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ENVIRONMENTAL ENGINEERING

1. Water Demand (Handout)
2. Sources of Water
3. Water Quality Parameters (Handout + Notes)
4. Treatment of Water
5. Water Distributions (Handout + Notes)
6. Quality Characteristics of Wastewater (Handout)
7. Disposal of Sewage (Handout)
8. Treatment of Sewage
9. Sewers & Sewerage System
10. Solid Waste Management (Handout)
11. Air Pollution (Handout)
12. Noise Pollution

1. WATER DEMAND

14/11/19

Pg. No. 84 (WB)

Q.1)

28000 — 4200 m^3/d

20y

$\boxed{\text{WTP}}$ Capacity = 6000 m^3/d

44000 — $4200 \times \frac{44000}{28000}$

$= 6600\ m^3/d$

$P_n = P_0 + n\bar{x}$

$44000 = 28000 + 20 \times \bar{x}$

$\therefore \bar{x} = 800$

Design Population

$$\frac{6000\ m^3/d}{4200\ m^3/d} \times 28000 = 40000$$

$40000 = 28000 + n \times 800$

$\therefore n = 15$ yrs

Pg. No. 85 (WB)

Q.8)

Years	Population	x	% Increase in Population (r_i)
1981	82	-	$r_1 = \frac{25}{82} \times 100 = 30.48\%$
1991	107	25	$r_2 = \frac{19}{107} \times 100 = 17.75\%$
2001	126	19	$r_3 = \frac{16}{126} \times 100 = 12.7\%$
2011	142	16	
⋮			

$$r = (30.48 \times 17.75 \times 12.7)^{1/3} = 19.01\%.$$

$$P_{2051} = P_{2011}\left[1 + \frac{r}{100}\right]^n$$

$$= 142\left[1 + \frac{19.01}{100}\right]^4$$

$$= 284.8$$

Pg. No. 85 (WB)

Q.9)

Year	Population	Increase (x)	Increment (y)
1940	200000	-	-
1950	380500	180500	-
1960	495500	115000	- 65500
1970	565700	70200	- 44800
1980	650300	84600	+ 14400
		$\bar{x}$ = 112575	$\bar{y}$ = -31966.67

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

$$= 650300 + [2 \times 112575] + \frac{2(2+1)}{2} \times (-31966.67)$$

$$= 779550$$

$$\text{Water Demand} = 779550 \times 225 \text{ l/d}$$

$$= 175.4 \times 10^6 \text{ l/d}$$

$$= 175 \text{ MLD}$$

Pg. No. 85 (WB)

Q. T1>

$P_s = 3,20,000$

$P_0 = 60,000$

$P_1 = 1,30,000$

$P_2 = 2,00,000$

$$m = \frac{P_s - P_0}{P_0} = \frac{320000 - 60000}{60000} = 4.33$$

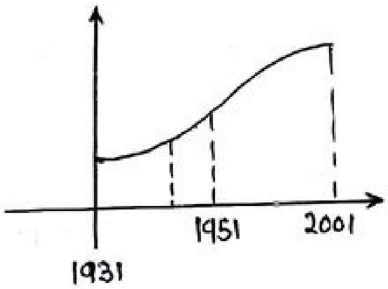
$$n = \frac{1}{10y} \ln\left[\frac{60000(320000 - 130000)}{130000(320000 - 60000)}\right]$$

$$= -0.108\, y^{-1}$$

//Their should not be any unit in exponential Power//

$$P = \frac{P_s}{1 + me^{nt}} = \frac{320000}{1 + 4.33\, e^{-0.108 y^{-1} \times 70y}}$$

$$= 319280$$



Pg. No. 85 (WB)

Q. T4>

$$P_s = \frac{2P_0P_1P_2 - P_1^2(P_0 + P_2)}{P_0P_2 - P_1^2}$$

$$= \frac{2 \times 40000 \times 160000 \times 280000 - (160000)^2(40000 + 280000)}{(40000 \times 280000) - (160000)^2}$$

$$= 320000$$

$$m = \frac{320000 - 40000}{40000} = 7$$

$$n = \frac{1}{t_1} \log_e\left[\frac{P_0(P_s - P_1)}{P_1(P_s - P_0)}\right] = \frac{1}{20} \log_e\left[\frac{40000(320000 - 160000)}{160000(320000 - 40000)}\right]$$

$$= -0.0972\, y^{-1}$$

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

$$= \frac{320000}{1 + 7 \times e^{-0.0972 \times y^{-1} \times 55y}}$$

$$= 309666$$

2. SOURCES OF WATER

Water can be extracted or obtained from following 2 types of Sources

1. Surface Sources
2. Sub - Surface Sources

1. Surface Sources

- Surface Sources include river, reservoirs & lakes.
- Water obtained from ocean & seas is highly ∴ uneconomical to treat for commercial supply. The sources such as Ponds cannot be used commerically as they contain less volume of water.

2. Sub-Surface Sources

- The Sub-Surface sources or under ground sources can be used for commercial purposes if such sources are available to Retent & Yield significant Quantity of water.
- Based upon water retention & Water Yield, following types of Geological Formation are defined.

I. Aquifer

It is a Geological formation which is able to store as well as Yield sufficient quantity of water.

Eg. Fine Sand, Coarse silt etc.

II. Aquiclude (Clay)

It is a Geological formation which is able to store significant quantity of water due to its High porosity but it is practically impermeable

Eg. Clay

III. Aquitard

- It is a Geological formation which is able to store significant quantity of water but deliviers it at very slow rate (in form of Seepage)

Eg. → Silty Clay, Sandy Clay etc.

IV. Aquifuge

It is a Geological formation which is neither permeable nor porous

Eg. → Rocks such as Marble, Granite, Quartz etc.

Out of above Geological formation, only Aquifer can be used for commercial supply of water.

* Types of Aquifers

1> Unconfined Aquifer or Non Artesian Aquifer

- If the Aquifer is not overlain by some confined clay or rock over it, it is referred as Unconfined Aquifer.

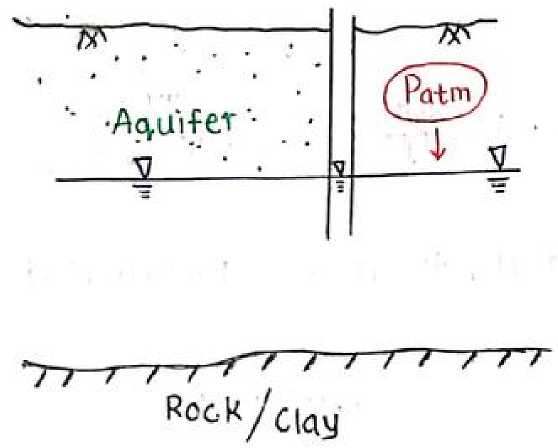
2> Confined Aquifer or Artesian Aquifer

When Aquifer is confined on its upper and under surface by impervious formations & is also somewhat inclined so as to expose the aquifier somewhere in the catchment area at a higher level, which creates a sufficient Hydraulic Head, is called as Confined Aquifer.

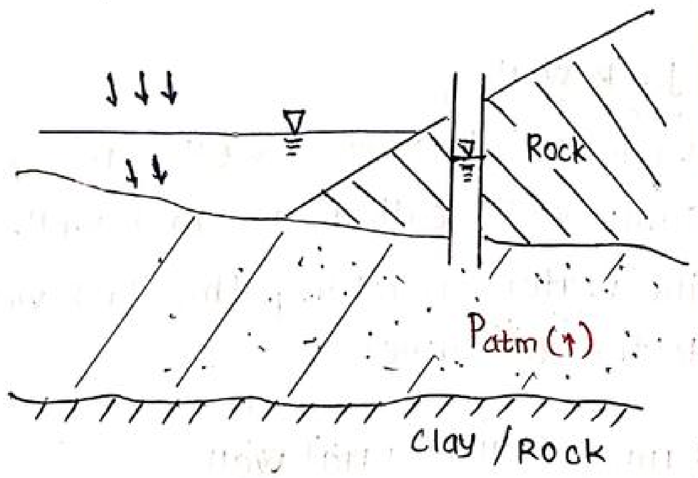
3> Perched Aquifer

It is a special case which is some times found to occur within an unconfined Aquifer. However it is not able to carry significant amount of water & thus it cannot be relied upon for commercial purposes.

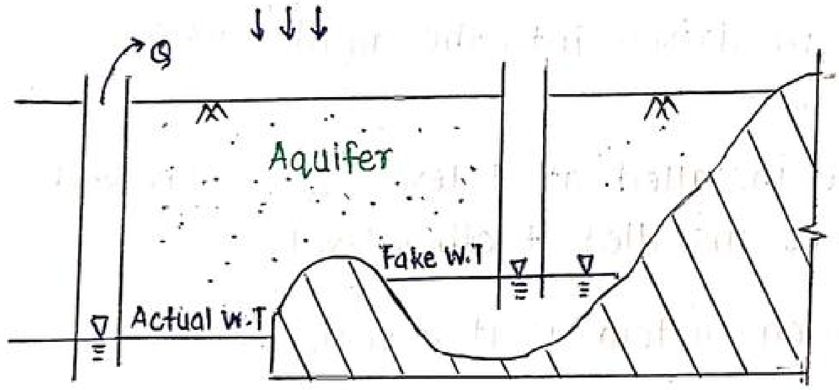
1) Unconfined Aquifers



2) Confined Aquifers



3) Perched Aquifers



* Extraction of Water from Aquifers

1. Infiltration Wells
2. Infiltration Galleries
3. Open & Tube wells
4. Springs

1. Infiltration Wells

These are shallow wells constructed in series along the banks of river In order to collect river water seeping through their bottom.
These are constructed of Brick masonary & Generally covered with

with Man holes provided for Inspection & maintainance.

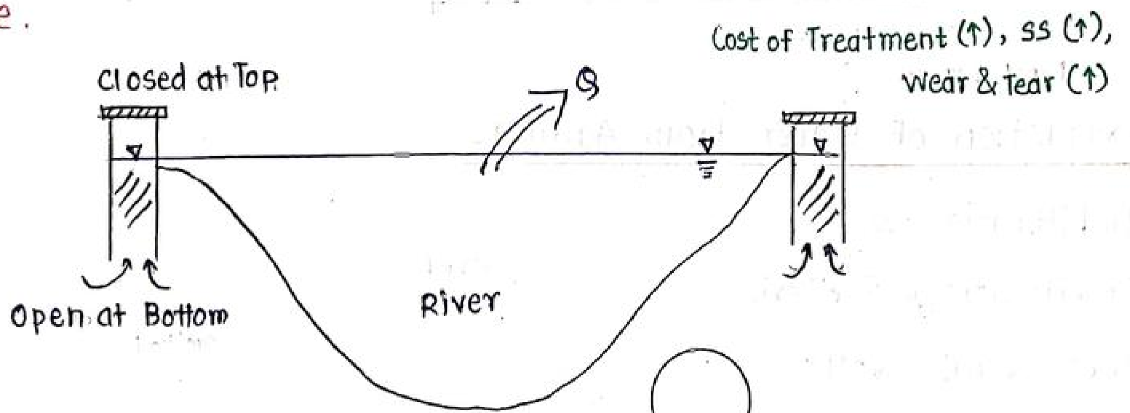
Jack Well

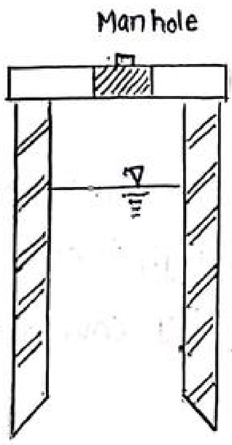
- Various infiltration wells are connected with pipe to a Main or Sump well called as Jack well.
- The water reaching the Jack well is lifted, treated & Distributed to the consumers.

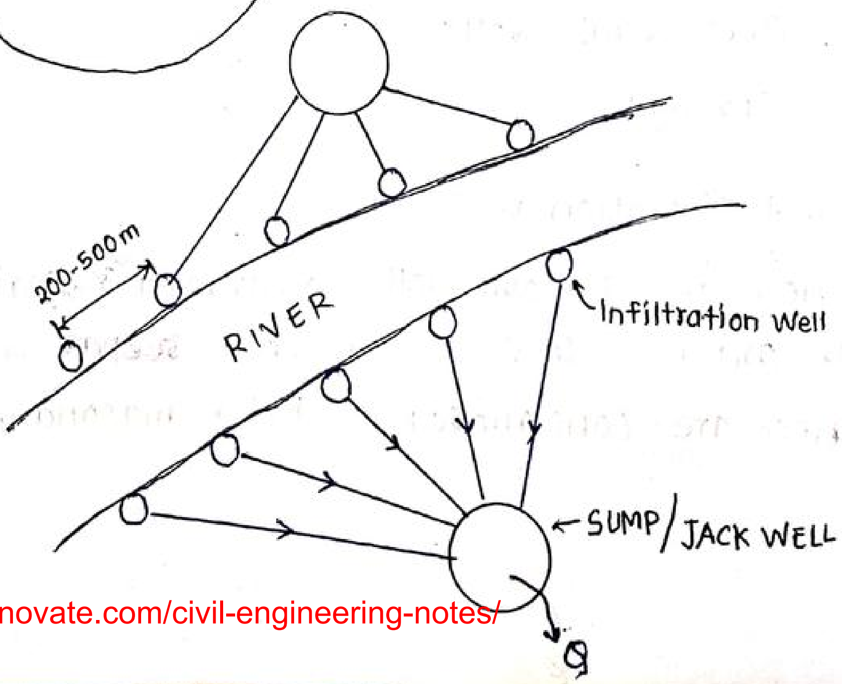
Ranney Well / Radial well

- It is a vertical well of 3m to 6m dia. with horizontal radial collector pipes.
- Horizontal perforated pipes are driven into the aquifer whose lengths are of order 60 - 80m
- About 10 collectors can be installed at 1 level & similar kind of set of such collector can be installed at other level.

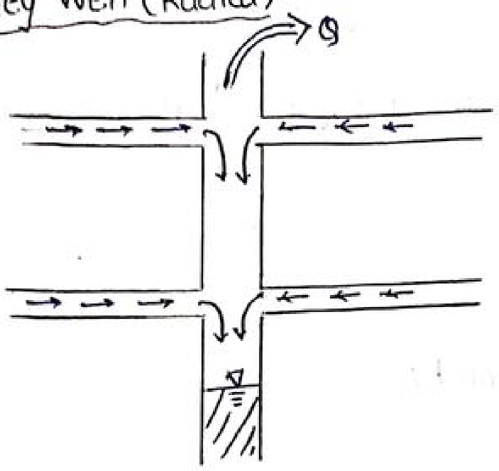
NOTE: It is also known as French System as it is very common in France.







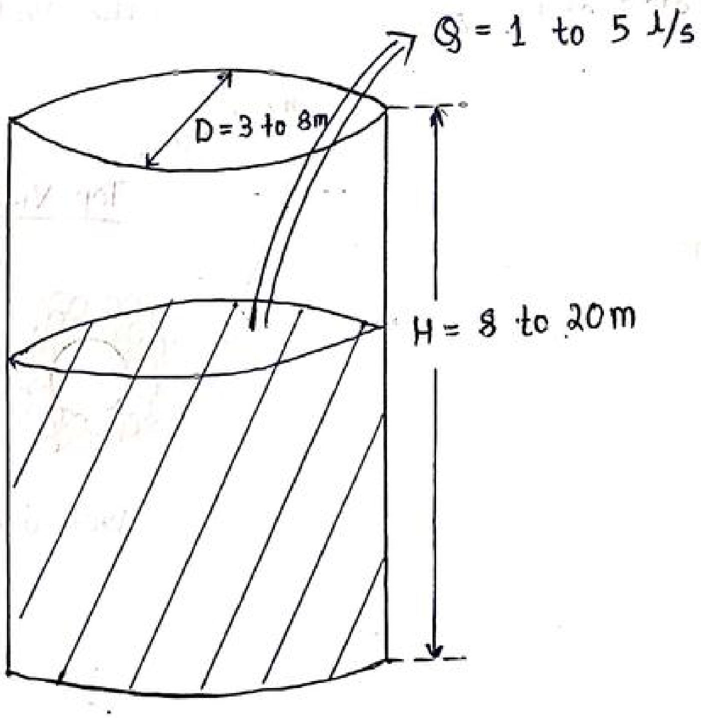
Ranney Well (Radial)



2. Open & Tube Wells

Open Wells / Dug wells

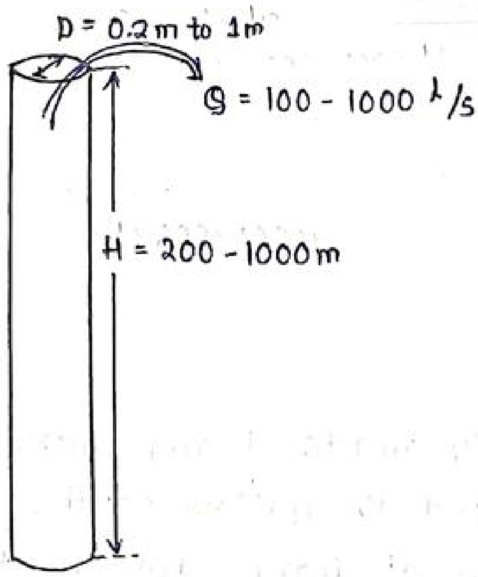
- Open well are generally made of masonary having comparatively bigger dia. & penetrated to shallow depth.
- They are used to obtain discharges for small communities which is in range of 1 to 5 lit/s.
- Yield of such wells is determined by Pumping Test & Recuperation Test



Open well / Dug well

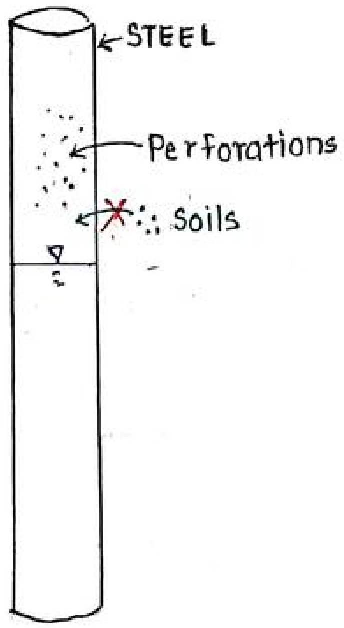
Tube Wells

These are Driven type wells which are used to generate Higher Discharges from great depths.
These are usually made of steel



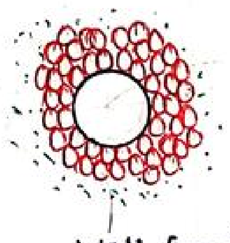
These are of 2 Types

i. Strainer Type Tube Well

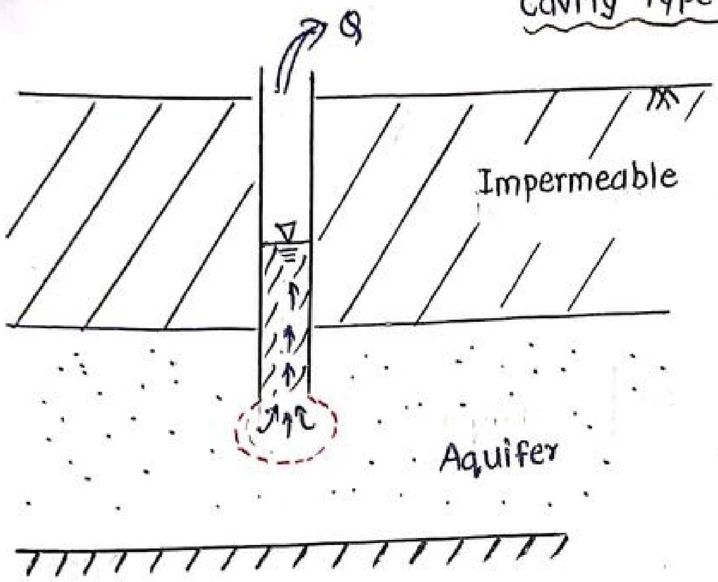


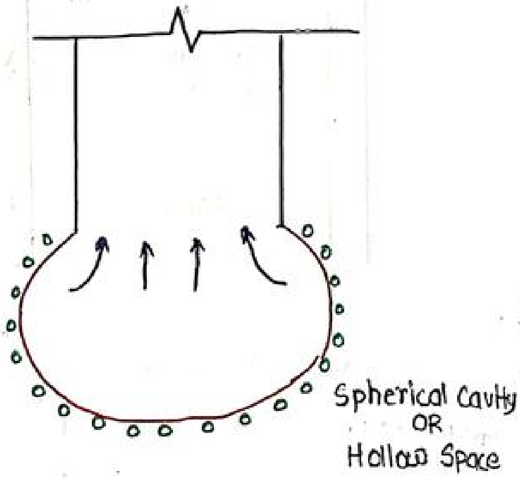
ii. Gravel Pack Type Strainer Tube Well

Top View



Cavity Type Tube Well





* Darcy's Law

15/11/19

$v \propto i$

$\boxed{v = ki}$ → Hydraulic Gradient

k → Coeff. of permeability

v → Flow or Superficial velocity

$Q = vA$

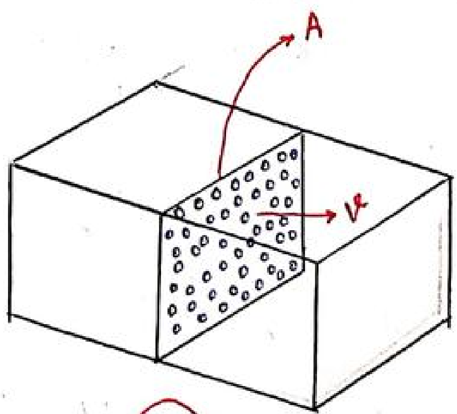
$Q = kiA$

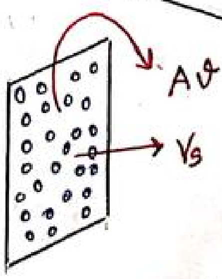
V_s → Actual / Seepage velocity

$$\boxed{V_s = \frac{v}{n}}$$

$v = V_s \times n$

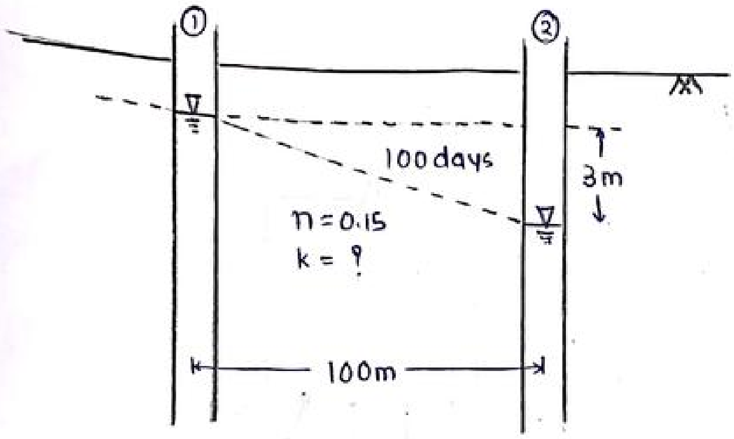
$$\boxed{Q = V_s \times n \times A}$$

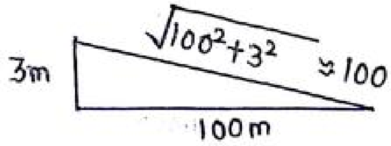




Pg. No. 87 (WB)

Q.9>





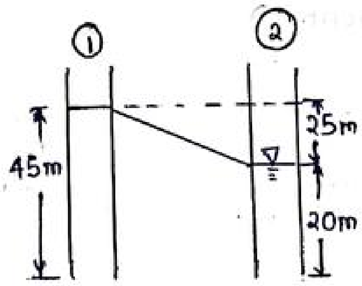
$$v_s = \frac{v}{n} = \frac{ki}{n} \qquad i = \frac{3}{100} = 0.03 \quad , \quad n = 0.15$$

$$1 \ m/d = \frac{k \times 0.03}{0.15}$$

$\therefore$ k = 5 m/day

Pg. No. 88 (WB)

Q.15> k = 30 m/day



$$i = \frac{25}{1500}$$

$$= 0.0167$$

* Yield of a Well

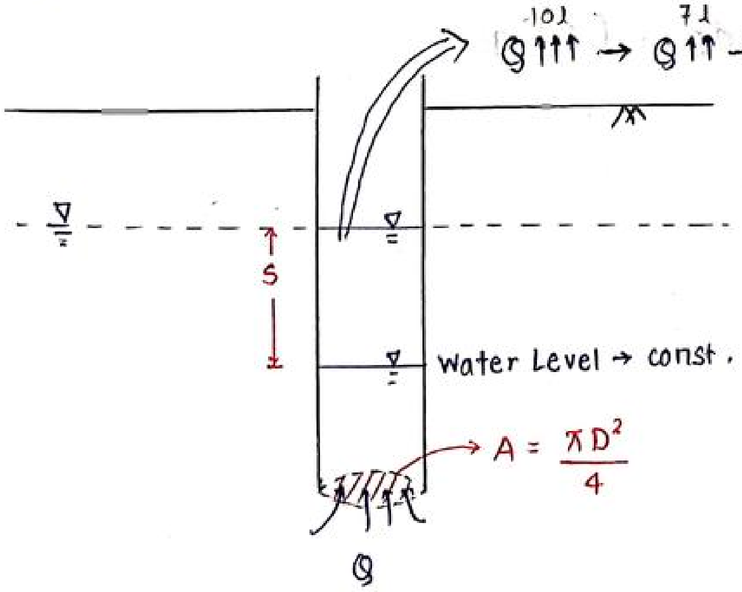
Yield of a Well

- Field Methods
  - Pumping Test / Pumping Out Test
  - Recuperation Test
- Theoretical Methods
  - Dupit's theory for Unconfined Aquifer
  - Thiem's theory for Confined Aquifer

* Pumping Test / Pumping Out Test

- In this test, a heavy drawdown is 1st caused in the well by withdrawal of water at very High Rate.
- The rate of withdrawal is gradually decrease untill the drawdown in the well becomes constant.
- At this particular moment (which is called as pumping equillibrium), the well constant 'C' is found out as follows

$$\overset{10l}{Q\uparrow\uparrow\uparrow} \rightarrow \overset{7l}{Q\uparrow\uparrow} \rightarrow \overset{3l}{Q\uparrow} \rightarrow Q$$ → Examples



At the state of pumping equilibrium :-

$$Q = kiA$$

$$Q = k \times \left[\frac{h_L}{L}\right] \times A$$

$$= \left[\frac{K}{L}\right] \times h_L \times A$$

In this case, $h_L = S$

$$\frac{k}{L} = C \rightarrow \text{well constant}$$

$$\boxed{Q = C \times S \times A}$$

$$C = \frac{Q}{AS}$$

C → unit → tim^{-1}
→ discharge per unit area per unit drawdown
→ constant for a season

Type of Soil

Q	Maxm Permissible Drawdown
15 l/s	13 m

Discharge (Q') at some drawdown (S') as :-

$$Q' = CAS'$$

$$Q' = C \times \frac{\pi}{4} D^2 \times S'$$

Find D

Pg. No. 88

Q.17) $C = 0.6\ h^{-1}$

$Q = 10 \times 10^{-3}\ m^3/s$

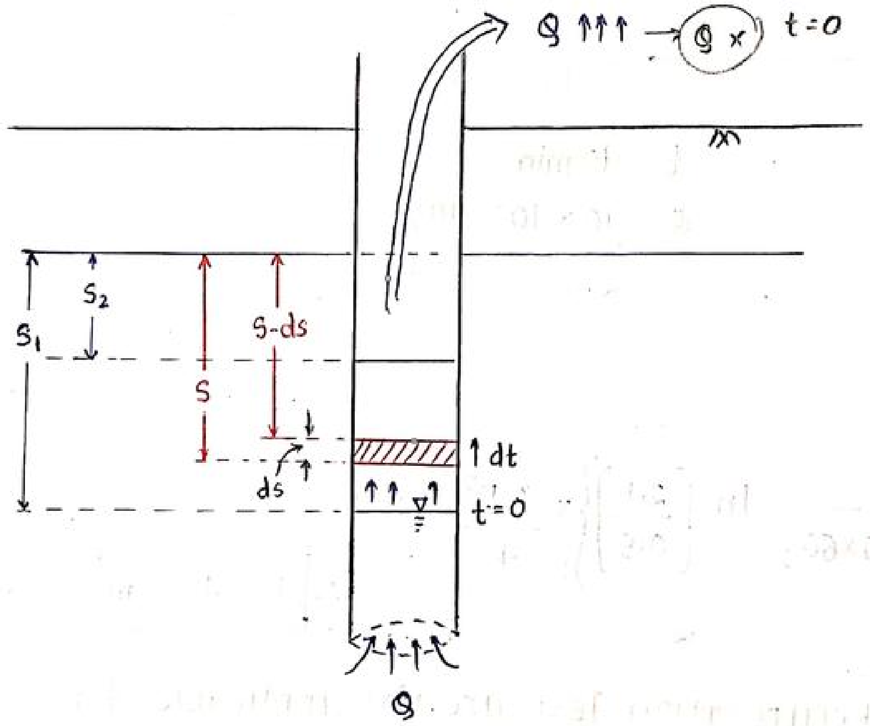
$S = 2.5\,m$

$$10 \times 10^{-3}\ m^3/s = 0.6\ h^{-1} \times \frac{\pi D^2}{4} \times 2.5\ m$$

$\therefore$ ~~D=0.525 m~~ $\therefore D = 5.52\ m$

* Recuperation Test

- In this Test, a heavy drawdown is 1st caused in the well by withdrawal of water at very high rate.
- After a significant drawdown is observed, the ~~wie~~ withdrawal of water is ~~t~~ stopped.
- The Recuperation or Regeneration of water in a well is observed in a particular std. time of 60 to 90 min.
- The well constant 'c' is found out as follows :



$$Q \cdot dt = -A \cdot ds$$

$$CAS \cdot dt = -A\, ds$$

$$\int_{S_1}^{S_2} \frac{ds}{S} = -C \int_0^t dt$$

$$\ln\left(\frac{S_2}{S_1}\right) = -Ct$$

$$\boxed{C = \frac{1}{t} \ln\left(\frac{S_1}{S_2}\right)}$$

$$\boxed{C = \frac{2.303}{t} \log_{10}\left(\frac{S_1}{S_2}\right)}$$

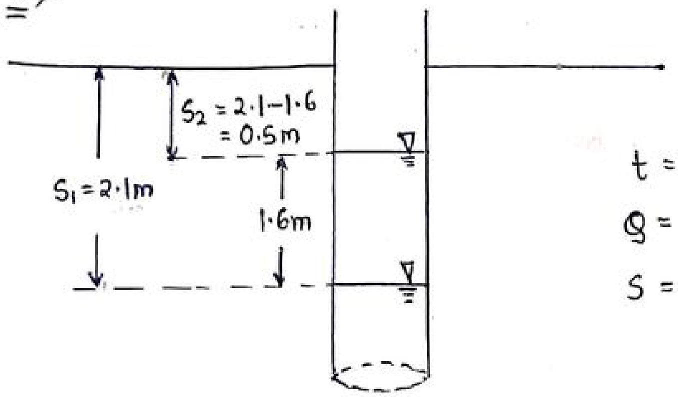
Discharge (Q') at some drawdown (S') is

$$\boxed{Q' = \left\{ \frac{1}{t} \ln\left(\frac{S_1}{S_2}\right) \right\} \times \frac{\pi D^2}{4} \times S'}$$

Std. Time = 60-90 min

Pg. No. 88 (WB)

Q. 16>



$t = 90$ min

$Q = 10 \times 10^{-3}$ m³/s

$S = 2$ m

$$10 \times 10^{-3} = \left\{ \frac{1}{90 \times 60_s} \ln\left(\frac{2.1}{0.5}\right) \right\} \times \frac{\pi D^2}{4} \times 2$$

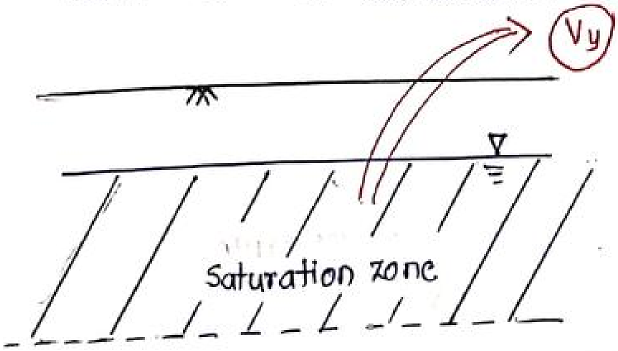
$\therefore \boxed{D = 4.89 \text{ m}}$

NOTE : • Pumping Test & Recuperation Test are not applicable for Strainer type of Tube well

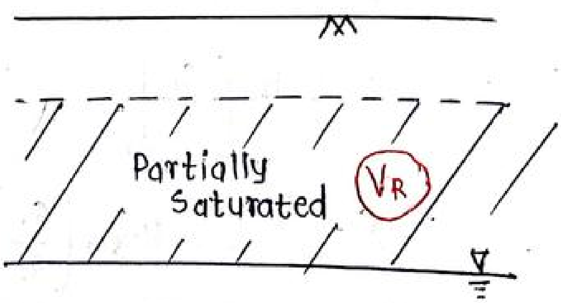
- Recuperation Test is more commonly used now a days because in Pumping Test, Pumping equillibrium is difficult to attain in many instances

* Specific Yield & Specific Retention

- Valid for a saturated soil mass



Saturated



Saturated

$V_y \rightarrow$ Volume of water withdrawn / Yielded
$V_R \rightarrow$ Volume of water Retained

$$V_v = V_w = V_y + V_R$$

$$\boxed{\frac{V_v}{V} = \frac{V_y}{V} + \frac{V_R}{V}}$$

$$\boxed{n = S_y + S_R}$$

Porosity (n), Specific Yield (S_y), Specific Retention (S_R)

Pg. No. 87 (WB)

Q.6)

$$n = S_y + S_R$$

$$0.4 = S_y + 0.15$$

$$\therefore S_y = 0.25$$

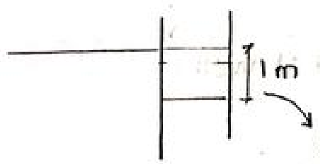
$$S_y = \frac{V_y}{V}$$

$$0.25 = \frac{V_y}{(150\,ha \times 3\,m)} \qquad \therefore V_y = 112.5\ ha\text{-}m$$

* Specific Capacity & Specific Storage

Specific Capacity

- It is a rate of flow from the well per unit drawdown in the well.
- It is determine for the fall of 1st meter, as it is not the same for all the drawdown



Specific Storage / Coefficient of Storage

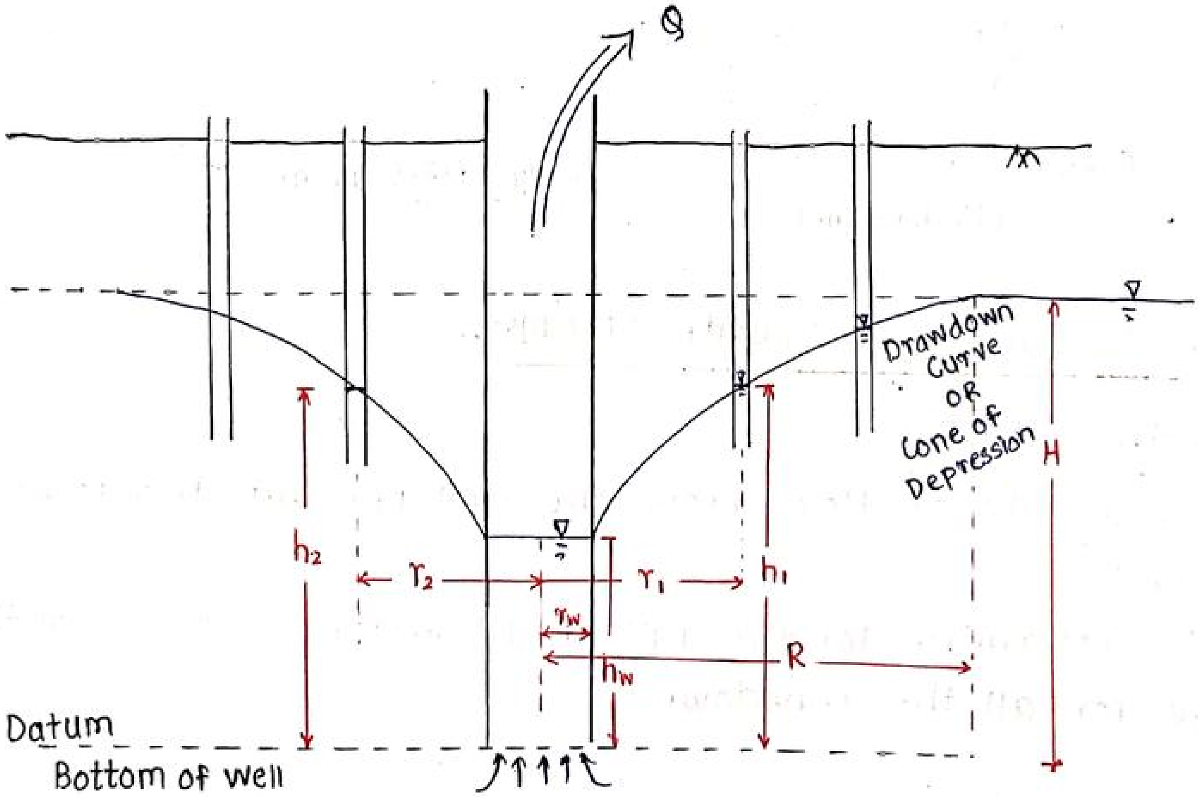
Discharg Obtained from unit volume of well or Aquifer per unit decline in the Hydraulic Head.

