mg/L

04. $K_{D(20)} = 0.1$ (to base 10).

$K_{D(20)} = 2.3 \times 0.1 = 0.230$ (to base e).

& $K_{D(30)} = K_{20}(1.047)^{T-20}$

$= 0.23(1.047)^{30-20} = 0.364.$

$y_5^{30} = L_0(1 - e^{-0.364 \times 5})$.

$110 = L_0(1 - e^{-0.364 \times 5})$.

$L_0 = 131.27$ mg/L

$y_5^{20} = 131.27(1 - e^{-0.23 \times 5})$.

$= 89.704$ mg/L

05. $y_1^{30} = 110$ mg/L

$y_5^{20} = ?$ $K_{20} = 0.23$

$K_{30} = 0.364.$

$y_1^{30} = L_0(1 - e^{-0.364})$.

$L_0 = 360.53$ mg/L.

$y_5^{20} = 360.53(1 - e^{-0.23 \times 5})$.

$= 246.37$ mg/L

13th nov, THURSDAY

08. $(BOD)_5 = 600$ mg/L ; $K = 0.23$/day.

$y_5^{20} = L_0(1 - e^{-0.23 \times 5})$.

$600 = L_0(1 - e^{-0.23 \times 5})$.

$\Rightarrow L_0 = 878.01$ mg/L

Proportion unoxidised after 20 days $= \frac{L_{20}}{L_0} \times 100$

$= \frac{L_0 e^{-kt}}{L_0} \times 100.$

$= e^{-0.23 \times 20} \times 100 = \underline{1.005\%}$

11. $y_5^{20} = 180$ mg/L , $K_{20} = 0.18$ per day.

$y_{2.5}^{T} = 180$ mg/L ;

$$y_5^{20} = L_0 \left(1 - e^{-0.18 \times 5}\right).$$

$$L_0 = \frac{180}{1 - e^{-0.9}} = 303.321.$$

$$y_{2.5}^{T} = L_0 \left(1 - e^{-K_T \times 2.5}\right).$$

$$180 = 303.321 \left(1 - e^{-K_T \times 2.5}\right).$$

$$\Rightarrow K_T = 0.36$$

$$K_T = K_{20} (1.047)^{T-20}.$$

$$0.36 = 0.18 (1.047)^{T-20}.$$

$$(T - 20) \ln 1.047 = \ln 2.$$

$$T = 20 + 15.09 = \underline{35.09^\circ C}$$

10. $K = 0.01$ /hour $= 0.01 \times 24 = 0.24$ /day.

$$y_5^{20} = L_0 \left(1 - e^{-0.24 \times 5}\right).$$

$$L_0 = \frac{190}{1 - e^{-1.2}} = \underline{271.89} \text{ mg/L}$$

12. $BOD_3 = 75\ mg/L$, $K = 0.345$

$$BOD_3 = L_o\left(1 - e^{-0.345 \times 3}\right).$$

$$L_o = 116.32\ mg/L$$

$$BOD_{10} = 116.32\left(1 - e^{-0.345 \times 10}\right).$$

$$= \underline{112.627}\ mg/L$$

Amount of BOD remaining $= 116.32 - 112.627.$

$$= \underline{3.69}\ mg/L$$

11. $y_5^{20^\circ C} = y_{2.5}^{T^\circ C}.$

$$L_o\left(1 - e^{-K_{20} * 5}\right) = L_o\left(1 - e^{-K_T \times 2.5}\right).$$

$$K_{20} * 5 = K_T \times 2.5.$$

$$K_{20} * 5 = K_{20}(1.047)^{T-20} * 2.5.$$

$$(1.047)^{T-20} = \frac{5}{2.5} = \underline{2}$$

21. $y_5^{20} = 250 = L_o\left(1 - e^{-K_{20} * 5}\right).$

$$y_t^{30} = 250 = L_o\left(1 - e^{-K_{30} * t}\right).$$

$$K_{20} \times 5 = K_{30} * t.$$

$$5 = (1.047)^{30-20} * t.$$

$$t = \underline{3.16}\ days$$

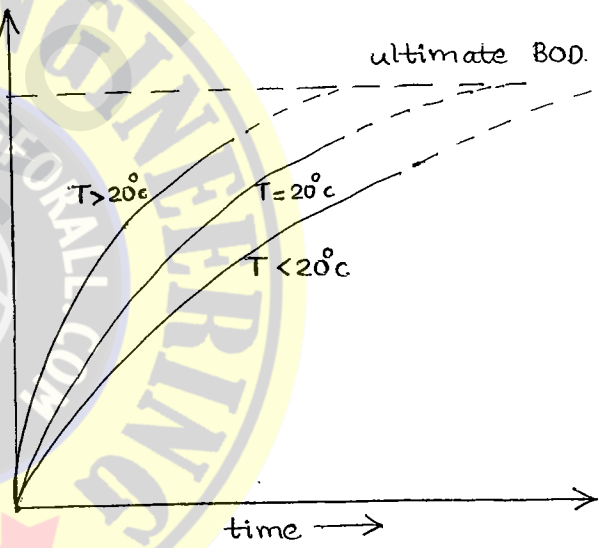
12 Organic matter remain unoxidised = BOD remaining.

$= L_o e^{-kt}$

$= 116.32 \times e^{-0.345 \times 10}$

$= \underline{3.69}$ mg/L

14 When temperature increases, organism growth increases and more oxygen is consumed for the decomposition of organic matter. As a result, with increase in temp, BOD also increases



23. $y_7^{35^\circ c} > y_6^{30^\circ c} > y_5^{20^\circ c}$

$y_5^{20} \simeq y_3^{27^\circ c} \simeq y_4^{30^\circ c}$.

$L_o (1 - e^{-K_{20} * 5}) = L_o (1 - e^{-K_{27} * 3})$

$= L_o (1 - e^{-K_{30} * 4})$

$K_{20} * 5 = K_{27} * 3 = K_{30} * 4$.

$= K_{20} (1.047)^7 = K_{20} (1.047)^{10}$.

$5 = 4.13 = 6.3$

∴ 3 days $27^\circ c$ BOD ≃ 5 days $20^\circ c$ BOD

25. $y_7^{20^\circ c} = 150$ mg/L

$L_o = 187.47$ mg/L

$y_5^{20^\circ c} = 187.47 (1 - e^{-0.23 \times 5}) = \underline{128.113}$ mg/L

→ Population Equivalent.

Total strength of waste water = Total BOD (kg/day).

= $Q_{(MLD)} * y_{(mg/L)}$.

= Population * per capita BOD

$$\text{Population equivalent} = \frac{\text{Total BOD}}{\text{Per capita BOD.}} = \frac{Qy}{\text{per capita BOD}}$$

As per IS code, per capita BOD = 70-90 g/person/day

6. $Q = 80 \times 10^6$ L/d = 80 MLD.

$y_5^{20} = 285$ mg/L

per capita BOD = 75 g/day.

$$\text{Population equivalent} = \frac{80 \times 285}{0.075} = \underline{304000} \text{ persons}$$

18. $Q = 1000\ m^3/day = 1$ MLD.

$y = 162$ mg/L

per capita BOD = 80 g/capita.

$$\text{Population equivalent} = \frac{1 \times 162}{0.080} = \underline{2025} \text{ persons}$$

→ Relative Stability, 'Sr'

$$S_r = \frac{y_t^{T_c}}{L_0} * 100 = \left(1 - e^{-K_T t}\right) 100$$

$T = 20^{\circ}c$, $S_r = (1 - (0.794)^t)\ 100$ $\quad K_{20} = 0.23$

$T = 30^{\circ}c$, $S_r = (1 - (0.695)^t)\ 100.$ $\quad K_{30} = 0.36.$

07. $S_r = \left(1 - e^{-0.23 \times 12}\right) * 100 = 93.67\ \%$

93.67% waste decomposed in 12 days.

1th nov, FRIDAY

COD - Chemical Oxygen Demand.

COD ≥ BOD

It is the amount of oxygen required to decompose biodegradable and non-biodegradable organic matter present in waste water using a strong chemical oxidant such as Potassium Dichromate.

COD is estimated in laboratory by using reflux apparatus (or) digesters.

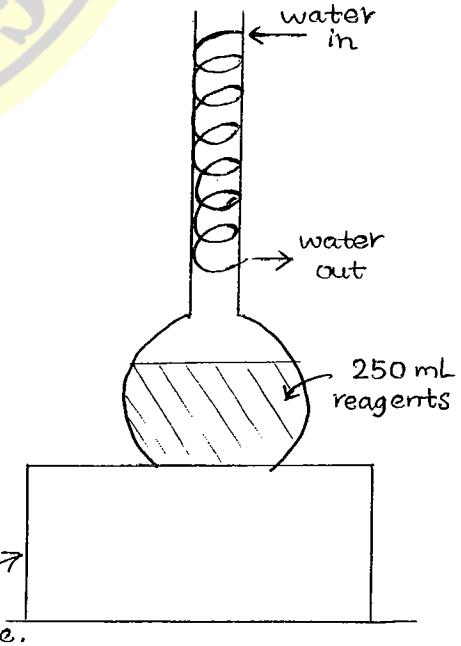
* Reagents :

(i) Potassium dichromate → oxidant

(ii) Mercuric Sulphate → inhibitor

(iii) Silver Sulphate → catalyst.

(iv) Conc. H_2SO_4. → Acidic condition.



⊙ After 2 hours of refluxion (heating), the left over dichromate estimated by titrating with FAS (Ferrous Ammonium Sulphate) using Ferroin indicator.

Colour : ~~Brick red~~ → green → brick red.

⊙ Based on $\frac{BOD}{COD}$ ratio, the nature of the waste water (sewage) can be determined.

(i) $\frac{BOD}{COD} > 0.6$ ⟶ Biodegradable waste

(ii) $\frac{BOD}{COD} < 0.6$ ⟶ Non biodegradable waste (synthetic waste)

(iii) $\frac{BOD}{COD} = 0$ ⟶ Toxic waste. (BOD = 0).

When the waste is of toxic nature, micro organisms are killed and DO remains the same, ∴ BOD = 0.

(iv) BOD = COD = 0 ⟶ devoid of organic matter.

TOD – Total Oxygen Demand.

Amount of oxygen required to oxidise biodegradable non biodegradable and few inorganic substances which are oxidisable present in sewage.

$$\boxed{TOD \geq COD \geq BOD.}$$

Th.OD – Theoretical Oxygen Demand.

It is the oxygen demand estimated by establishing relation among the substances which are oxidisable

Q. Find the ThOD of 70 mg/L of glucose in waste water. Use the following chemical reaction.

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

180 ; 72+12+96 ; 192 ; 6×32

Molecular wt. of glucose = 6×12 + 12 × 6×16 = 180

Molecular wt of oxygen = $6 \times 32 = \underline{192}$

180 parts of glucose demand 192 parts of CO_2

1 part glucose $\rightarrow \frac{192}{180}$ part oxygen.

70 mg/L of glucose $\longrightarrow \frac{192}{180} \times 70 = \underline{74.67}$ mg/L

$\therefore$ Th OD = 74.67 mg/L

$$\boxed{COD \geq Th.OD}$$

14th nov,
FRIDAY

04. TREATMENT OF SEWAGE

Treatment of sewage (waste water) classified into:

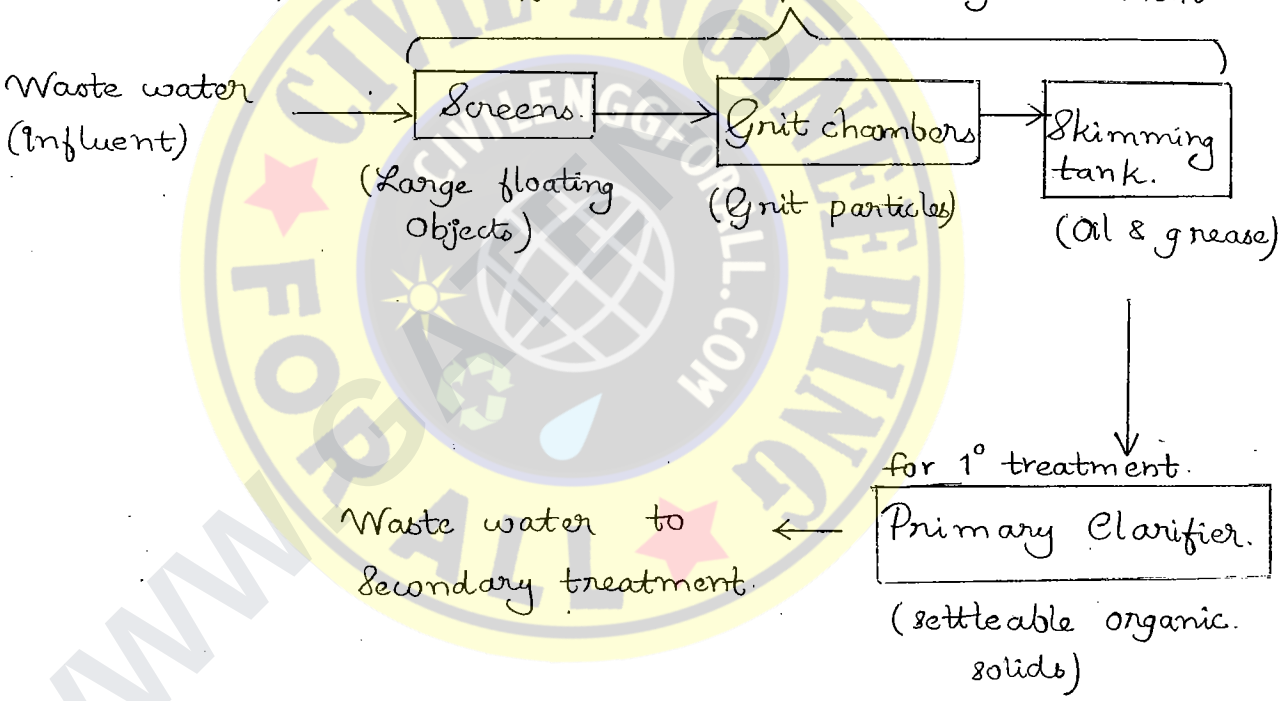
(i) Preliminary Treatment. → preparatory process

(ii) Primary Treatment → settleable organic solids are removed.

(iii) Secondary Treatment → for soluble BOD removal.

(iv) Tertiary Treatment → for recycling & reuse of waste water.

* Preliminary Treatment.



→ Screens

– Screen is the first unit operation employed in treating waste water.

– Screens are aimed to capture large floating objects such as paper, cloth, wooden pieces, cork etc.

– Screen is a device with uniform sized openings made up of parallel bars or rods, provided against flow with some inclination.

* Based on size of openings:

(i) Coarse Screens (size > 50 mm)

(ii) Medium Screens (size : 6mm to 40 mm)

(iii) Fine screens (size : 3mm to 6mm)

* Design of Screens:

- Approach velocity,

$V_a = 0.6 - 1.2$ m/s

- c/s area of screen

$= B \times H = \frac{Q_{in}}{V_a}$

- Depth of flow, H assumed and B is obtained

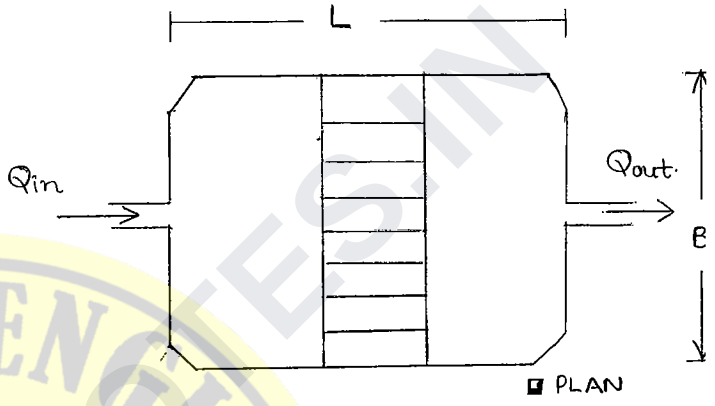
- Let s be the size of openings and n be the number of openings

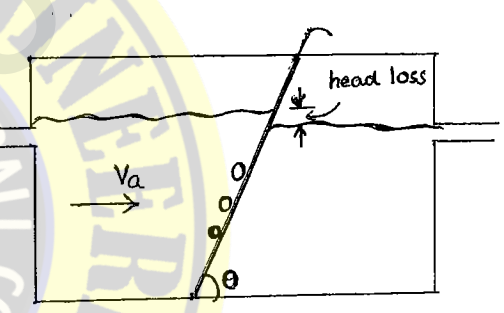
No. of bars $= n+1$

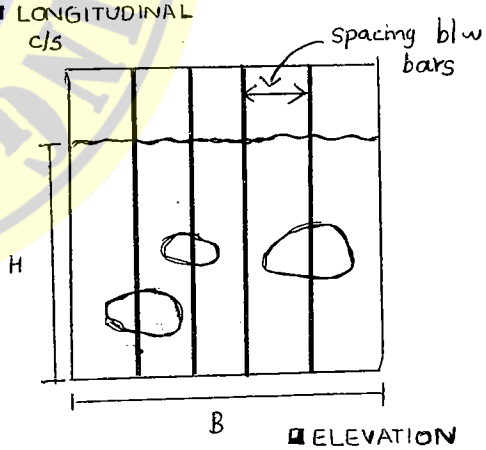
Diameter of bars $= \phi$

$$B = n \times s + (n+1)\phi$$

Normally, $\phi = 10$ mm & $S = 25$ mm



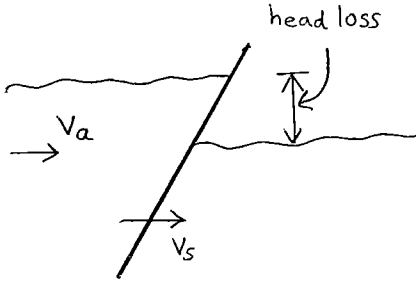


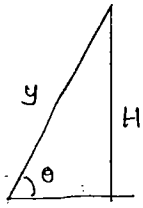


* Head loss through Screens

$$= \frac{V_s^2 - V_a^2}{2g} \times \frac{1}{0.7}$$

$$V_s = \frac{Q}{\text{net area}} = \frac{Q}{n \times s \times \frac{H}{\sin\theta}}$$





$y = \frac{H}{\sin\theta}$

Q. Find the number of bars. $\phi = 10\,mm$, $Q = 25$ MLD, $S = 25\,mm$, $V_a = 1\,m/s$, $H = 0.9\,m$. Also find head loss if $\theta = 60^\circ$ to horizontal.

$$Q = \frac{25 \times 10^6}{10^3 \times 86400} = \underline{\underline{0.2893}}\ m^3/s$$

$$B \times H = \frac{Q}{V_a} = \frac{0.2893}{1} = 0.2893\ m^2$$

$$B = \frac{0.2893}{H} = \frac{0.2893}{0.9} = 0.3215\ m.$$

$$B = ns + (n+1)\phi$$

$$1000 \times 0.3215 = n \times 25 + (n+1)10$$

$$\therefore n = 8.9 \approx \underline{\underline{9}}$$

$$\therefore \text{Number of bars} = n+1 = \underline{\underline{10}}$$

$$V_s = \frac{Q}{\text{net area}} = \frac{0.2893}{9 \times 0.025 \times \frac{0.9}{\sin 60}} = \underline{\underline{1.237}}\ m/s$$

$$\Rightarrow \text{Head loss} = \frac{V_s^2 - V_a^2}{2g} \times \frac{1}{0.7} = 0.038\ m = \underline{\underline{3.8\ cm}}$$

15th nov, SATURDAY

→ Grit Chamber

– Grit chamber is aimed to remove grit from waste water flow. If grit is not removed, it increase wear and tear of pumps and pipe used at treatment plant. Grit also settle down fast and occupy space left for others.