* Theoretical Methods

Assumptions

1) Soil is Homogeneous & isotropic
2) Flow is radially towards the well.
3) Soil is Semi infinite
4) Flow is Laminar

Dupit's Theory for Unconfined Aquifer



$$\boxed{Q = \frac{\pi k (h_2^2 - h_1^2)}{\ln\left(\frac{r_2}{r_1}\right)}}$$

k → coefficient of permeability

$$\boxed{Q = \frac{\pi k (H^2 - h_w^2)}{\ln\left(\frac{R}{r_w}\right)}}$$

R → Radius of influence

→ Max^m distance upto which the effect of withdrawal of ~~effect~~ water is observed in soil.

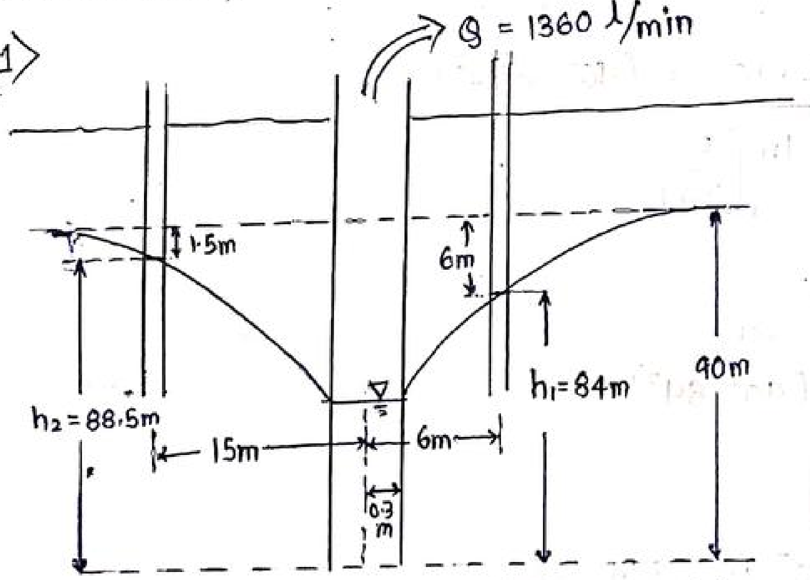
r_w → radius of well

h_w → Height of water in well

H → Original Ht. of water level

Pg. No. 88 (WB)

Q. 71>



$$Q = \frac{\pi k (h_2^2 - h_1^2)}{\ln\left[\frac{r_2}{r_1}\right]}$$

$$\frac{1360 \times 10^{-3}}{60}\left(\frac{m^3}{s}\right) = \frac{\pi k (88.5^2 - 84^2)(m^2)}{\ln\left[\frac{15}{6}\right]}$$

$\therefore k = 8.52 \times 10^{-6}$ m/s

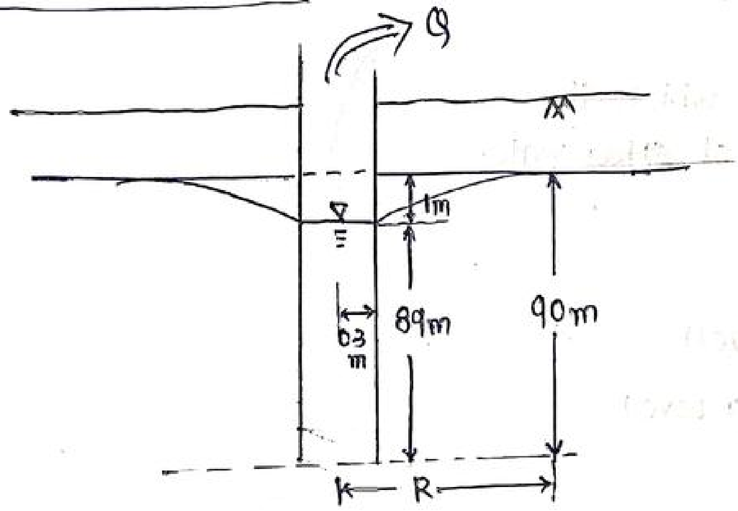
$$\frac{1360 \times 10^{-3}}{60}\left(m^3/s\right) = \frac{\pi \times 8.52 \times 10^{-6}\,(m/s) \times (84^2 - h_w^2)}{\ln\left[\frac{6}{0.3}\right]}$$

$\therefore h_w = 67.2\,m$

$S_w = 90 - 67.2$

$= 22.8\,m$

Specific capacity



$$\frac{1360 \times 10^{-3}}{60} = \frac{\pi \times 8.52 \times 10^{-6} \times (90^2 - 67.2^2)}{\ln\left[\frac{R}{0.3}\right]}$$

$\therefore R = 20.56\,m$

$$Q = \frac{\pi \times 8.52 \times 10^{-6} \times (90^2 - 89^2)}{\ln\left(\frac{20.56}{0.3}\right)}$$

$= 1.13 \times 10^{-3}\ m^3/s \times 10^3\ l/m^3 \times 60\ s/min$

$= 67.9\ l/min$

Maximum Rate

$$Q_{max} = \frac{\pi \times 8.52 \times 10^{-6} \times (90^2 - 0)}{\ln\left[\frac{20.56}{0.3}\right]}$$

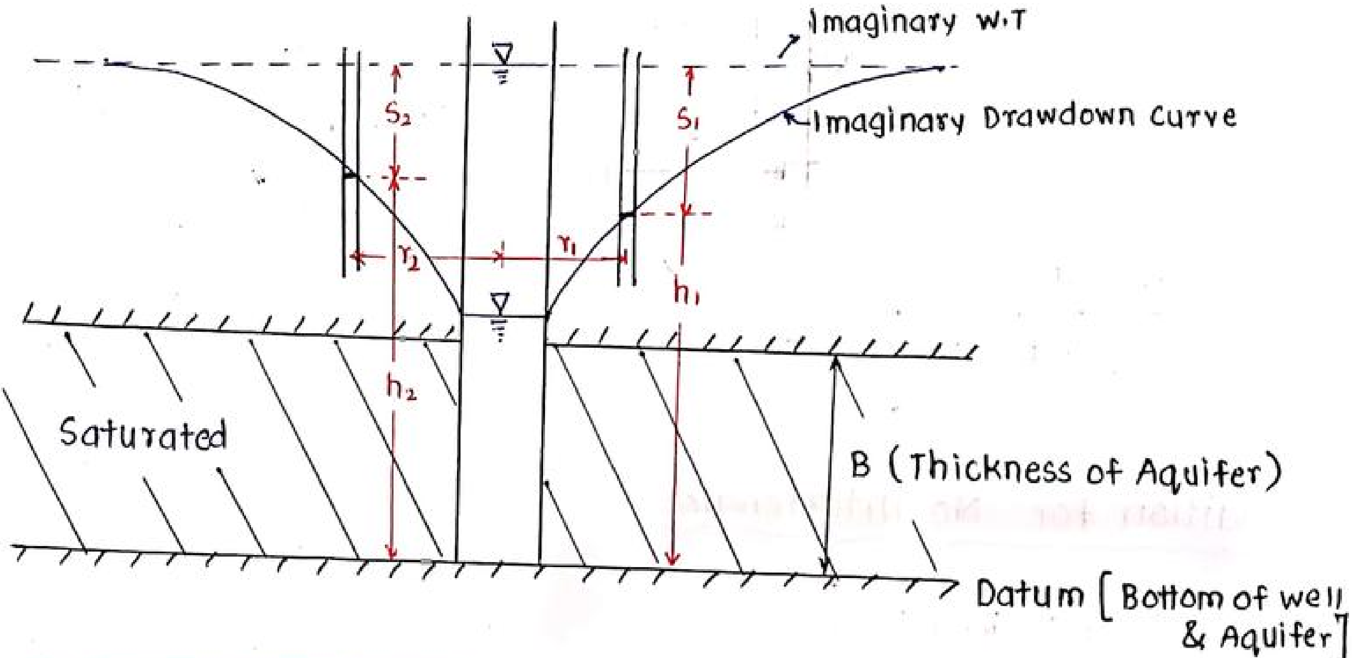
$Q = Q_{max}$ when $h_2 = h_{2max}$ & $h_1 = 0$

$$Q = \frac{\pi k (h_2^2 - h_1^2)}{\ln\left(\frac{r_2}{r_1}\right)}$$

$= 0.0512 \ m^3/s \times 10^3 \ l/m^3 \times 60 \ s/min$

$= 3078 \ l/min$

Theim's Theory of Confined Aquifer



$$\boxed{Q = \frac{2\pi k B (h_2 - h_1)}{\ln\left(\frac{r_2}{r_1}\right)}}$$

$h_2 + S_2 = h_1 + S_1$

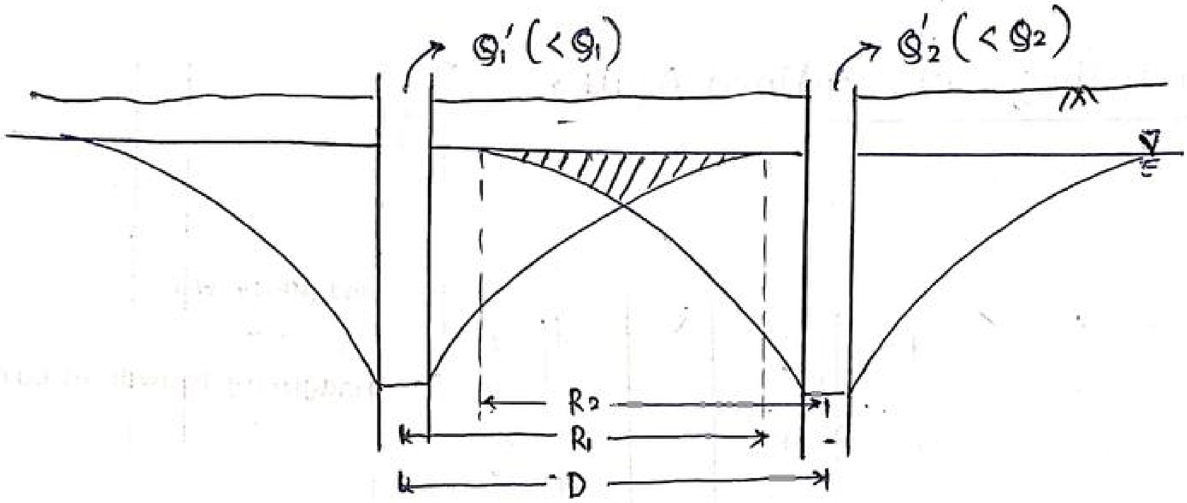
$h_2 - h_1 = S_1 - S_2$

$$\boxed{Q = \frac{2\pi k B (S_1 - S_2)}{\ln\left(\frac{r_2}{r_1}\right)}}$$

* Interference of wells

Two or more wells are said to be interfering each other when their drawdown curve intersect with each other. Due to interference, the discharge obtained through well decreases.

Condition for Interference



$$\boxed{D < R_1 + R_2}$$

Condition for No interference

$$\boxed{D \geq R_1 + R_2}$$

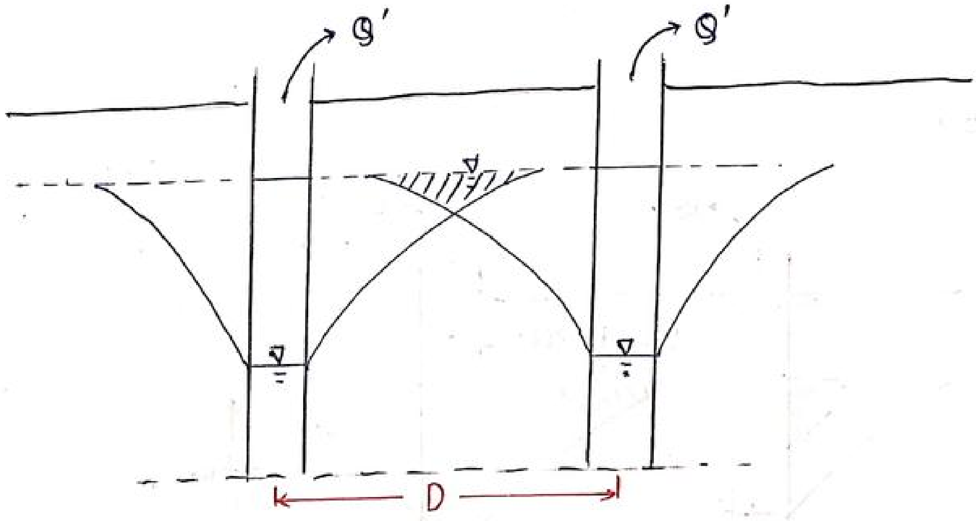
Q_1 & Q_2 → Discharge without interference

Q_1' & Q_2' → Discharge with interference

* Yield from Interfering Wells

CASE I : Two identical wells interfering in :-

a> Unconfined Aquifer



$$\boxed{Q' = \frac{\pi k (h_2^2 - h_1^2)}{\ln\left[\frac{r_2}{r_1} \times \frac{R}{D}\right]}}$$

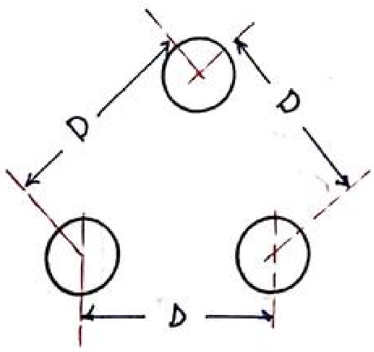
Q' → discharge through a well

b> Confined Aquifer

$$\boxed{Q' = \frac{2\pi k B (h_2 - h_1)}{\ln\left(\frac{r_2}{r_1} \times \frac{R}{D}\right)}}$$

CASE II : 3 identical well Interferring in :-

a> Unconfined Aquifer



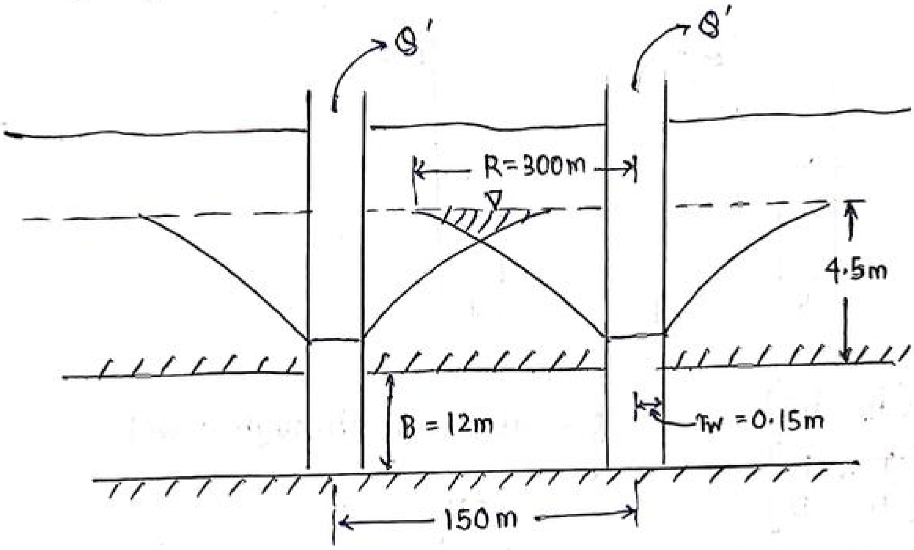
$$\boxed{Q' = \frac{\pi k (h_2^2 - h_1^2)}{\ln\left[\frac{r_2}{r_1} \times \frac{R^2}{D^2}\right]}}$$

b) Confined Aquifer

$$\boxed{Q' = \frac{2\pi k B(h_2 - h_1)}{\ln\left[\frac{r_2}{r_1} \times \frac{R^2}{D^2}\right]}}$$

Pg. No. 86 (WB)

Q.3)



Needed

$k = 1.5 \times 10^{-3}$ m/s

Discharge without Interference :-

$$Q = \frac{2 \times \pi \times 1.5 \times 10^{-3}\ m/s \times 12\ m \times 4.5\ m}{\ln\left(\frac{300}{0.15}\right)}$$

$Q = 0.0669\ m^3/s$

$$Q' = \frac{2 \times \pi \times 1.5 \times 10^{-3}\ m/s \times 12\ m \times 4.5\ m}{\ln\left[\frac{300}{0.15} \times \frac{300}{150}\right]}$$

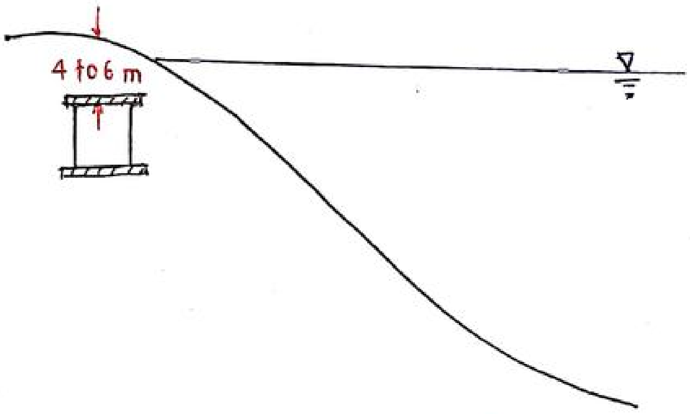
$= 0.0613\ m^3/s$

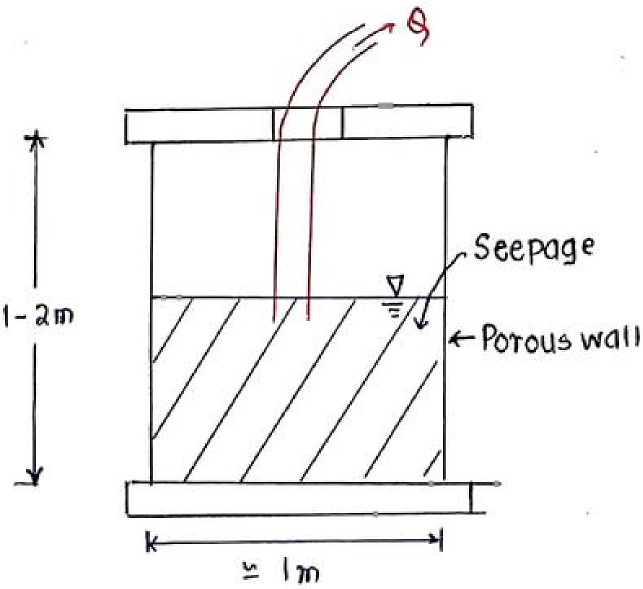
$$\%\ \text{Reduction} = \frac{Q - Q'}{Q} \times 100$$

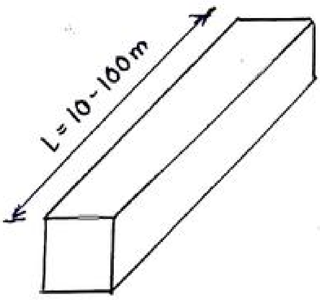
$= 8.37\ \%$

* Infiltration Galleries (Horizontal Well)

1. These are constructed at Shallow depths near the banks of River.
2. They are usually at the depth of 4m to 6m from Ground level.
3. It has height ranging from 1 to 2m, width approximatey 1m & length b/w 10-100m
4. water Enters the Gallery through seepage from its walls.
5. Suspended Solid ~~[illegible]~~ In the water collected in Gallery are negligible.
6. Drawdown observed in an infiltration Gallery is more than that in Infiltration well.







* Springs

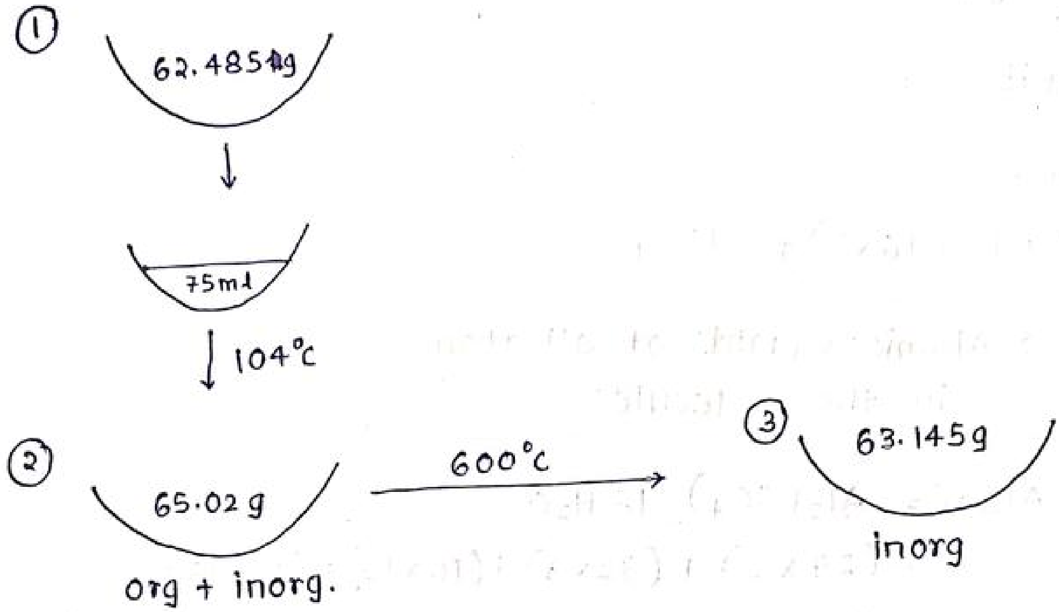
A Spring is a natural out cropping of ground water due to excess pressure in the Aquifer.

Springs are usually used to supply lesser quantity of water & can be either natural or Artificial.

3. WATER QUALITY PARAMETERS

Pg. No. 110 (WB)

Q. 10>



② - ③ → Organic

② - ① → Total

③ - ① → Inorganic

wt. of organic solids in 75 ml sample = $(65.02 - 63.145)\,g$

$= 1.875\,g$

$$\text{Conc}^n = \frac{1.875\,g}{75\,ml} \times \frac{10^3\,mg/g}{10^{-3}\,l/ml}$$

$= 25000\,mg/l$

* STOICHIOMETRY

1> Molecular Weight

Standard unit → g

M.w ⇒ $CaCO_3$

$$(40 + 12 + 16 \times 3)g = 100g$$

$$\boxed{M.w = \Sigma \text{ Atomic weights of all atoms in the molecules}}$$

MW ⇒ Alum = $Al_2(SO_4)_3\,18\,H_2O$

$$= (27 \times 2) + (32 \times 3) + (16 \times 12) + (18 \times 18)$$

$$= 666g$$

2> No. of Moles

$$\boxed{n = \frac{\text{Given wt. (g)}}{\text{Molecular wt. (g)}}}$$

Find the 'n' in 1000g of Alum

$$n = \frac{1000\,g}{666\,g} = 1.5$$

Find the ~~ext~~ no. of ~~molecules~~ m. moles in 300 g of $CaCO_3$

$$n = \frac{300}{100} = 3$$

$$1 \text{ mole} = 10^3 \text{ m moles}$$

no. of m moles = 3000

3) Equivalent Weight

$$\boxed{\text{Eq. wt.} = \frac{\text{Molecular weight (g)}}{\text{Valency}}}$$

Valency → No. of electron transfers taking / taken Place

Examples

① Eq. wt. of $CaCO_3 = \frac{100}{2}g$

$= 50g$

Ca^{2+} CO_3^{2-}

$2e^-$

② Eq. wt. of $NaCl = \frac{58.5g}{1} = 58.5g$

Na^+ Cl^-

$1e^-$

③ Eq. wt. of $MgSO_4 = \frac{120}{2}g = 60g$

Mg^{2+} SO_4^{2-}

$2e^-$

④ Eq. wt. of $H_2SO_4 = \frac{98}{2}g = 49g$

$H^+ \xrightarrow{1e^-} SO_4^{2-}$

$H^+ \xrightarrow{1e^-}$

⑤ Eq. wt. of Alum $= \frac{666g}{6} = 111g.$

$Al_2(SO_4)_3\,18H_2O \rightarrow 18H_2O$

$\rightarrow 2Al^{3+}$

$\rightarrow 3SO_4^{2-}$

Al^{3+} Al^{3+}

$6e^-$

SO_4^{2-} SO_4^{2-} SO_4^{2-}

(6) Eq. wt. of Ca^{2+}

$$= \frac{40g}{2} = 20g$$

(7) Eq. wt. of SO_4^{2-}

$$= \frac{96g}{2} = 48g$$

4) Gram Equivalent

$$\boxed{\text{Gram eq.} = \frac{\text{Given weight (in g)}}{\text{Equivalent weight}}}$$

Find the no. of g. eq in 300 g of $CaCO_3$.

$$\text{g. eq.} = \frac{300g}{50g} = 6$$

Find the no. of mg eq in 1000 g of Alum

$$\text{g. eq.} = \frac{1000g}{111g} = 9.009$$

$$1 \text{ g eq.} = 10^3 \text{ mg.eq}$$

$$\text{mg.eq} = 9009$$

GRAM EQUIVALENT PRINCIPLE

"1 g eq. of any compound reacts with 1 g eq. of any other compound to produce 1 g eq. each of the respective products in a reaction".

$$A + B + C + D \longrightarrow E + F + G + H$$

Given wt
eq. wt

wt. of G ?

$$\text{g. eq. of } C = \text{g eq. of } G$$

$$\boxed{\text{wt. of } G = \text{g eq. of } G \times \text{eq. wt. of } G}$$

$$H_2SO_4 \longrightarrow 2H^+ + SO_4^{2-}$$

98g 2g 96g

Given wt. = 600g

600g $600 \times \frac{2}{98}$ $600 \times \frac{96}{98}$

$= 12.24g$ $= 587.76$

<u>g. eq</u>

$\frac{600\ g}{49\ g}$ $\frac{12.24\ g}{1\ g}$ $\frac{587.76\ g}{48g}$

$= 12.24$ geq. $= 12.24$ g eq. $= 12.24$ g eq.

5) Normality (N)

It indicates strength of solution

$$\boxed{N = \frac{\text{No. of g. eq. of Solute}}{\text{Volume of Solution (litres)}}}$$

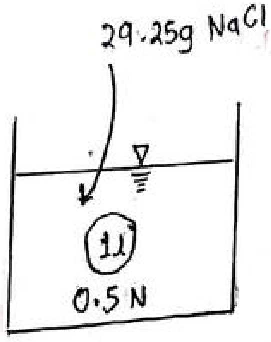
Prepare a 0.5 N Solution of NaCl

$$\left.\begin{array}{l}\text{Solute} \rightarrow \text{NaCl} \\ \text{Solvent} \rightarrow \text{water}\end{array}\right\} \text{Solution}$$

$$0.5\ N = \frac{\text{No. of g. eq. of NaCl}}{1\ l}$$

No. of g. eq. to be added = 0.5

wt. of NaCl required = $0.5 \times 58.5 = 29.25g$

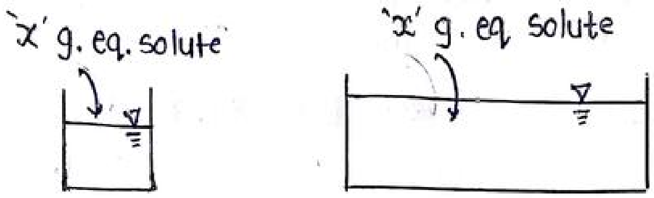


$$\boxed{N \propto \text{Strength}}$$

Normality Principle

$$N_1V_1 = N_2V_2 = x$$

[Apply only when no. of g. eq. in both systems is same]



$V_1\ l$ $\qquad$ $V_2\ (>V_1)$

$N_1 = \frac{x}{V_1}$ $\qquad$ $N_2 = \frac{x}{V_2}$

$x = N_1V_1$ $\qquad$ $x = N_2V_2$

6> Molarity (M)

It also indicates the strength of Solution

$$\boxed{M = \frac{\text{no. of moles of Solute}}{\text{Volume of Solution (litres)}}}$$

Relation between N & M

$$N = \frac{\text{no. of g. eq}}{V(l)}$$

$$= \frac{\text{wt. (g)}}{\text{eq. wt} \times V(l)}$$

$$= \frac{\text{wt. (g)} \times \text{Valency}}{\text{M. W (g)} \times V(l)} = \frac{\text{No. of moles}}{V(l)} \times \text{Valency}$$

$$= M \times \text{Valency}$$

* Chemical Water Quality Parameter

1) pH

- pH is defined as Potential exerted due to H^+ ions present in water.
- pH is a method of representation of concentration of H^+ ion present in water.
- Similarly pOH is Potential exerted due to OH^- ions present in water.
- Water is in equillibrium with H^+ & OH^- ions & at 25°C, water dissociate's as follows

$$H_2O \rightleftharpoons H^+ + OH^-$$

For an Equillibrium reaction at a particular temperature, the Reaction rate constant can be written as follows :

$$aA + bB \rightleftharpoons cC + dD$$

$k_f \rightarrow$ Rate const. in forward direction

$$k_f = \frac{[C]^c [D]^d}{[A]^a [B]^b}$$

$$k_B = \frac{[A]^a [B]^b}{[C]^c [D]^d} \quad ; \quad \boxed{k_f k_B = 1}$$

For water, k_f of Forward Reaction,

$$k_f = \frac{[H^+]^1 [OH^-]^1}{[H_2O]^1} = [H^+][OH^-]$$

$$= 10^{-14}$$

$$pH = \log_{10} \frac{1}{[H^+]} \quad \text{moles}/l$$

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

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

CASE I : $[H^+] = [OH^-] = 10^{-7}$ mol/l

$$pH = -\log_{10}[10^{-7}] = 7$$

$$POH = -\log_{10}[10^{-7}] = 7$$

Neutral water

CASE II : $[H^+] > 10^{-7}$ mol/l

$[OH^-] < 10^{-7}$ mol/l

$$\left.\begin{array}{l} pH < 7 \\ pOH > 7 \end{array}\right\} \text{Acidic Water}$$

CASE III : $[H^+] < 10^{-7}$ mol/l

$[OH^-] > 10^{-7}$ mol/l

$$\left.\begin{array}{l} pH > 7 \\ pOH < 7 \end{array}\right\} \text{Basic Water}$$

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

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

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

$$pH + pOH = 14$$

Acceptable Limit of pH of Water = 6.5 - 8.5

*. Measurement of pH

- pH can be measured exactly with the help of Specialized device called as Potentiometer & it can also be computed approximately with Help of indicator.
- Indicators are either weak acids or weak bases, which show a characteristic change in their colour at certain pH range.
- Those indicator which change their colour in acidic range is called as Acidic Indicator (Methyl Orange) & those which change their colour in Basic ~~mode~~ Range is called as Basic Indicator (Eg. Phenolphthalene)
- Indicator are used to determine the end points of Titration.

Pg. No. 91 (WB)

Q. 13>

$$[OH^-] = 10^{-5.6} \ m\,mol/l$$
$$= 10^{-5.6} \times 10^{-3} \ mol/l$$
$$= 10^{-8.6} \ mol/l$$

$$pOH = -\log_{10}[10^{-8.6}]$$
$$= 8.6$$

$$pH = 14 - POH = 14 - 8.6$$
$$= 5.4$$

Pg. No. 94 (WB)

Q. 31>

$$pH = 9.25$$

$$pH + pOH = 14$$

$$9.25 + pOH = 14$$

$$\therefore pOH = 4.75$$

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

1 mole of $OH^{-} = 17g$

$10^{-4.75}$ moles of $OH^- = 10^{-4.75} \times 17\ g$

$= 10^{-4.72} \times 17 \times 10^3$ mg

$= 0.302$ mg

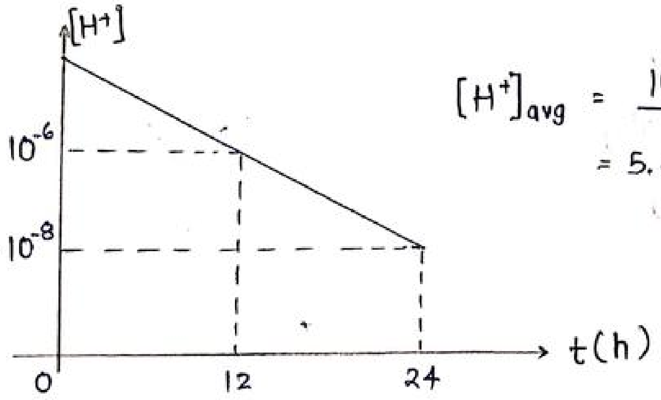
Concⁿ of $[OH^-] = 0.302$ mg/l

Pg. No. 94 (WB)

Q. 32>

$pH = 6, \quad [H^+] = 10^{-6}$ mol/l

$pH = 8, \quad [H^+] = 10^{-8}$ mol/l



$$[H^+]_{avg} = \frac{10^{-6} + 10^{-8}}{2}$$

$= 5.05 \times 10^{-7}$ mol/l

$pH_{avg} = -\log_{10}(5.05 \times 10^{-7})$

$= 6.296$

Pg. No. 94 (WB)

Q. 33>

$pH_A = 4.2$

$[OH^-]_B = 2 \times [OH^-]_A$

$pOH_A = 14 - 4.2 = 9.8$

$[OH^-]_A = 10^{-9.8}$ mol/l

$[OH^-]_B = 2 \times 10^{-9.8}$ mol/l

$pOH_B = 9.49$

$pH_B = 14 - 9.49 = 4.51$

2) Alkalinity

- Alkalinity is defined as ability of certain species to neutralized H^+ ions present in water. In naturally occuring water, certain Major species are present which ~~constitutes~~ constitutes more than 99% of Total Alkaline Species.
- They are $[OH^-]$, CO_3^{2-}, HCO_3^-
- The Minor species which constitute less than 1% of Total Alkaline species are

HS, NH_3, $H_2PO_4^-$, HPO_4^{2-}, $H_3SiO_3^-$, $H_2BO_3^-$ etc

$$OH^- + H^+ \longrightarrow H_2O$$

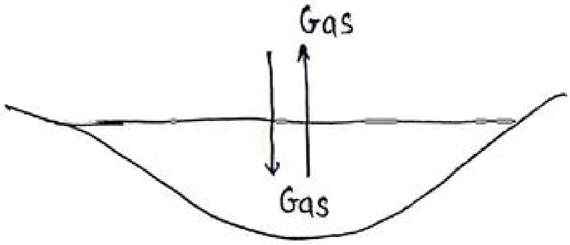
$$CO_3^{2-} + H^+ \longrightarrow HCO_3^-$$

$$HCO_3^- + H^+ \longrightarrow \underset{\text{weak acid}}{H_2CO_3}$$

OH^-, CO_3^{2-}, HCO_3^- are major species because they are obtained in water from mineral deposit from earth crust as well as atmospheric CO_2.

The Reaction with atmospheric CO_2 is governed by Henry's law

NOTE : Henry's law states that " The concentration of various gases in liquid is proportional to concentration of these gases in the atmosphere of that liquid."



$$CO_2 + H_2O \rightleftharpoons H_2CO_3$$

Carbonic Acid

$$H_2CO_3 \rightleftharpoons H^+ + \boxed{HCO_3^-}$$

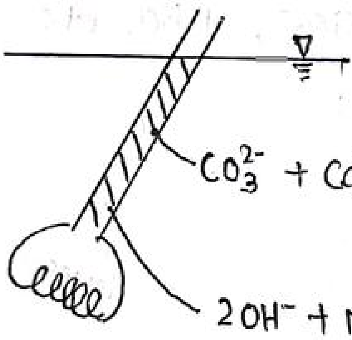
$$HCO_3^- \rightleftharpoons H^+ + \boxed{CO_3^{2-}}$$

$$CO_3^{2-} + H_2O \rightleftharpoons \boxed{HCO_3^-} + \boxed{OH^-}$$

Impacts of Alkalinity

16/11/19

1. High quantity of Alkaline species causes Bitter taste in water.
2. The Alkaline species tend to form Incrustation or deposition over various surfaces under proper conditions.



$$CO_3^{2-} + Ca^{2+} \xrightarrow{\Delta} CaCO_3 \downarrow$$

white (sticky)

$$2OH^- + Mg^{2+} \xrightarrow{\Delta} Mg(OH)_2 \downarrow$$

white

Representation of Alkalinity

Alkalinity is represented in terms of mg/l as $CaCO_3$ because -

i> It's equivalent wt. is 50g & ~~to~~ thus make the calculation easy

2> Alkalinity is ultimately compared with Hardness & thus written in form of $CaCO_3$

Acceptable Limit = 200 mg/l as $CaCO_3$

cause for Rejection = 600 mg/l as $CaCO_3$

Pg. No. 95 (WB)

Q.34)

$CO_3^{2-} = 210 mg$ $OH^- = 68 mg$

$HCO_3^- = 122 mg$

$$\text{Gram eq.} = \frac{\text{Given wt.}}{\text{Equivalent wt.}}$$

Species	weight	mg. eq	wt. of $CaCO_3$
CO_3^{2-}	210	210/30	7 mg.eq × 50g = 350mg
OH^-	68	68/17	4 mg.eq × 50g = 200mg
HCO_3^-	122	122/61	2 mg.eq × 50g = 100mg

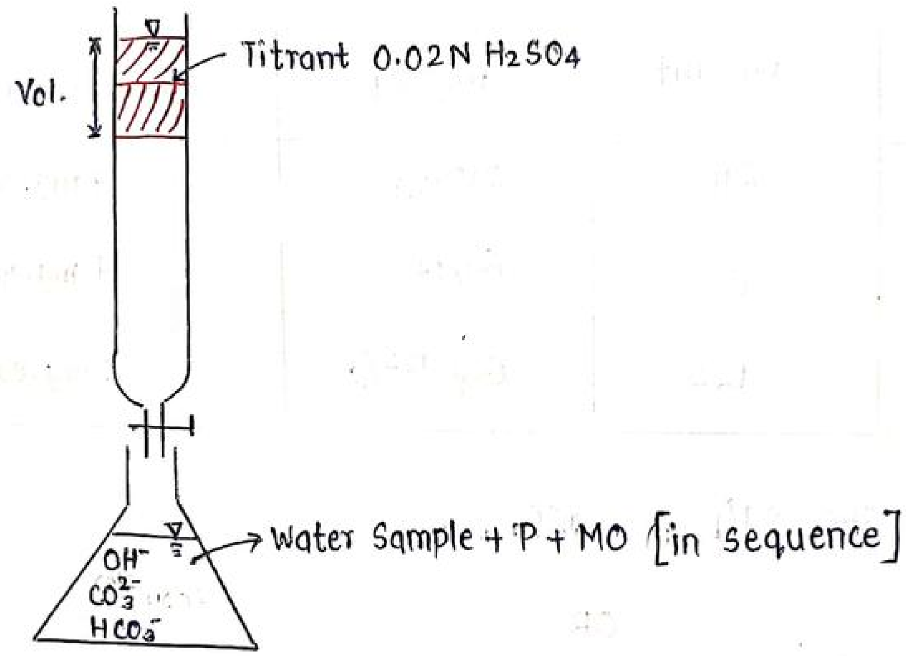
Alk = 650 mg/l as $CaCO_3$

OR

$$\text{Alk. (mg/l as } CaCO_3) = \frac{\text{mg/l of } CO_3^{2-}}{\text{Eq. wt. of } CO_3^{2-}} \times \text{eq. wt. of } CaCO_3 + \frac{\text{mg/l of } HCO_3^-}{\text{Eq. wt. of } HCO_3^-} \times \text{eq. wt. of } CaCO_3 + \frac{\text{mg/l of } OH^-}{\text{Eq. wt. of } OH^-} \times \text{eq. wt. of } CaCO_3$$

Lab Test of Alkalinity

- In laboratory, Alkalinity is measured by titrating the water sample against strong acid i.e $0.02\ N$ H_2SO_4 (as a standard) which reacts with Alkaline species present in water.
- Phenolphthalein & Methyl Orange are used as standard indicators in the Test.



P: Phenolphthalein MO : Methyl Orange

$$\text{Colourless} \xleftarrow{pH<8.2} P \xrightarrow{pH>10} \text{Pink}$$

$$\text{Red/Orange} \xleftarrow{pH<3.5} MO \xrightarrow{pH>4.5} \text{Yellow}$$

If $0.02\ N$ H_2SO_4 is used :

1 ml of acid reacts with 1 mg of alkalinity as $CaCO_3$

g. eq.

$$N = \frac{\text{No. of g.eq of } H_2SO_4}{\text{Vol. (litres)}}$$

No. of g.eq. of H_2SO_4

$= 0.02 \times 10^{-3}$

$= 2 \times 10^{-5}$

g. eq

$$g.eq = \frac{wt.(g)}{eq.wt}$$

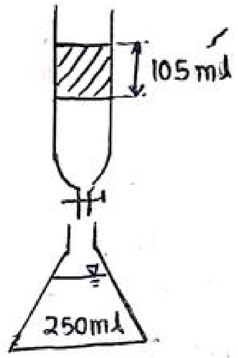
$$= \frac{10^{-3} g}{50\ g}$$

$$= 2 \times 10^{-5}$$

ESE

Q) 250 ml of water sample. It required 105 ml of 0.02 N H_2SO_4 for complete titration. Find the Total Alk. (mg/l as $CaCO_3$)

⇒



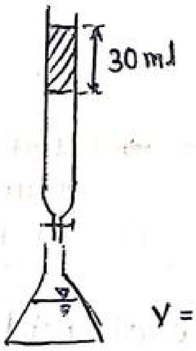
105 ml of acid reacts with 105 mg of Alk. as $CaCO_3$ in 250 ml sample.

$$\therefore \text{Total Alk.} = \frac{105\ mg}{0.25\ l} \text{ as } CaCO_3$$

$$= 420\ mg/l \text{ as } CaCO_3$$

Pg. No. 95

Q.35)



Y = 200 ml

$$Alk = \frac{30}{0.2} = 150\ mg/l$$

$$= 150\ mg/l$$

5 mg/l as $CaCO_3$ — OH^- HCO_3^- CO_3^{2-} — 145 mg/l as $CaCO_3$

pH = 10 , pOH = 4

$$OH^- = 10^{-4}\ mol/l \times 17 \times 10^3\ mg = 1.7\ mg/l \text{ (as } OH^- \text{ itself)}$$

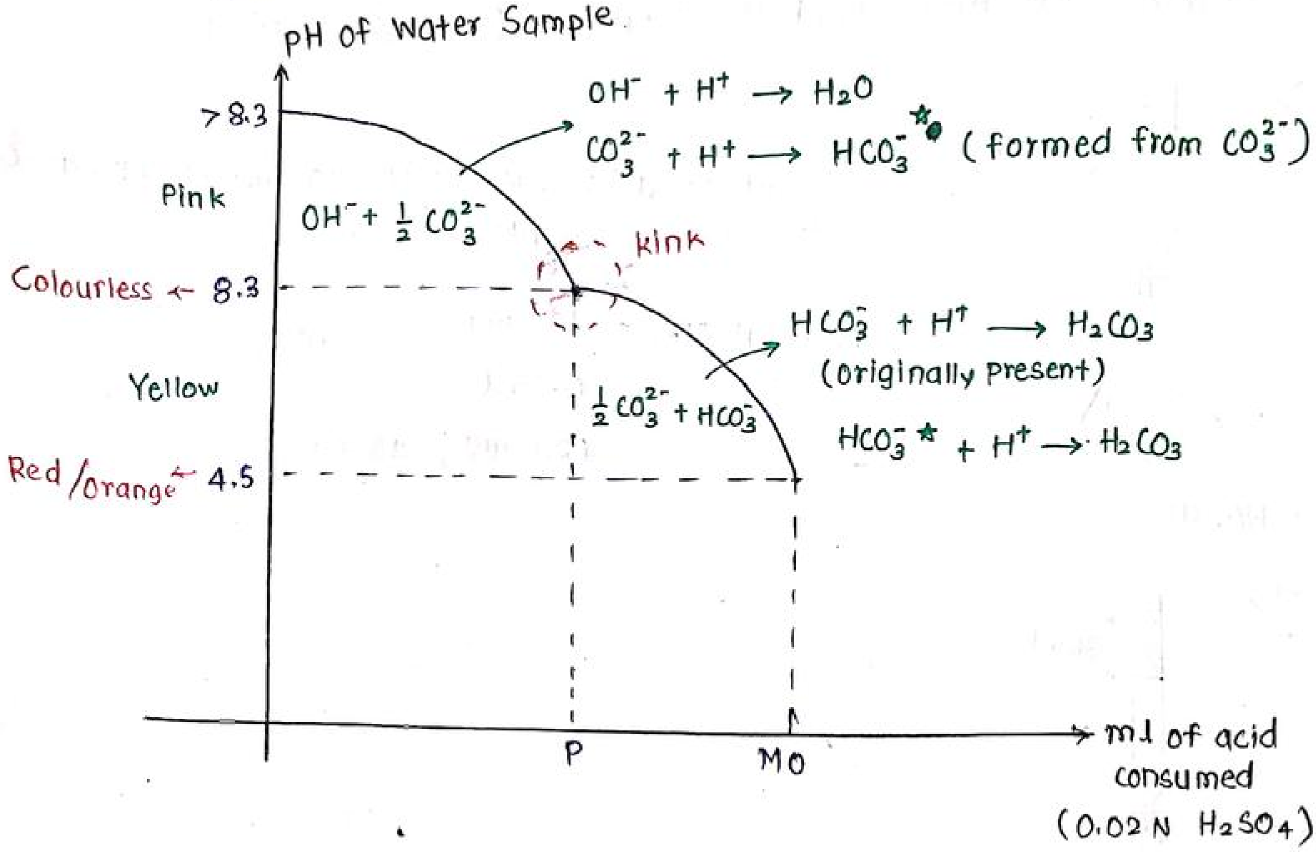
$$[OH^-] = \frac{1.7\ mg/l}{17} \times 50 = 5\ mg/l \text{ as } CaCO_3$$

If 1000 ml of sample is considered acid consumed upto pH of 8.3

$= 11 \times 5 = 55$ ml

∴ In 1 l sample, 55 mg of Alk. as $CaCO_3$ is neutralized till pH of 8.3.

Titration Curve



① The change in pH of water sample w.r.t Volume of acid consumed is depictated by Titration Curve.

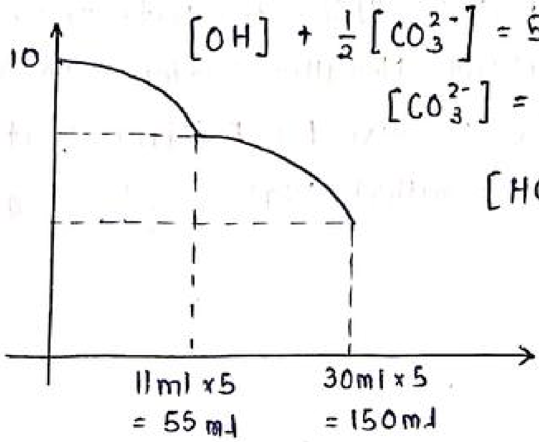
② Natural water usually have pH ranging from 8.5 to 11.5

③ At pH of around 8.3, OH^- & Half of carbonate reacts with H^+, at this pH, Phenolphthalein turns colourless from Pink. It is referred as 1st Stage of Titration. or Phenolphthalein end pt.

④ At pH of around 4.5, the Bicarbonates originally present as well as those formed from carbonate completely reacts with H^+ ion. At this pH, Methyl Orange turns red from Yellow. It is referred of End of Titration or Methyl Orange end point

Ans: Pg. No. 95

Q.35>



$[OH] + \frac{1}{2}[CO_3^{2-}] = 55$ mg/l as $CaCO_3$

$[CO_3^{2-}] = (55-5) \times 2 = 100$ mg/l as $CaCO_3$

$[HCO_3^-] + [CO_3^{2-}] = 145$ mg/l as $CaCO_3$

$[HCO_3^-] = 45$ mg/l as $CaCO_3$

3> Hardness

Hardness is the property by virtue of which water doesnot form sufficient Lather or Foam with Soaps & Detergents.

It is due to the presence of Multivalent Metallic cations presents in water

Eg. Ca^{2+} Fe^{2+}/Fe^{3+} Cr^{3+}/Cr^{2+}

Mg^{2+} Al^{3+} $Mn^{2+}/M^{5+}/Mn^{3+}$

~~Na^+~~ Zn^{2+} etc.

~~F^-~~

~~SO_4^{2-}~~

Types of Hardness

The type of Hardness is classified on the bases of the anions orginally associated with Multivalent Metallic cations in water.

It is of following 2 Types;

<i> Carbonate Hardness

It is due to the presence of carbonates & Bicarbonates orginally associated with Multivalent Metallic cations present In water. Such Hardness is easy to remove by Boiling & Thus it is also referred as Temporary Hardness.

<2> Non Carbonate Hardness

It is due to the presence of Sulphates, Chloride, Nitrates etc. Originally associated with Multivalent Metallic cations present in water. Such Hardness cannot be remove by Boiling & it requires certain specialize techniques called Water Softening techniques as follows :

(i) Lime - Soda Method
(ii) Ion Exchange / zeolite Method
(iii) Demineralization Method
(iv) Reverse Osmosis (R.O)

Impacts of Hardness

1> Hardness increases the Laundry Expenses

2> Hardness is the presence of minerals in water & thus it also imparts Taste to the Food.

3> Excessive Hardness produces laxative effect in water. The common Laxative are $MgSO_4$, $CuSO_4$, $CaSO_4$ etc.

4> Hardness causes Scaling in Boilers & incrushtation ~~in~~ over Pipe. Thus, for industrial water supply, zero Hardness water is recommended.

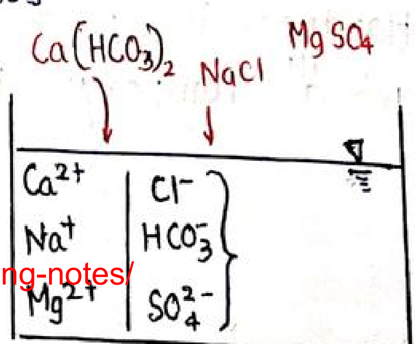
Calculation of Hardness

(1) Total Hardness

$$= \left[\frac{\text{mg/l of } Ca^{2+}}{\text{eq. wt. of } Ca^{2+}}\right] \times \text{eq. wt. of } CaCO_3$$

$$+ \left[\frac{\text{mg/l of } Mg^{2+}}{\text{eq.wt. of } Mg^{2+}}\right] \times \text{eq. wt. of } CaCO_3$$

$$+ \; \ldots\ldots + \ldots\ldots$$



② TH = CH + NCH

③ CH = $min^m \begin{cases} TH \\ Alkalinity \end{cases}$

④ NCH = TH - CH

Eg.

TH = 300 mg/l as $CaCO_3$

Alk = 180 mg/l as $CaCO_3$

$$CH = min \begin{cases} 300 \\ 180 \end{cases} = 180 \text{ mg/l as } CaCO_3$$

NCH = 300 - 180 = 120 mg/l as $CaCO_3$

TH = 180 mg/l as $CaCO_3$

Alk = 300 mg/l as $CaCO_3$

CH = 180 mg/l as $CaCO_3$

NCH = TH - CH = 0

Pg. No. 91

Q. 10 & 11 ~~$Na^+ = 56$ mg/l~~ $Cl^- = 165$ mg/l

✓ $Ca^{2+} = 40$ mg/l

✓ $Mg^{2+} = 30$ mg/l

✓ $Al^{3+} = 3$ mg/l

$HCO_3^- = 190$ mg/l

$$TH = \frac{40 \text{ mg/l}}{20} \times 50g + \frac{30 \text{ mg/l}}{12g} \times 50g + \frac{3 \text{ mg/l}}{9g} \times 50g$$

$= 241.67 \simeq 242$ mg/l as $CaCO_3$

$$\text{Alkalinity} = \frac{190 \text{ mg/l}}{61 \text{ g}} \times 50 \text{ g}$$

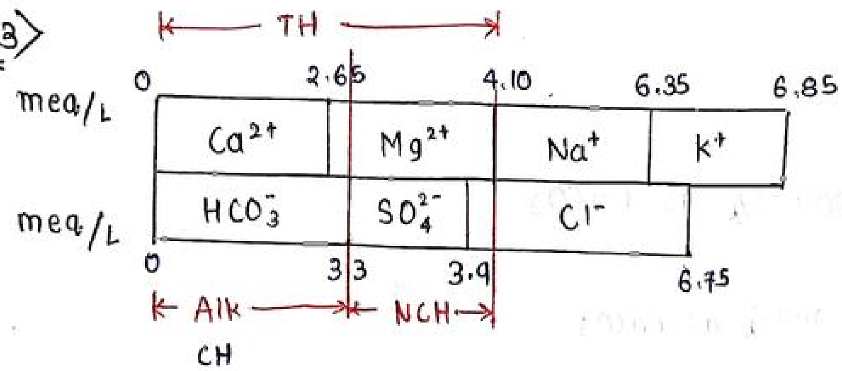
$$= 155.7 \simeq 156 \text{ mg/l as } CaCO_3$$

$CH = 156$ mg/l as $CaCO_3$

$NCH = 242 - 156 = 86$ mg/l as $CaCO_3$

Pg. No. 93 (WB)

Q. 23>



$(4.10 - 3.3) \times 50 = 40$ mg/l as $CaCO_3$

Limits of Hardness

1> AL = 200 mg/l as $CaCO_3$

2> CFR = 600 mg/l as $CaCO_3$

3> Preferred limit for Drinking = 75 - 115 mg/l as $CaCO_3$

Degree of Hardness

TH (mg/l as $CaCO_3$)	Degree
0 - 55	Soft
55 - 100	Slightly Hard
100 - 200	Moderately Hard
> 200	Very Hard / Hard

4) Total Dissolved Solids (TDS)

TDS can be measured accurately by Gravimetric Method & Approximately by Specific Conductivity Test.

This Test is carried out by Dionic water tester at a standard temperature of 25°C.

NOTE : As Temperature increases, electrical conductivity also increases

This Test is approximate because, the compound which get dissolved in water but do not get ionized cannot be measured by this Test

Eg. Phenol, Alcohols, ketone etc.

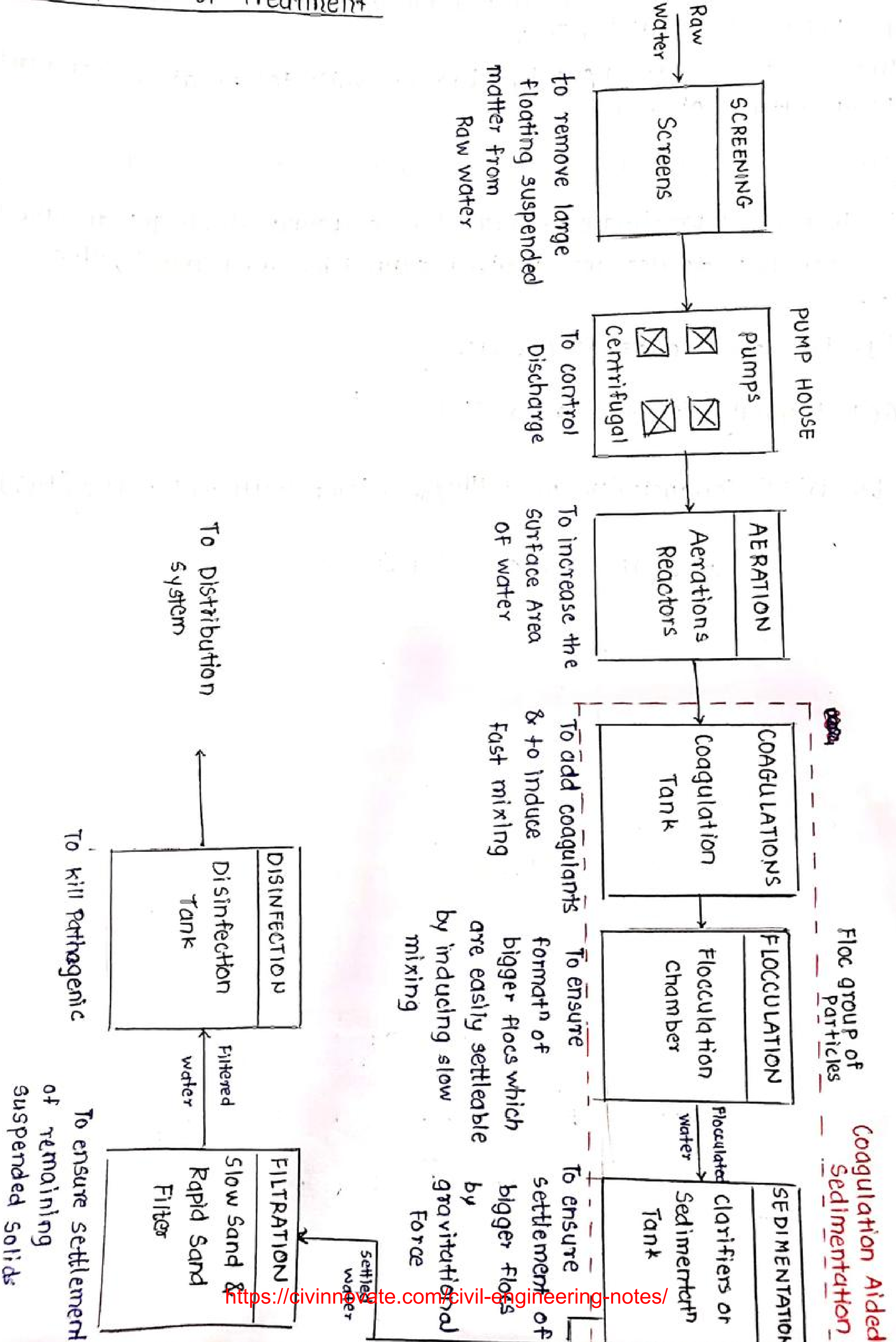
GOI Manuals Formula for TDS

$$\text{Electrical Conductivity in } \mu \text{ Mho/cm} \times \text{Temp Coefficient} = \text{TDS (mg/l)}$$

$$\text{Temp coeff} = k = 0.65 \text{ (at } 25°C)$$

4. TREATMENT OF WATER

* General process of Treatment



* Screening

- Screening is the 1st operation adopted in W.T.P as it removes Large Floating suspended matter which reduces wear & Tear of the Pumps & also reduces the Load on the Treatment Plant.
- Their are following 2 Types of Screens

(i) Fine Screens

- They are in the form of wire mesh with c/c spacing less than 20mm.
- These screens get choked easily, thus requireing frequent cleaning.
- Due to High operation cost of such screens they are not very common in W.T.P

(ii) Coarse Screens

- They are in the form of parallel iron or aluminium bars with c/c spacing between 20 to 100 mm.
- These are most commonly used as they allow higher discharges without the problem of frequent choking.
- They can be Mechanically Cleaned by Raking Mechanism

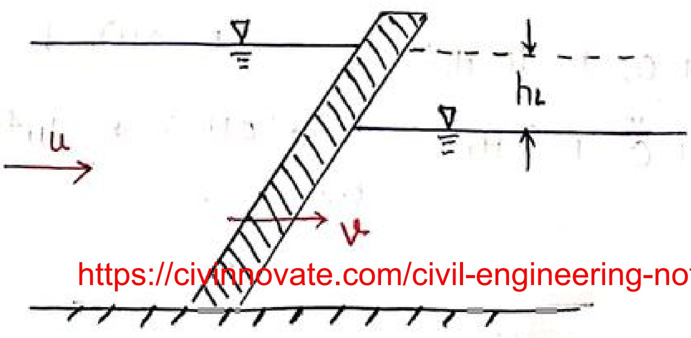
Design Data

1> The bars are of 10 - 25mm in diameter

2> The Screens are always placed inclined at an angle of 45 to 60° with horizontal in direction of flow.

3> Initial Head loss (h_L)

Head lost b/w U/s & D/s section when the screen is 100% clean.



$$\boxed{h_L = 0.0729\,(v^2 - u^2)}$$

$V \rightarrow$ velocity through the screen (m/s)

$u \rightarrow$ velocity at the U/s section (m/s)

$h_L \rightarrow$ m

4> Total screen area in contact with water = A

The screen should never be allowed to be clogged more than A/2.

5> Velocity through a 100% clean screen (v) should be between 0.8 - 1 m/s

* Aeration

- Aeration is a process of bringing the incoming water in intimate contact with Air.
- Aeration is accomplished with help of Aeration reactors in which the basic principle is to increase the surface area of water as much as possible

Advantages of Aeration

① It increases the dissolved oxygen content in water.

② It reduces the quantity of undesirable gases such as H_2S, NH_3, CH_2 etc from water.

③ It removes the volatile compound such as phenol, Humic acid etc. from water.

④ It helps the precipitates the Dissolved Iron & Magnese present in water.

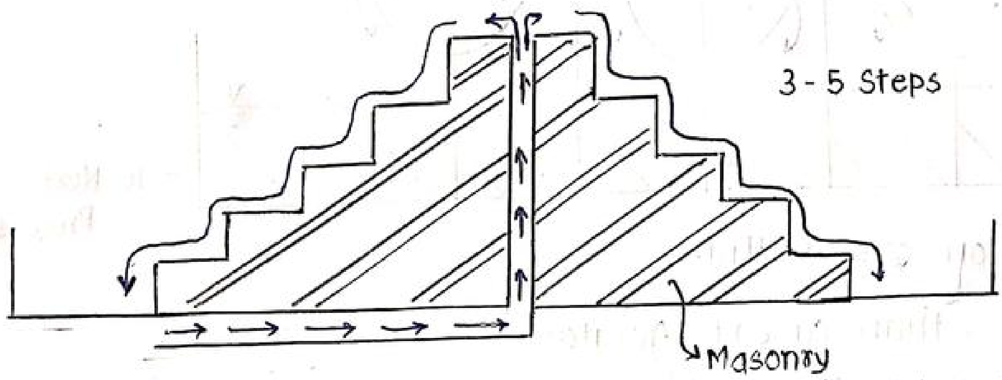
$$4Fe^{2+} + O_2 + 10H_2O \longrightarrow 4Fe(OH)_3 \downarrow + 8H^+$$

$$2Mn^{2+} + O_2 + 2H_2O \longrightarrow 2MnO_2 \downarrow + 4H^+$$

NOTE : ① Aeration of Ground water usually induces Acidity in water.
② Aeration Helps in Disinfection Process.

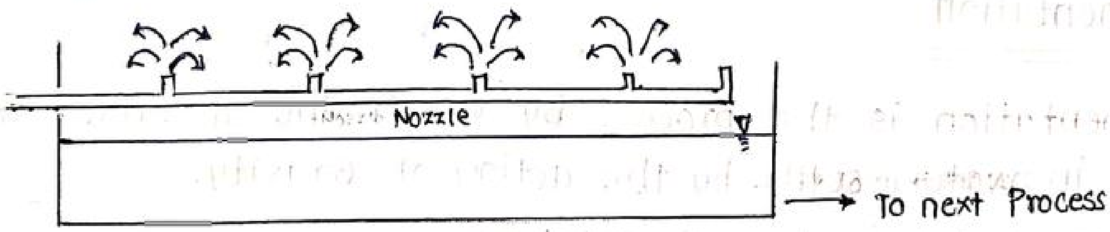
Types of Aerators

1> Cascade Aerators



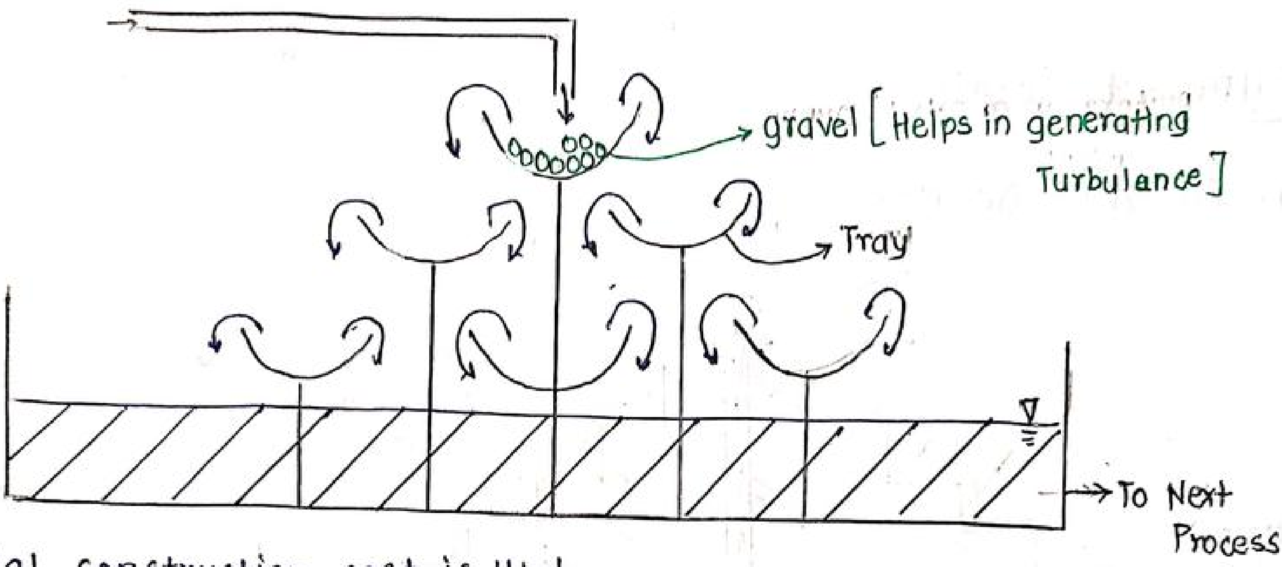
→ Robust & Durable
→ Operation & Maintaenance cost is least.

2> Spray Nozzle Aerator



→ Efficiency is Highest
→ Operation & maintenance cost is highest
→ Nozzles are subjected to frequent Choking
→ Only used for Groundwater

3> Trickling / Tray Tower Aerator



→ Initial construction cost is High.

→ Efficiency is less than cascade Aerator

→ Less Robust & Durable than cascade Aerator

NOTE : Screening is a compulsory process for surface water & Aeration is compulsory process for Ground Water as well as water obtained from bottom layers of Lake.

* Sedimentation

- Sedimentation is the process by which the suspended Solids present in water settle by the action of Gravity.
- Their are 2 Types of Sedimentation

① Plain Sedimention - Sedimentation without Coagulation & Flocculation.

② Coagulation Aided Sedimentation - Sedimentation assisted by coagulation & flocculation

Theory of Sedimentation

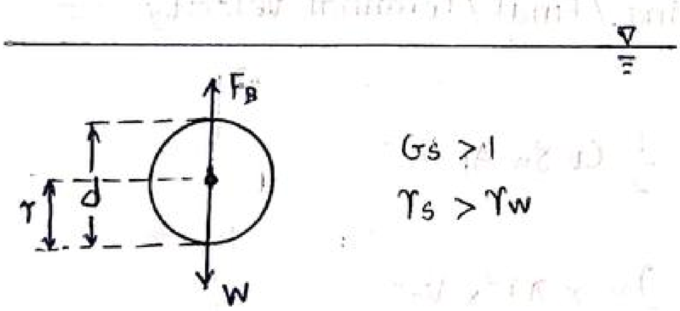
In general, the particles which have higher diameter & higher Sp. Gravities have a higher tendency to get settled.
In Natural water, impurities can be either Organic & Inorganic whose Sp. Gravity range as follows :

Types of Impurities	G_s	G_s design
Organic	1 - 2	1.2
Inorganic	2.6 - 2.9	2.65

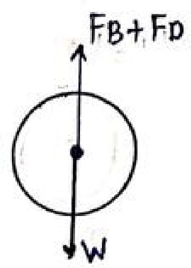
In water treatment, the settling behaviour of one particular is analyse & is applicable to all the particle in suspension. This is called as Type 1 Discrete Particle Settling in which particles do not interfere with eachother while settling.

Analysis of Type 1 Settling

$t = 0$ → Particle is introduced in water



$G_s > 1$
$\gamma_s > \gamma_w$



$W > F_B + F_D$

$$W = \frac{4}{3}\pi r^3 \times \gamma_s$$

$$F_B = \frac{4}{3}\pi r^3 \times \gamma_w$$

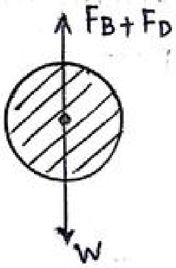
$$F_D = \frac{1}{2} C_D \, \rho_w . A_p . v^2$$

$$\uparrow F_D \propto v^2 \uparrow$$

$$\vec{F}_{ext} = m\vec{a} = m\frac{d\vec{v}}{dt}$$

$$\vec{F}_{ext} \propto d\vec{v}$$

At equilibrium :-



$$\vec{F}_{ext} = 0$$

$$\boxed{W = F_B + F_D}$$

$$d\vec{v} = 0$$

$V_s \Rightarrow$ constant

$V_s \rightarrow$ settling / Final / Terminal velocity

$$\frac{4}{3}\pi r^3 \gamma_s - \frac{4}{3}\pi r^3 \gamma_w = \frac{1}{2} C_D \rho_w A_p V_s^2$$

$$\frac{4}{3}\pi r^3 (\gamma_s - \gamma_w) = \frac{1}{2} C_D \frac{\gamma_w}{g} \times \pi r^2 \times V_s^2$$

$$\frac{d}{2} \times \frac{4}{3}(G_s - 1) = \frac{C_D}{2g} \times V_s^2$$

$$V_s^2 = \frac{4}{3} \frac{gd(G_s - 1)}{C_D}$$

$$\boxed{V_s = \sqrt{\frac{4}{3} gd \frac{(G_s - 1)}{C_D}}}$$

$V_s \propto d \uparrow$, $V_s \propto G_s \uparrow$

Determination of C_D

C_D is a function of diameter of particles & Reynold's number.

$$\boxed{Re = \frac{\rho V x}{\mu}}$$

x : characteristic dimension

In this case, $Re = \frac{\rho V_s d}{\mu}$

CASE I : $d \leq 0.1$ mm
& $Re < 1$

$$\boxed{C_D = \frac{24}{Re}}$$

$$V_s^{\cancel{2}} = \frac{\cancel{4}}{3} g d \frac{(G_s - 1)}{\cancel{24}\ 6} \times \frac{V_s d}{\nu}$$

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

Stoke's law for laminar flow.