* Grit composed of :

(i) Inorganic grit

imp

size : < 0.2 mm

Sp. gravity : ≥ 2.65

Eg: Sand, silt, clay, cynder, broken glass pieces, metal fragments etc.

(ii) Organic Substance (non biodegradable grit).

size : > 2 mm

Sp. gravity : 1.05 to 1.1

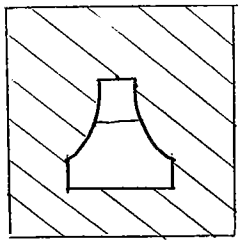
Eg: Coffee grounds, tea grounds, egg shells, bone chips, grains, seeds etc.

* Design of Grit Chamber:

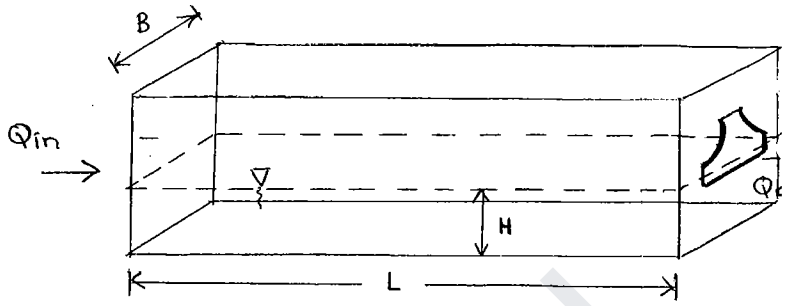
- Grit chamber is a long narrow rectangular channel with increased c/s dimension, deccelerate flow velocity and allow grit to settle down.

- Grit chamber is a settling tank with provision to control flow velocity. A velocity control device known as 'Proportional flow weir' is provided to the outlet of grit chamber to maintain constant flow velocity irrespective of flow rates.

- A 25% increase in flow velocity leads to scouring of settled grit. Similarly a 25% decrease in flow velocity leads to settling of other particles.



c/s @ outlet



- A flow velocity of 0.3 m/s is provided which neither allow scouring nor settling of others other than grit.

$$V_H = 0.3 \text{ m/s} \quad (0.25 \text{ m/s to } 0.4 \text{ m/s})$$

- Detention time = 1 to 3 min
- Volume of grit chamber = $Q * DT$
- c/s area $(B * H) = \frac{Q}{V_H}$
- Length of grit chamber = $V_H * DT$
- Settling velocity, $V_S = \frac{g(S-1)d^2}{18\gamma}$

$$V_S = 60.6(S-1)d\left[\frac{3T+70}{100}\right]$$; exclusively for Grit Chamber.

where $d \rightarrow$ cm

$T \rightarrow {}^{o}C$

$V_S \rightarrow$ cm/s.

- Length = 7.5 m to 20 m
  Width = 2.5 m to 7 m
  Depth = 2 m to 5 m.
- Settled grit is safe and disposed off by any method.

02. $L = 12\,m$, $B = 1.5\,m$, $H = 0.8\,m$; $Q = 720\ m^3/hr$

Surface loading rate, $V_o = \frac{Q}{L \times B} = \frac{720}{12 \times 1.5}$

$= 40\ m^3/hr/m^2$

$= 40,000\ L/hr/m^2$

Detention time, $t = \frac{V}{Q} = \frac{12 \times 1.5 \times 0.8}{720} = 0.02\ hr.$

$= 1.2\ min$

04. $V_H = 0.3\ m/s$ (for grit chamber)

c/s area $= \frac{Q}{V_H} = \frac{3}{0.3} = 10\ m^2$

05. $S = 2.7$, $d = 0.21\ mm$, $\gamma_w = 10^{-2}\ cm^2/s$

$$V_s = \frac{g(s-1)d^2}{18\gamma} = \frac{9.81 \times 10^2 \times (2.7-1)(0.021)^2}{18 \times 10^{-2}}$$

$= 4.08\ cm/s \Rightarrow 2.6 - 5.0$

P-28.

16. $V_H = 0.3\ m/s$, $H = 0.9\ m$, $S = 2.5$, $L = 7.5\ m$.

$\gamma_w = \frac{1.002 \times 10^{-3}}{1000} = 10^{-6}\ m^2/s$

$Q = $ c/s area $\times\ V_H$.

~~$= 7.5 \times 0.9 \times 0.3 = 2.025\ m^3/s$~~

$= B \times 0.9 \times 0.3 = 0.27\,B.$

$$V_o = \frac{Q}{L \times B} = \frac{0.27\ B}{7.5\ B} = 0.036\ m/s$$

$$V_o = V_s = \frac{g(S-1)d^2}{18\gamma} \quad (100\%\ \text{efficiency})$$

$$d = 2.098 \times 10^{-4}\ m = \underline{0.21\ mm}$$

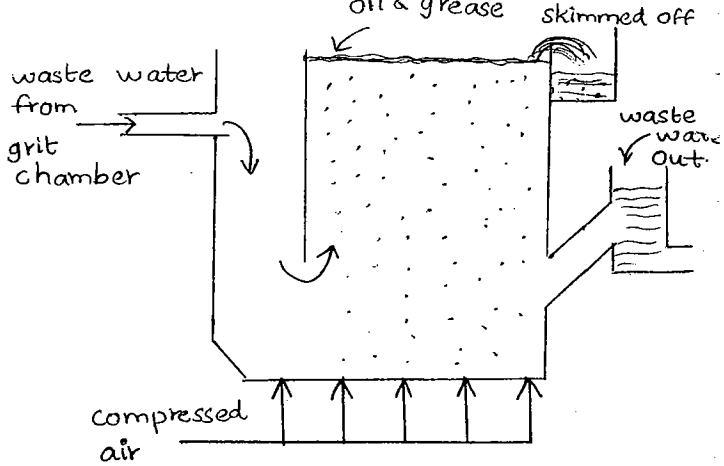
→ Skimming Tank (floatation chamber)

- Skimming tanks are aimed to remove oil and grease granular from waste water.

- Oil remaining affect secondary treatment and also cause problems to disposal system.

- Specific gravity of oil and grease is less than specific gravity of waste water, therefore they can't settle. Oil and grease removed by 'floatation'.

- Floatation is carried out by applying compressed air from bottom, so that oil and grease rise and float to top of skimming tank. From there oil and grease is skimmed off.



* Design:

- Surface area of skimming tank (L * B)

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

Q → flow rate (in m^3/day)

V_r → liquid rise velocity (0.25 m/min)

Detention time = 3 min to 5 min.

→ Primary Clarifier (or) Primary Sedimentation tank

– Primary clarifier is aimed to capture settleable organic solids (biodegradable) present in waste water.

– 70 to 80% biodegradable organic solids are captured.

⇒ 20 to 30% BOD removal.

– Primary clarifier follows TYPE III settling.

Type III settling is known as "hindered settling"

* Design

– Surface loading rate : 30 to 50 $m^3/day/m^2$.

– Detention time : 1.5 to 2 hour.

P-28

17. $Q = 5005\ m^3/day$

$V_o = 35\ m^3/m^2/day.$

$$SA = \frac{\pi}{4} d^2 = \frac{Q}{V_o} = \frac{5005}{35}$$

$$\Rightarrow d = \underline{\underline{13.49}}\ m$$

– Sludge (settled solids) obtained from primary clarifier should be treated prior to disposal.

15th nov,
SATURDAY

SECONDARY TREATMENT

- Secondary treatment of waste water is aimed to remove soluble BOD from waste water flow.
- Secondary treatment is carried out by:
  (i) Physico - chemical treatment methods.
  (ii) Biological treatment methods.

→ Physico - Chemical treatment methods:

- when $\frac{BOD}{COD} < 0.6$
- very expensive method.
- Different methods are:-
  (i) Micro screening
  (ii) Adsorption
  (iii) Absorption
  (iv) Filtration
  (v) Coagulation
  (vi) Chemical oxidation
  (vii) Air stripping
  (viii) Electrolysis.

→ Biological Treatment Methods.

- when $\frac{BOD}{COD} > 0.6$
- very economical.
- complex organic matter fed to micro organisms to obtain simple & stable end products.

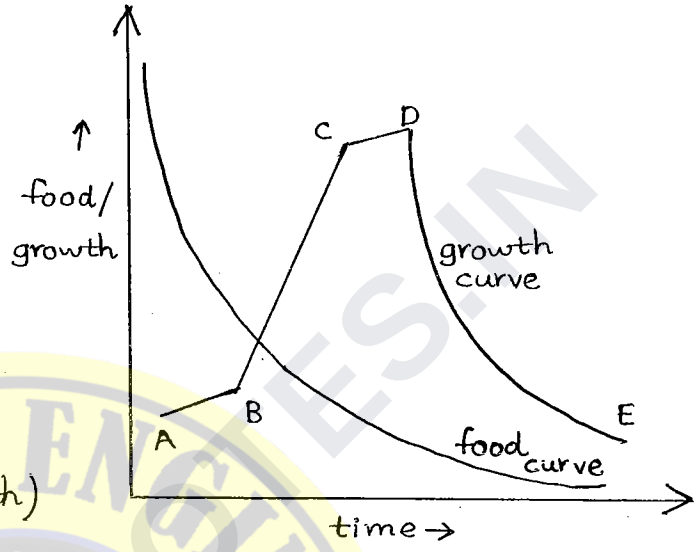
Complex Organic Matter (objectionable) ⟶ Microorganisms ⟶ Simple & Stable end products (inert : less objectionable)

AB → lag growth.

BC → log growth

CD → stationary growth

DE → declining growth (endogeneous growth)



* Micro organisms are classified :

1. based on food consuming capabilities –

(i) Autotrophic organisms

- Prepare food on their own
- cannot be used for biological treatment methods.

Eg: Algae.

(ii) Heterotrophic organisms.

- depend on external food source.

Eg: Bacteria.

2. based on oxygen consuming capacity.

(i) Aerobic organisms.

- depend on external oxygen source.

(ii) Anaerobic organisms

- depend on bound oxygen, doesn't depend on external O_2

(iii) Facultative organisms

– survive in both the condition.

3. based on <u>temperature</u> resistance.

(i) Psychrophylic organism (temperature $< 10^{\circ}C$)

(ii) Mesophylic organism ($20^{\circ}C <$ temp. $< 40^{\circ}C$)

(iii) Thermophilic organism. ($45^{\circ}C <$ temp. $< 60^{\circ}C$)

* Biological treatment Classified into:

(i) Aerobic Biological treatment.

Complex organic matter + Bacteria + Oxygen $\rightarrow \underbrace{CO_2 + NH_3}_{+ H_2O} +$ Energy

– very fast conversions. (4–5 hr)

– But it produces huge amount of biomass (sludge) as by-prod which cause disposal problems.

– costlier compared to other biological treatment metho.

(ii) Anaerobic Biological Treatment.

– complex organic matter fed to bacteria.

Complex organic matter + Bacteria $\longrightarrow CH_4 + CO_2 + H_2S +$ Energy

(70% methane – high calorific value) (biogas)

– very slow but doesn't produce large biomass.

– yield useful by-products.

⊙ Waste water treated aerobically and fast disposed off. The sludges obtained from this treatment subjected to anaerobic treatment.

→ Aerobic Treatment of Waste water

1. Suspended Culture Systems

Organisms chase food.(organic matter)

Eg: Activated Sludge Process. (ASP).

2. Attached Culture Systems

Food chase organisms.

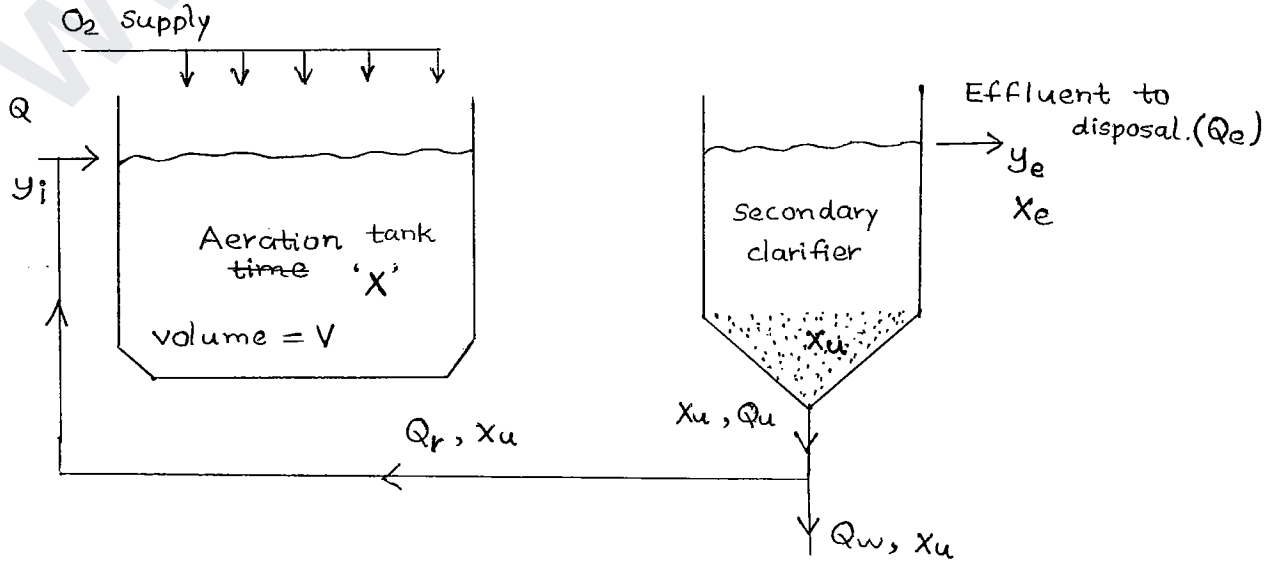
Eg: Trickling Filters.

Rotating biological contactors.

Biotowers. etc.

Activated Sludge Process :

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



17th nov, MONDAY

$Q \rightarrow$ waste water flow rate.

$y_i \rightarrow$ influent BOD (S_i).

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

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

(or)

MLVSS (Mixed Liquor Volatile Suspended Solids)

$Q_e \rightarrow$ flow rate effluent.

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

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

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

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

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

$Q_r \rightarrow$ flow rate of recycling sludge.

* Design Parameters of ASP

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

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

$$\boxed{\frac{F}{M} = \frac{Q\,y_i}{Vx}}$$

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

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

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

(10% change may be there in value)

2. Mean Cell residence time (MCRT), θ_c

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

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

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

$\{ Q = Q_e + Q_w \}$

θ_c : 4 to 15 days

3. BOD removal efficiency

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

η : 85% - 95%

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

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

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

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

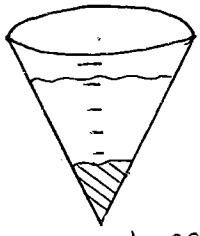
5. Aeration Period (Detention time).

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

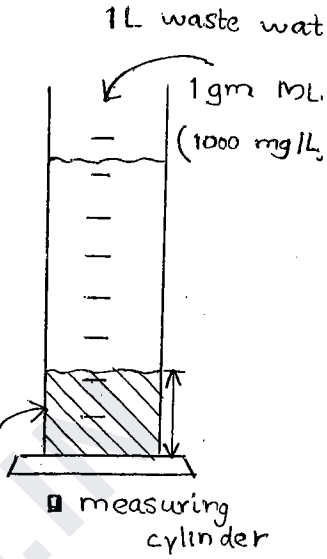
6. Sludge Volume Index, (SVI)

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

SVI : 50 to 150 mL/gm



imhoff cone.



7. Recycling Ratio

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

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

$\frac{Q_r}{Q}$: 0.25 to 0.5 $\quad X_a$: 1500 to 3000 mg/L

X_u : 9000 to 12000 mg/L

P-88

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

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

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

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

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

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

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

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

$= 8.58$ days

2. $Q = 50 \times 10^6$ L/d $= 50000$ m^3/d

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

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

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

$V = 10\,000$ m^3

301. $Q = 50000$ m^3/day.

$y_i = 180$ mg/L.

$\forall_L = 0.55$ kg of BOD/day/m^3.

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

$$0.55 = \frac{50 \times 180}{V} \Rightarrow V = 16363 \text{ m}^3$$

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

2 g/L MLSS occupies 176 mL.

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

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

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

14. $Q = 500\ m^3/hour.$

$y_i = 150\ mg/L$

$y_e = 10\ mg/L.$

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

$\theta_c = 240$ hours. $= 10$ days

$V = 4000\ m^3$

MLSS $= 2000\ mg/L. = X$

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

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

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

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

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

4 g/L of MLSS occupies 200 mL.

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

12. $V = L * B * H = 30 \times 14 \times 4.3$ $\quad X = 1500\ mg/L.$

$Q = 0.0796\ m^3/s$

$y_i = 130\ mg/L$

$X_u = 9100\ mg/L$

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

$= 6.3$ hours

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

$= 0.33\ day^{-1}$

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

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

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

$= 2054.21\ m^3/day$

08. SVI = 88 mL/g

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

10. $V = 400\ m^3$

1

Q. Following data pertains to ASP.

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

Influent BOD, $y_i = 150$ mg/L

Effluent BOD, $y_e = 20$ mg/L

MLSS $= X = 3000$ mg/L

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

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

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

Find the following:

a) Volume of aeration tank

b) aeration period.

c). BOD removal efficiency.

d) Volumetric loading rate.

e) Mass of solids wasted per day.

f). Volume of solids wasted per day.

g) SVI.

h) Recycling ratio.

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

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

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

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

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

$$= \underline{86.67 \%}$$

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

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

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

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

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

X_u = mg/L

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

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

(OR)

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

g) ~~SVI~~ =

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

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

Volume of solids wasted per day $= Q_w$.

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

(mLD) (mg/L)

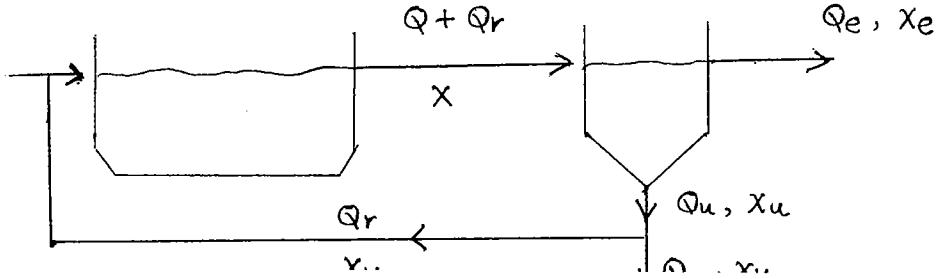
$\therefore Q_w = \frac{600}{10000} = 0.06 \text{ MLD} = 60 \text{ m}^3$

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

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

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

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



$\Rightarrow$ Mass in = mass out.

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

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

But $X_e = 0$ (not given).

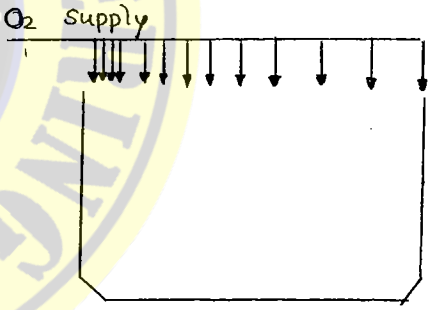
$$(10000 + Q_r)3000 = (Q_r + 60)10000$$

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

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

→ Modifications made to conventional ASP:

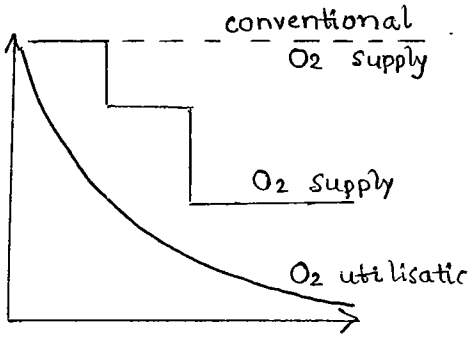
1. Tappered Aeration.

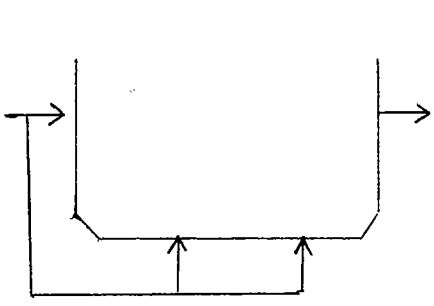


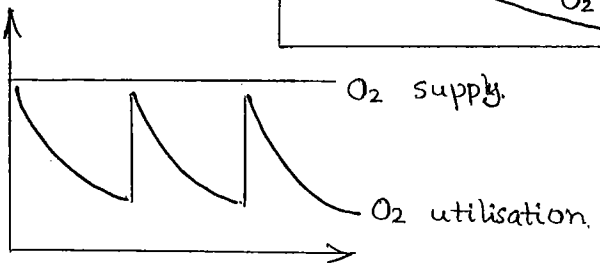
- Varying the O_2 supply from inlet to outlet.

2. Stepped Aeration.

- Food supply @ multiple points.
- This will bring down gap blw O_2 supply and utilisation.



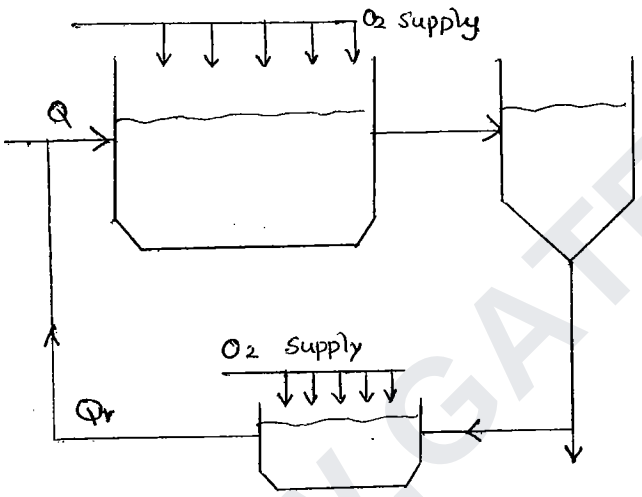


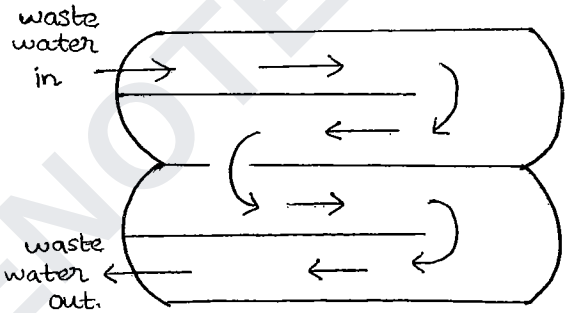


3. Extended aeration

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

4. Contact Stabilisation.



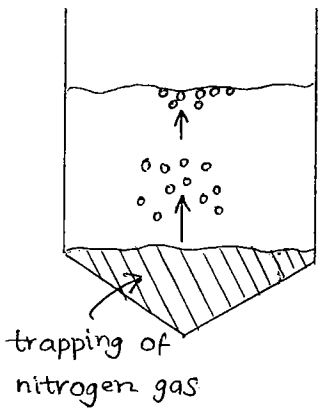


Plan of oxidation ditch.

→ Operational Problems associated with ASP

(i) Rising of Sludge.

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



Remedies:

- increase Q_r — decrease θ_c
- decrease Q

(ii) Bulking of Sludge.

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

– Remedies:

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

(ii) increase pH.

(iii) Increase DO

(iv) Add nutrients.

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

5th nov,
TUESDAY

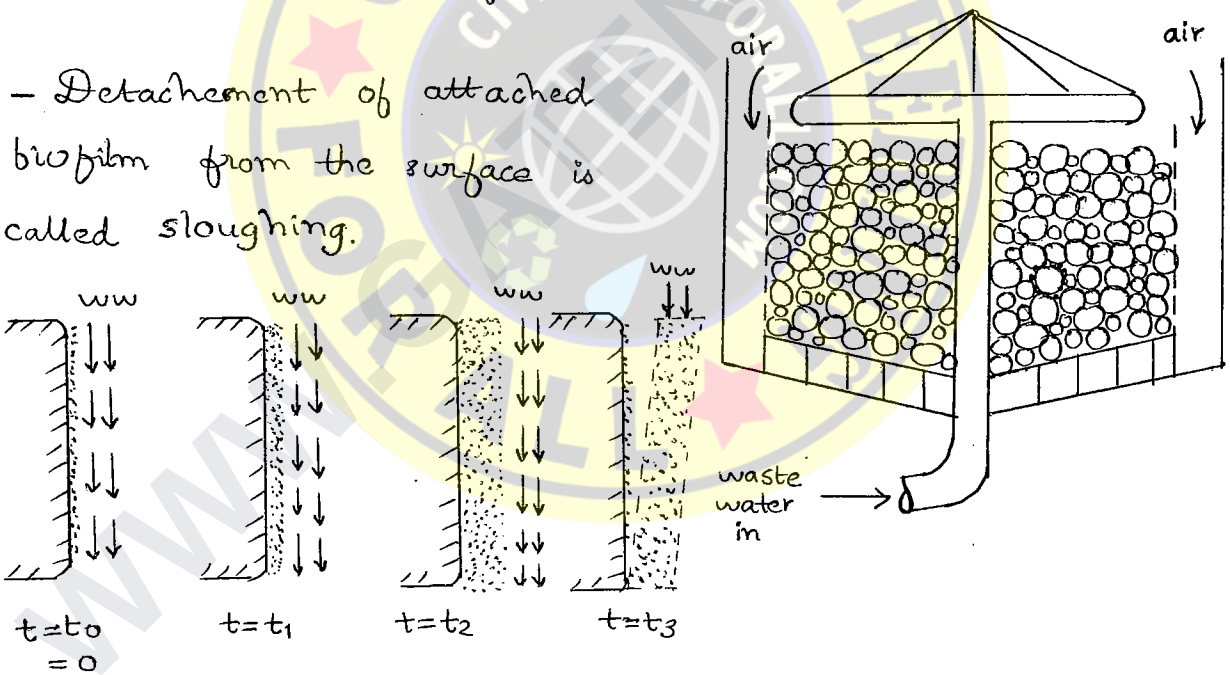
06. TRICKLING FILTERS

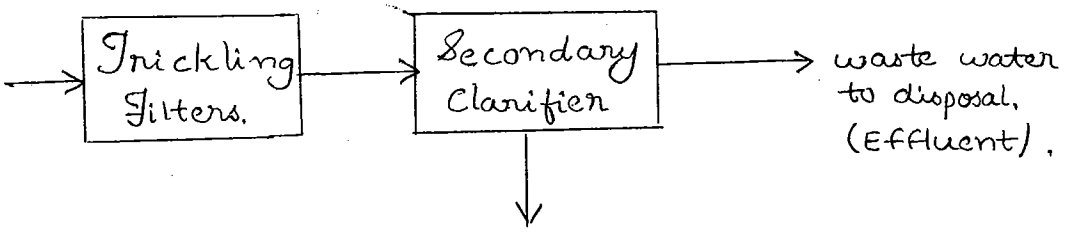
- Trickling Filters works on the principle of attached culture system.

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

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

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





- Circular tank of diameter 3.6 to 60 m

Depth = 1.8 m

→ Design of Trickling Filters.

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

* NRC Equation.

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

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

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

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

Qy_i → total BOD (kg/day)

F → recirculation factor.

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

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

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

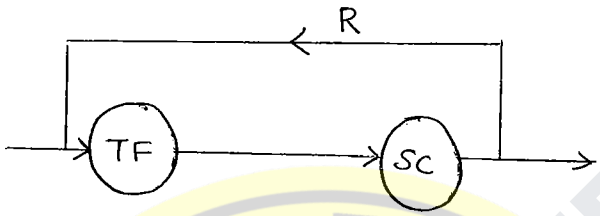
* Based on F, trickling filters are classified as:

1. Standard Rate Trickling Filters.

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

2. High Rate Trickling Filters.

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



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

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

D-92

Q.1. $y_i = 200$ mg/L, $y_e = 40$ mg/L

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

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

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

1 ha. m $= 10^4$ m³.

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

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

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

03. $Q = 6$ MLD.

$y_i = 150$ mg/L.

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

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

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

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

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

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

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

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

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

$= \approx 84.45\%$

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

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

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

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

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

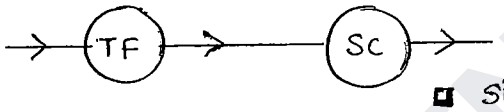
$= 83.33\%$

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

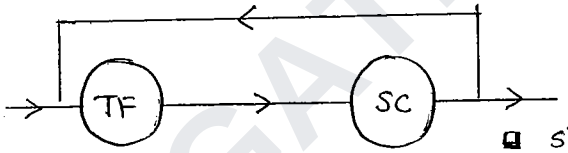
$$F = \frac{1 + R/I}{\left(1 + 0.1\,\frac{R}{I}\right)^2} = \frac{1 + 1.8}{(1 + 0.1 \times 1.8)^2} = \underline{\underline{2.02}}.$$

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

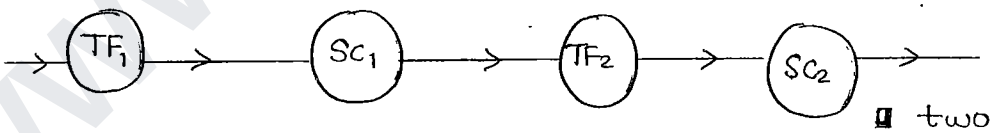
$$\Rightarrow V = \underline{\underline{633}}\ m^3$$



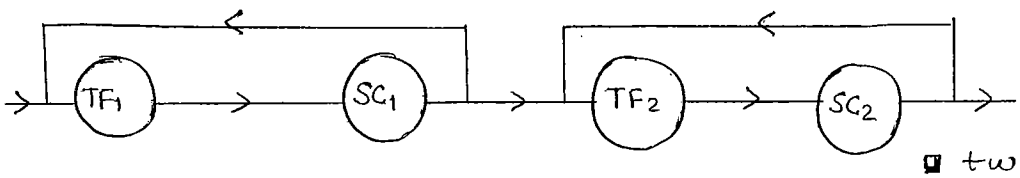
- single stage standard TF



- single stage high rate TF



- two stage std. rate TF



- two stage high rate TF

→ Operational Problems associated with TF

1. Psychoda Problems – fly nuisance.

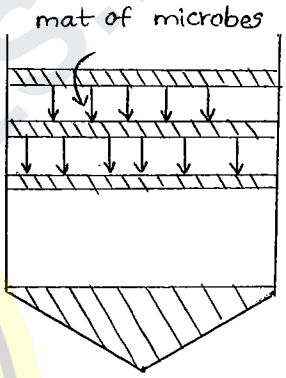
2. Ponding Problems – odour nuissance.

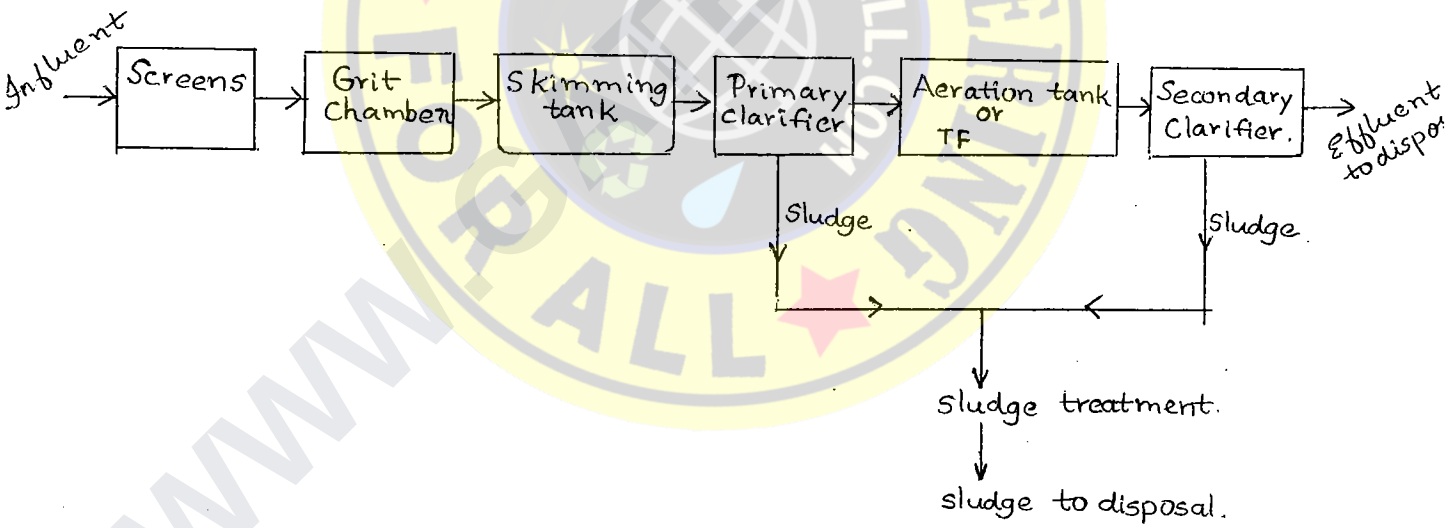
These problems are not present in ASP (relatively modern)

19th nov,
WEDNESDAY

SECONDARY CLARIFIER

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





19th nov,
WEDNESDAY

07. SLUDGE TREATMENT

– Sludge treatment is aimed to achieve:

1. Volume reduction
2. Strength reduction.

– Volume and strength reduction of sludges are achieved by:

1. Sludge thickening.
2. Sludge digestion
3. Sludge conditioning.
4. Sludge dewatering and drying.

– Sludge thickening is achieved by concentrating the sludges by expulsion of water.

– Sludge digestion is achieved by anaerobic decomposition to reduce strength.

– Sludge conditioning is done to improve drainability of sludges.

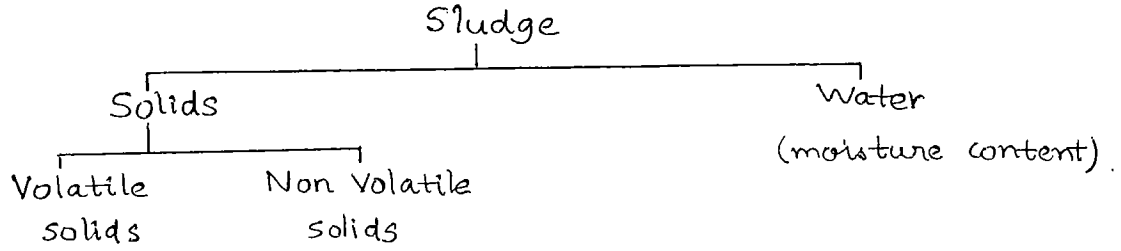
– Sludge dewatering and drying removes remaining moisture from sludge.

→ Composition of Sludge.

– Sludge is a mixture of water and solids.

– It contains 0.25% to 12% solids and.

88% to 99.75% water



→ Mass Volume Relation of Sludge.

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

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

A + B + C ⟶ D

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

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

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

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

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

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

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

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

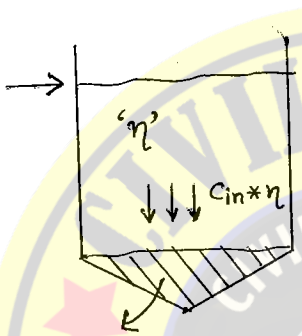
$$= \frac{98}{1} + \frac{2}{1.91} \Rightarrow S_{sludge} = 1.0096$$

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

→ Volume Reduction of Sludge.