CASE II : $0.1 < d \leq 1$ mm
$1 \leq Re < 10^3$

$$\boxed{C_D = \frac{24}{Re} + \frac{3}{\sqrt{Re}} + 0.34}$$

Hazen's law for transition flow

$$\boxed{V_s = \sqrt{\frac{4}{3} g d \frac{(G_s - 1)}{D}}}$$

CASE III : $d > 1mm$

$10^3 \le Re < 10^4$

$$\boxed{V_s = \sqrt{3.3\, g d (G_s - 1)}}$$

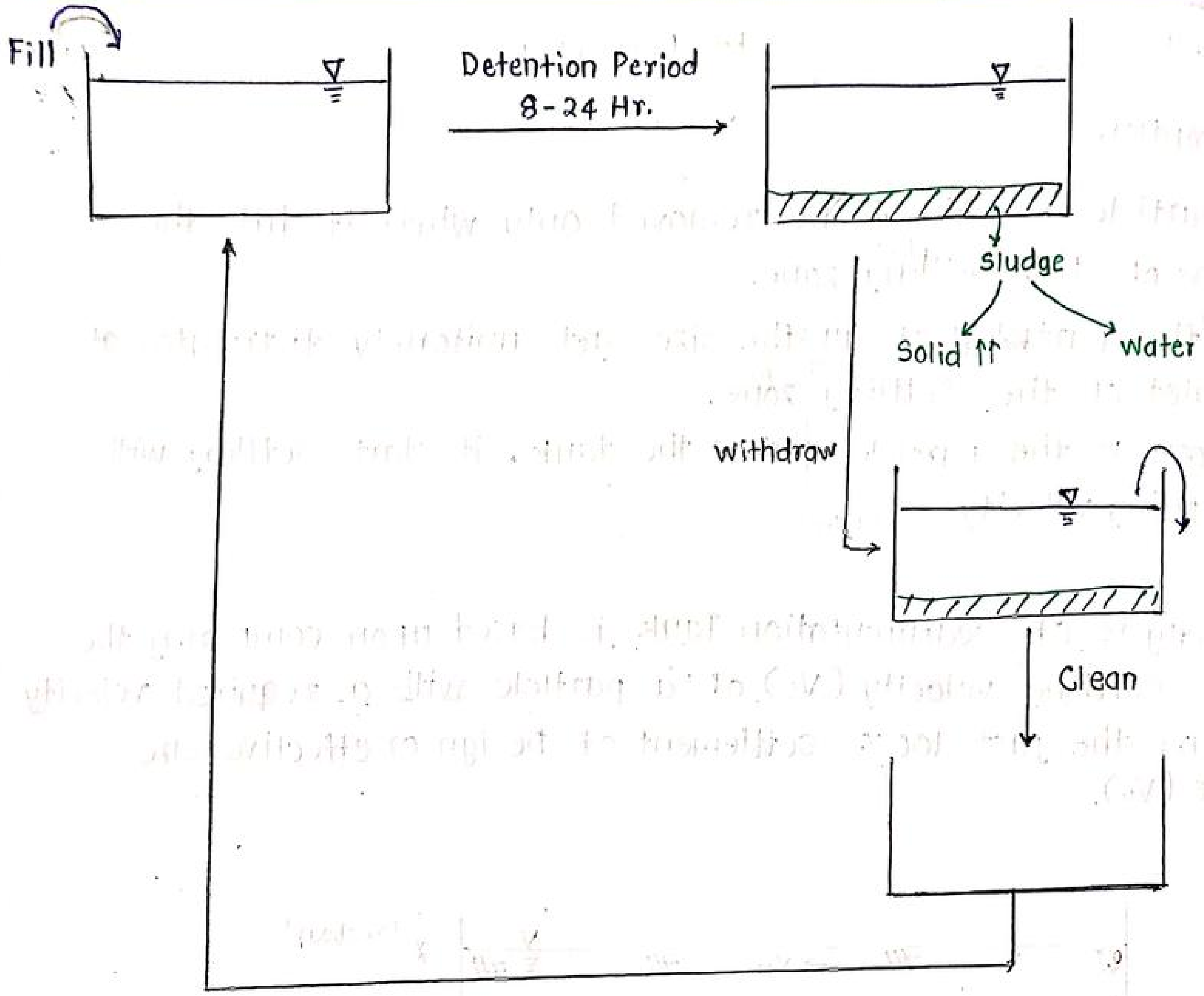
$$\boxed{V_s = 1.8 \sqrt{g d (G_s - 1)}}$$ Newton's law for Turbulent Flow

* Sedimentation Tanks / Clarifiers

- Sedimentation Tanks / Clarifiers
  - Quiescent Type or Fill & Draw Type
  - Continuos Flow type
    - Horizontal Flow Type or Rectangular Tank
    - Vertical Flow Type or Circular Tank

Quiescent Type or Fill & Draw Type Tank

- In this tank, water is filled & kept for Detention Period of 8 to 24 Hrs.
- This can handle only small discharges with less fluctuation.
- This is not adopted commerically because the other treatment unit are not in function during the Detention period of water in the tank



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Horizontal Flow Type / Rectangular Tank

- It is a continuos flow type tank in which water enters horizonatally, moves horizontally & exits horizontally.
- A Rectangular tank consists of following 4 zone
  1. Inlet zone
  2. Outlet zone
  3. Settling zone
  4. Sludge zone
- The Analysis & Design of Tank is based upon settling zone only

Analysis

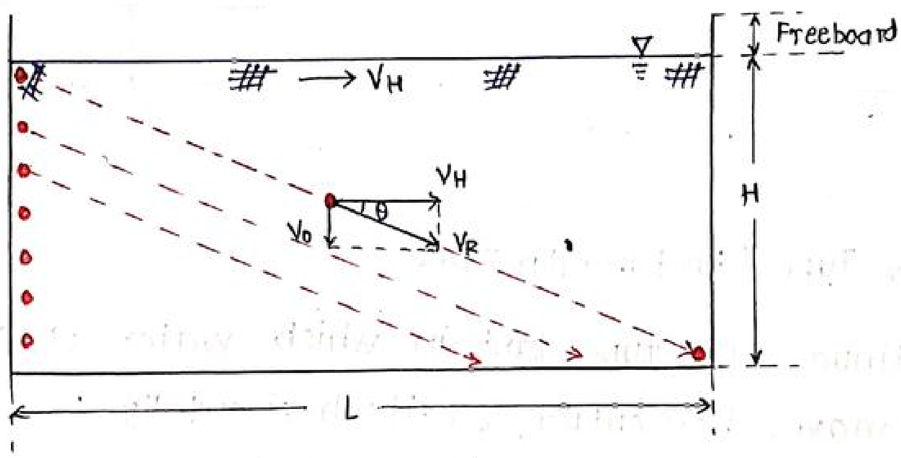
Assumptions

1> A Particle is said to be removed only when it hits the bottom of the settling zone.

2> All the particles of all the sizes get uniformly distributed at the inlet of the settling zone.

3> As soon as the particle enters the tank, it starts settling with its settling velocity

The Analysis of sedimentation Tank is based upon comparing the actual settling velocity (V_s) of a particle with a required velocity to ensure the just 100% settlement of Design or effective size particle (V_o).



Detention Time,

$$\boxed{D_t = \frac{\text{Volume of the Tank}}{\text{Discharge passing through Tank}}}$$

$$\boxed{D_t = \frac{V}{Q}}$$

$$\boxed{D_t = \frac{V}{Q} = \frac{LBH}{Q} = \frac{L}{V_H} = \frac{H}{V_o}}$$

$$\frac{LBH}{Q} = \frac{H}{V_0}$$

$$\therefore \boxed{V_Q = \frac{Q}{BL}}$$

V_0 → Surface overflow rate
→ Unit of velocity
→ $m^3/d/m^2$, $l/d/m^2$ etc.

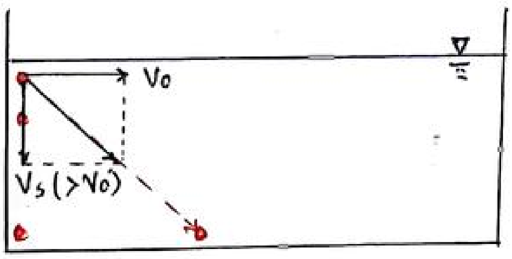
V_0 → Depends on Flow & Gravity
V_s → Depends on Dia. & G_s

CASE I : $V_s = V_0$

η = just 100 %

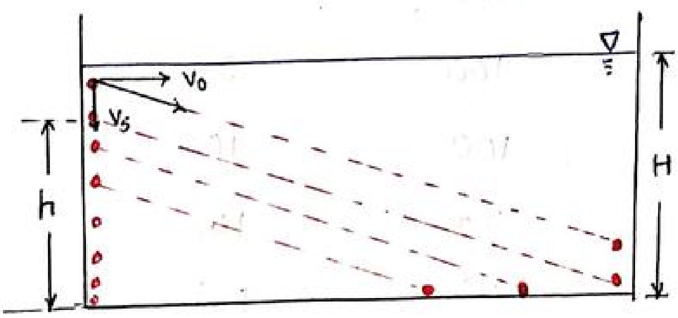
CASE II : $V_s > V_0$

η = 100 %



CASE III : $V_s < V_0$

$\eta < 100\%$



Efficiency of settlement of Particles having $V_s < V_o$

Let, $W \rightarrow$ wt. of particles entering the tank

$h \rightarrow$ Ht. upon which the particle gets settled

$\frac{W}{H} \rightarrow$ wt. of particles present per unit Height

$$\boxed{\eta = \frac{\text{wt. of particle settled}}{\text{wt. of particle entered}} \times 100}$$

$$\eta = \frac{\frac{W}{H} \times h}{W} \times 100$$

$$\boxed{\eta = \frac{h}{H} \times 100}$$

$H = V_o \times D_t$

$h = V_s \times D_t$

$$\eta = \frac{V_s \times D_t}{V_o \times D_t} \times 100$$

$$\boxed{\eta = \frac{V_s}{V_o} \times 100}$$

Pg. No. 103

Q.47>

Size (mm)	wt. (mg)	η %	wt. Settled (mg)
6	150	100	150
5	250	100	250
4	100	100	100
3	200	70	140
2	300	55	165
	1000 mg		805 mg

$$\eta = \frac{805}{1000} \times 100 = 80.5\,\%$$

Pg. No. 103

Q.48)

Particle Size (mm)	Settling velocity (mm/sec)	Quantity	Efficiency
0.1	0.2	10	57.14
0.2	0.25	15	71.42
0.3	0.3	5	85.71
$\boxed{0.4}$	$\boxed{0.35}$	20	100
0.5	0.4	30	100
0.6	0.5	20	100

wt. entered Let initial wt. = 1000 g	wt. Settled
100 g	57.14 g
150 g	107.13 g
50 g	42.855 g
200 g	200 g
300 g	300 g
200 g	200 g
1000 g	907.125

$$\therefore \text{Overall } \eta = \frac{907.125}{1000} = 90.71\%$$

Pg. No. 102 (WB)

Q.40>

Density of water = 1000 kg/m³

Density of Particle = 2650 kg/m³

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

$\eta = 90\%$, $\eta = \frac{V_s}{V_0} \times 100$

$$0.9\, V_0 = V_s$$

$$V_s = 0.9 \times 40 = 36$$

$$\frac{9.81 \text{ m/s}}{18}(2.65 - 1) \times \frac{d^2}{1.1 \times 10^{-6} \text{ m}^2/\text{s}} = \frac{40 \text{ m/d}}{86400 \text{ s/d}} \times 0.9$$

$$d = 22.57 \times 10^{-6} \text{ m}$$
$$= 22.57\ \mu\text{m}$$

Pg. No. 103 (WB)

Q.49>

$H = 3.5$ m

$L = 65$ m

$V_H = 1.22$ cm/sec

$G_s = 2.65$

$\nu = 0.01$ cm²/sec

$$V_s = V_0$$

Assuming that the settling velocity is described by Stoke's law

$$\frac{9.81\ m/s^2}{18}(2.65-1)\times\frac{d^2}{10^{-6}\ m^2/s} = \frac{3.5\,m}{65\,m}\times 122\times 10^{-2}\ m/s$$

$$d = 27\ \mu m$$
$$= 0.027\ mm$$

Stoke's Law is found to be valid if $d \le 0.1\,mm$ & $Re < 1$.

$\because d = 0.027\ mm < 0.1\ mm$, this criteria is satisfied.

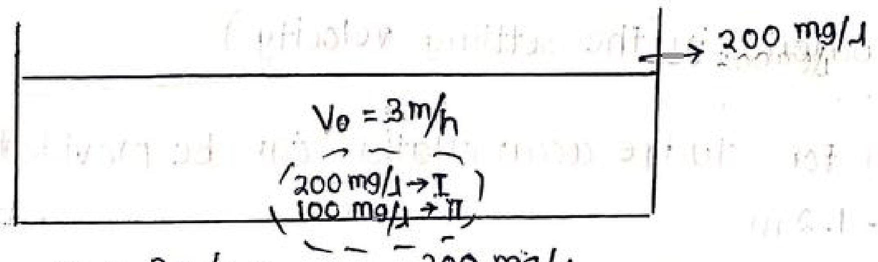
Check for Re

$$Re = \frac{6.56\times 10^{-4}\times 27\times 10^{-6}\ m/s\times m}{10^{-6}\ m^2/s}$$

$$= 0.017$$

Pg. No. 96

Q.1>



$I \rightarrow V_s = 3\,m/h$ — 200 mg/l

$II \rightarrow V_s = 1\,m/h$ — 300 mg/l

$\eta_I = 100\%$

$\eta_{II} = \frac{1}{3}\times 100\% = 33\%$

* Design of Rectangular Tank

1) Detention Time

(i) For P.S → D_t = 4-8 Hr.
(ii) For C.A.S → D_t = 2-4 Hr.

2) S.O.R

(i) For P.S → V_o = 15000 - 30000 $l/m^2/d$
(ii) For C.A.S → V_o = 30000 - 40000 $l/m^2/d$

3) Height of Settling zone = 1.8 - 6 m

4) $\frac{L}{B}$ = 2 to 5

5) V_H = 0.15 to 0.3 m/min
(To avoid Turbulence in the settling velocity)

6) Additional depth for sludge accumulation can be provided between 0.8 - 1.2 m

7) Free board = 0.3 m

NOTE: Usually, additional volume for inlet & outlet zone is provided in the Tank which is taken approximately 15 - 20% of the volume of settling zone.

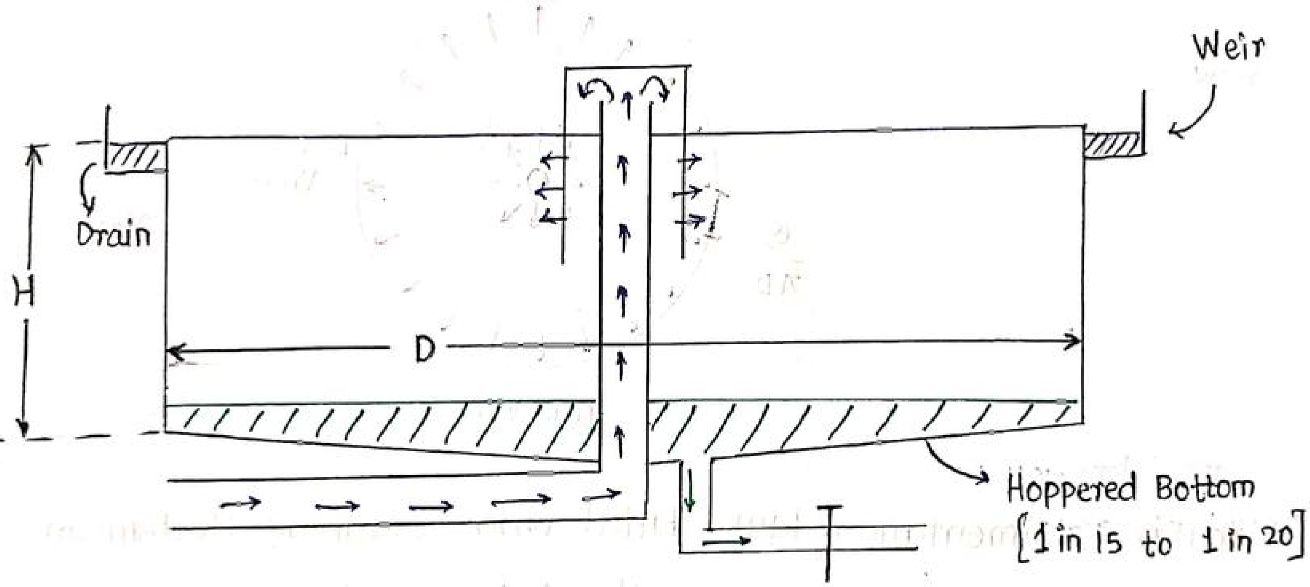
⁎ Vertical Flow Type / Circular Tank

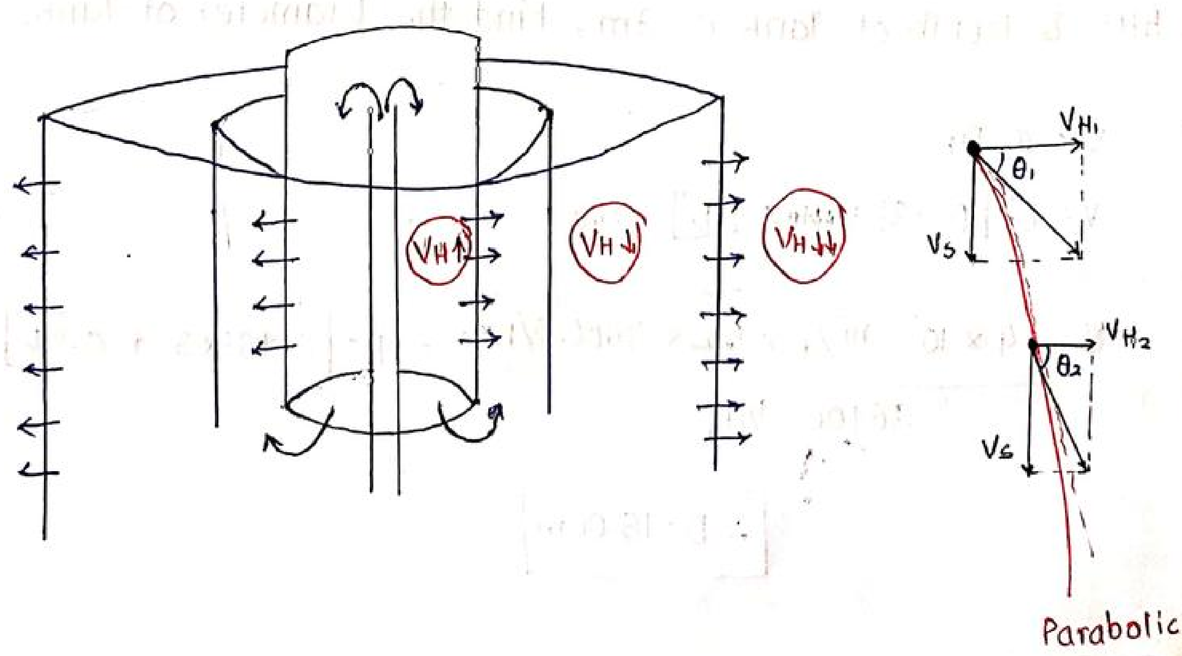
In this tank, water enters vertically at the center, flows radially outwards & exits Horizontally by overflowing through a weir.

The bottom of the Tank in kept inclined to accumulate the sludge at the Sludge Outlet.

This bottom is referred as Hoppered bottom which ultimately helps in Densification of Sludge there by reducing handeling cost of Sludge.

Since the Horizontal velocity of water continuosly decreases, the trajectory of settlement of Particle is not linear.





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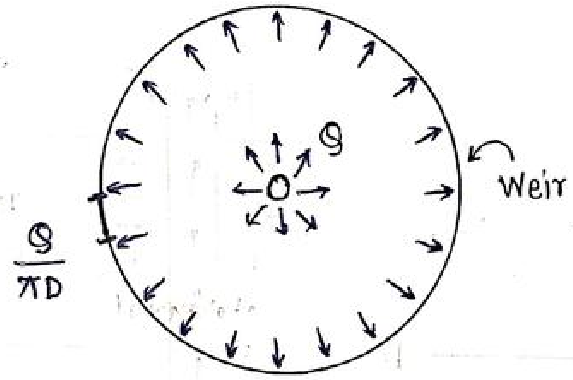
1> Volume of Tank , $V = Q \cdot D_t$

$$\boxed{V = D^2\left[0.785H + 0.11D\right]}$$

2> SOR , $\boxed{V_0 = \dfrac{Q}{\pi D^2/4}}$ $m^3/d/m^2$

3> Weir Overflow Rate

W.O.R , $\boxed{V_w = \dfrac{Q}{\pi D}}$ $m^3/d/m$



Top View

Q> A Circular Sedimentation Tank fitted with Scraping Mechanism is to treat a discharge of 4 MLD, the Detention time required is 5Hr. & Depth of Tank is 3m, Find the Diameter of Tank.

=> $V = Q \cdot D_t$

$V = D^2\left[0.785H + 0.11D\right]$

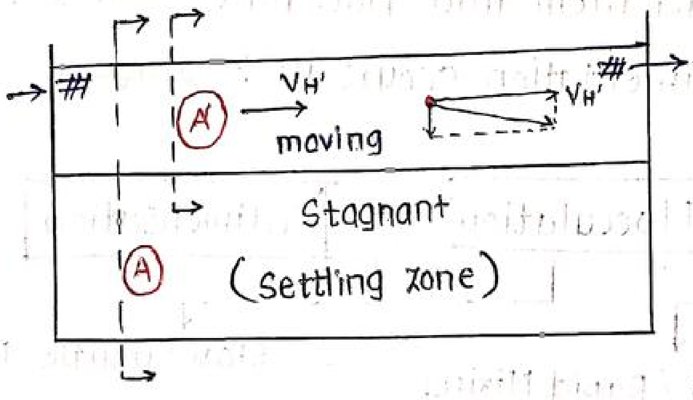
$$\frac{4 \times 10^3\ m^3/d \times 5h \times 3600\ s/h}{86400\ s/d} = D^2\left[0.785 \times 3 + 0.11D\right]$$

$$\boxed{\therefore D = 18.06m}$$

https://civinnovate.com/civil-engineering-notes/

* Short Circuiting in Sedimentation Tank

- Short circuiting in an operation trouble which occurs when water does not get uniformally distributed throughout the c/s of Tank.
- This happens when a large portion of water passes directly over the Top surface of Tank without being detend for the Intended Detention Time.
- The bottom layers of water become stagnant & do not contribute to the Discharge. It is measured by displacement efficiency as follows



$A > A'$

$V_H < V_H'$

$D_t > D_t'$

$D_t' \rightarrow$ Actual Flow through period

$D_t \rightarrow$ Theoretically required Detention Time

Displacement Efficiency,

$$\boxed{\eta_d = \frac{D_t'}{D_t} \times 100}$$

For no short circuiting, $\eta_d \geq 100\%$.

$\eta_d > 100\%$, if provided volume is larger than actually required.

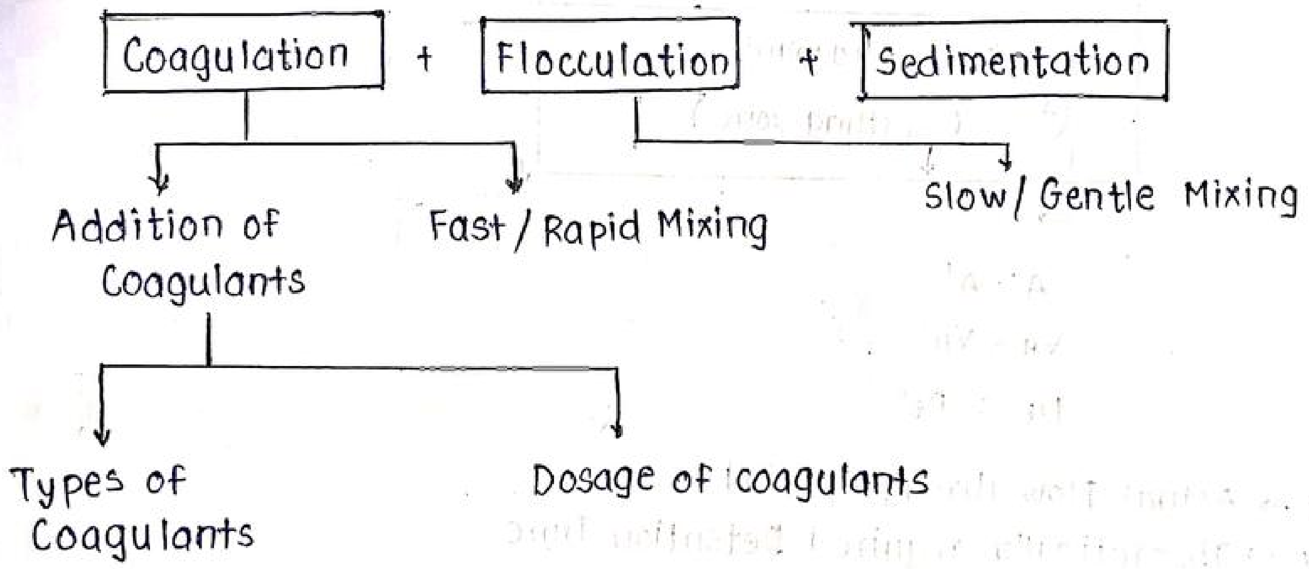
Remedial Measures

1. Provide large Vol. of Tank than required.
2. Provide Baffle walls.

* COAGULATION AIDED SEDIMENTATION

- Suspended Solid of Fine nature (colloidal solids) cannot settle down during Plain Sedementation process with Ordinary Detention Time.
- Such particles can however be removed very easily by increasing their sizes & converting them into flocculated masses.

Coagulation Aided Sedimentation occurs in 3 stages.



NOTE : (i) In modern W.T.P the combination of coagulation & Flocculation is designated by single term i.e Flocculation

(ii) Now a days, a unit is developed in which all the 3 stages can occur. It is called as ClariFlocculator.

* COAGULATION

- Coagulation is a process in which certain chemicals called as coagulants are added in water, to ensure the agglomaration of suspended solids present in water.
- The suspended solids in water vary considerably in source, size, composition, charge, shape & Density.
- Most of the suspended solids in water are negativly charged (clay, silt etc.) & thus they tend to repul eachother when they come close together.
- A coagulant must possess the following properties

<1> It must be non toxic.

<2> It must not get dissolved & produce stickly & porous ppt.

<3> It should be able to induce positive charges in water to neutralized the negative charges over various suspended solid.

- The commonly used coagulants are

- Alum
- Copperas
- Chlorinated Copperas
- Sodium Aluminate

As soon as the coagulant is added, high energy in the form of Rapid mixing is provided to disperse the coagulant uniformly in water.

NOTE : Due to sticky surface of coagulant ppt. they can also trap micro-organism thereby reducing the quantity of disinfectant required.

1. ALUM

- Alum when added in water reacts with Alkalinity present in water.
- Alum forms stickly Gelatinous ppt. of Aluminiom Hydroxide which is ultimately responsible for coagulation.
- In general Alum induces permanent Hardness in water.
- In general Alum induces Acidity in water. This is due to the ~~realize of CO_2~~ release of CO_2 which subsequently form a weak acid i.e H_2CO_3 which reduces the pH of water slightly
- The usual dosage of Alum in India is 10 - 30 mg/l.
- Alum works in a pH range of 6.5 to 8.5 & thus, it become suitable for Drinking Water Treatment.
- Since Alum reacts with Alkalinity, lime or soda can be used to artifically induce alkalinity in water if the alkalinity in water is less.

NOTE: Lime or soda are usually use because they are cheap, Non Toxic & their permissible limit in Human body is High

- Alum with Lime induces permanent Hardness but doesnot induce acidity.
- Alum with soda induces Acidity but doesnot induce permanent Hardness

Reaction of Alum

$$Al_2(SO_4)_3 18H_2O + \underset{(OH^-,\ CO_3^{2-},\ HCO_3^-)}{\text{Alkaline Species}} \longrightarrow \underset{\text{(sticky gel. ppt.)}}{Al(OH_3)\downarrow} + \text{other product}$$

$$Al_2(SO_4)_3 18H_2O + 3Ca(OH)_2 \longrightarrow 2Al(OH_3)\downarrow + \underset{\text{Perm. Hardness}}{3CaSO_4} + 18H_2O$$

$$Al_2(SO_4)_3 18H_2O + 3Na_2CO_3 \longrightarrow 2Al(OH_3)\downarrow + 3NaSO_4 + \underset{\text{Acidity}}{3CO_2} + 15H_2O$$

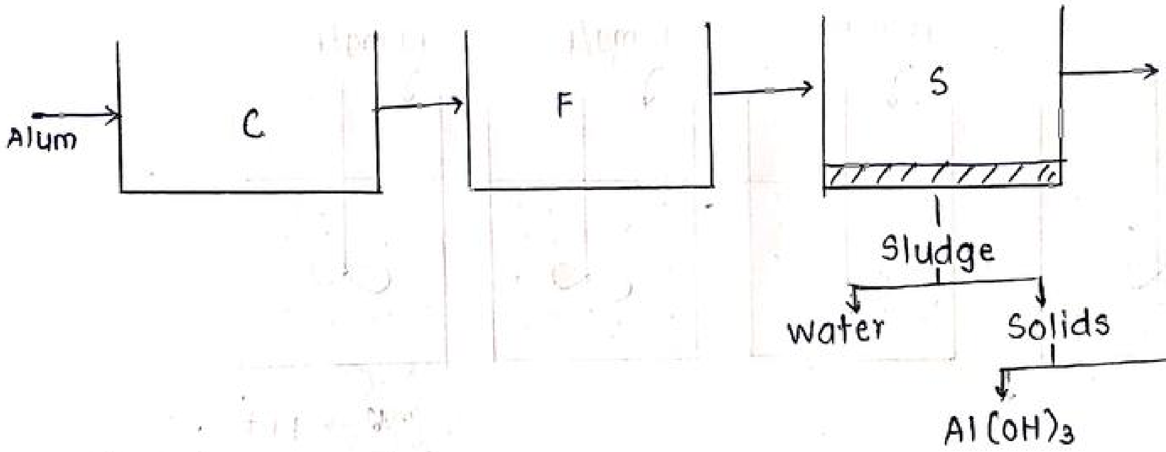
$$Al_2(SO_4)_3 18H_2O + 3Ca(HCO_3)_2 \longrightarrow 2Al(OH)_3\downarrow + \underset{\text{Perm. Hardness}}{3CaSO_4} + \underset{\text{Acidity}}{6CO_2} + 18H_2O$$

666 g of alum reacts with 486 g of $Ca(HCO_3)_2$ to produce 156 g of $Al(OH)_3$ ppt.

666g of alum reacts with $\frac{486}{81g} \times 50g = 300g$ of $CaCO_3$ alk. to produce 156 g of $Al(OH)_3$ ppt.

$\boxed{\text{1 g of alum reacts with 0.45g of } CaCO_3 \text{ alk. to produce 0.23g of } Al(OH)_3 \text{ ppt.}}$

Quantity of Sludge Produce



① Solid Content of Sludge

$$\boxed{S.C = \frac{M_{solids}}{M_{solid} + M_{water}} \times 100 = \frac{M_{solids}}{M_{sludge}} \times 100}$$

② Water Content of Sludge

$$\boxed{W.C = \frac{M_{water}}{M_{sludge}} \times 100}$$

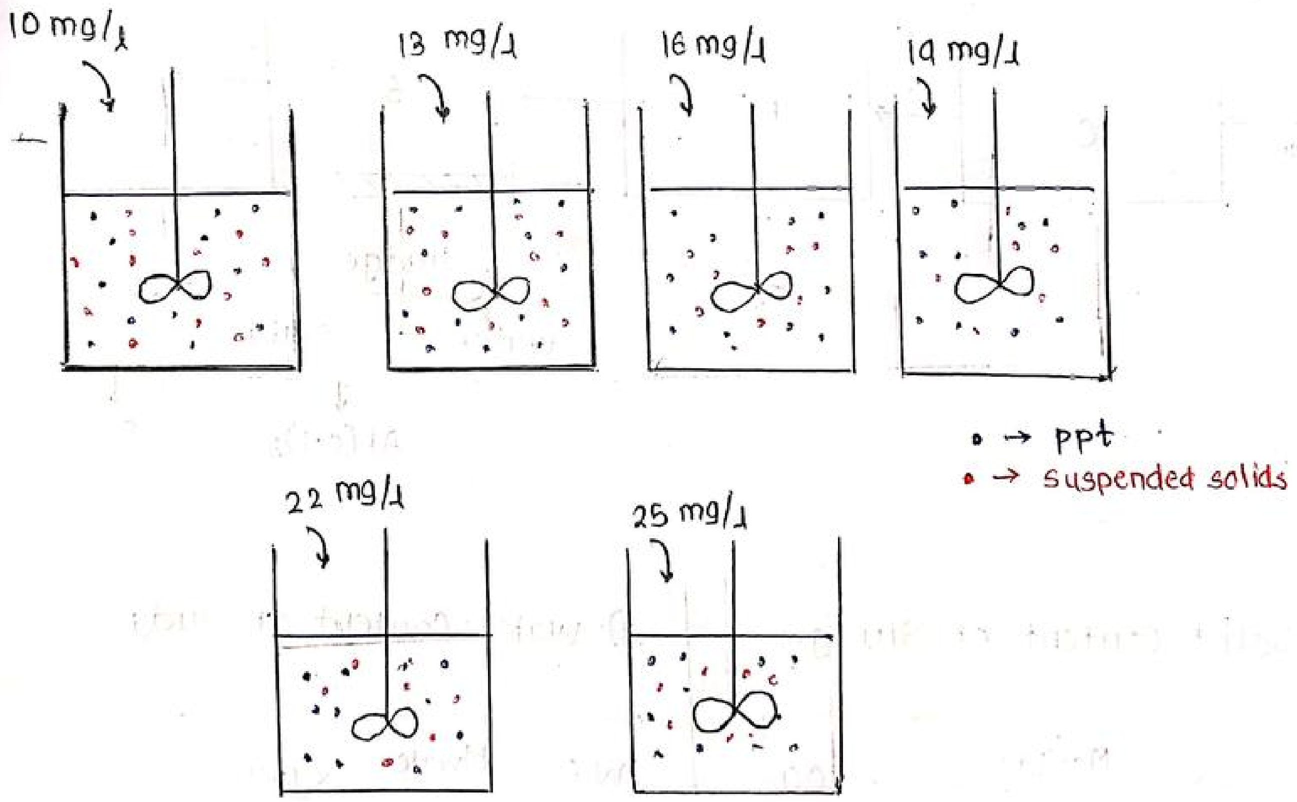
Eg. S.C = 3% → 100 kg sludge

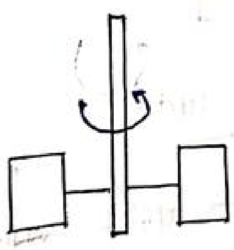
3 kg Solids ← → 97 kg. water

Optimum Dosage of Coagulants (Jar Test)

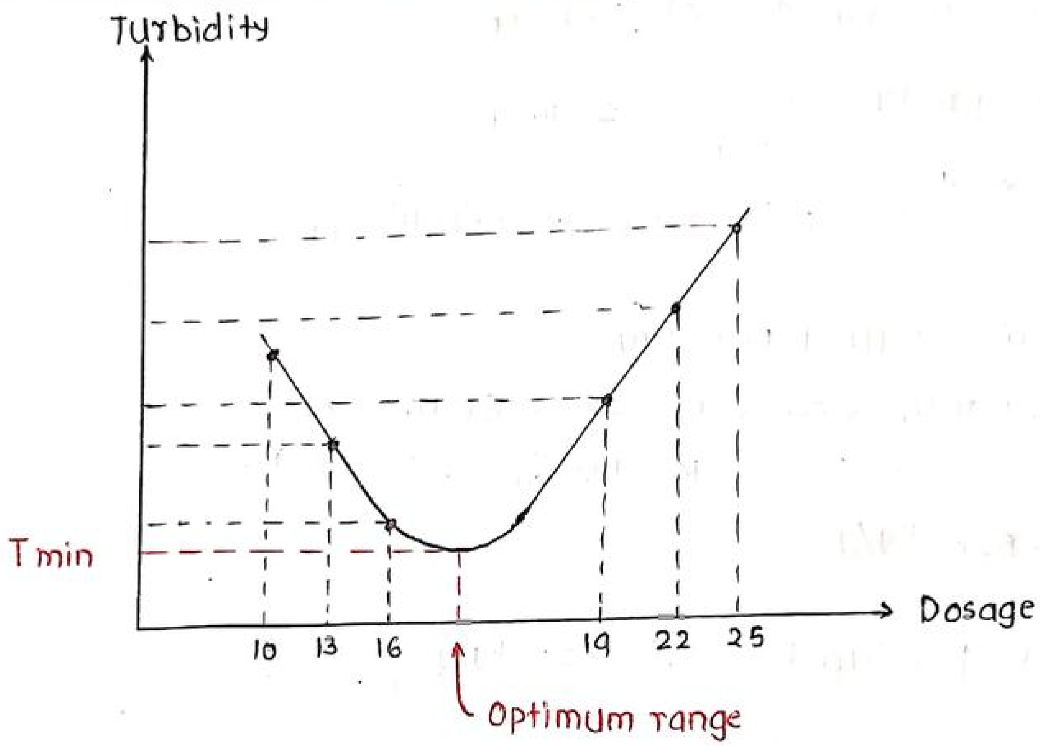
- The dosage of a coagulant can be convinently obtained with the help of Jar Test.
- In this Test 6 Jars are taken as a standard & processes of Coagulation, flocculation & Sedimentation are simulated in jar usually for about 20 mins.
- A Graph is plotted b/w Turbidity & Dosage and the dosage corresponding to minimum Turbidity is called as optimum dosage of coagulants

NOTE: Before conducting Jar Test a small quantity of Lime is added to ensure sufficient Alkalinity for reaction of Alum





20 min
- → Fast Mixing → 1 min
- → Slow Mixing → 4-5 min
- → Sedimentation



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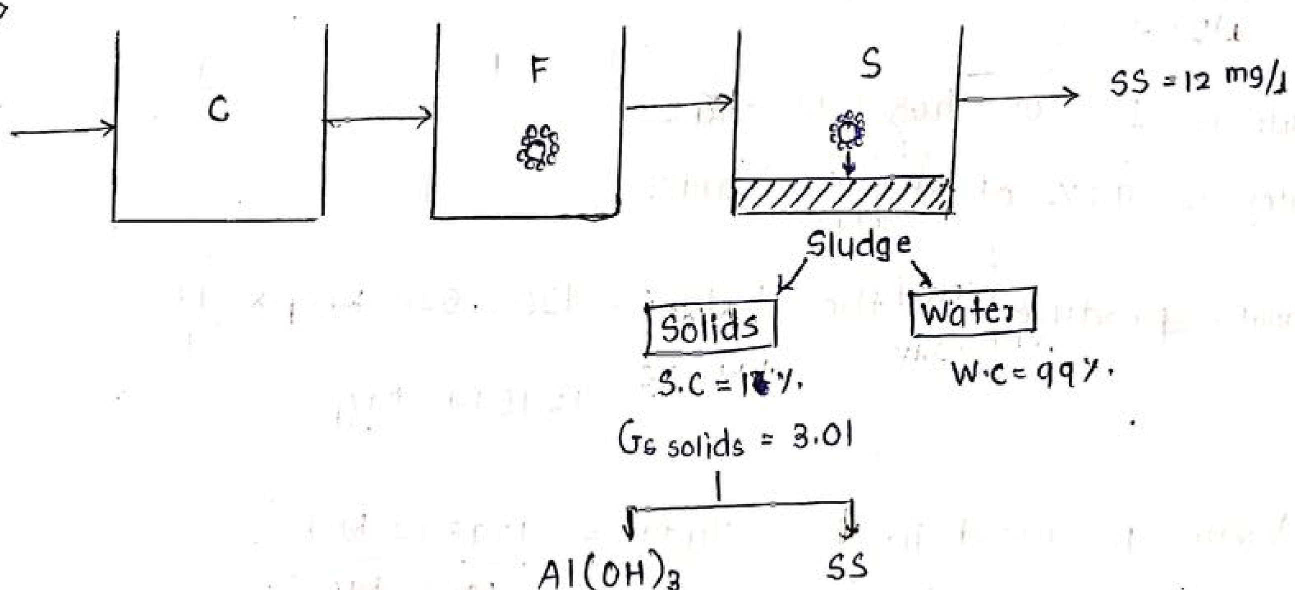
Q.51>



$(228.528\ kg/d) + (1080\ kg/d) = 1308.528\ kg/d$

Total quantity of S.S settled per day

$$= \frac{(37-12)\ mg/l \times 0.5 \times \cancel{m^3/s}\ 10^3\ l/s \times 86400\ s/d}{10^6\ mg/kg}$$

$= 1080\ kg/d$

1 g alum produces 0.23 g of $Al(OH)_3$ ppt.