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

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



Sludge (100 kg)
100%

Solids $(100-P_1)$ kg
$(100-P_1)$%

water. (P_1 kg)
P_1 %

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

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

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

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

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

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

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

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

Volume of sludge produced @ mc $P_1 = \dfrac{\left(\dfrac{100}{100-P_1}\right) * M}{\rho_{sludge}}$

Volume of sludge produced at m.c P_1 } $V_1 = \dfrac{100}{(100-P_1)} * \dfrac{M}{(\rho_{sludge})_1}$

Volume of sludge produced at mc. P_2 } $V_2 = \dfrac{100}{(100-P_2)} * \dfrac{M}{(\rho_{sludge})_2}$

$$M = \frac{V_1 (100-P_1)(\rho_{sludge})_1}{100} = \frac{V_2 (100-P_2)(\rho_{sludge})_2}{100}$$

$(\rho_{sludge})_1 \simeq (\rho_{sludge})_2$

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

$$\boxed{V_2 = \frac{(100-P_1) V_1}{(100-P_2)}}$$

P-96.

04. $V_2 = \dfrac{100-9}{100-9} * V_1$

$= \frac{1}{4} V_1 = 0.25\ V_1$

∴ Reduction in volume = 75%

11. $P_2 = \dfrac{100-98}{100-9}$; $P = \dfrac{P}{2}$

09. $S_{solids} = 2.2$.

$$\frac{100}{S_{sludge}} = \frac{2}{S_{solids}} + \frac{98}{S_{water}} = \frac{2}{2.2} + \frac{98}{1}.$$

$$\Rightarrow S_{sludge} = 1.011$$

$$P_{sludge} = S_{sludge} \times P_{water} = 1.011 \times 1000 = \underline{1011}\ kg/m^3$$

10. $$V_2 = \frac{100 - P_1}{100 - P_2} \times V_1$$

$$0.5 = \frac{100 - 98}{100 - P_2}$$

$P_2 = 96\%$ (moisture content).

$\therefore$ Solids content $= 100 - 96 = \underline{4\%}$

01 $Q = 4.5$ MLD; $C_{in} = 275$ ppm $= 275$ mg/L; $\eta = 55\%$

(i) Total mass of dry solids in sludge $= Q * C_{in} * .55$

$$M = 4.5 * 275 * 0.55$$

$$= \underline{680.625}\ kg/day$$

Weight of sludge produced per day @ mc = 96%

$$= \frac{100}{100 - 96} * 680.625 = \underline{17015.625}\ kg/day$$

(ii) Volume of sludge produced $= \frac{17015.625}{1.02 \times 1000} = \underline{16.682}\ m^3$

→ Strength Reduction

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

- Acid Fermentation.

Sludges ⟶ Acid Formers. (anaerobic acid forming bacteria) ⟶ Acids.
- Acetic acid
- Butyric acid.
- Propionic acid.

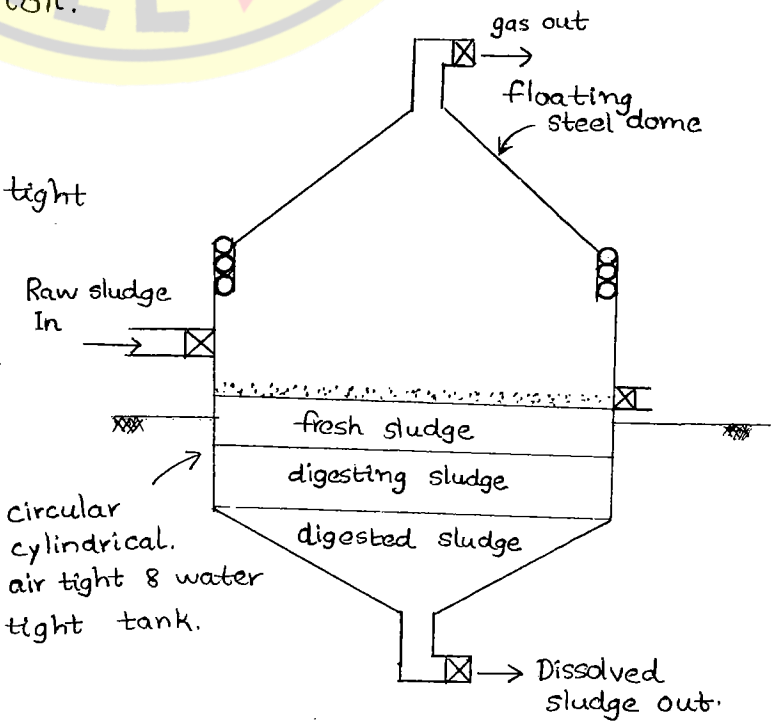
- Methane Fermentation

Acids ⟶ Methane formers. (anaerobic methane forming bacteria) ⟶ Gases.
- CH_4
- CO_2
- H_2S

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

→ Sludge Digestor

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



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

- Digestion period depends on type of micro organisms employed

  (i) Mesophilic organisms → 30 to 60 days (temp 20°c – 40°c)

  (ii) Thermophilic organism → 15 to 25 days (temp 45°c – 60°c).

- Rate of gas production:

  0.9 m^3/kg volatiles destroyed.

  70% → CH_4

  20–30% → CO_2

  Traces → remaining gases.

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

V_f → volume of fresh sludge/day.

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

V_d → volume of digested sludge/day.

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

P-96

08. Concentration of volatile suspended solids destroyed.

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

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

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

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

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

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

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

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

$= 249139.8\ kcal$

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

$= 20 * 300 * 0.65$

$= 3900\ kg/day.$

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

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

$S_{solids} = 1.645$

$S_{digested\ sludge} = ?$

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

$S_{ds} = 1.0625$

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

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

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

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

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

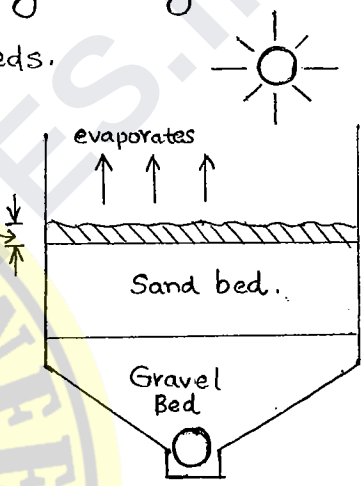
$$\frac{\pi}{4} d^2 = 151.425$$

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

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

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

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

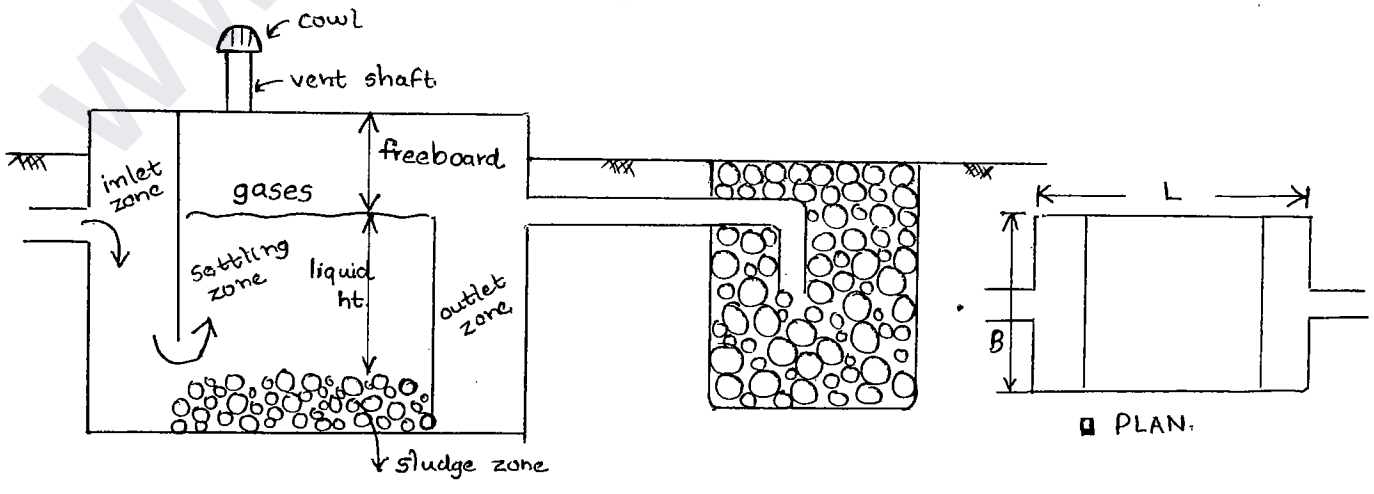


◘ Sludge drying bed.

20th nov,
THURSDAY

08. SEPTIC TANKS

- Septic tanks are used in unsewered areas (where there is no central provision to collect toilet waste from community)
- Septic tanks are best suitable if the number of users are less than 300.
- Septic tank is a settling tank with long detention time with an extra provision for anaerobic digestion of settled solids
- Septic tank undertakes following processes:
  (i) Settling of solids.
  (ii) Anaerobic digestion of settled solids.
- Septic tank is a rectangular air tight and water tight tank constructed with masonry or concrete.
- The effluents of septic tank applied to the ground with the help of soak pit. Soak Pit is an earthern pit filled with brick bats.
- Disperse effluents of septic tank into wide area with the help of soak pit.



LONGITUDINAL SECTION.

→ Design of Septic Tank :

* Settling Zone of Septic Tank :

- Detention time = 24 hrs to 48 hours

- Volume of settling zone (L * B * H) = Q_{DWF} * DT

- Surface area of septic tank (L * B) = Surface area of Settling Zone

$$= \frac{\text{Volume of Settling Zone}}{\text{Liquid Depth}}$$

* Sludge Zone of Septic Tank :

- Rate of sludge accumulation = 30 to 40 L/person/year

- Desludging period (cleaning period) = 1 to 3 years

- Volume of sludge zone = rate of sludge accumulation * no. of users * desludging period.

- Depth of sludge zone = $\frac{\text{Volume of Sludge Zone}}{\text{Surface area of Septic tank.}}$

- Total volume of Septic tank = Volume of Settling zone + Volume of sludge zone

$$V = Q_{DWF} * DT + ROS * \text{no. of users} * \text{cleaning period}$$

P-99. No: of users = 200

Per capita water supply = 135 lpcd.

DT = 24 hours.

Cleaning period = 1 years.

Rate of sludge production = 40 L/person/year.

L : B = 3 : 1.

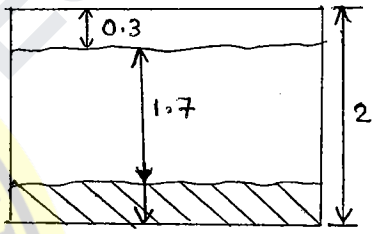
Total volume of septic tank = Q_{DWF} * DT + ROS * no. of users * cleaning period.

= 0.2 X 135 X 0.75 + 0.04 * 200

= 20.25 + 8 = 28.25 m^3

Surface area of septic tank

$$= \frac{\text{Volume of Septic tank}}{\text{Depth of septic tank}}$$



$$L * B = \frac{28.25}{1.7} = 16.61\ m^2$$

$$3B * B = 16.61\ m^2 \Rightarrow B = 2.35\ m \text{ \& } L = 7.06\ m$$

2. L = 2.25 B.

Depth of settling zone (liquid depth) = 2 m.

Freeboard = 0.3 m

No: of users = 300

Water supply = 100 lpcd = 0.1 m^3/p/d, factor = 0.8

ROS = 0.04 m^3/p/dayear

DT of 3 days @ startup.

Desludging is done when septic tank is 1/3rd full.

Hydraulic loading rate = 100 L/m^2/day.

NOTE:

At start up, total settling tank is used only for settling operation.

∴ Volume of septic tank = volume of settling zone @ start up.

$$= \frac{300 \times 100 \times 0.8}{10^3} * 3 = \underline{72\ m^3}$$

Total volume of septic tank = Volume of settling zone + volume of sludge zone

Volume of sludge zone = $\frac{1}{3}$ volume of septic tank.

$$= \frac{72}{3} = 24\ m^3.$$

72 = volume of settling zone + 24.

∴ Volume of settling zone = $48\ m^3$.

Liquid depth = 2 m.

Surface area of septic tank = $\frac{48}{2} = 24\ m^2$.

$$2.25\ B^2 = 24.$$

$$B = 3.266\ m$$

$$L = \underline{7.35\ m}.$$

> Volume of sludge zone = ROS * no. of users * desludging interval.

$$24 = 0.04 \times 300 \times \text{Desludging interval}.$$

$$\Rightarrow \text{Desludging interval} = \underline{2\ years}$$

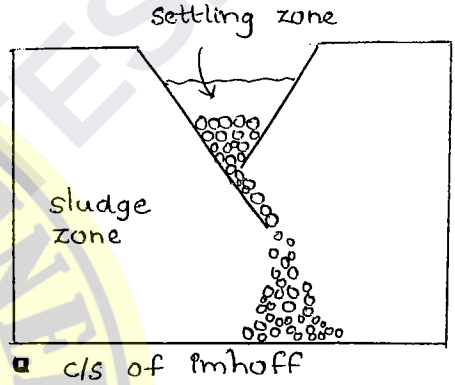
> Surface area of percolation trench (soak pit) = $\frac{Q}{HLR}$

$$= \frac{300 \times 100 \times 0.8}{100}$$

$$= \underline{240\ m^2}$$

→ Imhoff Tanks

- If no. of users are more than 300, Imhoff tanks are preferred to septic tank.
- Imhoff tank is a two compartment septic tank:
  1. Upper compartment – settling of solids.
  2. Lower compartment – anaerobic digestion of settled solids.



C/S of Imhoff

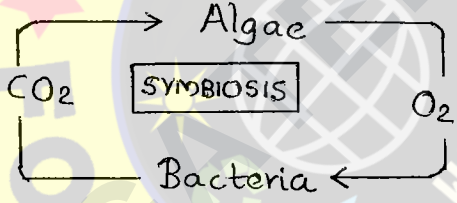
20th nov,
THURSDAY

09. OXIDATION PONDS

- Oxidation ponds are low cost waste water treatment units, stabilise the waste water (remove BOD from waste water under aerobic conditions.. ∴ oxidation ponds are aerobic waste stabilisation ponds.

- Oxidation ponds are shallow dug earthen ponds provided with high embankment. They are open rectangular earthen tanks.

- Oxidation ponds works on the principle of Algal-Bacterial Symbiosis.



- Oxidation ponds best work in tropical climatic zones. (temperature zones)

→ Design of Oxidation Pond.

1. BOD loading rate (or) Organic Loading rate.

- Surface area (or) Plan area of pond
(L * B)

$$SA = \frac{Q y_i}{OLR} = \frac{\text{Total BOD applied to pond}}{\text{BOD loading rate.}}$$

- BOD loading rate or organic loading rate depends on latitude of a place.

- Solar radiation at a place depends on latitude of place

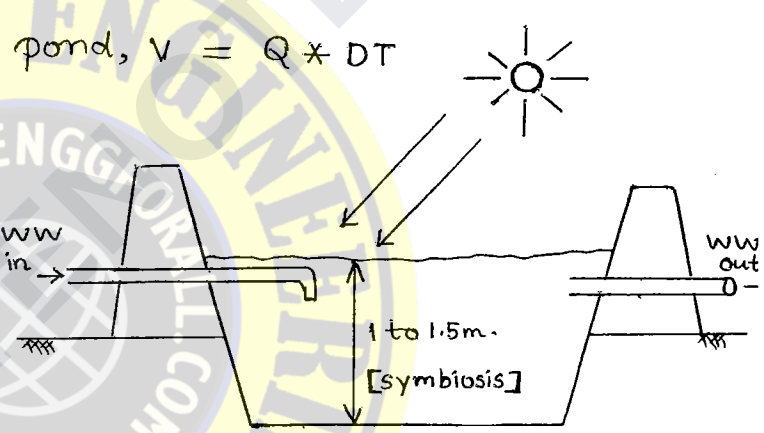
- BOD loading rate $\propto \frac{1}{\text{latitude of place.}}$

2. Detention time, 't'

- Detention time of 2 to 6 weeks

- $\boxed{t = \frac{1}{K} \ln \frac{Y_i}{Y_e}}$; K → pond rate constant.

- Volume of oxidation pond, V = Q * DT



P-101

Q.5 Population = 10,000

$Q = 200 \text{ lpcd} = 0.2 \text{ m}^3/\text{pcd} \times 10000 = 2 \text{ MLD.}$

$Y_i = 300 \text{ mg/L.}$

Organic loading rate = 310 kg/day/ha.

$$\text{Area of oxidation pond} = \frac{\text{Total BOD applied to pond}}{\text{BOD loading rate}}$$

$$= \frac{2 \times 300}{310} = 1.935 \text{ ha}$$

2. Population served = 10,000

Sewage flow = 150 lpcd.

y_i = 300 mg/L , y_e = 30 mg/L.

Total BOD applied to pond = $150 \times 10^{-6} \times 10000 \times 300$

= 450 kg/day.

Organic loading rate = 300 kg/ha/day.

$$\text{Area of pond} = \frac{450}{300} = 1.5 \text{ ha} = \underline{15000} \text{ m}^2$$

$$\text{Detention time} = \frac{1}{k} \ln \frac{y_i}{y_e} = \frac{1}{0.23} \ln \frac{300}{30}$$

$$= \underline{10 \text{ days}}$$

Volume of pond = Q * DT.

$= 0.15 \times 10000 \times 10 = 15000 \text{ m}^2.$

$$\frac{L}{B} = \frac{4}{1}$$

$$L * B = 15000$$

$$B = 61.23 \text{ m}$$

$$L = 245 \text{ m}.$$

$$H = \frac{\text{Volume}}{\text{Area}} = \frac{15000}{15000} = \underline{1 \text{ m}}$$

01 Population = 10,000

Per capita BOD = 40 g/day.

Per capita water supply = 100 lpcd.

Factor = 0.8

L : B = 4 : 1

H = 2m.

Q_{DWF} = population * per capita water supply * factor.

= 10000 * 100 * 0.8 L/day

= 0.8 MLD.

Total BOD = $Q y_i$ = population * per capita BOD.

$0.8 \times y_i = 10000 \times 40 \times 10^{-3}$

$y_i = 500$ mg/L.

> Surface area = L * B = $\frac{Q y_i}{BOD.LR} = \frac{0.8 \times 500}{200}$

= 2 ha = 2×10^4 m^2

$4B^2 = 20000$

$B = 70.71$ m

$L = 282.84$ m

> DT = $\frac{\text{volume of pond}}{Q} = \frac{L * B * H}{Q}$

= $\frac{2 \times 10^4 \times 2}{0.8 \times 10^6/10^3}$ = 50 days

> $\eta = \frac{y_i - y_e}{y_i} \times 100$

$80 = \frac{500 - y_e}{500} \times 100$

$y_e = 100$ mg/L.

As $y_e \leq 100$ mg/L, it can be used for Irrigation.

21st nov,
FRIDAY

10. DISPOSAL OF SEWAGE EFFLUENTS

- Effluents (treated waste water) are disposed by: ~~dilution~~
  (i) Dilution → water disposal.
  (ii) Land treatment → land disposal.

- Disposal by dilution is preferred under following conditions:

(i) water body should contain enough water with sufficient DO.

(ii) there should not be any community immediately on the downstream of point of disposal.

- Any effluent can be disposed into water body when it meet following disposal standards: (IS code)

Parameter	Domestic waste water	Industrial waste water.
BOD	$\leq$ 20 mg/L	$\leq$ 30 mg/L
COD	$\leq$ 250 mg/L	$\leq$ 250 mg/L
Oil & greese	$\leq$ 10 mg/L	$\leq$ 10 mg/L
Total Solids	$\leq$ 100 mg/L	$\leq$ 100 mg/L

- After disposing the waste water into fresh water course, the left overs in the waste water are removed by the water body on its own → 'self purification capacity'

– Self purification of water bodies are due to:

(i) Dilution – mixing

(ii) Dispersion. – spread.

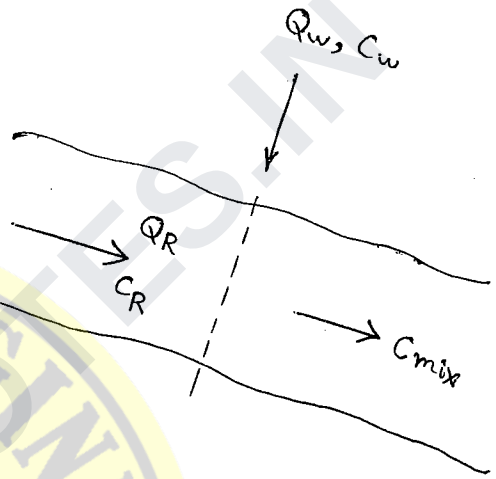
(iii) Sedimentation

(iv) Oxidation.

(v) Reduction.

(vi) Disinfection. – due to sunlight.

$$C_{mix} = \frac{Q_R C_R + Q_w C_w}{Q_R + Q_w}$$



P–106.

01 $Q_w = 12,000\ m^3/day$ $Q_R = 40,000\ m^3/day$

$y_w = 50\ mg/L$ $y_R = 3\ mg/L$

$$y_{mix} = \frac{12000 \times 50 + 40000 \times 3}{12000 + 40000} = 13.846\ mg/L$$

11. $Q_w = 50\ m^3/s$ $Q_R = 500$

$y_w = 200\ mg/L$ $y_R = 8\ mg/L$

$$y_{mix} = \frac{50 \times 200 + 500 \times 8}{500 + 50} = 25.45\ mg/L$$

12. $Q_w = 8640\ m^3/d = 0.1\ m^3/s$ $Q_R = 1.2\ m^3/s$

$T_w = 25^\circ C$ $T_R = 15^\circ C$.

$$T_{mix} = \frac{0.1 \times 25 + 1.2 \times 15}{0.1 + 1.2} = 15.77^\circ C$$

03. $Q_w = 1.1\ m^3/s$ $\quad Q_R = 8.7\ m^3/s$

$(DO)_w = 2\ mg/L$ $\quad (DO)_R = 8.3\ mg/L$

$$(DO)_{mix} = \frac{1.1 \times 2 + 8.7 \times 8.3}{1.1 + 8.7} = 7.592\ mg/L$$

04. a) $$y_{mix} = \frac{12000 \times 50 + 40000 \times 3}{12000 + 40000} = 13.846\ mg/L$$

b) $$y_{mix} = L_o (1 - e^{-kt})$$

$$13.846 = L_o (1 - e^{-0.23 \times 5})$$

$$\Rightarrow L_o = 20.262\ mg/L$$

c) $$(DO)_{mix} = \frac{12000 \times 2 + 40000 \times 7}{12000 + 40000} = 5.846\ mg/L$$

08. Rate of dissipation = rate of removal = $0.12\ mg/L/hr$.

$Q_w = 30 \times 10^6\ L/day$ $\quad Q_R = 5\ m^3/s$

$= 0.35\ m^3/s$

$C_R = 0$

$C_w = 25\ mg/L$

$$C_{mix} = \frac{0.35 \times 25 + 5 \times 0}{0.35 + 5} = 1.6233\ mg/L$$

Time taken to remove chemical residue by river $= \frac{1.6233}{0.12}$

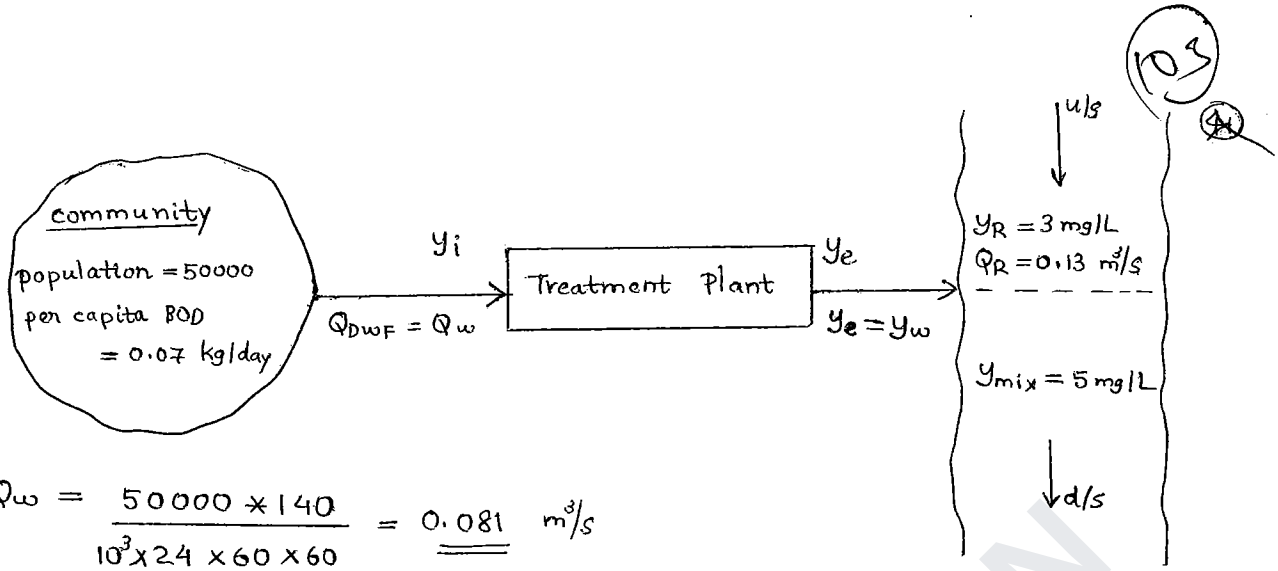
$= 13.53$ hours

$= 48701.3\ s$

Distance upto which chemical residue persist $D = V * t = 0.2 \times 48701.3$.

$= 9740.26\ m = 9.74\ km$

05.



$$Q_w = \frac{50000 * 140}{10^3 \times 24 \times 60 \times 60} = 0.081 \ m^3/s$$

$$y_i = \frac{50000 \times 0.07}{140 \times 50000} \times 10^6 = 500 \ mg/L$$

$$y_{mix} = \frac{Q_R y_R + Q_w y_w}{Q_R + Q_w}$$

$$5 = \frac{0.13 * 3 + 0.081 * y_w}{0.13 + 0.081}$$

$$y_w = y_e = 8.21 \ mg/L$$

Percentage purification required @ treatment plant. $\eta = \frac{y_i - y_e}{y_i} * 100$

$$= \frac{500 - 8.21}{500} * 100 = 98.358 \ \%$$

13. $Q_w = 2 \ m^3/s$ $\quad Q_R = 12 \ m^3/s$

$(L_o)_w = 90 \ mg/L$ $\quad (L_o)_R = 5 \ mg/L$

$$(L_o)_{mix} = \frac{Q_R \cdot (L_o)_R + (Q_w)(L_o)_w}{Q_R + Q_w} = \frac{12 * 5 + 2 * 90}{12 + 2}$$

$$= 17.14 \ mg/L$$

$$\text{Velocity} = \frac{Q}{\text{c/s area}} = \frac{12+2}{50} = 0.28 \ m/s$$

Time taken to travel 10 km = $\frac{\text{distance}}{\text{velocity}} = \frac{10000}{0.28}$

$= 0.413$ days

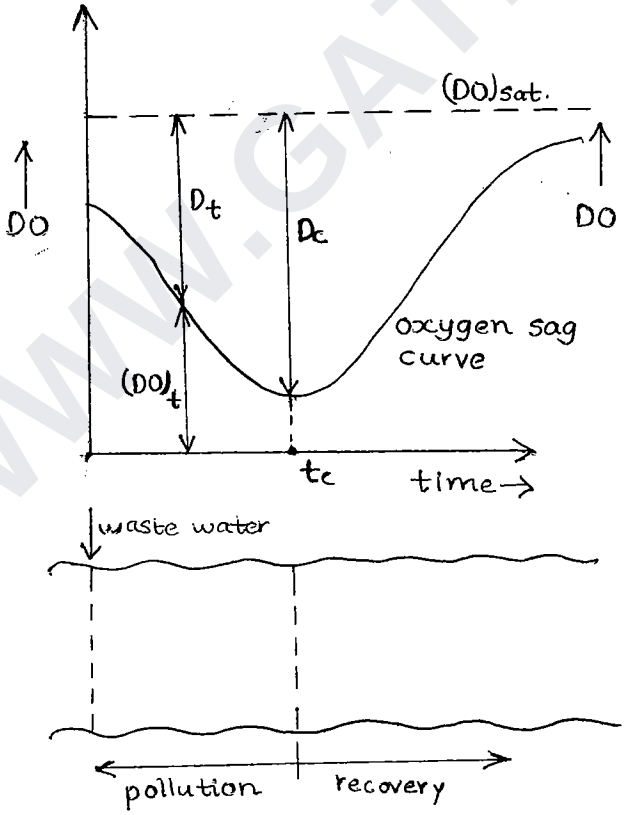
BOD remaining after 0.413 days (Organic matter remaining after 0.413 days).

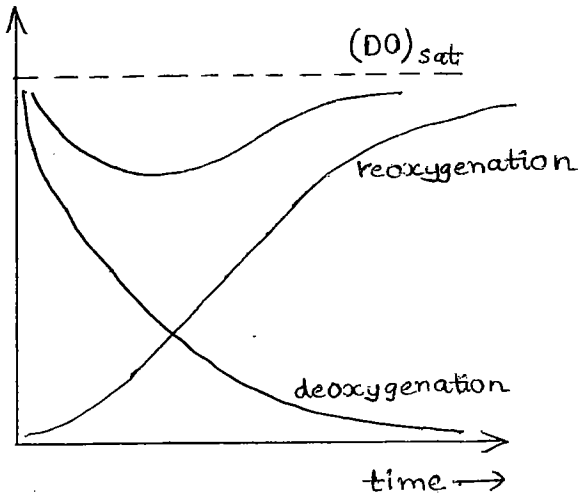
$L_{0.413} = (L_o)_{mix} \cdot e^{-kt}$

$= 17.4 \times e^{-0.23 \times 0.413}$

$= 15.46$ mg/L

– The river undergoing self purification process experiences deoxygenation on one side to meet oxygen demand of waste and re-oxygenation on the other hand due to continuous exposure to free atmosphere.





→ Streeter-Phelps Equations for Stream Sanitation

⊙ Oxygen deficit at any time t, $D_t = \frac{K_1 L_o}{K_2 - K_1}\left(e^{-K_1 t} - e^{-K_2 t}\right) + D_o e^{-K_2 t}$.

⊙ DO at any time t, $(DO)_t = (DO)_{sat} - D_t$.

⊙ Critical oxygen deficit, $D_c = \frac{K_1}{K_2} L_o e^{-K_1 t_c}$

⊙ Critical time, 't_c' $= \frac{1}{K_2 - K_1} \ln\left[\frac{K_2}{K_1}\left(1 - D_o \frac{K_2 - K_1}{K_1 L_o}\right)\right]$.

where, $L_o = \frac{y_{mix}}{1 - e^{-K_1 * 5}}$

D_o = initial oxygen deficit.

K_1 → deoxygenation rate constant.

K_2 → reoxygenation rate constant.

Q. A stream with a flow rate of 50 m^3/s having no BOD with DO 8 mg/L receives 1 m^3/s of waste water with no DO and having a BOD of 30 mg/L. Find oxygen deficit at 5 km downstream and critical oxygen deficit. Also find out DO in the river stream at 30 km. downstream if velocity of flow is 0.5 m/s. DO @ saturation is assumed to be 9 mg/L.

Given,

$K_1 = 0.5\ d^{-1}$

$Q_w = 1\ m^3/s$ $\quad Q_R = 50\ m^3/s$ $\quad K_2 = 1\ d^{-1}$

$y_w = 30$ mg/L $\quad y_R = 0$

$(DO)_w = 0$ $\quad (DO)_R = 8$ mg/L.

$$y_{mix} = \frac{30\times1 + 0}{50+1} = 0.588 \text{ mg/L}$$

$$L_0 = \frac{y_{mix}}{1-e^{-K_1 t}} = \frac{0.588}{1-e^{-0.5*5}} = 0.641 \text{ mg/L}$$

$$DO_{mix} = \frac{0\times1 + 50\times8}{50+1} = 7.843 \text{ mg/L}$$

Initial oxygen deficit, $D_0 = (DO)_{sat} - (DO)_{mix}$

$= 9 - 7.843 = 1.157$ mg/L

Time taken to travel 5 km $= \frac{5000}{0.5} = 10000$ s

$= 0.116$ day

Oxygen deficit at 5 km d/s ($t = 0.116$ day) $= \frac{0.5\times0.641}{1-0.5}\left(e^{-0.5\times0.116} - e^{-1\times0.116}\right) + 1.15\, e^{-1\times0.116}$

$= 1.064$ mg/L

Time taken to travel 30 km $= \frac{30000}{0.5} = 0.694$ day.

Oxygen deficit 30 km d/s ($t = 0.694$ d) $= \frac{0.5\times0.641}{1-0.5}\left(e^{-0.5\times0.694} - e^{-1\times0.694}\right) + 1.16\, e^{-1\times0.69}$

$= 0.71$ mg/L

DO at 30 km d/s $= 9 - 0.71 = 8.3$ mg/L

Critical time, $t_c = \frac{1}{K_2 - K_1} \ln\left[\frac{K_2}{K_1}\left(1 - D_0 \frac{K_2 - K_1}{K_1 L_0}\right)\right]$

$= \frac{1}{1-0.5} \ln\left[\frac{1}{0.5}\left(1 - 1.157\left(\frac{1-0.5}{0.5\times0.641}\right)\right)\right]$.

$\Rightarrow t_c = 0$

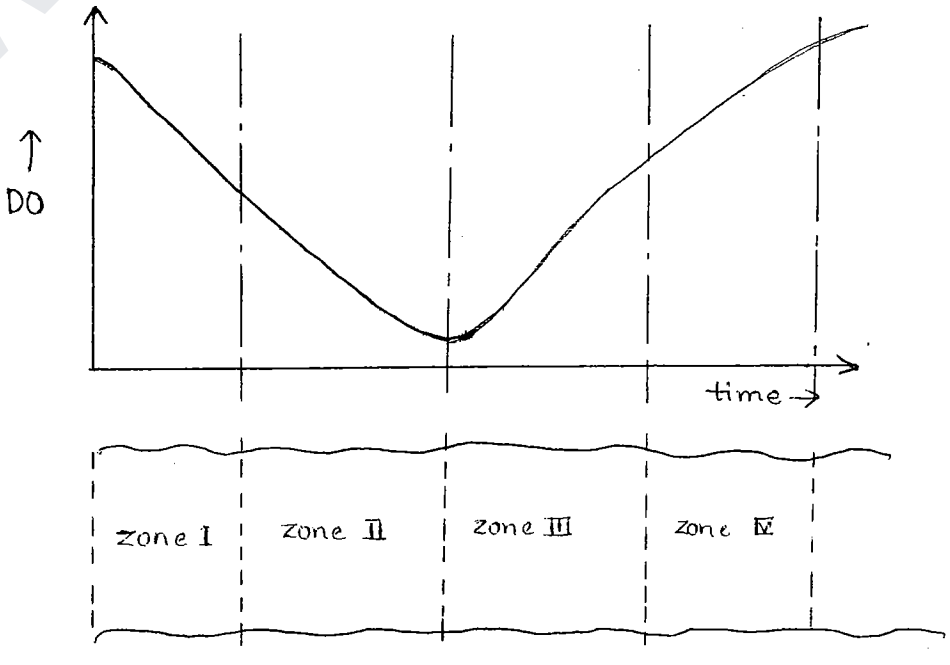
Critical Oxygen deficit, $D_c = \frac{K_1}{K_2} L_o e^{-K_1 t_c}$.

$= \frac{0.5}{1} \times 0.64 \; e^{-0.5 \times 0}$

$= 0.32$ mg/l

→ Zones of Pollution in a river:

Dilution factor	Type of treatment recommended for domestic waste.
> 500	No treatment.
300 – 500	Preliminary treatment.
150 – 300	Primary treatment.
< 150	Primary treatment followed by secondary treatment.



* Zone I :

- Zone of degradation
- DO levels reduced to 40% of original
- exccept fish, all organisms disappear.

deoxygenation > reoxygenation.

* Zone II :

- Zone active decomposition.
- DO levels reduced to zero.
- heavily polluted zone.
- obnoxious gases (foul) evolve.
- all aquatic organism disappear.

* Zone III :

- Zone of recovery
- DO level rose to 40% original.
- fish appear in water

reoxygenation > deoxygenation.