$\therefore$ 23 mg/l will produce $\frac{0.23}{1} \times 23$ mg/l

$= 5.29$ mg/l of $Al(OH)_3$ ppt.

Total qty. of $Al(OH)_3$ settled per day

$$= \frac{5.29 \text{ mg/l} \times 0.5 \times 10^3 \text{ l/s} \times 86400 \text{ s/d}}{10^6 \text{ mg/kg}}$$

$= 228.528$ kg/d

Total mass of solids produced $= 1308.528$ kg/d

$$G_{s\,solids} = \frac{\rho_{solids}}{\rho_{water}} = \frac{M_{solids}}{V_{solids} \times 1000 \text{ kg/m}^3} = \frac{1308.528 \text{ kg/d}}{V_{solids} \times 1000 \text{ kg/m}^3}$$

$V_{solids} = 0.4347$ m³/d

M_{solids} is 1 % of the Total mass

M_{water} is 99 % of the Total mass

M_{water} produced in the sludge $= 1308.528 \text{ kg/d} \times \frac{99}{1}$

$= 129544$ kg/d

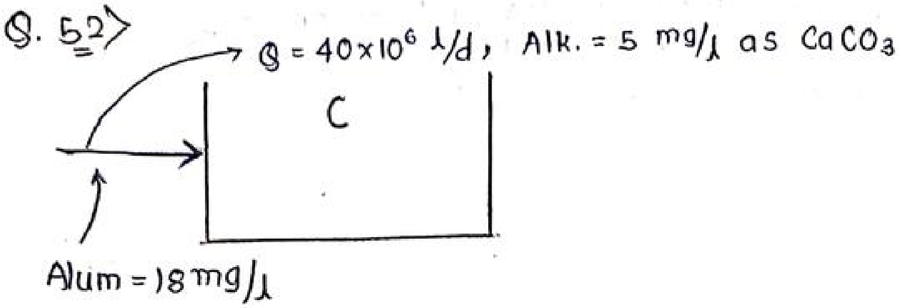
V_{water} produced in the sludge $= \frac{129544 \text{ kg/d}}{1000 \text{ kg/m}^3}$

$= 129.544$ m³/d

$V_{sludge} = V_{solids} + V_{water}$

$= 129.97 \approx 130$ m³/d

Q. 52)



1 g alum requires 0.45 g alk. of $CaCO_3$

$\therefore$ 18 mg/l will require $\frac{0.45}{1} \times 18 = 8.1$ mg/l as $CaCO_3$

Alkalinity to be added ~~extremely~~ externally = $8.1 - 5 = 3.1$ mg/l as $CaCO_3$

Qty. of Filter Alum (100% Pure)

$$= \frac{18 \text{ mg/l} \times 40 \times 10^6 \text{ l/d} \times 365 \text{ d/y}}{10^9 \text{ mg/ton}}$$

$= 262.8$ ton/y

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

Quick lime — Hydrated lime

Alk. = 3.1 mg/l as $CaCO_3$

mg. eq/l of Alk. req. = $\frac{3.1}{50}$

$\frac{3.1}{50}$ mg. eq/l of CaO is required

$\therefore$ Concⁿ of CaO required to be added in water $= \frac{3.1}{50} \times 28$

$= 1.736$ mg/l of CaO

$$\text{Qty. of CaO req.} = \frac{1.736 \text{ mg/l} \times 40 \times 10^6 \text{ l/d} \times 365 \text{ d/y}}{10^9 \text{ mg/ton}}$$

$= 25.34$ ton/y

If CaO is 85% Pure

Qty. of CaO req. = $\frac{25.34}{0.85}$ = 29.81 ton/y

2. Copperas

- Copperas is Hydrated Ferrous Sulphate [$FeSO_4 . 7H_2O$]
- Copperas also reacts with alkalinity in water. The Alkalinity is in form of Lime.
- Copperas reacts only when pH of water is greater than 9.
- Usually Lime is added along with Copperas and:-

CASE I: When Lime is added first

1 mole of copperas consumes 1 mole of Lime.

CASE II: When copperas is added first // 1st reacts with water & then with Lime //

1 mole of copperas consumes 2 mole of Lime

- Copperas produces Stickly Gelatinous ppt. of Ferric Hydroxide $Fe(OH)_3$
- 1 mole of copperas produces 1 mole of ppt.
- Copperas induces reddish brown colour in water which make it unsuitable for the treatment of Drinking water.
- Copperas induces permanent Hardness in water due to production of $CaSO_4$
- Its usual dosage for surface water is 10 - 30 mg/lit.
- The method of addition of Lime & Copperas is referred as Lime Copperas method.

3. Chlorinated Copperas

- It is formed by Chlorinating Copperas at a high temperature which produce $FeCl_3 + Fe_2(SO_4)_3$
- Ferric Chloride is effectiveat a pH of 3.5 - 6.5 & Ferric Sulphate is effective at pH greater than 9.
- Thus Chlorinated Copperas has wider working pH range than copperas.
- It cannot be used for Drinking water.
- It induces permanent Hardness in water.
- It also reacts with Lime (Alkalinity)
- It doesnot induce any colour in Acidic condition

4. Sodium Aluminate ($Na_2Al_2O_4$)

- It is formed as follows

$$Na_2O + Al_2O_3 \xrightarrow{\Delta} Na_2Al_2O_4$$

- It reacts with Calcium or Magnesium present in water & forms sticky Gelatinous ppt. of Calcium Aluminate & Magnesium Aluminate
- It doesnot require alkalinity for its reaction
- It requires Hardness for its reaction.
- Its manufacture is costly.
- It has a wider working pH Range (4 to 11)
- Since Hardness is required in drinking water, it cannot be used for treatment of drinking water.
- It can be used for industrial water supplies

$$\underset{\text{(Hardess)}}{Na_2Al_2O_4 + Ca^{2+} \text{ or } Mg^{2+}} \longrightarrow \underset{\text{(Sticky gel. ppt.)}}{CaAl_2O_4 \text{ or } MgAl_2O_4} + 2Na^+$$

Aluminium Salt	Iron Salt
1) Costlier	1) Cheaper
2) Flocs are lighter	2) Flocs are heavier
3) Reacts Slower	3) Reacts Faster
4) Working pH Range coincides with pH of Drinking water	4) Working pH range don't coincide with pH of Drinking water.
5) Lesser working pH Range	5) Wider working pH Range
6) Deteriorate Rate is less	6) Deteriorate Rate is High.

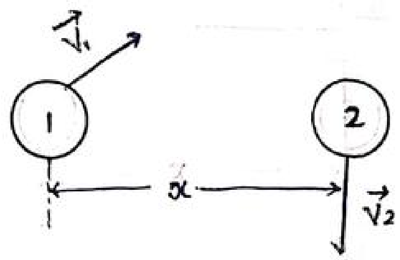
* FAST MIXING / RAPID MIXING

- In order to disperse the coagulant uniformly & to prevent the localization of concentration of coagulant, fast mixing is induced.
- Fast Mixing is induced by Mechanical or Flash mixers in which paddles are installed on vertical shaft, which rotate at a high speed thereby generating huge turbulence.
- The disturbance in water is generated by the drag force imparted by the surface area of the paddle.
- Thus, the Power required to rotate the shaft is against the work done by the drag on the paddle

* Mixing Theory

- Proper mixing is said to take place when large No. of contact opportunity are provided in the system.
- Intensity of mixing is denoted by parameter called as Temporal mean velocity gradient denoted by 'G.'

- As Intensity of mixing increases, value of 'G' increases



$$G = \frac{\overrightarrow{\Delta V}}{x} = \frac{|\vec{V_1} - \vec{V_2}|}{x}$$

NOTE :

How much time it will require for one collision?

$$\boxed{t = \frac{x}{\overrightarrow{V_{rel}}}}$$

How many collision will occur per unit time?

$$\boxed{G = \frac{1}{t} = \frac{\overrightarrow{V_{rel}}}{x}} \quad s^{-1}$$

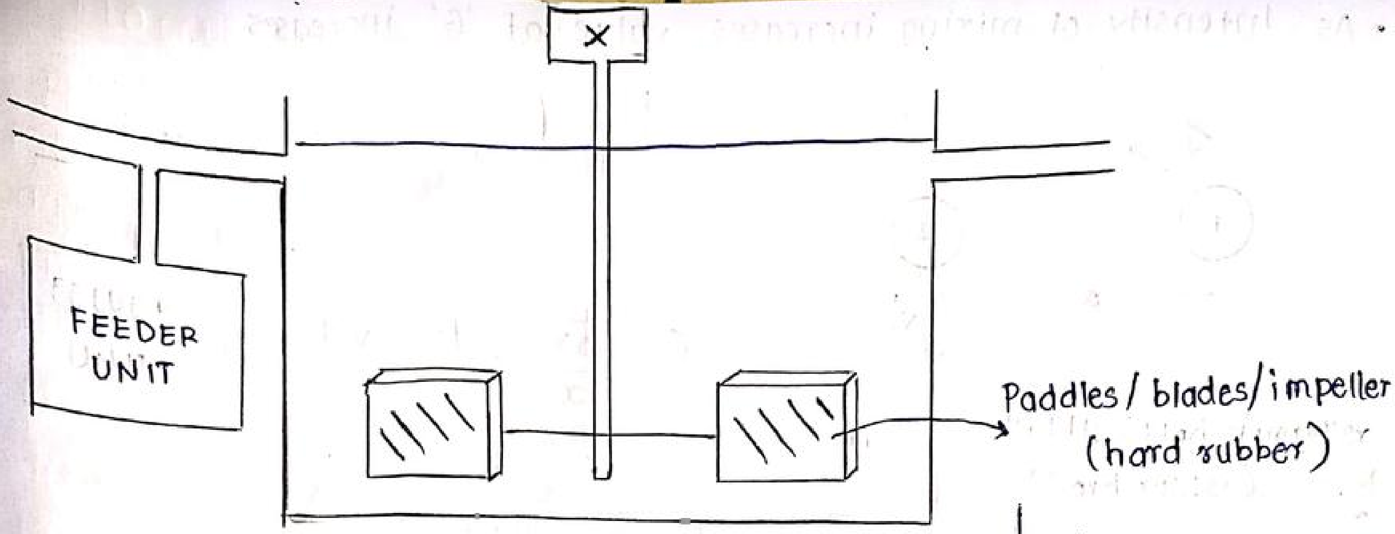
- The value of G is a flash mixer is maintained by providing sufficient power to the shaft which just ensures proper mixing.
- The intensity of mixing however depends on
  (i) Power given to the shaft
  (ii) Viscosity of water
  (iii) Volume of Tank

The relation b/w G, P, μ & V can be obtained as follows

$$G = P^a \mu^b V^c$$

$$[T^{-1}] = [ML^2T^{-3}]^a [ML^{-1}T^{-1}]^b [L^3]^c$$

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

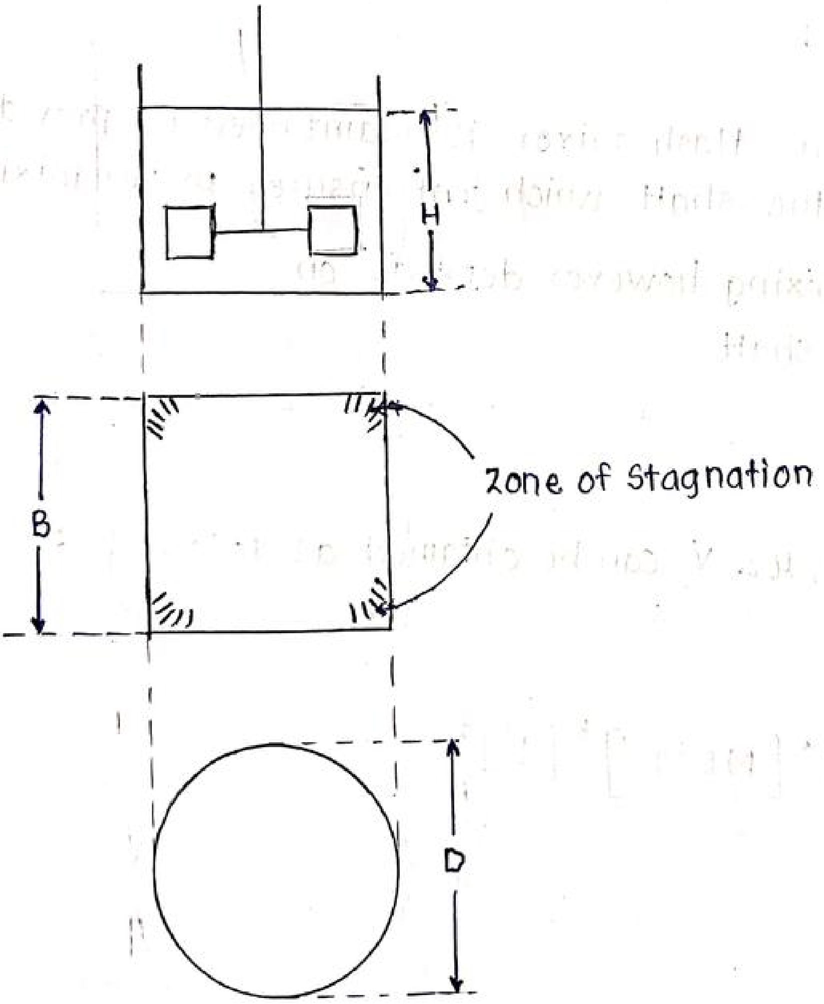


- → insert
- → less abrasion
- → Cheap
- → Light

(i) $G = 300 - 900\ s^{-1}$

(ii) $D_t = 20 - 60\ s$

(iii)



$\frac{H}{B}$ or $\frac{H}{D}$ = 1 to 3

Design Steps

Given Q & μ

1) Assume D_t
2) Find $V = Q.D_t$
3) Assume $\frac{H}{B}$ OR $\frac{H}{D}$
4) Find H & B OR D
5) Assume G
6) Find $P = \mu V G^2$

$\nu = 1.01 \times 10^{-6}$ m^2/s (at 25°C)

$\rho_w = 1000$ kg/m^3

$\mu = \rho_w \times \nu$

$= 1.01 \times 10^{-3}$ $N\text{-}s/m^2$

Pg. No. 97

Q.8) $\rho_w = 1000$ kg/m^3, $\nu = 10^{-6}$ m^2/s, $Q = 28800$ m^3/d, $G = 900$ s^{-1},

$D_t = 2\,min$, $P = ?$

$$\mu = \nu \rho_w = 10^{-6}\ \frac{m^2}{s} \times 1000\ kg/m^3$$

$$P = \mu V G^2$$

$$= \left(10^{-6}\ m^2/s \times 1000\ kg/m^3\right) \times \left[\frac{28800}{86400}\ \frac{m^3}{s} \times 120\,s\right] \times \left(900^2\ s^{-2}\right)$$

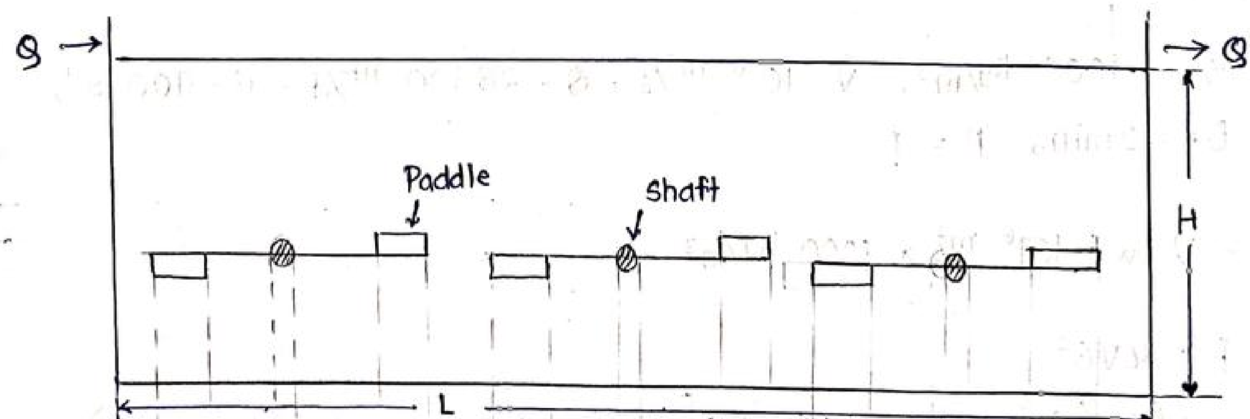
$$= 32400\ \frac{kg\text{-}m}{s^2} \times \frac{m}{s}$$

FLOCCULATION

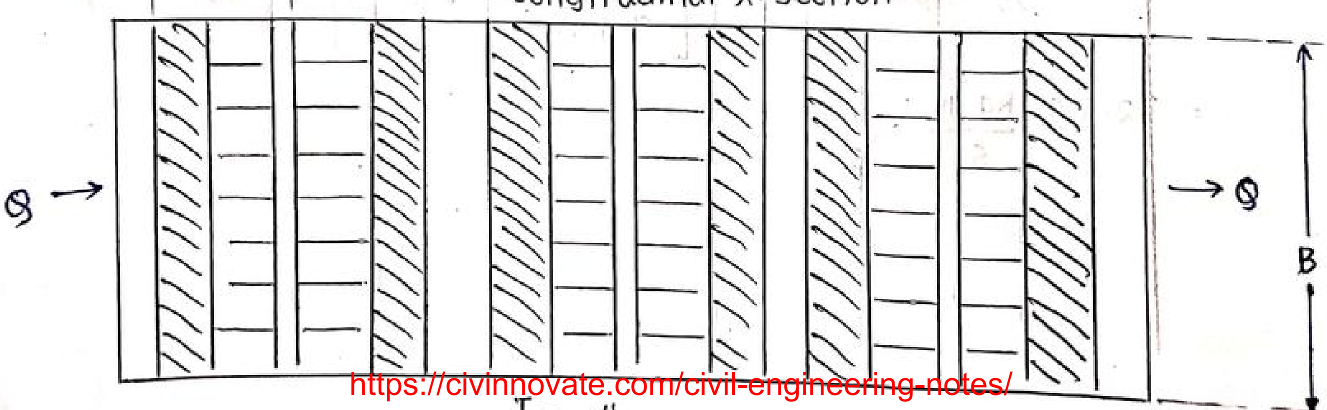
- It is a process in which coagulent particles are brought in intimate contact with each other so as to promote their agglomaration.
- This results in increased sizes thereby increasing the efficiency of sedimentation.
- In order to increase opportunities of contact, slow & gentle mixing is induced in water.
- Flocculation is conducted in flocculation chamber fitted with paddles on wall mounted shafts.

NOTE: The wall mounted shafts have lesser maintenance cost due to lesser deflections.

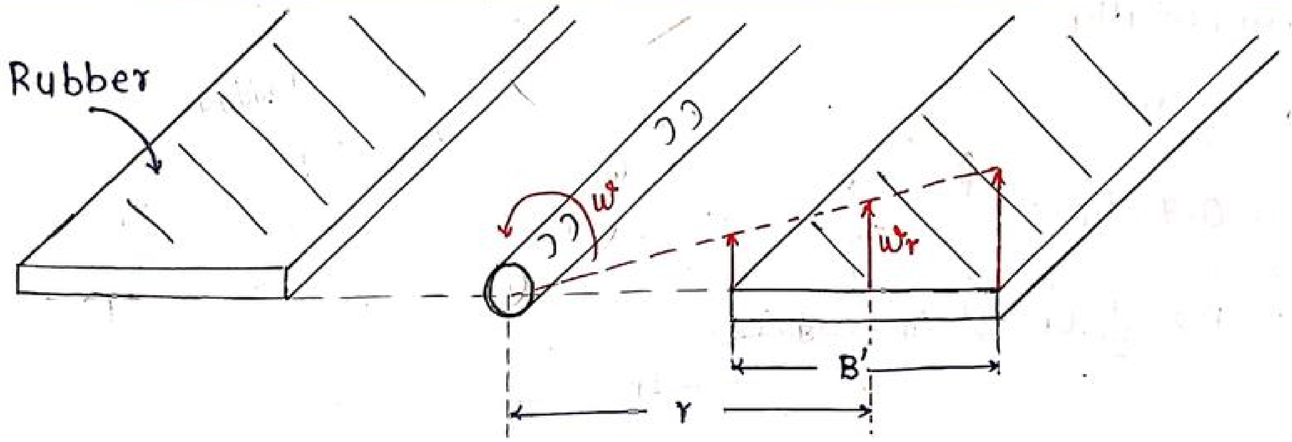
Design of Flocculation Chamber:



Longitudinal x-section



Top View



1) Power required to rotate each shaft

$$\boxed{P = F_D \times v_r}$$

F_D → Area Force

v_r → relative velocity b/w paddle & water

$$\boxed{P = \frac{1}{2} C_D \rho_w A_p v_r^3}$$

$C_D = 1.8$ (For Paddles)

A_p = Surface area of paddles in contact with water

In this case

$A_p = 2 \times B' \times B$

If 4 paddles are installed

$A_p = 4 \times B' \times B$

In General :-

$\vec{V_r} = |\vec{V_p} - \vec{V_w}|$

$\vec{V_p}$ → Avg. velocity of the paddle

$\vec{V_p} \rightarrow \omega r$

Experimentally,

$$\vec{V_w} = \frac{1}{4}\vec{V_P}$$

$$\vec{V_r} = 0.75\,\vec{V_P}$$

$$\therefore \boxed{P = \frac{1}{2} C_D \rho_w A_P (0.75 V_P)^3}$$

2) 2 or 4 ~~can be~~ Paddles can be symmetrically placed on each shaft

3) The depth of Tank is around 3 to 4.5 m

4) The Detention Time is betn 10 to 30 min

5) The Temporal Mean velocity Gradient 'G' is kept between 10 - 75 s^{-1}

NOTE: If 'G' value is very less, it may lead to settlement of flocs.
If 'G' value is very high, it may lead to disintegration of flocs.

Pg. No. 103

Q.53)

(i) Power consumption

$$P = \frac{1}{2} C_D \rho_w A_P (0.75 V_P)^3$$

$$\omega = \frac{2\pi N}{60} = \frac{2\pi \times 2.5}{60} = 0.2617 \text{ rad/s}$$

$$P = \frac{1}{2} \times 1.8 \times 1000 \frac{kg}{m^3} \times (2 \times 12 \times 0.3) m^2 \left(0.75 \times 0.261 \times 1.8 \text{ m/s}\right)^3$$

$$P = 286.07 \text{ W}$$

Total Power required = $4 \times 286.07 = 1144.28$ W

$$D_t = \frac{V}{Q} = \frac{(4.5 \times 30 \times 12)\, m^3}{75 \times 10^3\, m^3/d} \times 60 \times 24\ min/d$$

$$= 31.1\ min$$

$$G = \sqrt{\frac{P}{\mu V}} = \sqrt{\frac{1144.28\ W}{1.31 \times 10^{-3} \left(\frac{N\text{-}s}{m^2}\right) \times (4.5 \times 30 \times 12)\, m^3}}$$

$$G = 23.22\ s^{-1}$$

Type of Floc Formed

The various types of flocs which can be formed are

1. Small & Light
2. Small & Heavy
3. Large & Light
4. Large & Heavy ✓

- The desirable type of flocs are large & Heavy & to ensure the formation of such flocs 'G' & 'Dt' should be kept in range as per Design data
- In flocculat^n we define a term called as ~~conju~~ conjugation opportunity which signifies total No. of Opportunity of contact in the Tank

$Co_{min} = 10\, s^{-1} \times 10\, min \times 60\ s/min = 6000$

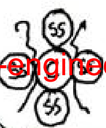
$Co_{max} = 75\, s^{-1} \times 30\, min \times 60\ s/min = 1,35,000$

$$\left\{ \begin{array}{l} G = 10 - 75\ s^{-1} \\ D_t = 10 - 30\ min \end{array} \right\}$$

$G \rightarrow$ No. of collision occuring per unit time

$G \times D_t$ → Total No. of collision occuring in tank
→ Conjugation opportunity

G ↑↑↑ Dt ↓↓↓



G ↓↓↓ Dt ↑↑↑

$2\, s^{-1}$ 50000 s

Flocs Float

~~Settle~~

Tapered Flocculation

18/11/19

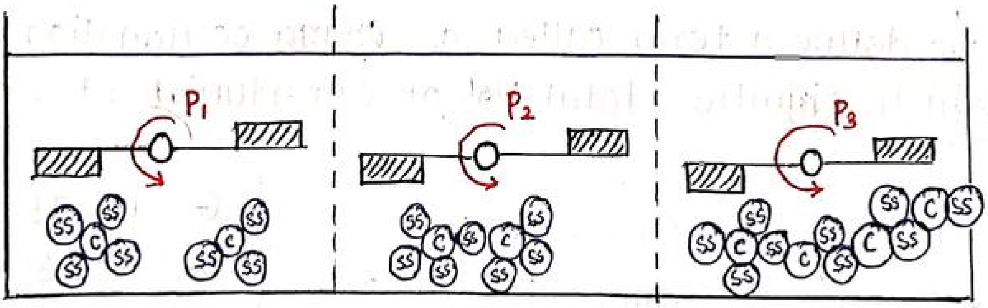
- In the conventional flocculation process, each shaft is given equal power while in Tapered flocculation process, the power given to each shaft is varied along the length of the Tank.
- Where highest power is given to inlet shaft & it is subsequently reduce to other shaft.
- This type of flocculation ensures the heaviest possible & the largest possible flocs.
- As per GOI manual.

$$\frac{G_{inlet\ shaft}}{G_{outlet\ shaft}} = 2$$

NOTE: ① If G is very high & Dt is very less, the particles will remain small but they can be comparativly higher because the small colloids can penetrate the porous structure of the precipitate

② If G is very less & Dt is very high, the flocs become comparativly larger but since the colloidal impurities cannot penetrate the porous structure of precipitate they tend to remain lighter.

$P_1 > P_2 > P_3$



largest & Denest Possible

* Mechanism of Coagulation & Flocculation

i) Ionic layer Compression

- Very small suspended solids like clay are negatively charged & thus they tend to repul eachother when they come close together.
- This particle also have an attractive force b/w them which is called as Vander waal's Force.
- When a coagulant like Alum is added, it dissociates & produces positive charges in water as follows.

$$Al_2(SO_4)_3\ 18\,H_2O \rightarrow 2\,Al^{3+} + 3\,SO_4^{2-}$$

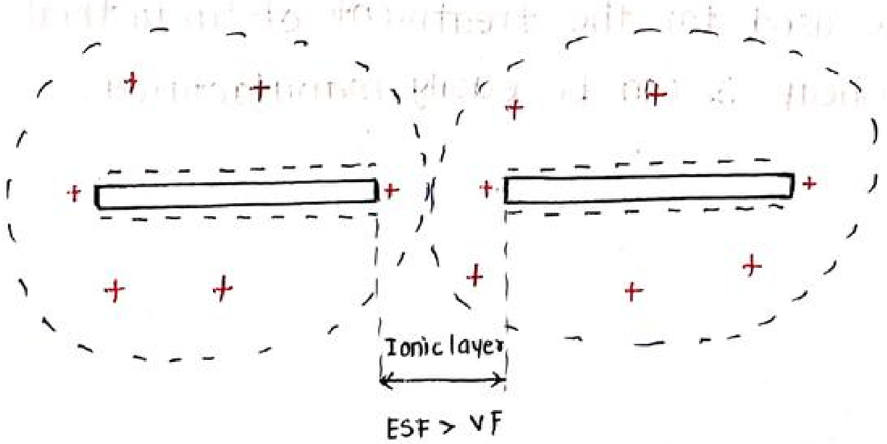
Aqua Metallic ion

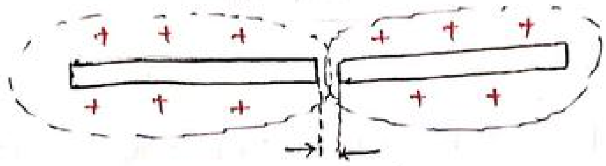
$$Al^{3+} + OH^- \longrightarrow Al(OH)^{2+}$$

$$Al(OH)^{2+} + OH^- \longrightarrow Al(OH)_2^{+}$$

$$Al(OH)_2^{+} + OH^- \longrightarrow Al(OH)_3 \downarrow$$

- With introduction of such positive charge in abundance, the repulsive electrostatic Force or zeta potential decreases & Vanderwaal Forces be predominant.
- This reduces the ionic layer & particle can coalese.





ESF < VF

2> Adsorption & Charge Neutralization

- In the flocculation chamber, the suspended solids gets adsorps over the sticky surface of the precipitate & simultaneous charge neutralization of attached suspended happen.
- This is combinely referred as adsorption & Charge neutralization

3> Sweep Coagulation

In the flocculation chamber, the flocs grow bigger & bigger eventually forming an easily settleable mass this is referred as sweep coagulation.

4> Inter particle bridging

Their are certain compound of Polymeric nature which are also capable of Sticking on to surface of suspended solid thereby forming an inter connected large size easily settleable floc.

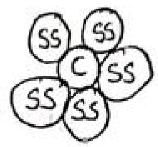
Such Polymers are not used in the treatment of drinking water, however they can be used for the treatment of industrial water as they are Cheap & can be easily manufactured.

* Conditions for Coagulation & Flocculation

1) High Alkalinity High Turbidity

Coagulant $react^n$ ✓

Sweep coag ✓



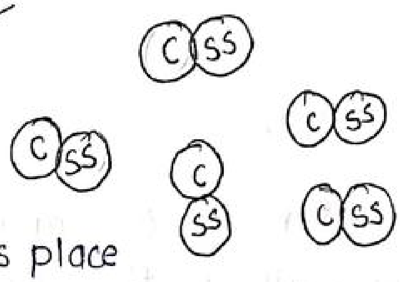
– Best condition

pH reduction takes place

2) High Alkalinity Low Turbidity

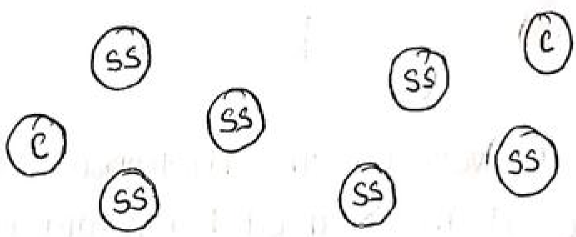
cogulant $react^n$ ✓

Flocs are smaller



pH reduction takes place

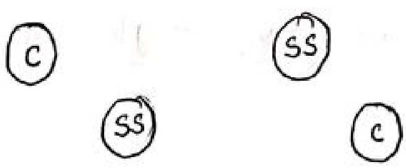
3) Low Alkalinity High Turbidity



Coagulant reaction (↓)

4) Low Alkalinity Low Turbidity

Coagulant reaction (↓)



- worst condition

Filtration

- Filtration is a Physical Process in which the suspended & colloidal impurities are removed from water by passing it through a porous medium.
- Filtration is employed to remove Turbidity, colour, precipitate iron & magnese from Aerated water, Precipitated hardness from Chemically soften water etc.
- Filtration is conducted in Filter which are broadly classified into Gravity & Pressure Filter.

Gravity Filter

- In this Filters, the driving Force to overcome the frictional resistance encountered by the flowing water is the Gravitational Force.
- Eg. Slow Sand Filter, Rapid sand Filter, Dual Media & Multimedia Filter, Double Filter etc.

Pressure Filter

- In this Filter, the driving Force to overcome the frictional resistance is the externally applied pressure. It is not used for commercial purposes.
- For commercial or mass Filteration, Gravity Filters are used.

The most commonly used Gravity Filters are Slow Sand & Rapid Sand Filters. Both these Filters are down flow, single medium Granular Gravity Filter.

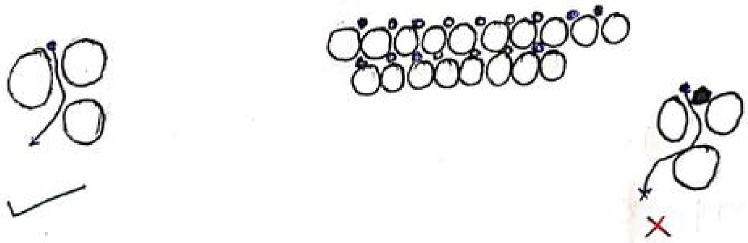
Sand is generally used as medium because it is widely available, cheap & effective in removing impurities. However sand is being replaced now a days by other materials such as crushed coconut shell, activated Carbon, Garnet, Geosynthetic material like HDPE etc.

NOTE : Among Slow Sand & Rapid Sand Filters, Rapid Sand Filters are most commonly used in India.

* Theory of Filteration

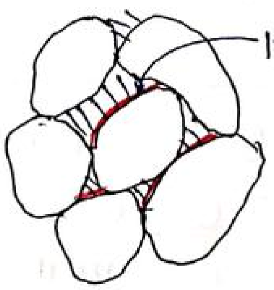
1> Mechanical Straining

- The suspended particles which are bigger than the size of the void get arrested in these voids.
- The water passing these voids becomes free from such particles
- Over a period of time, a filter is able to remove the impurities of sizes even lesser than the size of voids of the filter. This is called as Mechanical Straining.