* Zone IV :

- Clear water zone
- reoxygenation only occur; no deoxygenation.
- River is recovered
- DO levels rise to original.
- all aquatic organisms appear.

→ Land Treatment (Land Disposal)

– This method is followed :

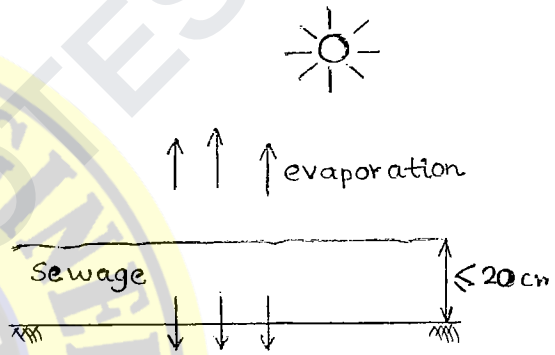
(i) when there is no fresh water body in the vicinity of community.

(ii) Soil is capable of absorbing waste (sandy or loamy soil are best)

(iii) Ground water table deep below the ground.

* Sewage Sickness

Continuous application of sewage on same land leads to sewage sickness.



– Remedies :

(i) Give rest to the soil.

(ii) Plough the soil after drying.

– Area of land required for sewage disposal

$$= \frac{Q}{\text{consuming capacity of soil}}$$

09. $Q = 8$ MLD. ; Consuming capacity $= 80000$ L/ha/day

Area of land required $= \frac{8 \times 10^6}{80000} = \underline{\underline{100 \text{ ha}}}$

10. $Q = 4.5$ MLD.

Area of land used for disposal $= \cancel{0.6 \times 140 = 84 \text{ ha}}$ x ha

~~Consuming capacity~~ = 40 % extra for rest and rotation $= 0.4x$

$$x + 0.4x = 140$$

$$x = \frac{140}{1.4} = 100 \text{ ha.}$$

∴ Area of land used for sewage application = 100 ha.

$$\text{Consuming capacity} = \frac{4.5 * 10^6}{100} = 45000 \text{ L/ha/day}$$

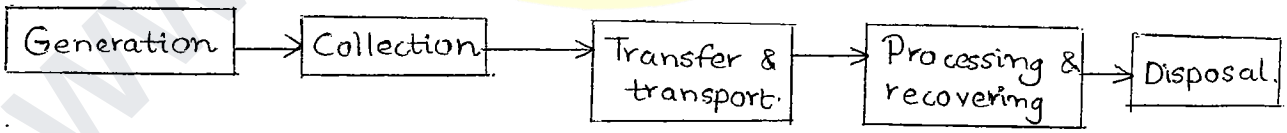
22nd nov,
SATURDAY

11. SOLID WASTE MANAGEMENT

Waste generated by the community which is essentially solid and discarded as useless (or) unwanted is known as 'Solid waste'.

- Mishandling of solid waste or piling up of solid waste in the community cause spread of "roddent borne diseases"
- Roddent bornes diseases are those which are spread in the community through roddents such as rats, mice and bats.
- Common diseases caused by roddents are:
  (i) Plague
  (ii) Lyssa viral infection.
  (iii) Hanta viral infection.
  (iv) Leptosporasis.

→ Solid Waste Management System:



- Generation: Indian communities generate solid waste at 0.4 to 0.8 kg/person/day
- Collection: Bin System:
- Transfer & Transport: Hauling system
- Processing & Recovery: Segregation of solid waste.

→ Classification of Solid waste:

1. Municipal Solid waste.
   (i) Residential solid waste.
      Eg :- food waste, rubbish, ashes etc.
   (ii) Commercial Solid waste.
      Eg:- food waste, rubbish, construction & demolition waste.
   (iii) Solid waste from open areas.
   (iv) Treatment plant waste.
      Eg:- Dry sludges.
2. Industrial Solid waste.
3. Hazardous waste - ignitable, corrosive, reactive, toxic.

→ Examination of Solid waste.

1. Physical Examination
   (i) Identification of individual substances.
   (ii) Size distribution of solid waste.
   (iii) Specific gravity of solid waste.

$$\frac{100}{S_{msw}} = \sum \frac{P_i}{S_i}$$; S_{msw} → Sp.gr. of municipal solid waste.

   (iv) Moisture content of solid waste.

2. Chemical Examination
   (i) Energy content

$$\text{Total energy as discarded} = \frac{\sum P_i E_i}{100}$$

$$\text{Energy on dry-basis} = \text{Energy as discarded} * \left[\frac{100}{100 - \%\ mc}\right]$$

$$\text{Energy on ash free dry basic} = \text{Energy as discarded} * \left[\frac{100}{100 - \%\ mc - \%\ ash}\right.$$

(ii) Organic content.

(iii) Ash content.

P-110

1. Energy as discarded $= \frac{\sum P_i E_i}{100} = \frac{15\times4650 + 45\times16750 + 10\times16300 + 10\times32600 + 10\times6500 + 5\times(8600+70)}{100}$

$= 14740$ kJ/kg

Energy on dry basis $= 14740\left(\frac{100}{100-21}\right) = 18658.2$ kJ/kg

Energy on ash free dry basis $= 14740\left(\frac{100}{100-21-5}\right) = 19918.9$ kJ/kg

2. $\frac{100}{\rho_{MSW}} = \frac{50}{300} + \frac{30}{500} + \frac{10}{65} + \frac{10}{125}$

$\rho_{MSW} = \frac{217.15}{\cancel{109.79}}$ kg/m³

4. CO_2 & CH_4 are treated as Green House ~~house~~ gases.

120 g of MSW $\longrightarrow$ 50 g CO_2 + 25g CH_4 = 75 g green house gase

120g MSW $\rightarrow$ 75g green house gases.

500 ton/day MSW $\longrightarrow \frac{75}{120} * 500 = 312.5$ ton/day green house gases

Total green house gas emission/day = 312.5 t/day

Per capita green house gas emission $= \frac{312.5\times10^6}{10^6}$ g/person/day.

$= 312.5$ g/person/day

5. Molecular weight of $C_{50}H_{100}O_{40}N = 50\times12 + 100 + 40\times16 + 14.$

$= \underline{1354}$

Molecular wt of $\left(\frac{4\times50 + 100 - 2\times40 - 3}{8}\right) CH_4 = 27.125\,(12+4)$

$= 434$

1354 parts MSW $\longrightarrow$ 434 parts of CH_4

1 t MSW $\longrightarrow \frac{434}{1354} \times 1$ t of CH_4.

Mass of CH_4 produced $= \frac{434}{1354} = 0.320\,t = 320$ kg.

$P_{CH_4} = \frac{\text{Mass of } CH_4}{\text{vol. of } CH_4} \Rightarrow$ Vol. of $CH_4 = \frac{320}{0.7167} = \underline{447.2}$ m³

→ Disposal of Solid waste.

* Methods :

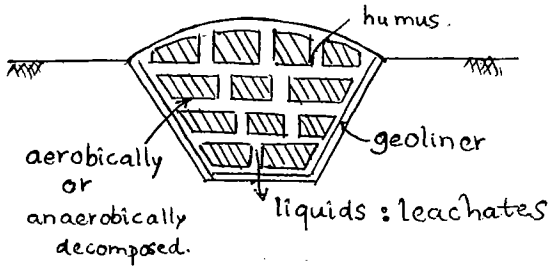
1. Landfilling

- Biodegradable solid waste is disposed by this method.

(i) Area filling

(ii) Trench filling

(iii) Depression filling.



13. Population = 65000

Per capita generation = 2kg / capita / day.

$P_{SW} = 650$ kg/m³.

Depth of land fill = 5 m.

Total solid waste generated/year = $65000 \times 2 \times 365$

$= 47.45 \times 10^6$ kg/year.

Volume of solid waste generated per year $= \frac{47.45 \times 10^6}{650} = 73000\ m^3$.

Area of fill required/annum $= \frac{\text{Volume}}{\text{depth}} = \frac{73000}{5} = 14600\ m^2$

$= 1.46$ ha

2. Incineration

- Combustable solid waste disposed by this method.
- burning in the presence of air.

3. Pyralosis

- Burning in the absence of air.
- plastic and rubber waste disposed by this method.
- this yield oil, tar etc.

4. Pulvarisation

- non biodegradable and non combustible solid waste disposed by this method.
- cutting or shredding into small pieces.

5. Sea disposal

- Barged into sea and disposed into deep sea water.

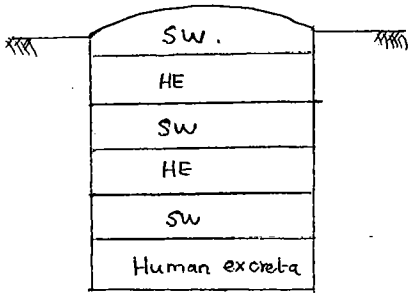
6. Composting

Human excreta + Solid waste → aerobically or anaerobically decomposed → compost (manure)

- biodegradable solid wastes are ~~impro~~ decomposed by this method.

(i) Indore method of composting
— aerobic method.

(ii) Bangalore method of composting.



22nd nov, SATURDAY

12. AIR POLLUTION & CONTROL

Presence of one or more contaminants in ambient atmosphere, affect human, animal, plant life and also damage property.

→ Composition of Air

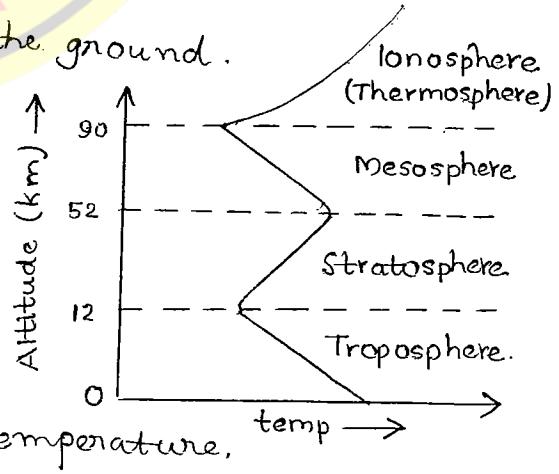
Nitrogen	⟶	78.09 %
Oxygen	⟶	20.95 %
Argon	⟶	0.93 %
CO_2	⟶	0.032 %

Remaining gases ⟶ traces.

→ Structure of Atmosphere.

Atmosphere is the space above the ground.

(i) Troposphere
(ii) Stratosphere
(iii) Mesosphere.
(iv) Ionosphere



Above classification is based on temperature.

→ Sources of Air Pollution

1. Based on Origin of Source.

(i) Natural Sources.

Eg: dust storms, forest fires, ocean spray, volcanoes etc.

(ii) Anthropogenic Sources (man made).

Eg: Industries, automobiles, thermal power stations etc.

2. Based on Spatial Distribution

(i) Point Source. Eg: industry at a place.

(ii) Areal Source. Eg: industrial area (group of industries).

3. Based on position of Source.

(i) Stationary source.

Eg: Industries.

(ii) Mobile source.

Eg: truck on highway.

→ Classification of Air Pollutants

1. Based on Origin

(i) Primary air pollutant.

Eg: particulates, CO, SO_x, NO_x, hydrocarbons etc.

(ii) Secondary air pollutant.

Eg: ozone, PAN, PBN, PPN, smog (due to photochemical reactions)

2. Based on Nature.

(i) Organic pollutant

Eg: hydrocarbons.

(ii) Inorganic pollutants.

Eg: SO_x, NO_x, CO etc.

3. Based on State of matter.

(i) Particulates

a) liquid Eg: mist.

b) Solid Eg: Dust

(ii) Gases.

Eg: CO, SO_x, NO_x etc.

→ Concentration of Air Pollutants.

Concentration of air pollutants in atmosphere are monitored and expressed in terms of $\mu g/m^3$ / ppm / ppb

06. Concentration of CO = 9 ppm = 9 mg/L

$$= \frac{9 \times 10^{-3}}{10^{-3}} \frac{\mu g}{m^3} = 9 \times 10^6 \ \mu g/m^3$$ } WRONG

1 ppm = 1 mg/L (only for water)

1 ppm ≠ 1 mg/L (for air)

$PV = RT$; T (in K) and P (in atm).

R → universal gas constant $= 82.05 \times 10^{-6} \ \frac{atm.m^3}{mol.\ K}$

At standard temperature and pressure (T = 0°c & P = 1 atm).

$$V = 82.05 \times 10^{-6} * \frac{273}{1}$$

$$= 0.224\ m^3 = 22.4 \times 10^{-3}\ m^3$$

M grams of any gas at STP occupy $= 22.4 \times 10^{-3}\ m^3$ of space.

1 m^3 of any gas at STP weights $= \frac{M}{22.4 \times 10^{-3}}$ gm.

$$1 \text{ ppm} = \frac{1 \text{ part of any gas}}{10^6 \text{ parts of air}} = \frac{1\ m^3 \text{ of any gas}}{10^6\ m^3 \text{ of air}}.$$

$$1 \text{ppm} = \frac{M}{22.4 \times 10^{-3} \times 10^6}\ g/m^3 = \frac{M}{22.4 \times 10^3}\ g/m^3$$

$$= \frac{M \times 10^6}{22.4 \times 10^3}\ \mu g/m^3 = \frac{M * 10^3}{22.4}\ \mu g/m^3$$

$$1 \text{ ppm @ STP} = \frac{M}{22.4} \times 10^3 \ \mu g/m^3$$

$$1 \ \mu g/m^3 \text{ @ STP} = \frac{22.4}{M \times 10^3} \text{ ppm}$$

$$\boxed{1 \text{ ppm @ temp \& pressure} = \frac{M}{\text{constant}} * 10^3 \ \mu g/m^3}$$

M → molecular wt.

$$1 \ \mu g/m^3 \text{ @ T \& P} = \frac{\text{constant}}{M * 10^3} \text{ ppm}$$

$$\boxed{\text{constant} = 82.05 \times 10^{-6} \times \frac{T}{P} \times 10^3 = 82.05 \times 10^{-3} \frac{T}{P}}$$

$$\boxed{1 \text{ ppb @ T \& P} = \frac{M}{\text{const.}} \ \mu g/m^3}$$

06. 9 ppm of CO @ STP $= 9 * \frac{M}{22.4} * 10^3 \ \mu g/m^3$.

$$= 9 * \frac{(12+16)}{22.4} * 10^3 = 11250 \ \mu g/m^3.$$

$$= \underline{11.25} \ mg/m^3$$

07. Concentration of chloroform ($CHCl_3$) $= 0.4 \ \mu g/m^3$

$$1 \ \mu g/m^3 = \frac{\text{constant}}{M} \text{ ppb}$$

$$\text{constant} = 0.08205 * \frac{298}{1} = 24.04$$

$$0.04 \ \mu g/m^3 = \frac{24.04}{(35.5 \times 3 + 12 + 1)} \text{ ppb} = \underline{0.08} \text{ ppb}$$

Q.1 Convert 120 $\mu g/m^3$ of SO_2 into ppm and ppb at temp 10°c and pressure 1 atm. (S = 32, O = 16)

$$1\ \mu g/m^3 = \frac{\text{const}}{M}\ ppb = \frac{23.22}{(32+16\times2)} = 0.3628\ ppb.$$

$$\text{const} = 0.08205 \times \frac{283}{1}$$

$$= 23.22 \Rightarrow 120\ \mu g/m^3 = 43.54\ ppb$$

=

Q.2 Convert 40 ppb of NO_2 into $\mu g/m^3$ and ppm at temp 20°c and pressure 1 atm.

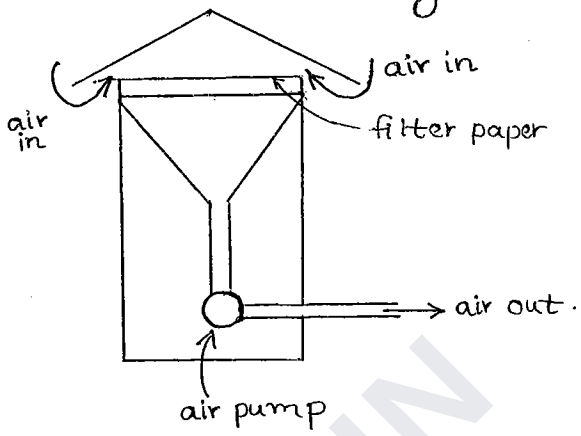
4th nov, WEDNESDAY → Major Air Pollutants.

1. Particulates. (TSP or SPM)

TSP → total suspended particle
SPM → suspended particulate matter

- 0.01 μm to 200 μm
- Causes respiratory diseases.
- Particulates are of two types:
  - (i) Solid particulates.
    Eg: Dust, smoke, fumes, fly ash etc.
  - (ii) Liquid particulates.
    Eg: mist, spray etc.
  - (iii) Aerosoles
    Solid or liquid particulates of size < 1 μm which always remain in suspended in atmosphere.

- Particulates in atmosphere are measured by 'High Volume Air Sampler'.



TSP (or) SPM ($\mu g/m^3$)

$$= \frac{(W_{final} - W_{initial})_{filter\ paper}}{\text{Volume of air sampled.}}$$

P-120

11. $W_{initial} = 9.787\ g.$

$W_{final} = 10.283\ g.$

Initial rate of air sampling $= 1.5\ m^3/min.$

Final rate of air sampling $= 1.4\ m^3/min.$

∴ Average rate of air sampling $= \frac{1.5+1.4}{2} = 1.45\ m^3/min$

Volume of air sampled in 24 hours $= 1.45 \times 24 \times 60$

$$TSP = \frac{(10.283 - 9.787)\ 10^6}{1.45 \times 24 \times 60} = 237.55\ \mu g/m^3$$

2. Carbon Monoxide, CO

- CO is an inert gas.
- CO reacts with haemoglobin in blood to form toxic compound carboxyhaemoglobin.

$$CO + Hb \longrightarrow CoHb \text{ (carboxyhaemoglobin)}$$

- CoHb is Asphyxiant (toxic chemical substance) which causes Asphyxia (breathlessness) and at higher concentrations may cause even death.
- CO is analysed by NDIR analyser (Non ~~Destructive~~ Dispersive Infrared)

3. Oxides of Sulphur, SO_x

- SO_2 damage buildings (marble structures), affect plant life, irritate mucus membrane of respiratory tract. (respiratory irritant).
- SO_x are measured by West & Gaek Method.
- SO_2 & SO_3 are most dangerous.

Q A thermal power station consuming coal @ 5t/hr containing 2% sulphur. Find SO_2 emission from plant in g/s.