2> Sedimentation

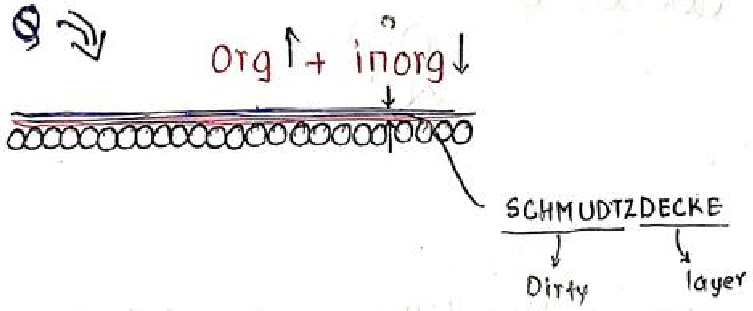
- The small voids between the filter media act as tiny sedimentation units & very small impurites get settled over the medium surface.
- Impurities get deposited over the surface & the whole media needs to be replaced after certain Time.

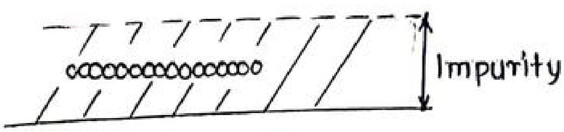
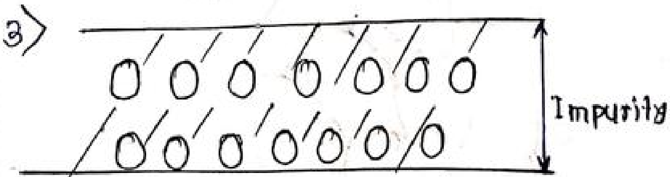


3> Biological Metabolism

- The micro-organisms requires organic impurites as food for their survival.
- Most of the impurities that enter the filter are of organic nature. This organism convert the organic matter into decomposed form by the process of Biological Metabolism.
- As the Time progresses a layer is formed at the top of the medium consisting of decompose organic matter & large quantity of micro organism. This layer is called as SCHMUDTZDECKE or mud layer.

NOTE : SCHMUDTZDECKE is predominant feature observed in a slow Sand Filter.



Slow Sand Filter	Rapid Sand Filter
1> Q ↓	1> Q ↑
2>  less size of media.	2>  larger size of media.
3> 	3> 
4> Surface Cleaning	4> Full depth is cleaned.
5> Frequency of cleaning → 1 - 3 months	5> Frequency ≈ 24 hr.
6> η ↑	6> η ↓

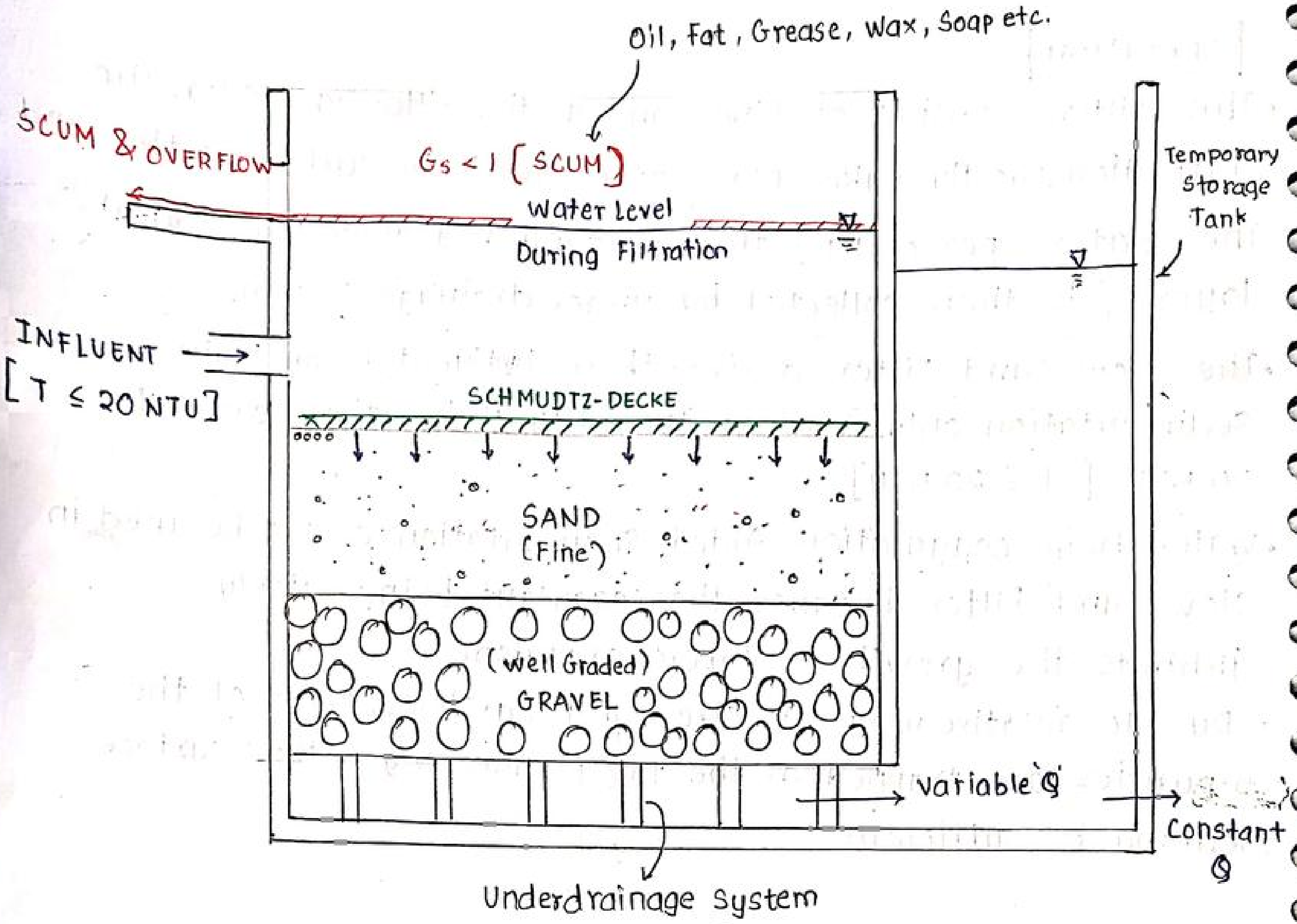
SLOW SAND FILTER

Operation

- This Filters consists of Fine sand as the filtering media. The Filtration mostly takes place at or near the surface of the sand
- The water percolating through sand bed enters the Gravel layer & is then collected by under drainage system.
- ~~The~~ Slow sand Filter derive their influent from Plain Sedimentation only & when Turbidity is less than equal to 20 NTU [$T \leq 20$ NTU].
- Water from coagulation Aided Sedimentation cannot be used in Slow Sand Filter because the coagulant being Stickly inhibits the growth of micro-organisms.
- Due to relatively small size sand particles, most of the impurities are trapped at the top portion only & thus surface cleaning is sufficient

Cleaning

- During Cleaning, schmudtzdecke along with Top 1.5 to 3 cm of sand layer is removed & remaining surface is manually checked for leftover impurities.
- SCHMUDTZDECKE redevelops in 2 to 3 days & Filters is said to be ready for operation.
- Always 1 unit is ~~kept~~ to be kept as Stand by which ~~is~~ ~~to~~ will be used when the other Filter is getting cleaned.



- SLOW SAND FILTER

Design Data

1) Designed for maximum Daily Demand

2) Rate of Filtration,

$$\boxed{F_r = 100 - 200\ l/h/m^2}$$

$$\boxed{F_r = \frac{\text{Discharge passing through the Filter}}{\text{Surface Area of the Filter}}}$$

3) Depth of the Filter Tank = 2.5 – 4.5 m

4) Sand Depth ≃ 1m

5> Sand Characteristics

$D_{10} = 0.2 - 0.3\,mm$

$C_u = 3-5$

$C_c = 1-3$

6> Always one unit is to be kept as Standy.

7> No. of units required depends upon to Total Surface Area required.

Surface Area Required	working	~~sand~~ Stand by	Total
less than $20\,m^2$	1	1	2
$20 - 250\,m^2$	2	1	3
$250 - 650\,m^2$	3	1	4
$650 - 1200\,m^2$	4	1	5
$> 1200\,m^2$	5	1	6

8> $\frac{L}{B} = 1$ to 4

9> Freeboard = 0.3 m

Pg. No. 103 (WB)

Q. 54>

Population = 40000

$q = 150\ l/c/d$

Design Demand $= 150\ l/c/d \times 40000\ c \times 1.8$

$= 10.8 \times 10^6\ l/d$

Assume $F_r = 150\ l/h/m^2$

$$150\ l/h/m^2 = \frac{10.8 \times 10^6\ l/d}{\text{Total S.A required}} \times \frac{1}{24\ h/d}$$

Total S.A required $= 3000\ m^2$

Surface Area of each unit $= \frac{3000}{5} = 600\ m^2$

Provide 5 working + 1 Stand by of $600\ m^2$ each

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

$L = 4B$

$4B^2 = 600\ m^2$

$B = 12.25m$

$L = 4B = 49m$

Assume, $H = 3m$ { in which sand depth ≈ 1m & Free Board = 0.3m }

RAPID SAND FILTER

- Their are 2 aspects of operation of a Rapid Sand Filter:- Filteration & Back washing

Filteration

- During Filtration valve No. 1 & 4 are opened from where settled water is fed into the Filter through valve No. 1 & Filtered water is collected through Valve No. 4.
- The size of the medium particles used in a Rapid Sand Filter is bigger than that of Slow Sand Filter. Thus impurities are able to penetrate upto the bottom most layers

of medium.

- Hence Surface Cleaning is not sufficient & is accompained by Back washing.

Back Washing

- During Back washing, valve No. 1 & 4 are closed and valve No. 2, 5 & 6 are opened as a result of which, compressed air & Back wash water are forced into the medium resulting in increase porosity of the medium.
- As the porosity increases, the entrapped impurities get removed in the back wash water.
- Once this process is complete, Valve No. 2, 5 & 6 are closed & the filter is again loaded with settled water. However, for a small duration valve No. 4 is kept close & Filter water is disposed into the sewer through valve No. 3.
- This is done for the readjustment of sand partlole under the Hydraulic Head of water.
- The entire Process of Back washing is complete ~~around~~ in around 30 min.
- SCHMUDTZDECKE does not get formed as the cleaning is done quite Frequently.

Design Data

1) It is design for maximum daily demand + Back water Demand.

2) The quantity of water required for Backwashing is around 2-5% of the water passed through the filter each day.

3) Frequency of Back washing is preferrably 24 Hrs. & in no case it should ~~not~~ exceed 48 Hrs.

4) The Rate of Filtration (F_r) is between 3000 to 6000 $l/h/m^2$

NOTE :

$$F_{r\,RSF} = 30 \times F_{r\,SSF}$$

5) The Time required for Back washing is around 30 min.

6) Area of each unit is 10 - 80 m^2

7) Atleast 1 unit is to kept as stand by unit.

8) $\frac{L}{B} = 1$ to 4.

9) The discharge during back washing is 6 to 16 times the discharge during Filteration.

10) The velocity of water during Back washing (upflow velocity) is

$$V_B = 15 - 90 \text{ cm/min}$$

NOTE : If velocity exceeds this limit sand gets lost from the system. If V_B is very less, the porosity of sand bed doesnot get increased.

11) The Characteristic ~~xxxx xxx~~ of flow during filtration is laminar & that during Back washing is transition

NOTE : Excess Turbulence in the system, causes displacement of Gravel Particles

12) Sand Characteristics

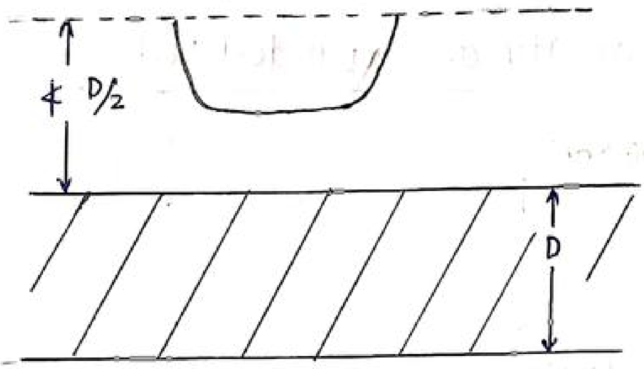
$D_{10} = 0.35$ to 0.55 mm

$C_u = 1.2$ to 1.6

$C_c = 1$ to 3

NOTE : Comparativly, the sand of ~~SSF~~ RSF is poorly graded than the sand of SSF. This ensures higher discharge passing capacity & the impurities penetrate throughout the depth of Filter.

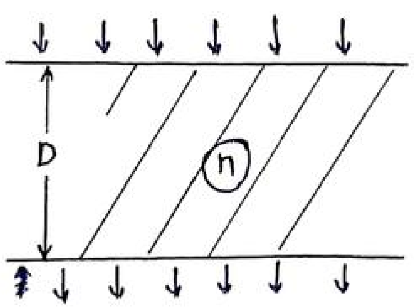
13) The Top of the wash water trough should be kept atleast at the Distance of (D/2) from the top of Normal sand surface.



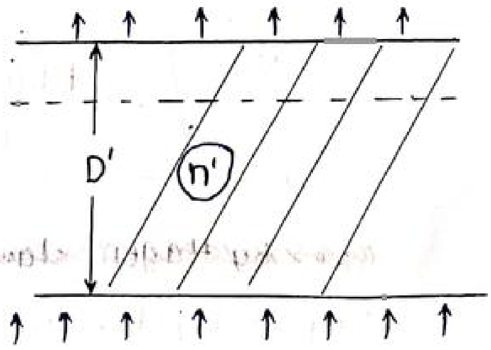
Expansion of Filter Media During Backwashing

- During Back washing, the depth of the filter medium increases thereby increasing the porosity of the medium.
- If the impurities are neglected, the loss in head during filteration is equal to loss in head during Backwashing. This is due to equal surface area of sand particles in both the cases.

Filtration



Back washing



If impurities are not considered :

$$h_{L\,filtration} = h_{L\,backwashing}$$

$$h_{L\,\text{filtration}} = (1-n)(G_s-1)\,n$$

$$h_{L\,\text{backwashing}} = (1-n')(G_s-1)\,n'$$

$$\therefore \quad (1-n)(G_s-1)D = (1-n')(G_s-1)\,n'$$

$$\boxed{(1-n)D = (1-n')D'}$$

G.O.I Manual Formula for porosity of expanded Bed

$$\boxed{n' = \left[\frac{V_B}{V_s}\right]^{0.22}}$$

$V_B \rightarrow$ Back washing / upflow velocity

$V_s \rightarrow$ Settling velocity of sand particles

$n' \propto V_B$

$n' \propto \dfrac{1}{V_s}$

$n' \propto \dfrac{1}{G_s}$

$n' \propto \dfrac{1}{d}$

$$\boxed{V_s = \sqrt{\frac{4}{3}gd\,\frac{(G_s-1)}{C_D}}}$$

$C_D \rightarrow$ By Hazen's law

Pg. No. 104 (WB)

Q.55> Population = 275000

$q = 200$ lit/c/d

$F_r = 15\ m^3/m^2/h$

Backwash period = 30 min

Backwash Freq. = 24 hr.

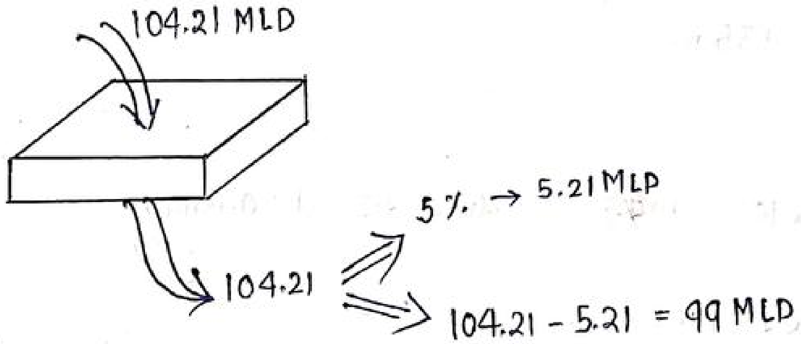
Surface Area of each Filter = $40\ m^2$

Maximum Daily Demand = $1.8 \times 2,75,000 \times 200\ l/d$

$= 99 \times 10^6\ l/d$

$= 99$ MLD

Design Discharge Through the Filter = $\frac{99}{0.95} = 104.21$ MLD



$F_r = 15\ m^3/m^2/h = \frac{104.21 \times 10^6\ l/d}{\text{Surface area required}}$

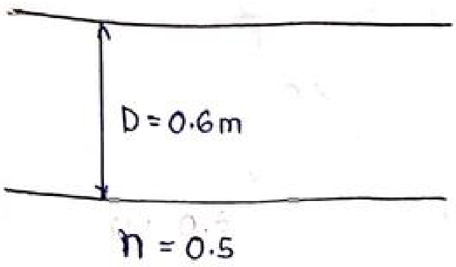
Surface area required $= \frac{104.21 \times 10^6}{15 \times 10^3\ l/m^2/h \times 23.5\ h/d}$

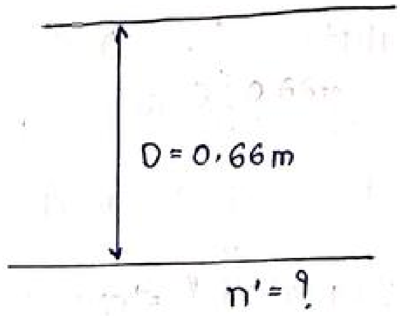
$= 295.63\ m^2$

No. of Filters required $= \frac{295.63\ m^2}{40\ m^2}$

$= 7.39 \approx 8$ units + 1 Stand by

∴ Total 9 units





$h_{L\,Filt} = h_{L\,Backwashing}$ [if impurities are neglected]

$$(1-n)D = (1-n')D'$$

$$(1-0.5)\times 0.6 = (1-n')\,0.66$$

$$n' = 0.545$$

$$h_{L\,Back} = (G_s-1)(1-n')D'$$
$$= (2.5-1)(1-0.545)\,0.66$$
$$= 0.455\text{ m}$$

$C_D = 5.02$

$\nu = 1.0316\times10^{-6}\ m^2/s$, $G_s = 2.5$, $d = 0.6\text{ mm}$

$$n' = \left[\frac{V_B}{V_S}\right]^{0.22}$$

$$V_B = V_S \times n'^{1/0.22}$$
$$= 18.4\ cm/min$$

$$V_S = \sqrt{\frac{4}{3}gd\frac{(G_s-1)}{C_D}}$$
$$= \sqrt{\frac{4}{3}\times 9.81\times 0.6\times10^{-1}\frac{(2.5-1)}{5.02}}$$
$$= 0.0484\ m/s$$
$$= 290.4\ cm/min$$

* Operational Troubles in RSF

1> Bumping of Filter Media

It is due to careless operation & sudden discharge of the Back wash water. The Intensity of Back wash water must be Gradually increase to its maximum value.

2> Sand Boils

These are caused when disproportionate discharges of Back wash water enters the sand layer from Gravel layer. This is due to poor distribution of wash water which decreases the efficiency of Back washing.

3> Defective Gauges

Their are 2 types of Gauges :- Flow Gauges & Pressure Gauges. These frequently go out of service & Hence necessary repairs are to be made at regular intervals.

4> Air Binding

The initial loss of Head through a Freshly cleaned Filter is in the range of 15 to 30 cm. which goes on increasing as more and more impurities are trapped in the voids of the medium. A Stage comes during operation of the Filter when Head loss experience by water is so high that it start releasing the dissolved gases. This is due to excess Head loss which significantly reduces the Pressure Head. Bubbles of these gases rise & a large quantity gets trapped in the system. This phenomenon is called Air Binding.

It reduces the volume available ~~from~~ for discharge &

entrapment of impurities. To reduce Air binding back washing must be done more frequently & more efficiently.

5) Inadequate depth of medium

It is the loss of sand during back washing eventually leading to its reduced quantity. Sand depth is never allowed to be depleted by more than 10 cm.

6) Incrustation over medium surface

This problem arises when sand gets coated with material which is difficult to remove during regular Back washing. Its remedial measure is to add small quantity of NaOH into the Backwash water. If the incrustation still exist, the media needs to be changed completely.

7) Cracking of the Filter

This is due to excess turbulence caused over the medium surface causing displacement of sand particles.
This reduces effective depth of Filter.

8) Mud Ball Formation

Mud from the atmosphere enters the filter & such material is difficult to remove during Back washing. It gets coated over the sand & Gravel particles thereby forming mud balls.
Its remedial measure is to add a small quantity of NaOH in Back wash water along with discharge of compressed Air.

9> Slime Growth over Filter

Slime is Gooey viscous mass such as oil, fat etc. which may get coated over the filter medium. This reduces the efficiency of entrapped impurities. A small quantity of NaCl is added in Backwash water to remove such material.

DOUBLE FILTER

- In this system, R.S.F is used before S.S.F which provides a sufficient Discharge ~~disch~~ passing capacity Capacity as well as High effeciency.
- The Slow sand Filter does not get choked as Rapid Sand Filter removes all the impurities.

NOTE : The Rapid Sand Filter used in this System is also called as Roughning Filter.

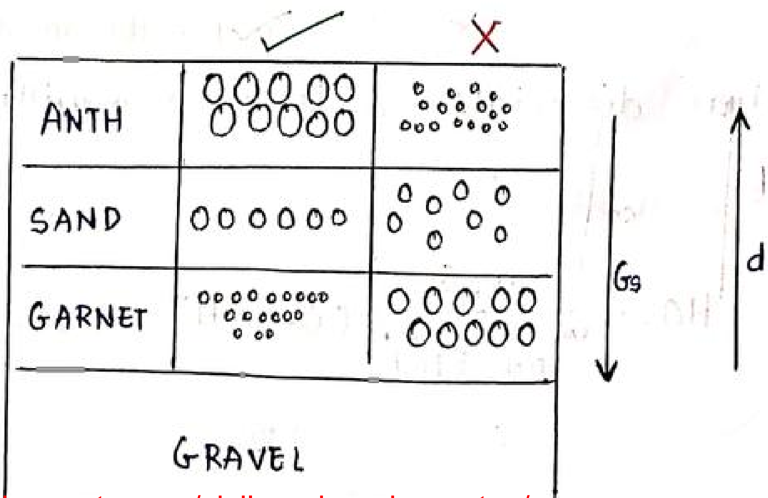
DUAL MEDIA & MULTI MEDIA FILTER

When 2 or more than 2 medium particles are used in the filter it is referred as dual media or Multi media Filter resp.

① To effectively utilize medium particles, the coarser material is kept above the Finer material.

② To avoid the ~~causing~~ crushing of bottom most ~~material lighter~~ ~~materi~~ ~~to~~ material, lighter material is placed over heavier material

Sand → $G_s = 2.65$
Anthracite → $G_s = 1.2$
Garnet → $G_s = 4.2$



$$\boxed{fr_{MMF} \simeq fr_{DMF} > fr_{RSF} > fr_{rSSF}}$$

$$fr_{MMF} \simeq fr_{DMF} = 8000 - 10000 \ l/h/m^2$$

* Chlorination

// IMP for GATE //

- Chlorine is commercially adopted because :-

1> It is easy to manufactured in various form such as solid, liquid & Gas out of which liquid form is mostly used.

2> Chlorine directly reacts with water & thus it doesnot change chemical characteristic of water.

3> Its storage is convinent b/w 10 - 92°C

- Chlorine when added in water reacts immediately as follows

$$Cl_2 + H_2O \xrightarrow{pH > 5} \underset{\substack{\text{Hypochlorous} \\ \text{Acid} \\ \text{(weak acid)}}}{HOCl} + \underset{\text{(Strong acid)}}{HCl}$$

- The Strong acid HCl produce & then dissociates as follows

$$HCl \longrightarrow H^+ + \underset{\text{(Chloride ion)}}{Cl^-}$$

(not responsible for disinfection)

- HOCl then dissociates & it is in equillibrium with OCl^- as follows

$$HOCl \underset{pH = 5 \text{ to } 7}{\overset{pH > 8}{\rightleftharpoons}} OCl^- + H^+$$

- Cl_2, $HOCl$ & OCl^-, all these forms are capable of Carrying out Disinfection & are called as free forms of Chlorine or freely available Chlorine.

CASE I : pH ≤ 5 → only Cl_2 is present

CASE II : pH = 5 to 7 → $HOCl$ ↑↑↑↑
OCl^- ↓↓↓↓

CASE III : pH > 8 → OCl^- ↑↑↑↑
$HOCl$ ↓↓↓↓

CASE IV : pH = 7 to 8 → OCl^- & $HOCl$, both are appreciable in quantities

- Drinking water has a pH range of 6.5 to 8.5 & thus, HOCl is present in predominance in drinking water (between 6.5 to 7)
- Out of HOCl & OCl^-, HOCl is more distructive to microorganism & thus pH of water is maintained between 6.5 to 7 during Disinfection.
- HOCl is 80 times more reactive than OCl^- with microorganism

$$\boxed{K_{HOCl} \approx 80 \times K_{OCl^-}}$$

Reaction of Chlorine with Ammonia

- HOCl reacts with microorganism instantly & Hence it is considered as powerful disinfectant. However, wheh left as residue to safeguard against future contamination both HOCl & OCl^- produce high bitter Taste.
- As per G.O.I manual, Chlorine residual in free form should not exceed 0.2 mg/l.
- However this residual is insufficient to safeguard against

future contamination. Thus Chlorine is allowed to react with ammonia which form Chloroamines as follows.

$$NH_3 + Cl_2 \underset{}{\overset{pH > 7.5}{\rightleftharpoons}} NH_2Cl + HCl$$

$$NH_2Cl + Cl_2 \underset{}{\overset{pH = 5 \text{ to } 7}{\rightleftharpoons}} \underset{\text{Dichloramine}}{NHCl_2} + HCl$$

$$NHCl_2 + Cl_2 \underset{}{\overset{pH < 4.4}{\rightleftharpoons}} NCl_3 + HCl$$

This Chloroamines are 25 times less reactive than HOCl with microorganism. However they do not cause any bad taste even if left in high concentration as residual.

This Chloroamines provide protection against future contaminatⁿ.

This Chloroamine are called as combined forms of Chlorine

$$\boxed{K_{HOCl} \simeq 25 \times K_{Chloroamines}}$$

CHICK'S LAW

This disinfection with Chlorine is observed to follow the following law :-

$$\boxed{N_t = N_o e^{-kt}}$$

$N_o \rightarrow$ Initial Qty. of microorganism (at $t = 0$)

$N_t \rightarrow$ No. of microorganism remaining after time 't'

$k \rightarrow$ reaction rate constant

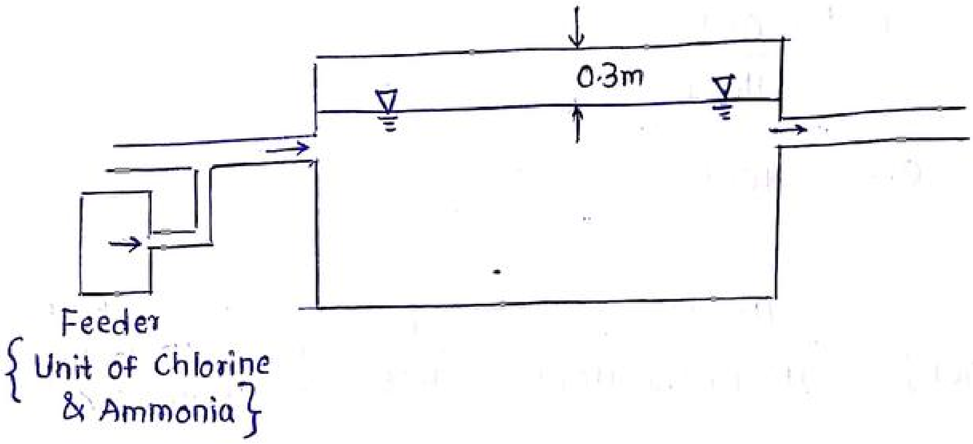
For a required degree of killing of microorganism, find 't'.

Find Volume of Tank, $V = Qt$

$$\frac{L}{B} = 2 \text{ to } 4$$

H = 1.5 to 5 m

Freeboard = 0.3m



CHICK - WATSON MODEL

Efficiency of disinfection depends on :-

1> Concentration of disinfectant (c)

2> Time of contact (t)

$$\eta_d = f(c, t)$$

For two systems with equal degree of disinfection :-

$$\boxed{t_1 C_1^{n_1} = t_2 C_2^{n_2}}$$

n_1 & n_2 → Solubility / Dilution coefficients

19/11/19

Pg. No. 100

Q.27>

$$HOCl \rightleftharpoons H^+ + OCl^-$$

$$k = 2.5 \times 10^{-8} \text{ mol/l.}$$

$$k = \frac{[H^+]^1 [OCl^-]^1}{[HOCl]^1} = 2.5 \times 10^{-8}$$

$[H^+] = 10^{-7}$ mol/l

$$2.5 \times 10^{-8} = 10^{-7} \times \frac{OCl^-}{HOCl^*}$$

$$OCl^- = 0.25 \times HOCl^* .$$

$$\frac{[HOCl]}{[HOCl] + [OCl^-]} = \frac{HOCl}{HOCl + 0.25 HOCl} = \frac{1}{1.25} = \underline{\underline{0.8}}$$

Pg. No. 100 (WB)

Q. 30>

$$HOCl \rightleftharpoons H^+ + OCl^-$$

$$K = 2.7 \times 10^{-8} \text{ mol/l} = \frac{[H^+][OCl^-]}{[HOCl]}$$

$$[OCl^-] + [HOCl] = x$$

$$0.1x \quad + \quad 0.9x$$

$$= [H^+] \times \frac{0.1}{0.9}$$

$$[H^+] = 2.43 \times 10^{-7}$$

$$pH = -\log_{10}(2.43 \times 10^{-7})$$

$$= 6.6$$

Pg. No. 101 (WB)

Q. 33>

$$OCl^- + H^+ \xrightleftharpoons{k} HOCl$$

$$K = 10^{7.5} = \frac{[HOCl]}{[H^+][OCl^-]}$$

$pH = 7.5$

$H^+ = 10^{-7.5}$ mol/l

$[HOCl] = [OCl^-]$

$[HOCl] + [OCl^-] = 2$ mg/l (as Cl_2)

$[OCl^-] = 1$ mg/l as Cl_2

$= \frac{1}{71}$ mg.eq/l $= \left[\frac{1}{71 g} \times 51.5 g\right]$ mg/l

$= 0.725$ mg/l (as OCl^- itself)

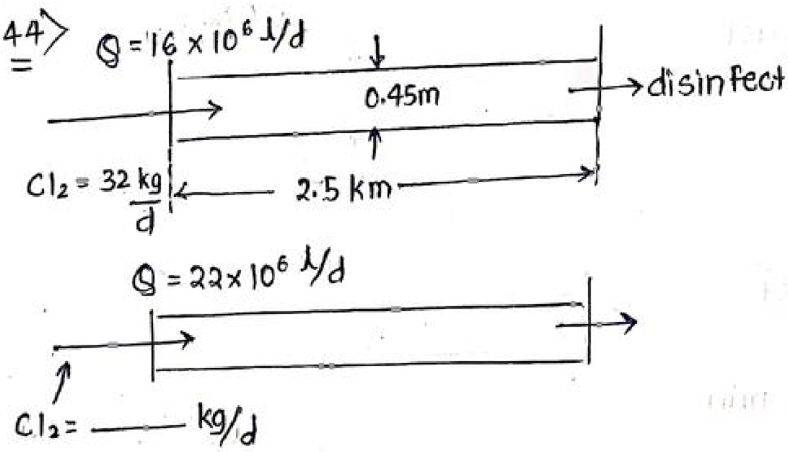
$= 0.725 \times 10^{-3}$ g/l

$= \frac{0.725 \times 10^{-3}}{51.5}$ mol/l

$= 1.408 \times 10^{-5}$ mol/l.

Pg. No. 102 (WB)

Q.44)



$C_1 = \frac{32 \text{ kg} \times 10^6 \text{ mg/kg/d}}{16 \times 10^6 \text{ l/d}}$

$= 2$ mg/l

$t_1 C_1^{n_1} = t_2 C_2^{n_2}$

$t_1 C_1 = t_2 C_2$

$$\frac{L}{Q_1/A} \times C_1 = \frac{L}{Q_2/A} \times C_2$$

$$\frac{C_1}{Q_1} = \frac{C_2}{Q_2}$$

$$\frac{2\ mg/l}{16\ MLD} = \frac{C_2}{22\ MLD}$$

$$C_2 = 2.75\ mg/l$$

$$\text{Total } Cl_2 \text{ required} = \frac{2.75\ mg/l \times 22 \times 10^6\ l/d}{10^6\ mg/kg}$$

$$= 60.5\ kg/day$$

Pg. No. 126 (WB)

Q.40)

$$Q = 2670\ m^3/d$$

$$N_t = N_0\, e^{-0.145t}$$

$$v = ?$$

$$\eta = 98\ \%$$

$$2 = 100\, e^{-0.145t}$$

$$t = 26.97\ min$$

$$V = 2670\ \frac{m^3}{d} \times \frac{26.97\ min}{24 \times 60\ min/d} = 50\ m^3$$

* Types of Chlorination

1> Plain Chlorination

- When Chlorination is the only treatment given to the water, it is called as Plain Chlorination.
- It removes the organic matter, microorganism, taste & odour from water. This can only be done to less turbid water when full fledged treatment cannot be given to water.

Eg. - Supplying water to Army troopes during war.
- Water to mountainer, explorer.

2> Pre-Chlorination

- It is a process of applying Chlorine to water during coagulation itself.
- It is adopted when the concentration of pathogenic organism along with organic matter is huge.
- It is predominantly adopted in these areas where chances of outbreak of epidemic is High. However prechlorination is always followed by Final & Post Chlorination to ensure final safety of water.

3> Post Chlorination

- It is simply called as Chlorination & it is a standard Process of applying Chlorine at the end.
- When both Pre & Post Chlorination are done, they are combinedly called as Double Chlorination.

4> Super Chlorination

- When excess chlorine is added such that it leaves high residual beyond a break point, it is referred as super Chlorination.

• It produces a free residual of 3 to 5 mg/l & thus Subsequent dechlorination is to be done before it's consumption.
• Dechlorination can be done by following Dechlorinating agent.

<1> Activated Carbon
<2> Ammonium Hydroxide
<3> Sodium Thiosulphate ($Na_2S_2O_3$) {Cheapest & most effective}
<4> Sodium Sulphite

* Various forms of Chlorine

1> Molecular form of Chlorine

• It doesnot form sludge while carrying out disinfection.
• It quality doesnot get deteriorated over a period of time due to storage.
• It doesnot change Chemical characteristics of water & Hence it is convinently use for final Chlorination.

2> Bleaching Powder & Calcium Hypochlorites

• These are Predominantly used for Prechlorination
• Bleaching Powder is also popular in laundry industry
• Both these compound change the chemical characteristic of water & Hence they are not used for final Chlorination.
• For 100% Pure Bleaching Powder, percentage freely available Chlorine by wt. is approximately 40%.
• For 100% Pure Calcium Hypochlorite, percentage freely available chlorine is approximately 72%.

$$CaOCl_2 \longrightarrow Ca^{2+} + OCl^- + Cl^-$$

127g   51g

$$\% \ FAC = \frac{51.5}{127} \times 100 = 40\%$$

$$Ca(OCl)_2 \longrightarrow Ca^{2+} + 2OCl^-$$

143 g   103g

$$\% \ FAC = \frac{103}{143} \times 100 = 72\%$$

3) Halazone Tablet

- National Environmental Engineering Institute (Nagpur) has developed special tablets which posses the properties of Chlorine & Ozone.
- They are cheaper & more effective than ordinary Chlorine tablets.

4) Chlorine Dioxide Gas (ClO_2)

- It is the most powerful disinfectant form of Chlorine which is manufactured by heating Chlorine & Oxygen at high temperature.
- It's manufactureing is costly & hence it is not used commercially

$$\boxed{KClO_2 \simeq 2 \times KHOCl}$$

Arrange in increasing order of reactivities.

$HOCl$, $NHCl_2$, $KMnO_4$, ClO_2, OCl^-

$$\boxed{KMnO_4 < OCl^- < NHCl_2 < HOCl < ClO_2}$$

* Demand & Dosage of Chlorine

Chlorine is required for the following reaction in the following sequence

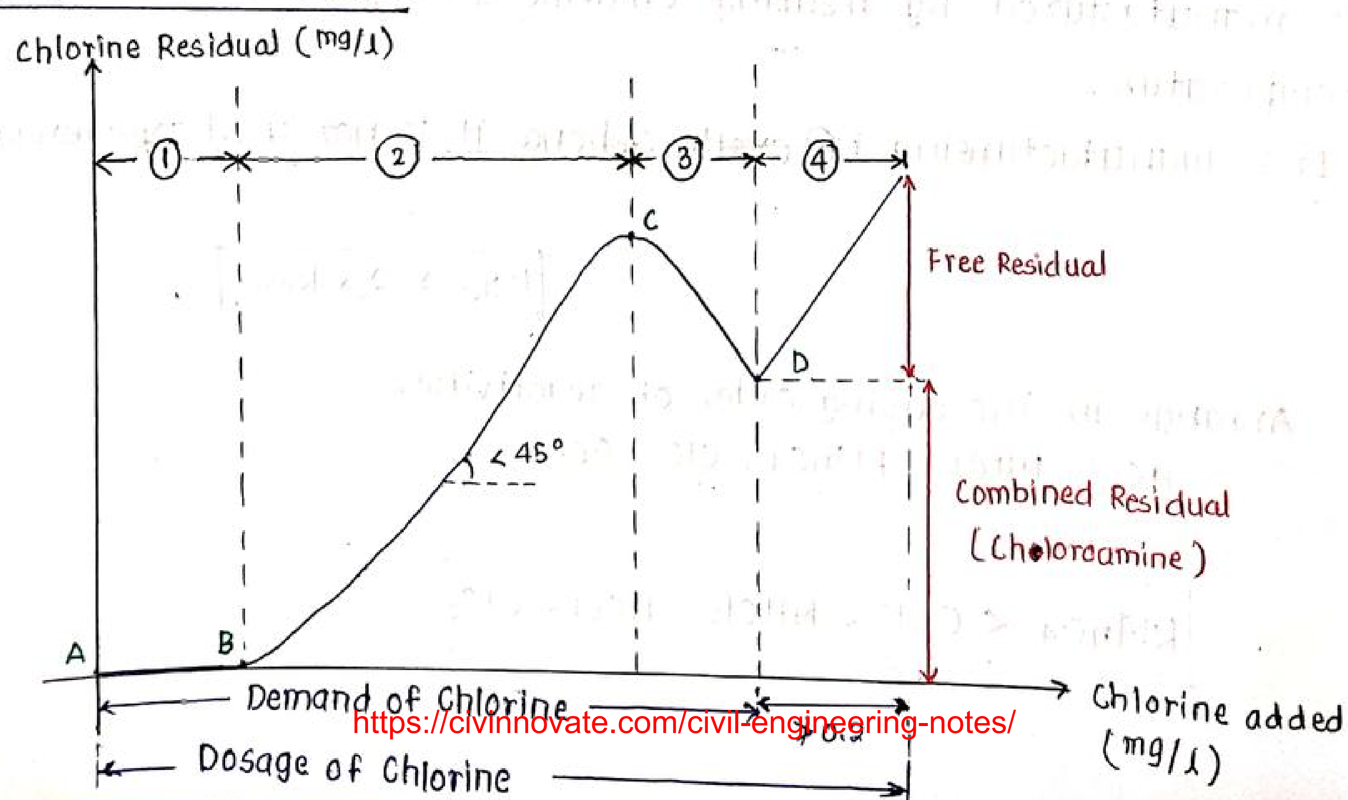
<1> Killing of M/O + Reducing compounds
(HOCl & OCl^-) (Metallic ions) Fe^{2+} / Fe^{3+} / Mn^{2+} / Mn^{3+} / Mn^{5+} → $FeCl_2$ → $FeCl_3$

<2> Killing of M/O + Formation of Chloro-amines & Chloro-organics
(HOCl + OCl^-)

<3> Killing of M/O + Destruction of chloro organics
(HOCl + OCl^-)

- Chlorine consumed in all the above 3 reactions represents the chlorine demand of water. Once this demand gets satisfied, any further chlorine added simply appears as free residual.
- As per GOI manual the free residual should not exceed 0.2 mg/l after a contact period of 10 mins.

Chlorination Curve.



- Chlorination Curve is a curve b/w the amount of Chlorine added & the residual obtain.
- The Residual chlorine can be found out by various tests as follows.

1) Diethyl Paraphenylene Diamine (DPD Test)

- It is also called as Palin's reagent.
- It is most commonly used as it is cheap & can measure free as well as combine residual [Chloroamines & Chloro organics]

2) Starch - iodide Test

3) Chlorotex test

- Both Starch iodide & Chlorotex Test can measure free residual only.

4) Orthotolidine Test

- This test is not used for Chlorine residual measurement because orthotolidine with chlorine forms carcinogens (cancer causing agent)

→ Between 'A' to 'B' their is no residual observed as Chlorine is been consumed by reducing agent.

→ At pt. 'C', bad smell starts coming out due to start of destruction of organic matter. The destroyed chloro-organics leave the system in the form of gases.

→ At pt. 'D', Bad smell stops coming out indicating complete oxidation of organic matter.

→ After pt. 'D', any futher chlorine added simply appears as free chlorine residual.

→ Pt. 'D' is called as break pt. & Process of obtaining this pt. is called as break pt. Chlorination.

→ Break pt. Chlorination is defined as demand which is required to satisfy the requirement of chlorine.

→ Theoretically no ~~cho~~ chlorine is to be added beyond break point but practically a small quantity of chlorine is further added to leave a free residual of not greater than 0.2 mg/l.

$$\boxed{\underset{(mg/l)}{\text{Dosage of Chlorine}} = \underset{(mg/l)}{\text{Demand of Chlorine}} + \underset{(mg/l)}{\text{Free Residual}}}$$

* Minor Methods of Treatment

1) Water Softening

Water softening is defined as reduction or removal of hardness from water. Following methods can be used for water softening.

① Lime Soda Method

It is the most commonly adopted method in water treatment method in which Lime & Soda reacts as follows :

Lime reacts with all the CH present in water & converts into ppt

$$\underset{\text{dissolved}}{Ca(HCO_3)_2} + Ca(OH)_2 \longrightarrow \underset{\text{ppt.}}{2CaCO_3\downarrow} + 2H_2O$$

$$MgCO_3 + Ca(OH)_2 \longrightarrow CaCO_3\downarrow + Mg(OH)_2(\downarrow)$$

$$Mg(HCO_3)_2 + Ca(OH)_2 \longrightarrow Mg(OH)_2\downarrow + Ca(HCO_3)_2^{*}$$

$$Ca(HCO_3)_2^{*} + Ca(OH)_2 \longrightarrow 2CaCO_3\downarrow + 2H_2O$$

$$\underset{\text{(Alkaline)}}{Ca^{2+}CO_3^{2-}} + Ca(OH)_2 + CO_2 \longrightarrow 2CaCO_3\downarrow + H_2O$$

(ii) Lime also reacts with NCH of other MMC & converts them into NCH of Ca.

$$\underbrace{Mg^{2+}\left\{\begin{matrix} Cl^- \\ SO_4^{2-} \\ NO_3^- \text{ etc.}\end{matrix}\right.}_{\text{NCH of Mg}} + Ca(OH)_2 \longrightarrow Mg(OH)_2\downarrow + \underbrace{Ca^{2+}\left\{\begin{matrix} Cl^- \\ SO_4^{2-} \\ NO_3^- \text{ etc.}\end{matrix}\right.}_{\text{NCH of Ca.}}$$

After reaction of lime is complete, NCH of Ca remains.

Reaction of Soda

$$Ca^{2+}\left\{\begin{matrix} Cl^- \\ NO_3^- \\ SO_4^{2-} \text{ etc.}\end{matrix}\right. + Na_2CO_3 \longrightarrow CaCO_3\downarrow + Na^+\left\{\begin{matrix} Cl^- \\ SO_4^{2-} \\ NO_3^- \text{ etc.}\end{matrix}\right.$$

Soda reacts with NCH of Ca only

$\boxed{\text{Source of NCH of Ca}}$

- Converted from NCH of other MMCs
- Originally Present

Recarbonation

- The Hardness in the supplied water must be between 75 - 115 mg/l as $CaCO_3$ in the form of carbonate Hardness.
- To ensure this hardness in water the precipitates are converted into a dissolved form by process called as Recarbonation.
- A Recarbonation Tank converts the ppt. from settled water into their dissolved forms by passing CO_2 gas in water

• The reaction of precipitates with CO_2 is as follows :

$$\underset{SS}{CaCO_3 \downarrow} + CO_2 + H_2O \longrightarrow \underbrace{\underset{DS}{Ca(HCO_3)_2}}_{CH}$$

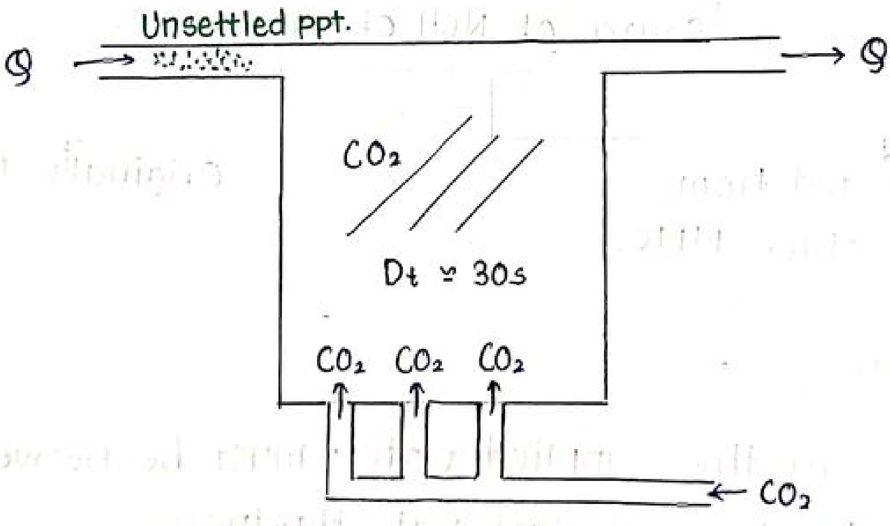
$$\underset{SS}{Mg(OH)_2 \downarrow} + 2CO_2 \longrightarrow \underbrace{\underset{DS}{Mg(HCO_3)_2}}_{CH}$$

• This process ensures sufficient Hardness in water, lesser load of Suspended solids onto the Filter & effective Disinfections in the Disinfection units.

• Lime & Soda are added along with Alum in the coagulation Tank itself because.

(a) It ensures sufficient alkalinity for reaction of Alum.

(b) Lime & Soda gets dispersed uniformly owing to Fast Mixing.



Q.57>

		Lime	Soda	Eq. wt.
$CaCO_3$ alk.	200 ppm	✓	×	50 g
$Mg(HCO_3)_2$	120 ppm	✓	×	73 g
$CaSO_4$	100 ppm	×	✓	68 g
Fe_2O_3	40 ppm	×	×	×
$MgCl_2$	150 ppm	✓	✓	47.5 g
$MgSO_4$	100 ppm	✓	✓	60 g
NaCl	25 ppm	×	×	×
SiO_2	30 ppm	×	×	×

$$\text{Qty. of Lime required/year} = \left[\left(\frac{200\ mg/l}{50g} \times 37g\right) + \left(\frac{120\ mg/l}{73g} \times 37g \times 2\right) + \left(\frac{150\ mg/l}{47.5} \times 37g\right) + \left(\frac{100\ mg/l}{60g} \times 37\right)\right] \times \left[\frac{80000\ l/d \times 365\ d/y}{10^9\ mg/Ton}\right]$$

$$= 13.08\ Ton/y$$

$$\text{Qty. of Soda required/year} = \left[\left(\frac{100\ mg/l}{68} \times 53\right) + \left(\frac{150\ mg/l}{47.5} \times 53\right) + \left(\frac{100\ mg/l}{60} \times 53\right)\right] \times \frac{80000 \times 365}{10^9}\ ton/y$$

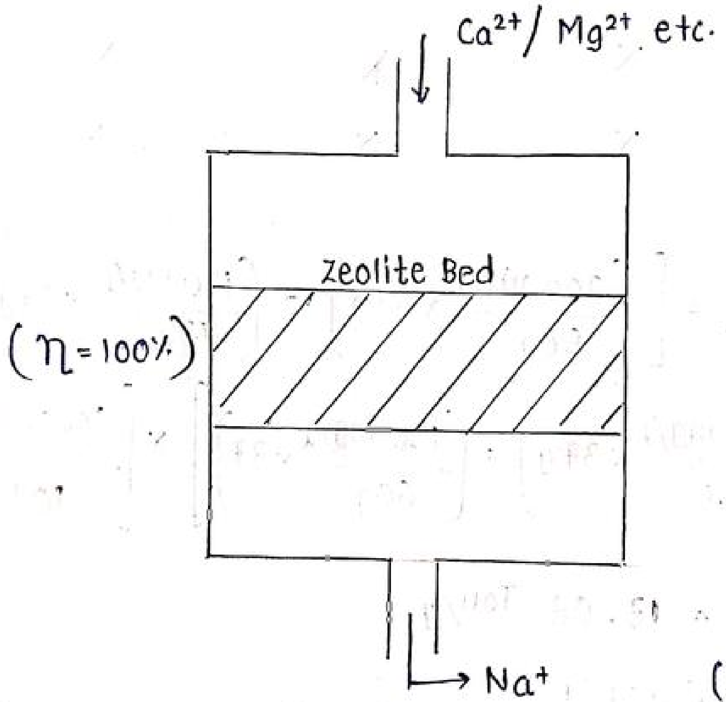
$$= 9.74\ Ton/y$$

(ii) Ion Exchange / Zeolite

- Zeolite is a compound which can exchange its sodium ions with Calcium & Magnesium ion in water.
- It can completely remove Hardness as long as sodium ions are Present in zeolite bed.
- As it's efficiency is 100%, it find its usuage in thermal Power Plants, Boilers etc. It is not used for Drinking water.

Sodium Zeolite → $Na_2O \cdot xAl_2O_3 \cdot ySiO_2 \cdot zH_2O$

Sodium Zeolite → NaZ

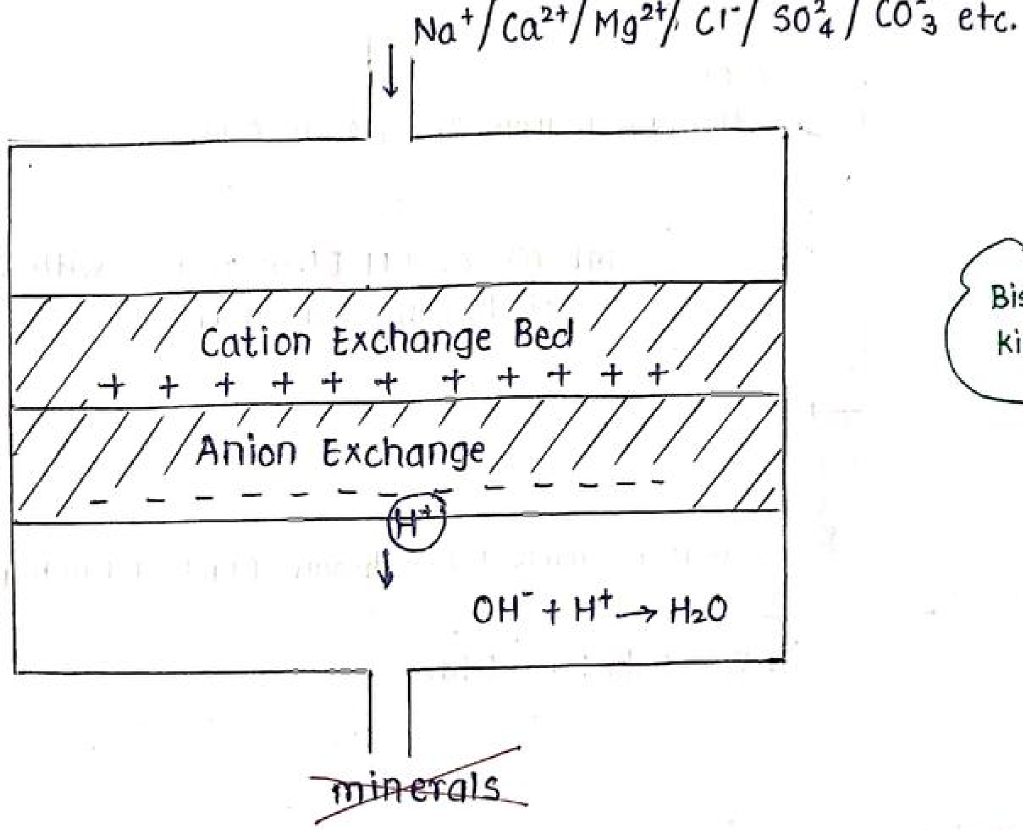


Zeolite
- Green Sand (naturally)
- Permutit (Artifically)

(iii) Demineralization Method

- It is a general process of removal of all the minerals from water. In this method, treatment is carried out in 2 stages.
- In the 1st Stage, a cation exchange bed replaces all the positively charge ions with H^+ Ions & in the 2nd Stage, all the negatively charged Ions are replaced with OH^- ions.
- Thus this method removes all the minerals from water & generates water of zero Hardness.

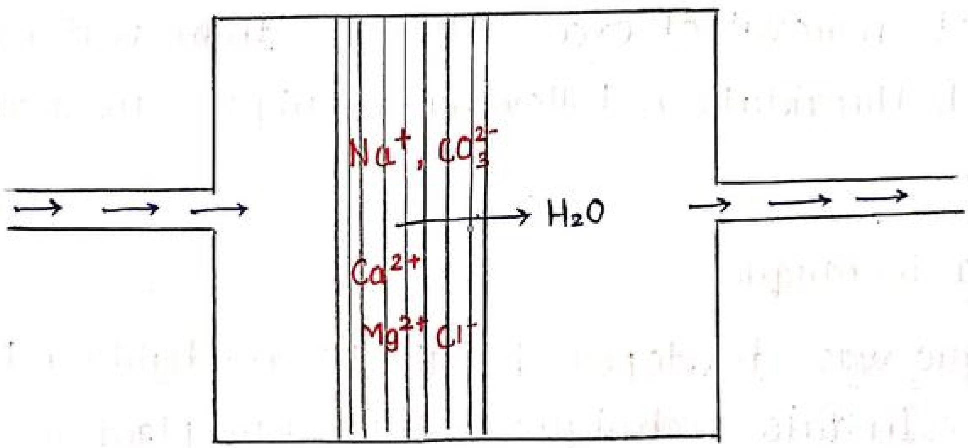
• It is not used for drinking water. but it can be used for Thermal Power Plants, boilers etc.



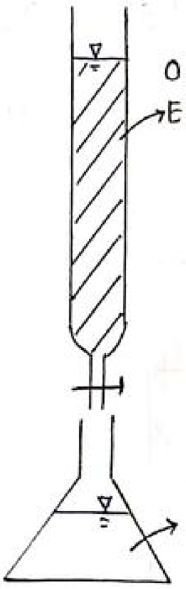
Bisleri, Kinley

(iv) Reverse Osmosis (R.O)

In this method, water is passed through a semipermeable membrane which permits the flow of water molecules only & block the flow of all impurities. This is also a general method to remove hardness from water.



Lab Test for Hardness



0.01 M
Ethylene Diamine Tetra Acetic Acid

$\boxed{\text{1 ml. of 0.01 M EDTA reacts with 1 mg of hardness as } CaCO_3}$

water sample + Eriochrome Black -T (indicator) [EBT]

Wine Red → Blue

2) Fluoridation

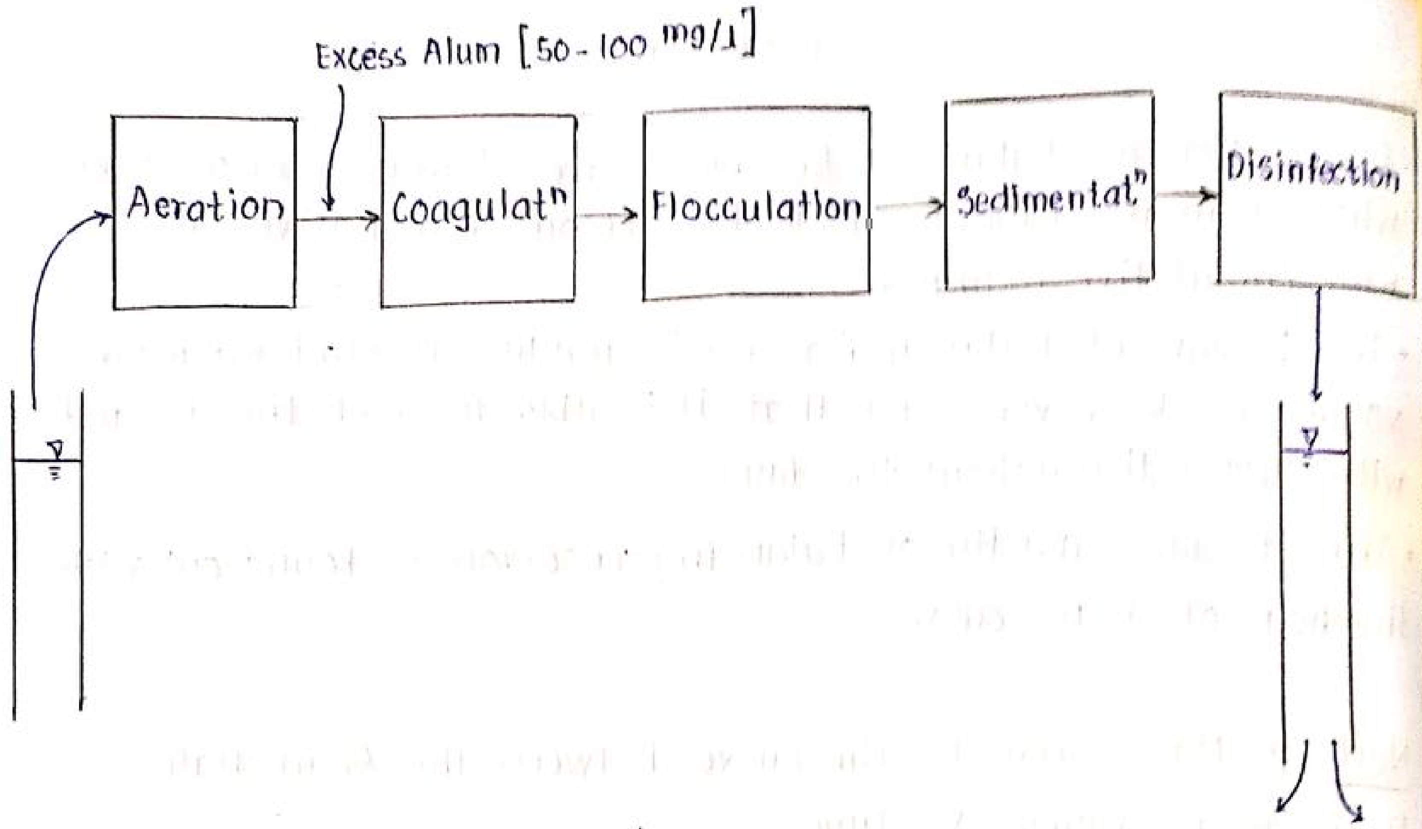
- The addition of external fluoride in water when concentratn is less than 1 mg/l is referred as Fluoridation.
- Fluorides are usually added with the help of Sodium Fluoride (NaF)

3) Defluoridation

- The Process of removal of excess fluorides from water is referred as Defluoridation. Following techniques are used for Defluoridation.

(i) Nalgonda Technique

This technique was developed by NEERI for fighting Nalgonda crisis. In this technique a treatment plant is setup at various location in which Alum is added in excess quantities. (10 to 50 mg/l). The excess Alum forms stable ppt. with Fluorides there by getting removed in Sedimentation.



(ii) Prashanti Technique

In this method, water containing high amount of fluoride is passed through granular beds of Activated alumina, Activated Silica, Activated Carbon, Bone Char etc. all these material have a high surface area & are positively charged in nature. Out all these material Activated Alumina is most effective.

(iii) R.O Process

(iv) Demineralization

4> Treatment with Copper Sulphate

Copper Sulphate kill the algae & also checks its growth even before it is produced. It is usually added to reservoirs & lakes to control the growth of Algae.

NOTE: Algae deteriorate the quality of water & causes Algal Symbiosis.

5. DISTRIBUTION SYSTEM

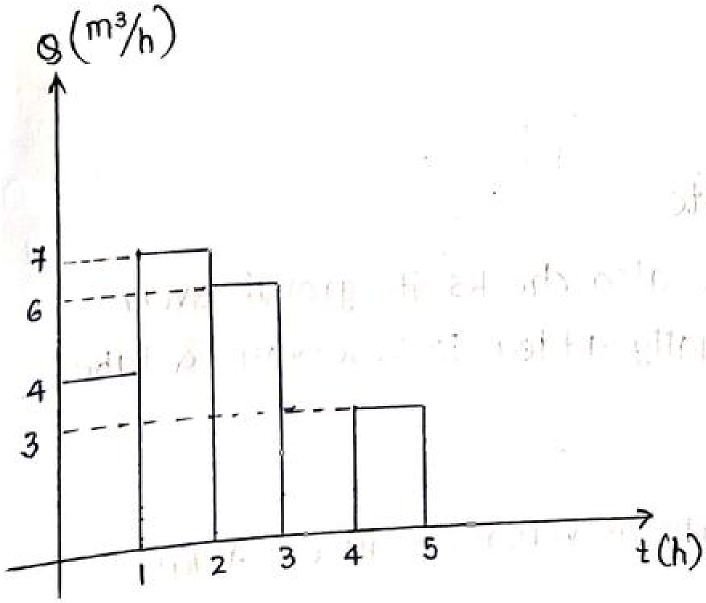
- Distribution or Balancing Reservoirs are storage reservoir ~~wher~~ whose Primary function is to balance out cumulative supply with cumulative demand.
- The Design of balancing reservoir implies to find minimum Volume of Reservoir such that it is able to meet the demand with supply throughout the day.
- The Storage capacity of balancing reservoir is found out with the help of mass curve.

NOTE: (1) Mass Curve is the curve between the Accumulated Discharge i.e Volume vs Time.

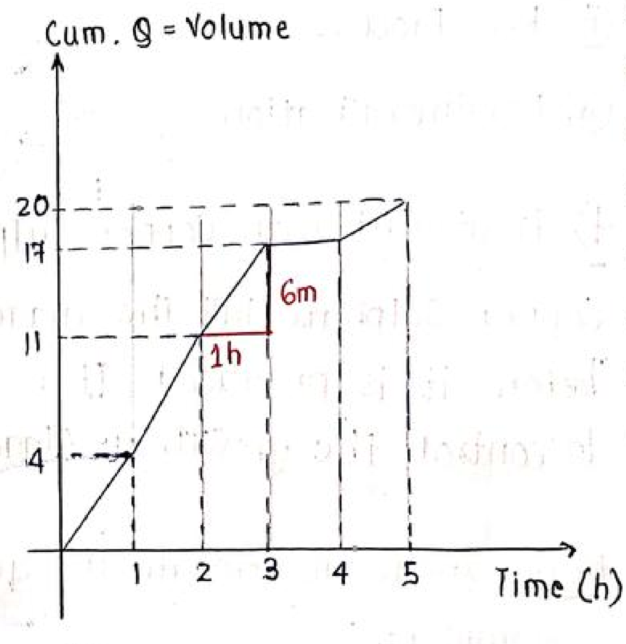
(2) The slope at any pt. on the mass curve represents the intensity at that point.

(3) The Area upto a point in Hyetograph represents the ordinate of mass curve at that point.

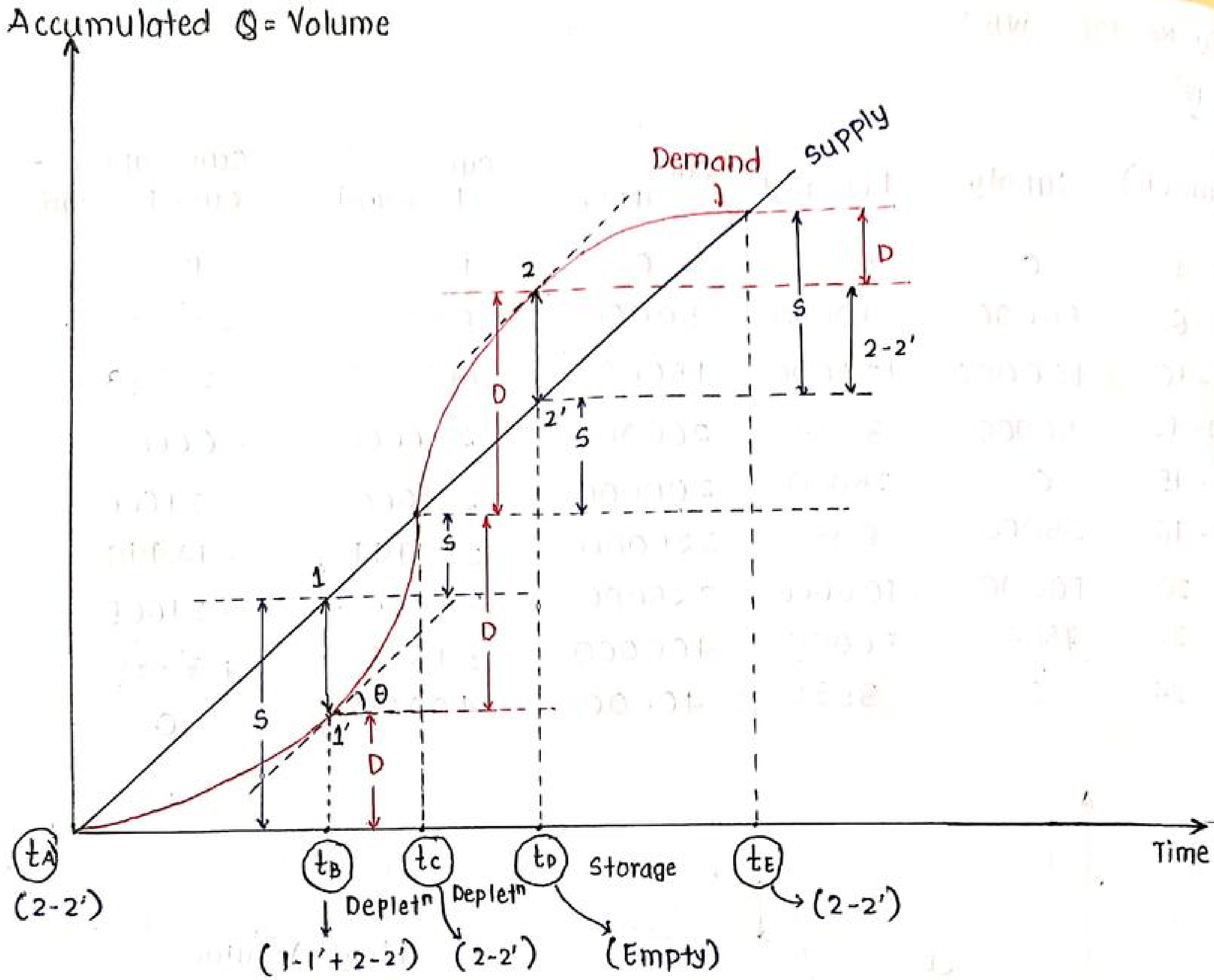
(4) A Mass curve can be increasing, it can be constant but it can never be decreasing



Hyetograph



Mass Curve



Supply Rate → Constant

Demand Rate → Variable

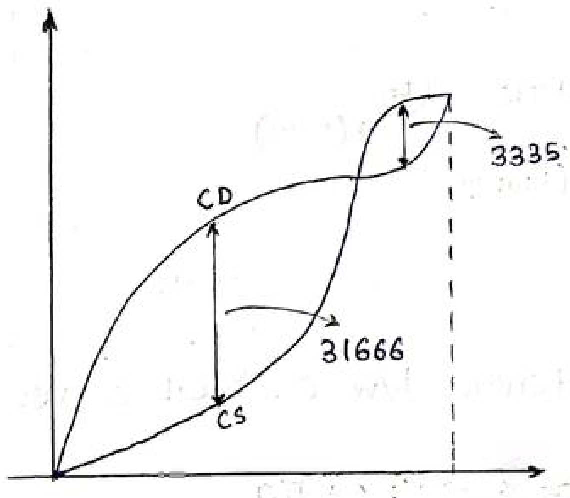
V_{min} = Sum of Max^m ordinate difference b/w C.S & C.D curves

$$\boxed{V_{design} = \underbrace{(1.2 - 1.3)}_{\text{Range}} \times V_{min}}$$

Pg. No. 108 (WB)

Q.17>

Time (h)	Supply	Demand	Cum. supply	Cum. Demand	Cum. Supply - cum Demand
0 - 4	0	13333	0	13333	-13333
4 - 6	50000	40000	50000	53333	-3333
6 - 10	100000	120000	150000	173333	-23333
10 - 12	50000	33333	200000	206666	-6666
12 - 15	0	25000	200000	231666	-31666
15 - 16	25000	8333	225000	239999	-14999
16 - 20	100000	106666	325000	346665	-21665
20 - 23	75000	50000	400000	39665	+3335
23 - 24	0	3335	40000	40000	0



Min.^m Volume

$V_{min} = 31666 + 3335$
$= 35001$ Litres

Water levels	Water Levels + 31666
0 - 13333 = -13333	18333
-13333 + 50000 - 40000 = -3333	28333
-3333 + 100000 - 120000 = -23333	8333
-23333 + 50000 - 33333 = -6666	25000
-6666 + 0 - 25000 = -31666	0
-31666 + 25000 - 8333 = -14999	16667
-14999 + 100000 - 106666 = -21665	10001
-21665 + 75000 - 50000 = +3335	35001
+3335 + 0 - 3335 = 0	31666

P.

20/11/19

Pg. No. 112 (WB)

Q.24>

$$BOD_1 = \frac{(9.2-6.9)\ mg/l}{(5/300)} = 138\ mg/l$$

$$BOD_2 = \frac{(9.1-4.4)\ mg/l}{(10/300)} = 141\ mg/l$$

$$\therefore\ BOD_{avg} = 139.5\ mg/l$$

Pg. No. 113 (WB)

Q.27> $BOD_5\ (20°C) = 100\ mg/l$

$BOD_1\ (37°C) = ?$

$k\ (20°C) = 0.1\ d^{-1}$

As per G.O.I manual when $k = 0.1 d^{-1}$ at $20°C$. For municipal sewage, it is taken w.r.t base 10.