Sulphur SO_2 emission $= \frac{2}{100} \times \frac{5 \times 10^6}{3600} = \underline{\underline{27.78}}$ g/s

$$S + O_2 \longrightarrow SO_2$$

32 parts S $\longrightarrow$ 64 parts SO_2

1 part S $\longrightarrow \frac{64}{32} = 2$ part SO_2.

$\therefore$ SO_2 emission $= 27.78 \times 2 = \underline{\underline{55.56}}$ g/s

4. Oxides of Nitrogen, NO_x

- NO & NO_2 concentrations are more critical.
- most of secondary air pollutants are produced due to NO_x. Eg:- Ozone, PAN etc.
- NO_x damage alveoli (respiratory component in lungs), plant life etc.
- NO_x are estimated by Jacob - Oscheisser method.

21. Rate of NO_x emission $= 2$ g/km/vehicle.

No: of vehicles $= 50000$

Distance travelled $= 20000$ km

$\therefore$ Quantity of NO_x produced $= 2 \times 10^{-6} \times 50,000 \times 20,000$

$= \underline{\underline{2000}}$ t

5. Hydrocarbons

- exposure to high conc. of hydrocarbon cause lung cancer
- estimated by Gas chromatography.

Emission let into atmosphere by various sources remain in atmosphere temporarily for certain period. During that period, they travel in atmosphere and their concentrations decreases due to dispersion and dilution.

Dispersion and dilution of air pollutants in atmosphere depends on meteorological factors (weather factors).

→ The following meteorological factors affect dispersion and dilution :-

Dispersion → spread.
dilution → mix.

imp

1. Temperature

- Temperature is the measure of heat energy.
- Lapse Rate : Change of temperature with altitude.

$\frac{dT}{dz}$ → lapse rate.

* Two types of Lapse Rate :-

(i) ALR – Adiabatic Lapse Rate.

- Any warm air parcel if it rise in ~~to~~ atmosphere, for every 1 km rise, change of temperature is $-9.8^{\circ}C$

$$\frac{dT}{dz} = -9.8^{\circ}C/1\,km = -1^{\circ}C/100\,m$$

- ALR is always constant. It doesn't change from time to time and place to place.

(ii) ELR – Environmental Lapse Rate

- Lapse rate of atmosphere at a given time at a given place.

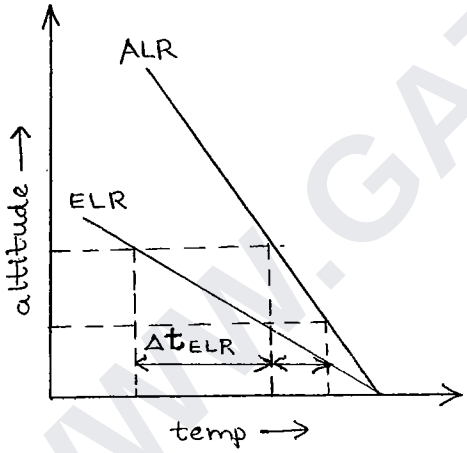
- It changes from time to time at a given place. It is also known as 'prevailing lapse rate'.

- Dispersion and dilution of pollutants in atmosphere depends on difference blw ELR and ALR.

Case 1: ELR > ALR (4.00 pm – 7.00 pm).

- ELR is known as 'Super – Adiabatic Lapse Rate'.

- during this period, atmosphere is said to be in unstable condition, which is good for pollutant dispersion and dilution.



1 km ↑ 100m

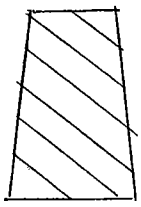
smoke $T_s = 30^\circ C$ $T_a = 24^\circ C$

$T_s = 39^\circ C$ $T_a = 37.5^\circ C$

ALR $-1^\circ C/100m$

$T_s = 40^\circ C$ $T_a = 39^\circ C$

$ELR = -1.5^\circ C/km$.



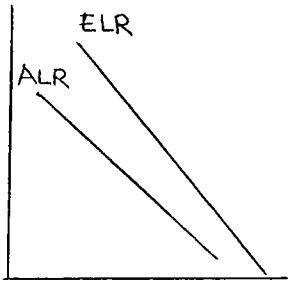
$T_s - T_a$ value ↑ and ∴ smoke rises in atmosphere.

Case 2: ELR < ALR. (8.00 am – 11.00 am).

- ELR now known as 'Sub-adiabatic Lapse Rate'

- during this period, atmosphere is said to be stable.

- Stable atmosphere is bad for pollutant dispersion and dilution.



200m $T_s = 38^\circ C$ $T_a = 38^\circ C$

100m $T_s = 39^\circ C$ $T_a = 38.5^\circ C$

ALR $-1^\circ C/100m$ $T_s = 40^\circ C$ $T_a = 39^\circ C$.

$ELR = -0.5^\circ C/100m$.

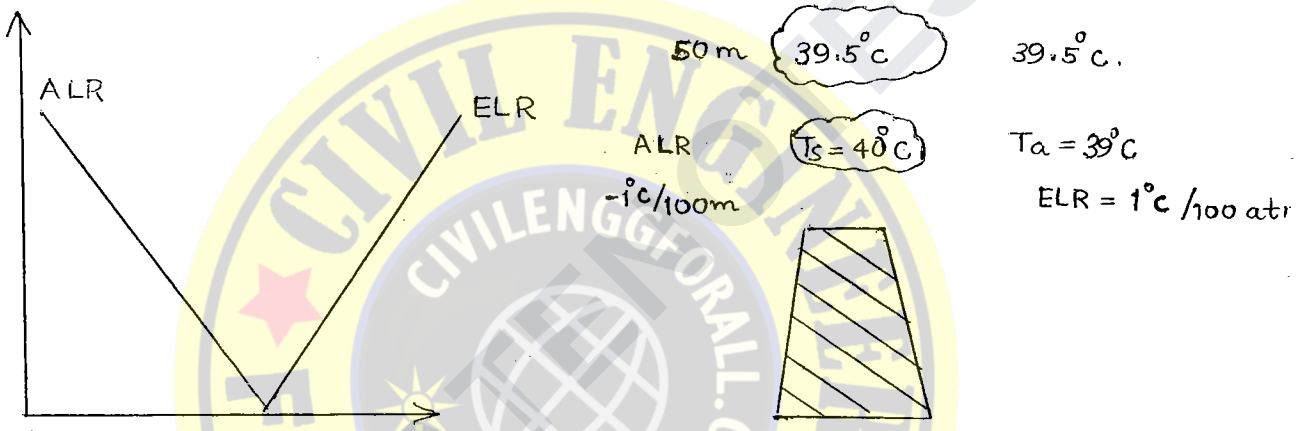
case 3 : ELR : −ve lapse rate. (occurs at midnight)

- Temperature increases with altitude, then $\frac{dT}{dz}$, lapse rate is known as inversion.

$$\frac{dT}{dz} = +ve \Rightarrow -ve \text{ lapse rate. (inversion)}$$

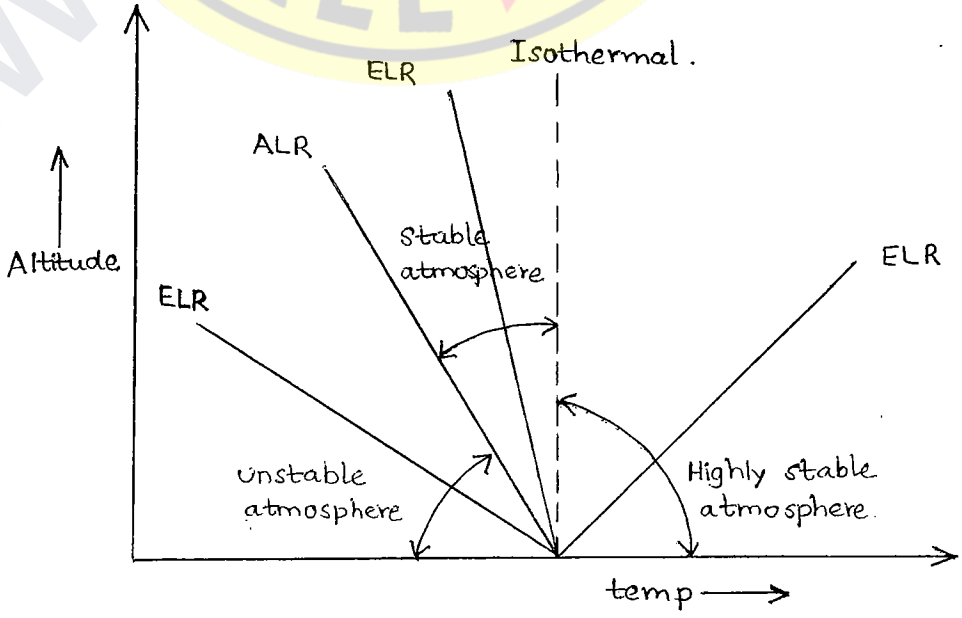
- During inversion, atmosphere is said to be highly ~~unstable~~ stable
- Highly stable atmospheric, is worse for pollutant dilution and dispersion.

■ smoke won't rise above 50 m.



case 4 : ELR = ALR (ideal condition)

- atmosphere is said to be neutral.



- Lapse rate (ELR) is the measure of atmospheric stability

27.

z	T°C
4m	21.25
444m	15.7

ALR = −1°C/100 m.

$$ELR = \frac{dT}{dz} = \frac{T_2 - T_1}{z_2 - z_1} * 100$$

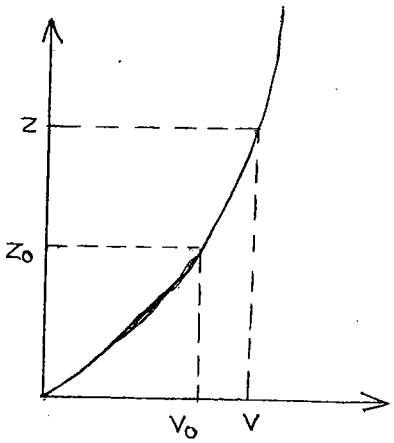
$$= \frac{15.7 - 21.25}{444 - 4} * 100 = -1.26^\circ/100\,m.$$

ELR > ALR ⇒ Super adiabatic lapse rate.

2. Winds.

- air in motion is wind
- horizontal movement (transport) of pollutants in atmosphere depends on winds.
- winds in atmosphere represent unstable condition.
- no wind condition gives quite and calm atmosphere, ie, stable atmospheric condition.
- Wind at a place is measured by:
  - (i) Anemometer → wind speed.
  - (ii) Wind Vane → wind direction.
- Wind speed at any altitude z can be found by $\frac{1}{7}$ th law:

$$\boxed{\frac{V}{V_0} = \left(\frac{z}{z_0}\right)^{1/7}}$$



Q. An anemometer installed at a ht. of 3m recorded a wind speed of 2m/s. What is the wind speed at a ht. of 300m above ground.

$$\frac{V}{2} = \left(\frac{300}{3}\right)^{1/7}$$

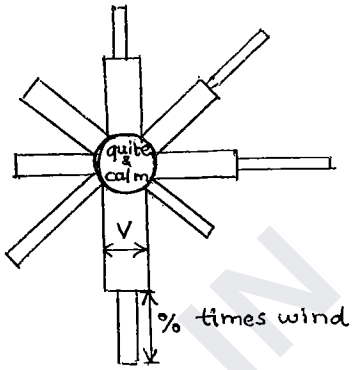
$$\Rightarrow V = 3.86 \text{ m/s}$$

– Wind data at a place is ~~measured~~ represented by using Wind Rose Diagram.

3. Pressure

(i) High Pressure System (Antic cyclone).

– quite and calm atmosphere
– stable atmospheric condition



wind rose diagram.

(ii) Low Pressure System (Cyclone).

– gusty winds with highly unstable atmospheric condition

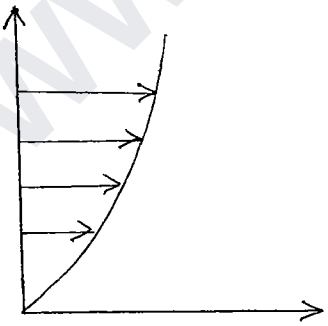
4. Humidity

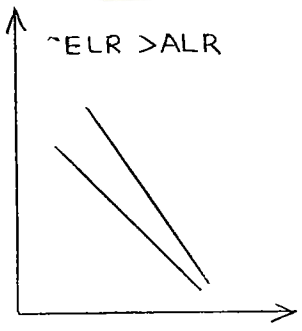
– moisture in atmosphere.
– secondary pollutants in atmosphere formed in humid atmospheric conditions
– No. secondary pollutants formed in dry atmospheric conditi

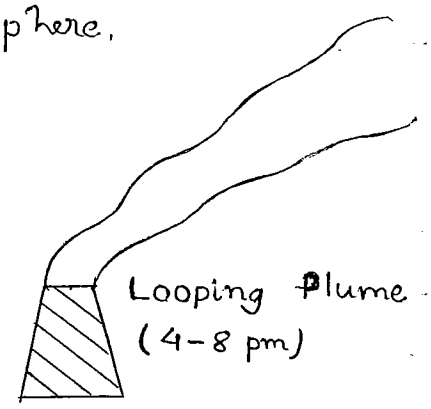
→ Plume Rise models :

– Plume : puff of smoke let into atmosphere.

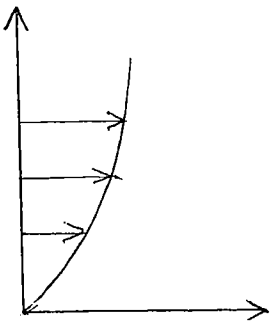
(i)

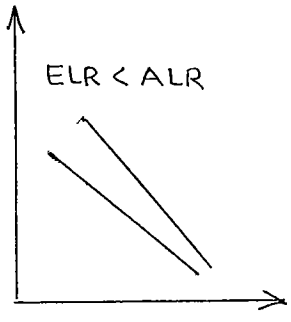


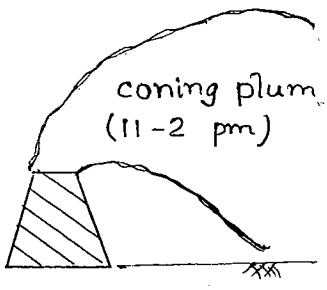




(ii)

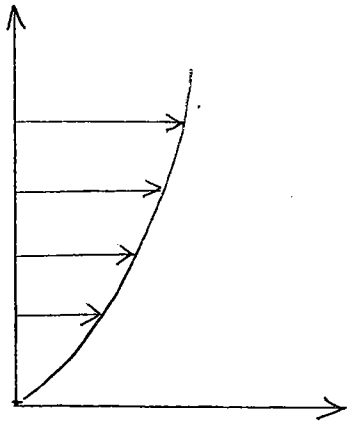


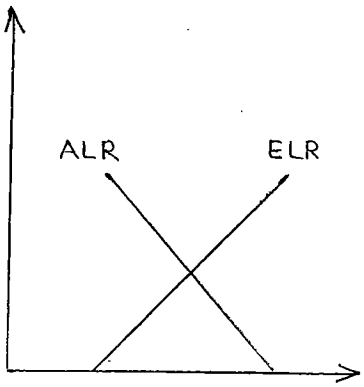




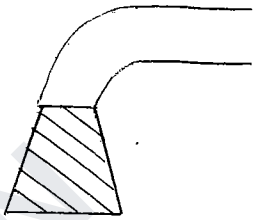
- Inversion (mid-night).

(iii)

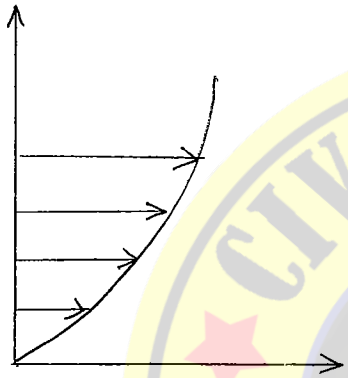


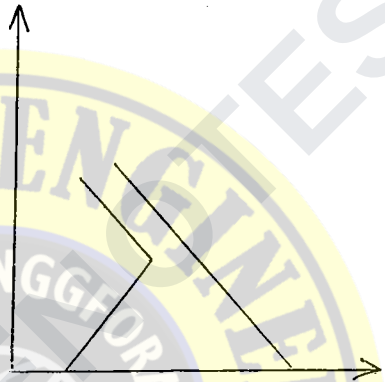


Fanning Plume.

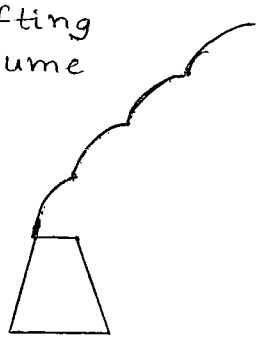


(iv)

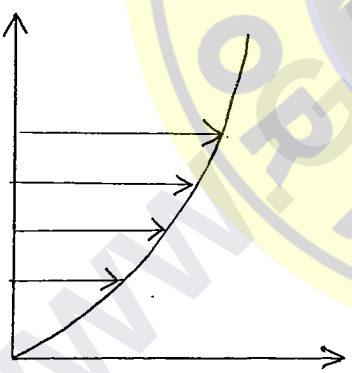


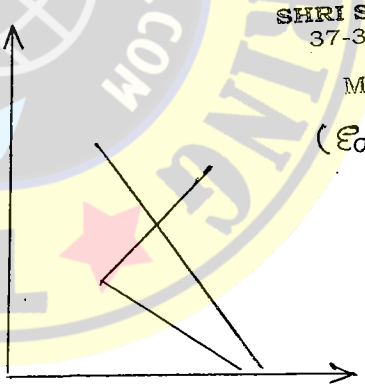


Lofting flume

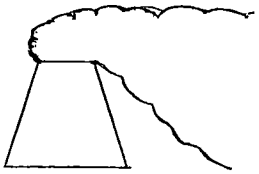


(v)

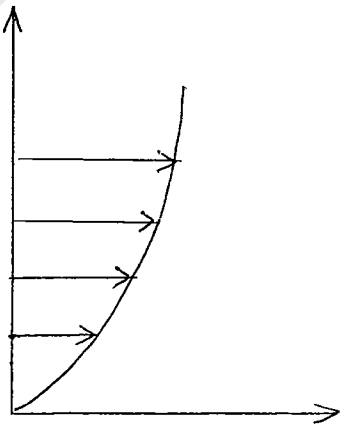


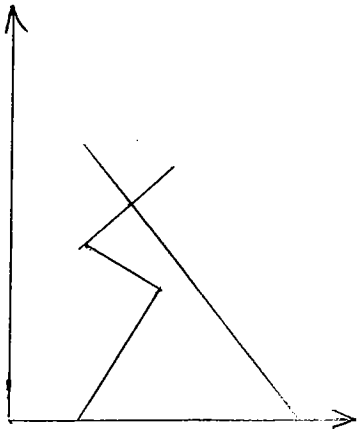


Fumigation.
(Early morning)

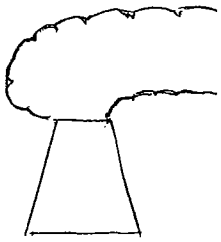


(vi)





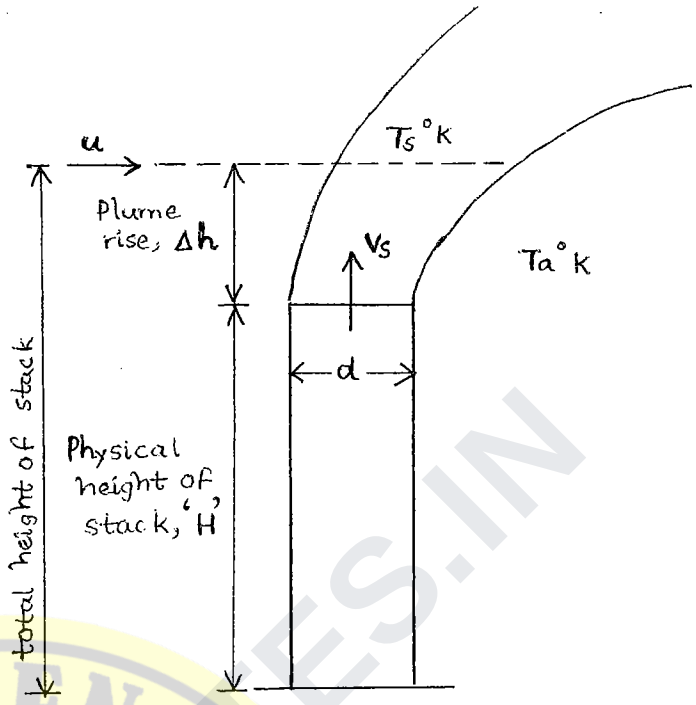
Trapping Plume.



27th nov, THURSDAY

To find plume rise, 'Δh,' Hollend formula is used.

$$\Delta h = \frac{V_s d}{u}\left(1.5 + 2.68\times10^{-3} p d \frac{\Delta T}{T_s}\right)$$



V_s → stack gas velocity.

d → diameter of stack

u → wind speed (wind velocity).

p → atm. pressure (in millibars)

$\Delta T = T_s - T_a$.

T_s → stack gas temp. (°k)

T_a → atm. temp (°k)

P-121.
Q.23

$$\Delta h = \frac{V_s d}{u}\left(1.5 + 0.0096 \frac{Q_h}{V_s d}\right).$$

$$= \frac{8\times1}{4}\left(1.5 + 0.0096 \times \frac{2000}{8\times1}\right). = 7.8 \text{ m}$$

* Ringlemen Charts:

- density of smoke is measured by Rinklemen Charts.

* Isokinetic Sampling:

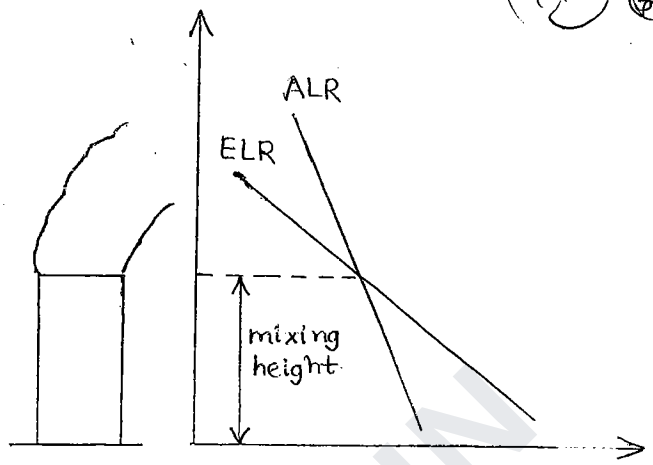
- Collection of smoke using the probe across the c/s of stack

→ Physical Stack height

- Physical stack height is fixed at a place by 'maximum mixing depth' (or) 'mixing height'

→ Air Pollution Control

1. Preventive measures :-
2. Using control technologies.



* Preventive measures (Source correction methods) :

(i) Change of raw materials.

(ii) Change of technology (or) change of processes

(iii) Using tall stacks.

(iv) Zoning and planning.

(v) Strict legislation.

* Control Technologies :

(i) Particulate Control Technologies -

a) Gravity settling chambers.

b) Centrifugal seperators.

c). Electrostatic precipitator.

d). Fabric filters.

(ii) Gaseous Control Technologies -

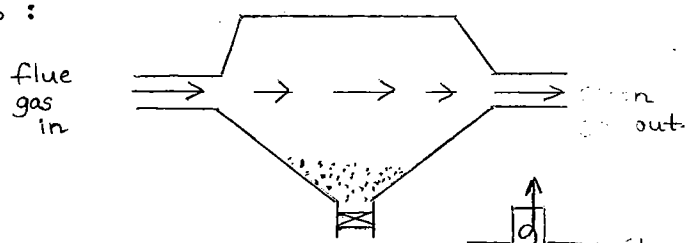
a) Adsorption.

b) Absorption

c) Combustion.

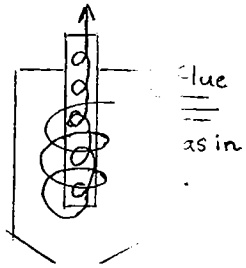
* Gravity Settling Chambers :

- $d_p > 50\ \mu m$
- $\eta < 50\%$

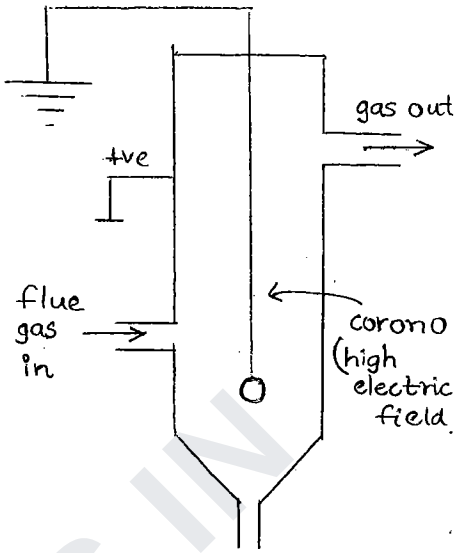


* Centrifugal (or) Cyclone Seperators

- d_p : 5 μm – 25 μm
- η : 50-90 %



* Electrostatic Precipitators

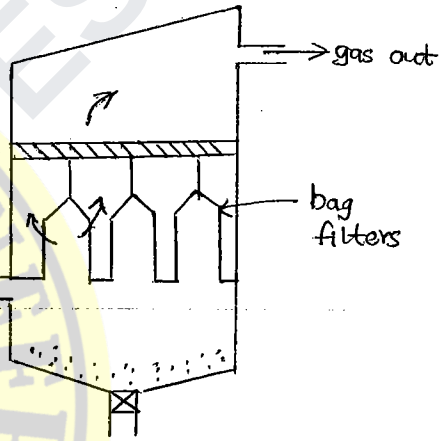


- $dp \geq 1\ \mu m$
- η : 95 to 99 %

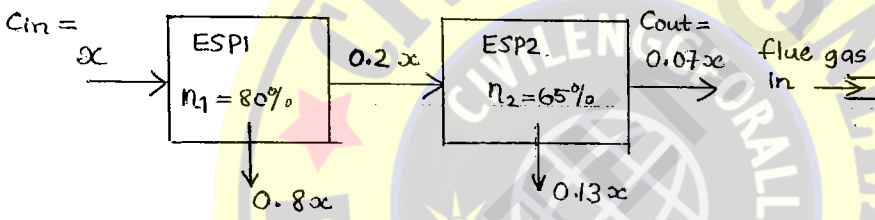
$$\eta = \left(1 - e^{\frac{-Aw}{\phi}}\right) * 100$$

* Fabric Filters.

- $dp < 1\ \mu m$
- $\eta > 99\%$



05.



$$\eta_{overall} = \frac{x - 0.07x}{x} * 100$$

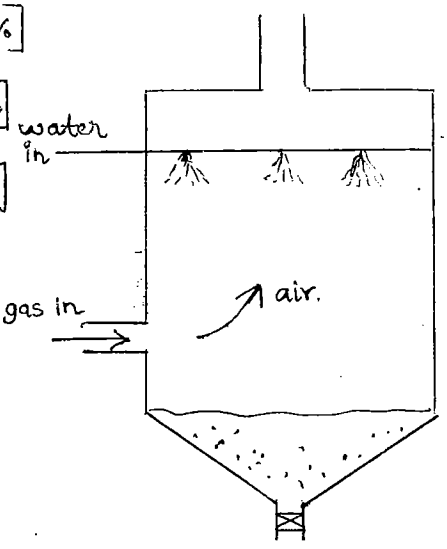
$$= 93\%$$

* Particulates can also be removed by Wet Control Devices

(i) Tray Scrubber. [dp > 10 μm, η < 80%]

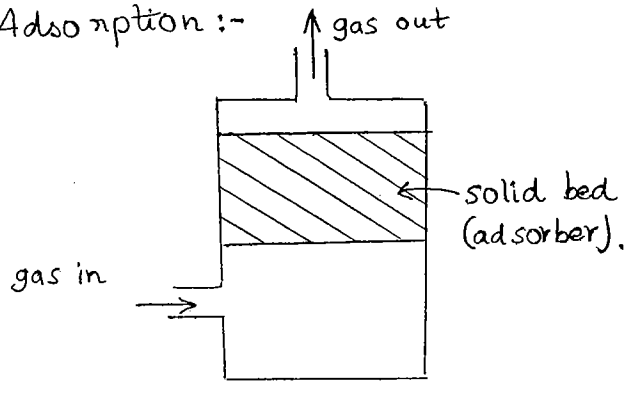
(ii) Cyclone Scrubber. [dp > 2.5 μm, η < 85%]

(iii) Venturi Scrubber [dp > 0.5 μm, η < 90%]

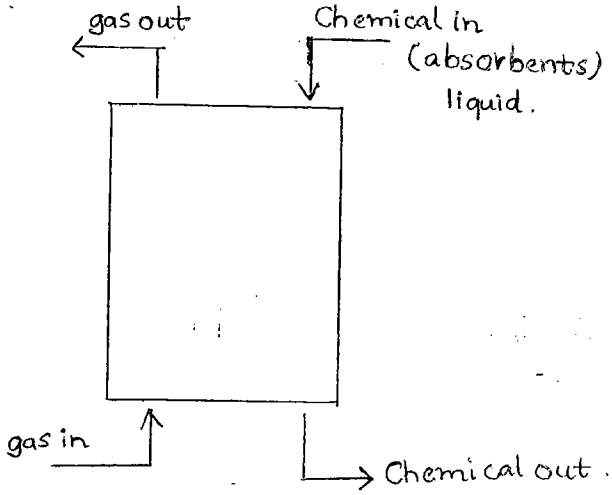


* Gaseous control technologies:

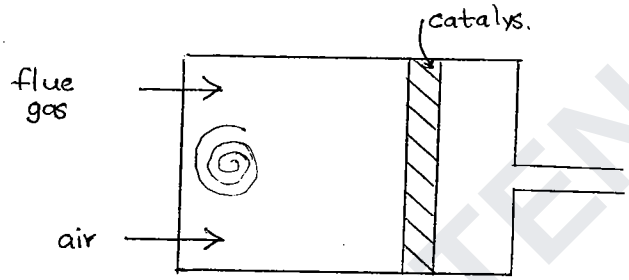
1. Adsorption :-



2. Absorption



3. Combustion



27th NOV,
THURSDAY

13. NOISE POLLUTION

- Noise is an unwanted sound.

- Sound is measured in terms of SPL (Sound Pressure Leve and expressed in dB (deciBell).

$$\boxed{\text{SPL "L"} = 20 \log_{10} \frac{P}{P_o}}$$

where $P \rightarrow$ pressure in μPa.

$P_o \rightarrow$ reference pressure. (20 μPa)

Sound is sensed by humans when pressure ≥ 20 μPa

$$1\ \mu\text{bar} = 1 * 10^5\ \mu\text{Pa}$$

P-124

Q.5 $P = 2000\ \mu\text{bar} = 2.000 \times 10^5\ \mu\text{Pa}.$

$$SPL = 20 \log_{10} \frac{2000 \times 10^5}{20} = \underline{\underline{140\ dB}}$$

Q.8 $P = 2 \times 10^{-4}\ \mu\text{bar} = 2 \times 10^{-4} \times 10^5\ \mu\text{Pa}.$

$$SPL = 20 \log_{10} \frac{20}{20} = \underline{\underline{0}}$$

- When many noise sources emitting different sounds $L_1, L_2, \ldots L_n$ at a time at a given place.

$$\boxed{\left.\begin{array}{l}\text{Total sound produced} \\ \text{by different sources}\end{array}\right\} L_{total} = 20 \log_{10} \frac{P_{rms}}{P_o}}$$

$$P_{rms} = \sqrt{P_1^2 + P_2^2 + \ldots P_n^2}$$

$$L_1 = 20 \log_{10} \frac{P_1}{P_0} \Rightarrow P_1 = ?$$

$$L_2 = 20 \log_{10} \frac{P_2}{P_0} \Rightarrow P_2 = ?$$

$$L_n = 20 \log_{10} \frac{P_n}{P_0} \Rightarrow P_n = ?$$

P-124

i. $L_1 = 20 \log_{10} \frac{P_1}{P_0}$

$60 = 20 \log_{10} \frac{P_1}{20} \Rightarrow P_1 = 20000 \ \mu Pa.$

$L_1 = L_2$

$\therefore P_1 = P_2 = 20000 \ \mu Pa.$

$P_{rms} = \sqrt{20000^2 + 20000^2} = 28284.27$

Total noise $= 20 \log_{10} \frac{P_{rms}}{P_0} = 20 \log_{10} \frac{28284.27}{20} = \underline{\underline{63 \text{ dB}}}$

* L_1 & L_2 are two sounds

$L_1 > L_2$

$L_1 - L_2$	Add to L_1
0 to 1	3 dB
2 to 3	2dB
4 to 8	1dB
>9	0dB

Q. There are 5 AC in a room emitting 59 dB, 63 dB, 66 dB, 69 dB, and 63 dB sound reptly. Find total sound produced by AC in a room.

59 63 63 66 69

66

69

72 $\Rightarrow \underline{\underline{72 \text{ dB}}}$

9.

L_1	L_2
50 dB	50 dB

$L_1 - L_2 = 0 \Rightarrow$ Add 3 dB to 50 dB.

$\therefore$ Total noise $= 50 + 3 = \underline{\underline{53\ dB}}$

* A source emitting different sounds at different time periods:

$$\text{Average sound of source, } \bar{L} = 20 \log_{10} \frac{1}{N} \Sigma \left(10^{\frac{Li}{20}}\right)$$

where N → no. of sound measurements.

3.

$$\bar{L} = 20 \log_{10} \frac{1}{N} \Sigma\, 10^{\frac{Li}{20}}$$

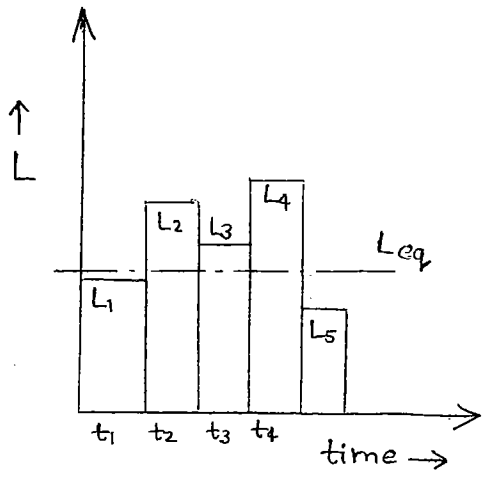
$$= 20 \log_{10} \left\{ \frac{1}{4} \left(10^{20/20} + 10^{56/20} + 10^{66/20} + 10^{42/20} \right) \right\}$$

$$= 20 \log_{10} 690.53 = \underline{\underline{56.78\ dB}}$$

→ Equivalent Noise, L_{eq}

$$L_{eq} = 10 \log_{10} \Sigma \left[10^{\frac{Li}{10}} * Ti \right]$$

where $Ti = \frac{ti}{T}$



04.

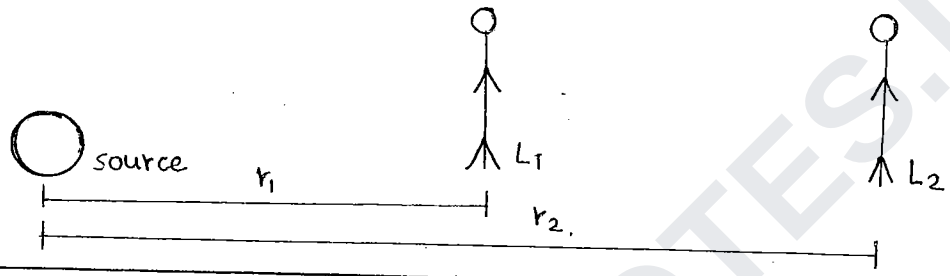
$L_1 = 80$ dB	$t_1 = 10$ min
$L_2 = 60$ dB	$t_2 = 80$ min
$L_3 = 100$ dB	$t_3 = 5$ min.

$T = 95$ min.

$$L_{eq} = 10 \log_{10} \Sigma \left(10^{L_i/10} * T_i\right).$$

$$= 10 \log_{10} \left\{ 10^8 * \frac{10}{95} + 10^6 * \frac{80}{95} + 10^{10} * \frac{5}{95} \right\}$$

$$= 87.3 \text{ dB}$$



$$L_2 = L_1 - 20 \log_{10} \frac{r_2}{r_1}$$

10. $L_2 = 80 - 20 \log_{10} \frac{2r_1}{r_1} = 73.97$ dB

→ Noise Standards :

- given by Ministry of Environment & Forest

Type of Area	Day (dB)	Night (dB)
Sensitive	50	40
Residential	55	45
Commercial	65	55
Industrial.	75	70