Thus,

$$BOD_5\ (20°C) = BOD_u\left(1 - 10^{-0.1 \times d^{-1} \times t}\right)$$

$$100\ mg/l = BOD_u\left(1 - 10^{-0.1 d^{-1} \times 5d}\right)$$

$$BOD_u = 146.24\ mg/l$$

$$K_{DT°C} = K_{D20°C}\,(1.047)^{T-20}$$

$$= 0.1 d^{-1}\,(1.047)^{37-20}$$

$$= 0.218\ d^{-1}$$

$$BOD_1(37°C) = 146.24\ mg/l\left(1-10^{-0.218 \times 1}\right) = 577\ mg/l.$$

Pg. No. 111 (WB)

Q.15)

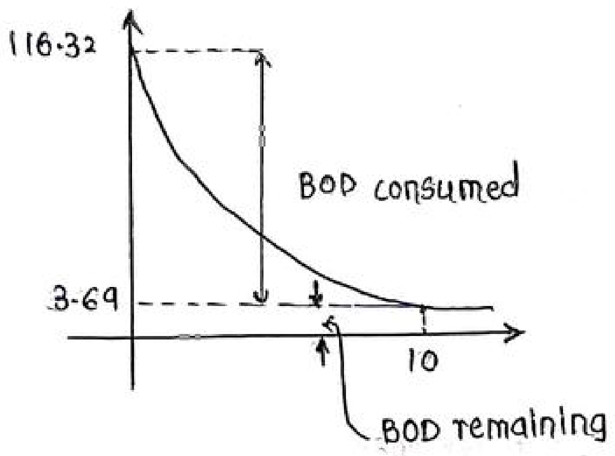
$$BOD_3 = 75\ mg/l = BOD_u\left(1 - e^{-0.345 d^{-1} \times 3d}\right)$$

$$k\ (\text{base } e) = 0.345\ d^{-1}$$

$$BOD_u = 116.32\ mg/l$$

$$L_{10} = 116.32\ mg/l \times e^{-0.345 d^{-1} \times 10d}$$

$$= 3.69\ mg/l.$$



Pg. No. 110 (WB)

Q.7)

$$BOD_5\ (20°C) = 180\ mg/l$$

$$k\ (\text{base}) = 0.18\ d^{-1}\ (\text{at } 20°C)$$

$$k_{T°C} = k_{20°C}\ (1.047)^{T-20}$$

$$BOD_{2.5}\ (T°C) = 180\ mg/l$$

$$BOD_5\ (20°C) = BOD_{2.5}\ (T°C)$$

$$L_0\left(1 - e^{-0.18 d^{-1} \times 5d}\right) = L_0\left(1 - e^{-k_{T°C} \times 2.5}\right)$$

$$e^{-0.18 \times 5} = e^{-k_{T°C} \times 2.5}$$

$$0.18 \times 5 = k_{T°C} \times 2.5$$

$$\therefore k_{T°C} = 0.36\ d^{-1}$$

$$0.36\,d^{-1} = 0.18\,d^{-1}(1.047)^{T-20}$$

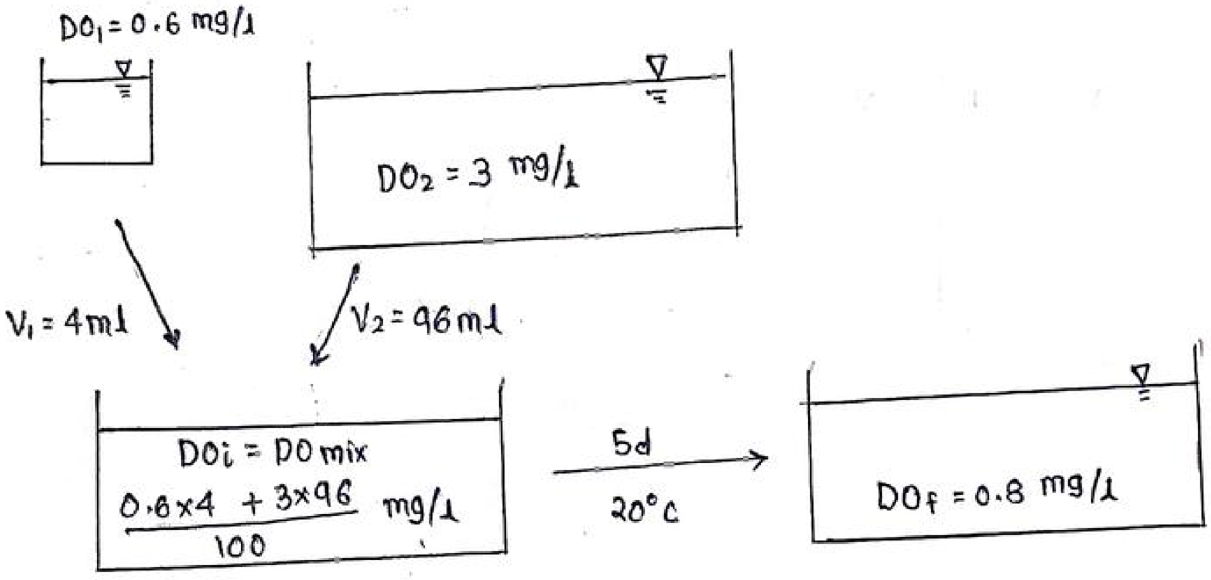
$$2 = (1.047)^{T-20}$$

$$\log_{10} 2 = (T-20)\log_{10}(1.047)$$

$$\therefore\ T = \underline{\underline{35^\circ C}}$$

Pg. No. 113 (WB)

Q. 25>



$= 2.904$ mg/l

$$\text{BOD w/w } (5d, 20^\circ C) = \frac{2.904 - 0.8}{4/100} = 52.6 \text{ mg/l}$$

$$k = 0.234\, d^{-1} \text{ (base 'e')}$$

$$BOD_5\,(20^\circ C) = BOD_u(1 - e^{-k\times t})$$

$$52.6 \text{ mg/l} = BOD_u(1 - e^{-0.234\times5})$$

$$\therefore\ BOD_u = 76.3 \text{ mg/l}$$

Pg. No. 109 (WB)

Q.1)

$$= \frac{162 \text{ mg/l} \times 10^{6} \text{ l/d} \times 10^{-3} \text{ g/mg}}{80 \text{ g/c/d}}$$

$$= 2025$$

7. DISPOSAL OF SEWAGE

Pg. No. 115 (WB)

Q. 10>

$L_o = 28.28$ mg/l

$D_o = 0.56$ mg/l

$f = 4.25$

$$\left(\frac{28.28}{D_c \times 4.25}\right)^{3.25} = 4.25\left[1 - \frac{3.25 \times 0.56}{28.28}\right]$$

$$\therefore D_c = 4.35 \text{ mg/l}$$

Pg. No. 115 (WB)

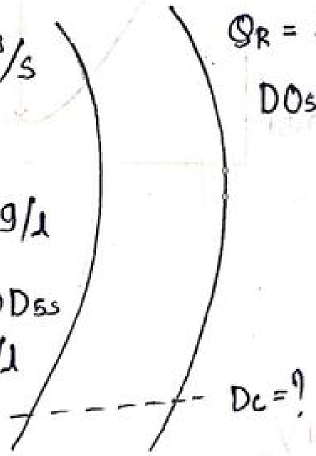
Q. 11>

$Q_s = 250$ m³/s

$DO_s = 0$

$BOD_{5s} = 350$ mg/l

$BOD_{us} = 1.5 \times BOD_{5s}$
$= 525$ mg/l



$Q_R = 2000$ m³/s

$DO_{sat} = 12$ mg/l

$f = 3$

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

$L_o \rightarrow$ Ultimate BOD of the mix

$$L_o = \frac{BOD_{us} \times Q_s + BOD_{uR} \times Q_R}{Q_s + Q_R}$$

$$= \frac{(525 \text{ mg/l} \times 250 \text{ m}^3/\text{s}) + 0}{2250 \text{ m}^3/\text{s}}$$

$$L_o = 58.33 \text{ mg/l}$$

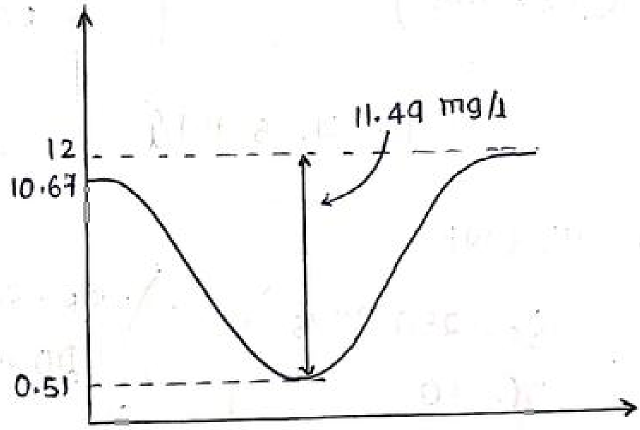
$$DO_{mix\ t=0} = \frac{DO_R \times Q_R + DO_s \times Q_s}{Q_R + Q_s}$$

$$= \frac{12\ mg/l \times 2000\ m^3/s + 0}{2250\ m^3/s} = 10.67\ mg/l$$

$$D_o = 12\ mg/l - 10.67\ mg/l$$
$$= 1.33\ mg/l$$

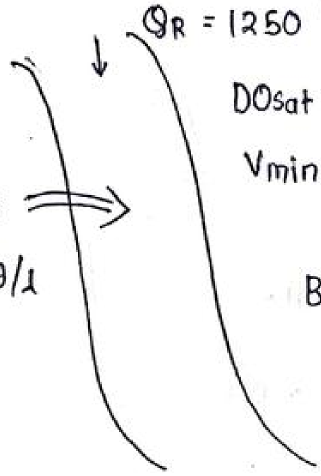
$$\left[\frac{58.33}{D_c \times 3}\right]^2 = 3\left[1 - \frac{2 \times 1.33}{58.33}\right]$$

$$D_c = 11.49\ mg/l$$



Pg. No. 115

Q.12)



$Q_R = 1250\ m^3/s$

$DO_{sat} = 9.17\ mg/l$

$V_{min} = 0.15\ m/s$

$Q_s = 100\ m^3/s$

$BOD_{5s} = 260\ mg/l$

$DO_s = 0$

$BOD_{u\,mix} = 1.25 \times BOD_{5\,mix}$

$f = 4$

$K_D' = 0.11\ d^{-1}$ (base 10)

$K_R' = 0.44\ d^{-1}$ (base 10)

$BOD_{mix} = 1.25 \times BOD_{5mix} = L_o$

$$BOD_{5mix} = \frac{BOD_{5S} \times Q_S + BOD_{5R} \times Q_R}{Q_S + Q_R}$$

$$= \frac{260 \ mg/l \times 100 \ m^3/s + 0}{1350 \ m^3/s}$$

$$= 19.25 \ mg/l$$

$L_o = 1.25 \times 19.25 = 24.07 \ mg/l$

$$DO_{mix} = \frac{0 + 9.17 \times 1250}{1350} = 849 \ mg/l$$

$D_o = DO_{sat} - DO_{mix} \ t = 0$

$= 0.68 \ mg/l$

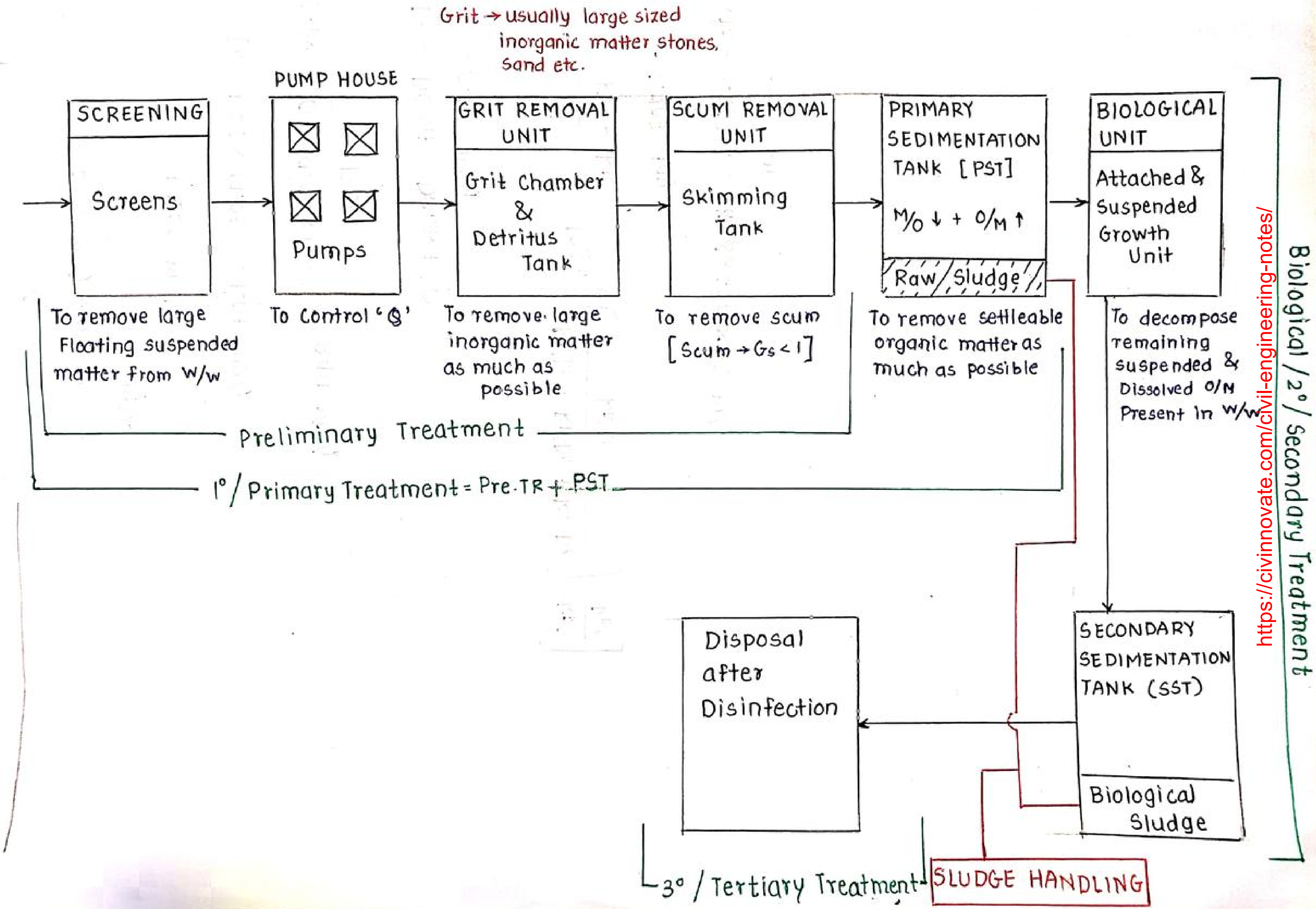
$$t_c = \frac{1}{K_R' - K_D'} \log_{10}\left[\left(\frac{K_D' L_o - K_R' D_o + K_R' D_o}{K_D' L_o}\right)\frac{K_R'}{K_D'}\right]$$

$= 1.707$ day

$d_c = 1.707 \times \cancel{8Al6} \ 86400 \times 1.5 \ m/s$

$= 22.13$ km

8. TREATMENT OF SEWAGE



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Overview

- The Preliminary Treatment consist of Screening, Grit removal Unit & Skimming Tank. It is designed to reduce the BOD of waste water by about 15-30%.
- The Preliminary Treatment combined with PST is called as Primary Treatment. It is designed to remove 60 to 70% Suspended solid & 30 to 40% BOD associated with it.
- In Secondary or 2° Treatment, microorganism will decompose the organic matter & form Biological sludge. The effluent from Secondary Treatment usually content 2 to 5% of the original B.O.D
- Tertiary or 3° Treatment consists of removal of microorganism usually by ultraviolet radiation.

GRIT CHAMBER

This are long narrow sedimentation basins with less Detention Time.

They are designed to ensure settlement of inorganic particles only.

The material collected at the bottom of the chamber can be easy disposed.

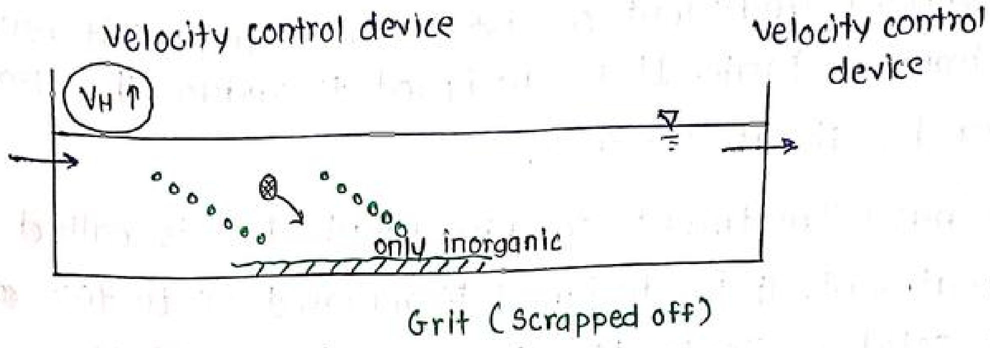
Either Rectangular or Parabolic sections can be used.

Velocity control devices are installed at the entry and the exit of the Chamber.

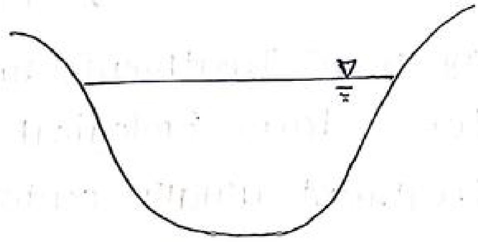
To control the velocity in Rectangular Tank, Proportional or sutro weir is used.

To control the flow in parabolic Channel, Parshall Flume is used.

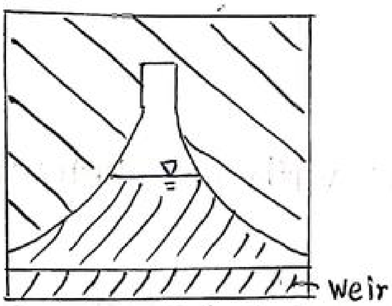
Design Data



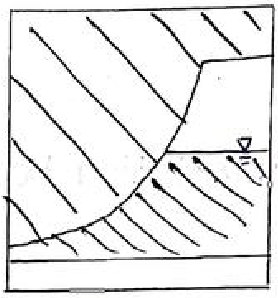
Rectangular



Parabolic

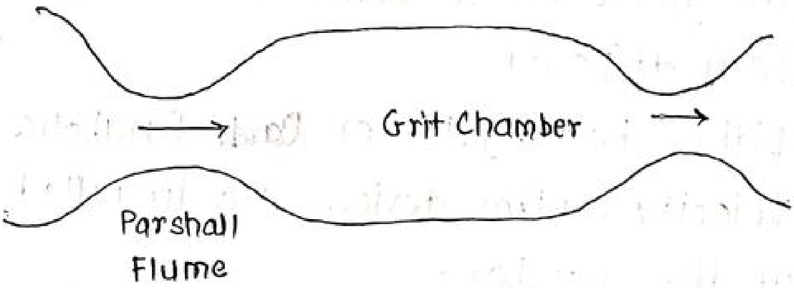


'Proportional' weir



'SUTRO' weir

Top View



1) $D_t = 40 - 60\ s$

2) $V_H = 0.15 - 0.3\ m/s$

3) $d > 0.2\ mm$

4) Stoke's law is not valid

5) $G_{s\,design} = 2.65$

6) $H = 1 - 2.5m$

7) Free board $= 0.3m$

8) $\frac{L}{B} = 4$ to 8

Modified Shield's Velocity

Shield proposed a minimum Horizontal velocity to be maintained if the particle of size 'd' & Specific Gravity (G_s) is to be kept in suspension & not allowed to get settled. This is called as modified shield velocity & is given as follows :

$$V_{Hmin} = 3 \text{ to } 4.5\sqrt{gd(G_s-1)}$$

$$V_{Hmin} = 4\sqrt{gd(G_s-1)}$$

Detritus Tank

The basic difference between Detritus Tank & Grit Chamber is that, Grit chamber is designed for removal of inorganic matter only but detritus Tank is designed for settlement of most of the inorganic matter.

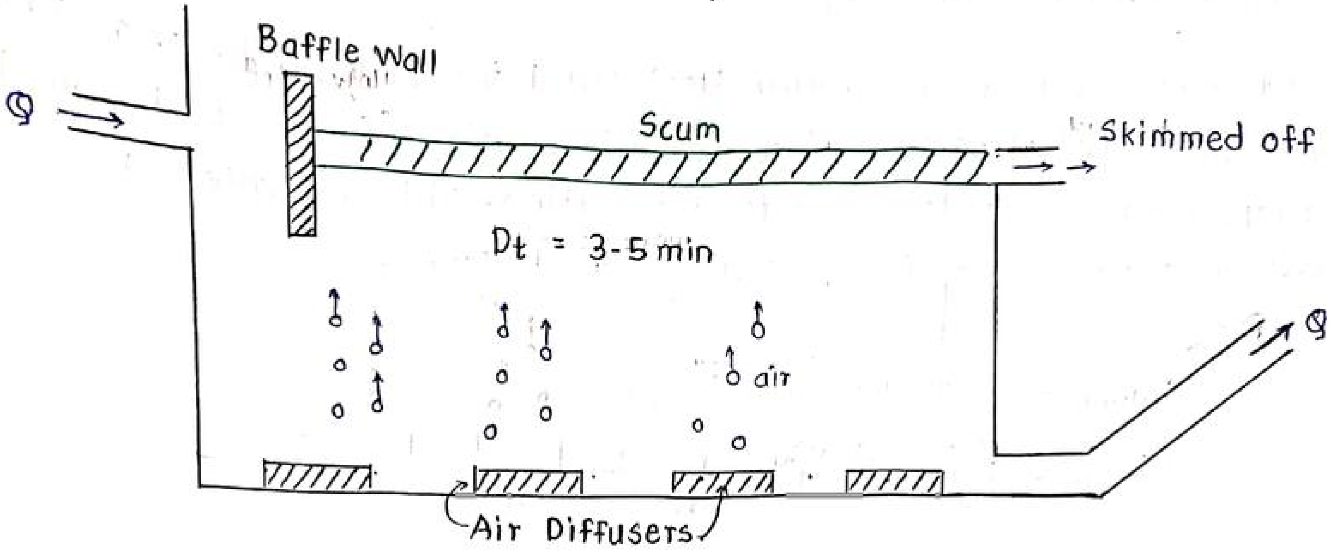
D_t = 3 - 5 min

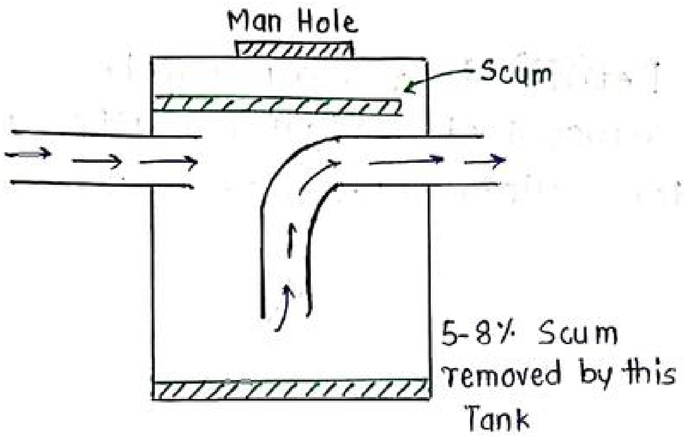
V_H = 0.09 - 0.15 m/s

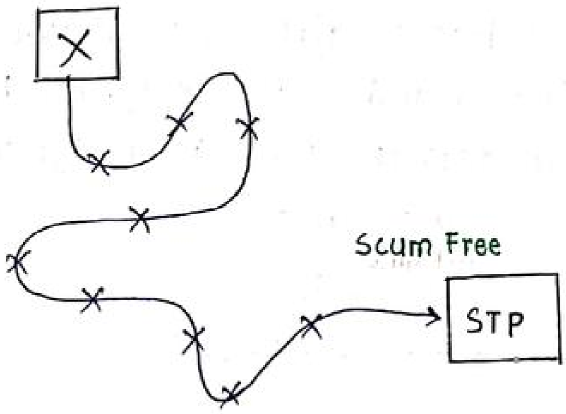
SKIMMING TANK

21/11/19

- Skimming Tank removes the scum particles such as oil, fat, grease, soap etc. which enters the waste water system.
- In a Skimming Tank, Compressed Air is blown by Aerating device which tends to drag the lighter scum particles to the top. It has a Detention Time of 3 to 5 min.
- Skimming Tank is an optional unit as a scum is removed even before it enters S.T.P by constructing oil & grease traps throughout the sewarage system.







BIOLOGICAL UNIT

• All the Biological units are designed to work under Aerobic Condition because :

① Aerobic Decomposition is faster & thus the Detention Time is lesser

② Aerobic Decomposition doesnot produce bad smell or Gases.

• The Biological units are of 2 Types

① Attached Growth Units – In these units, a medium is used to retain & to grow micro-organism under Aerobic conditions

Eg. Trickling Filter, Rotating Biological Contactor & Bio Tower.

(2) Suspended Growth Unit - In these unit, organic matter & micro organism are kept in suspension & are not attached to any medium. To obtain a high rate of decomposition, the contact b/w micro organisms & Organic matter is increased.

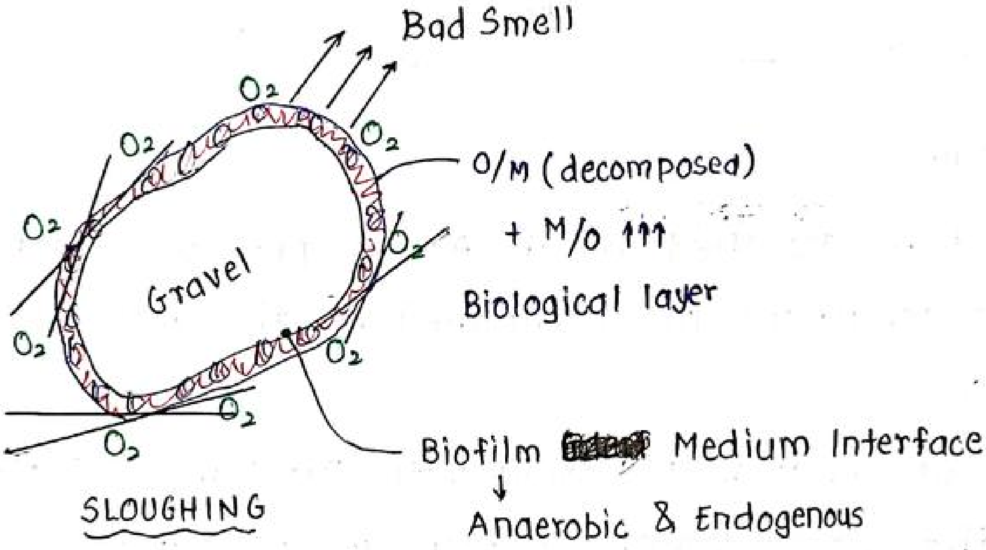
Eg. Aeration Tank of Activated Sludge Process, Oxidation Ditch / Aeration lagoon, Sequential Batch Reactor (SBR)

TRICKLING FILTER

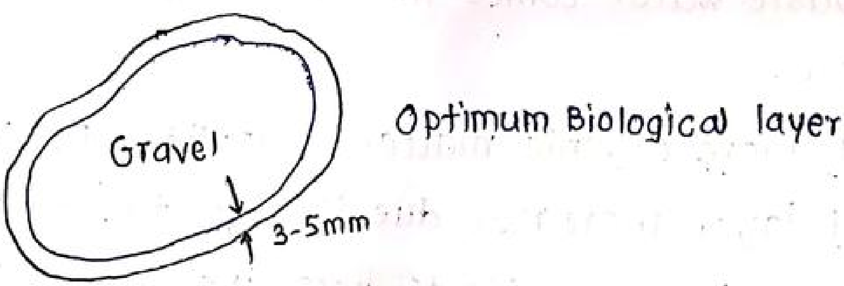
System Biology

- In a fixed film attached Growth Aerobic process like a Trickling Filter, Aerobic conditions are maintained for the decomposition of Organic matter.
- The surface of medium is coverd with Bio Film or Biological layer & as the waste water tricklers in a medium, organic matter in waste water comes in contact with micro organism & gets Decompose.
- As more and more organic matter is applied, the thickness of this Biological layer increases due to which the conditions at the bio film medium interface starts becoming Anaerobic because of unavailability of Oxygen.
- Due to Anaerobic conditions, foul smell gets produced & due to unavailability of food, Endogenous respiration predominates at the bio film medium interface.
- As the thickness of this bio film increases & becomes excessive & it gets sheared off & this process of Shearing is called as Slouging.
- Due to Slouging action bio film detached from the surface as it's thickness increases.

- As sloughing action increases, Aerobic conditions become predominant in the Filter.
- In Trickling Filter this sloughing action is controlled by controlling the discharge & in Rotating Biological Contactor (RBC) sloughing action is controlled by rotation.



SLOUGHING



Operation of Trickling Filters

- Trickling Filters are circular in cross-section in which sewage is applied at the top of the medium by rotating Distributory Arms.
- This Arm ensures uniform application throughout the circular section.
- 2 or 4 Arms are usually provided which rotate at 0.5 to 2 rpm which ensures uniform sloughing.

The media is usually Gravel & is well graded b/w 20 to 75 mm in size.

As the waste water Trickles down through the gravel, the biological layer gets exposed to the Air there by ensuring sufficient oxygen for decomposition, this ensures Aerobic decomposition.

Types of Trickling Filter

Their are 2 Types of Trickling Filter

1) Standard Rate or conventional Trickling Filter

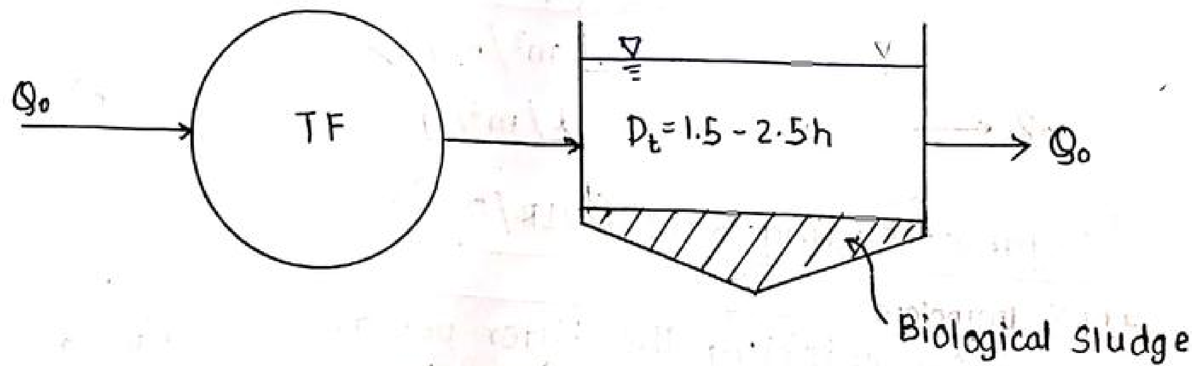
In this Trickling Filter, the Discharge over the filter cannot be controlled as their is no provision for recirculation.

Due to no or less control over sloughing action, the operational troubles like odour nuisance, Fly nuisance & ponding, are commonly observed in RSTF.

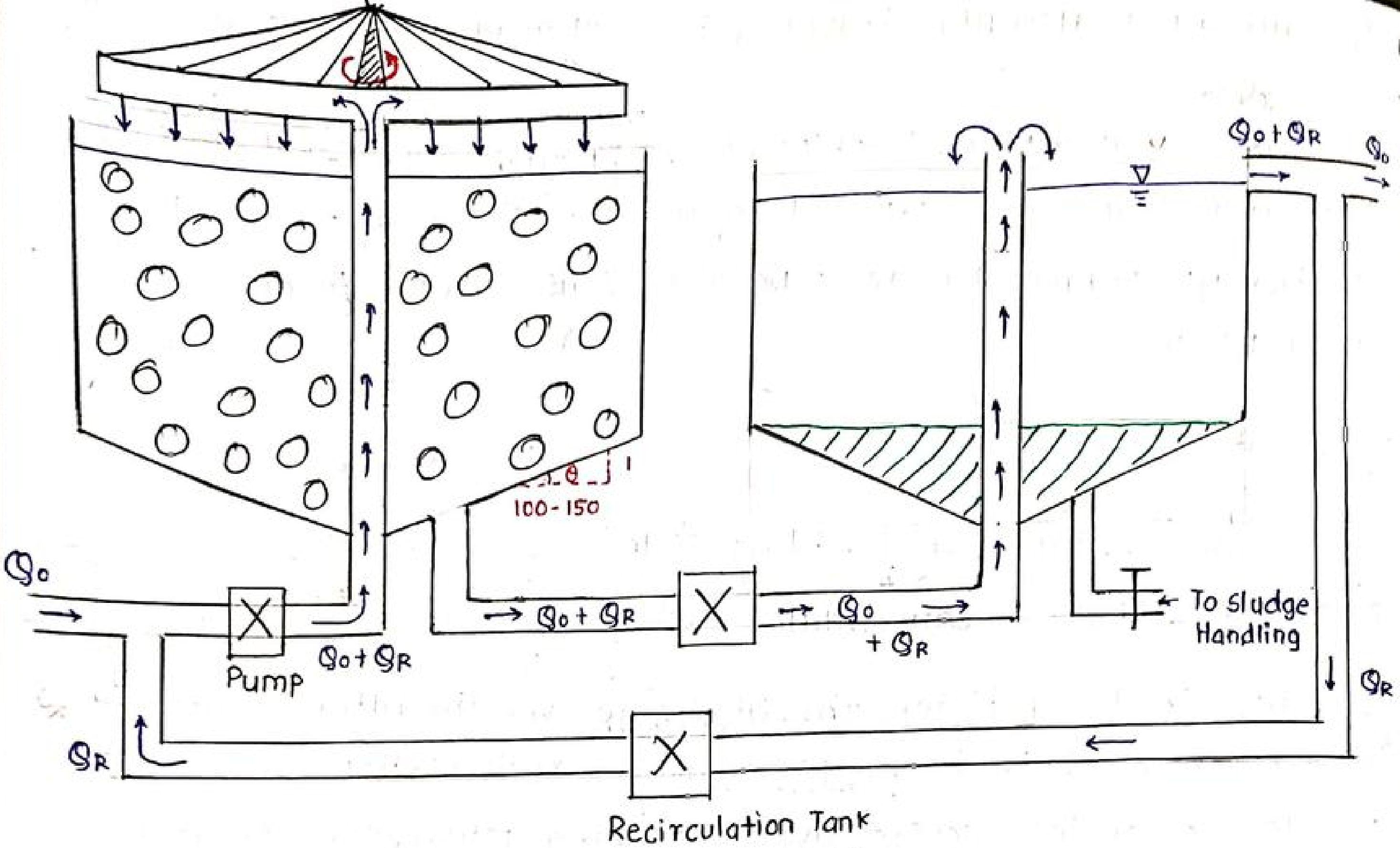
It is not used now a days

2) High Rate Trickling Filter.

In HRTF, their is a provision for recirculation of effluent due to which sloughing action can be controlled. Thus operational troubles of SRTF are not observed in HRTF.



—Standard Rate / Conventional T. F



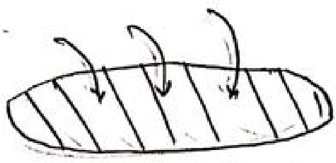
SINGLE STAGE HRTF

Design Parameters

1) Hydraulic / Surface Loading Rate (HLR/SLR)

$$\boxed{HLR = \frac{\text{Discharge Passing through the Filter}}{\text{Surface Area of the Filter}}}$$

$$S.A = \frac{\pi D^2}{4}$$



$m^3/m^2/d$

$l/m^2/d$

2) Volumetric / Organic Loading Rate (VLR/OLR)

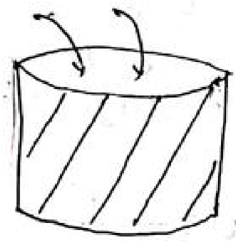
$$\boxed{VLR = \frac{\text{kg. of BOD entering the Filter per day}}{\text{Volume of the Filter}}}$$

$kg/m^3/d$

$kg/l/d$

mg/l × l/d , $V = \frac{\pi D^2}{4} \times H$

mg/d → kg/d



Parameters	SRTF	HRTF
1) HLR ($m^3/m^2/d$)	1 - 4	10 - 40 (including recirculation)
2) OLR (kg of BOD/m^3/d)	0.08 - 0.32	0.32 - 1.0 (excluding recirculation)
3) Recirculation Ratio ($R = Q_R/Q_D$)	0	0.5 - 3.0
4) Depth of Filter	1 - 3 m	0.9 - 2.5 m
5) Maximum Diameter	60 m	60 m

Efficiency of Trickling Filter

- Given by National Research Council (NRC) Formula

$$\boxed{\eta_{TF} = \frac{BOD_i - BOD_e}{BOD_i} \times 100}$$

CASE I : η of SRTF

$$\boxed{\eta = \frac{100}{1 + 0.44\sqrt{OLR}}}$$

OLR or VLR ⇒ kg of BOD/m^3/d

OLR → unit loading rate

CASE II : η of HRTF

⟨i⟩ Single Stage HRTF

$$\eta = \frac{100}{1 + 0.44\sqrt{\frac{OLR}{F}}} = \frac{100}{1 + 0.44\sqrt{\frac{W}{VF}}}$$

$$OLR = \frac{\text{kg of BOD entering the Filter/day}}{\text{Volume of the Filter } (m^3)} = \frac{W}{V}$$

$F \rightarrow$ Recirculation Factor

$$\boxed{F = \frac{1+R}{[1+(1-f)R]^2}}$$

$R = Q_R/Q_D$

$f \rightarrow$ Treatability Factor

$f = 0.9$ (For sewage in India)

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

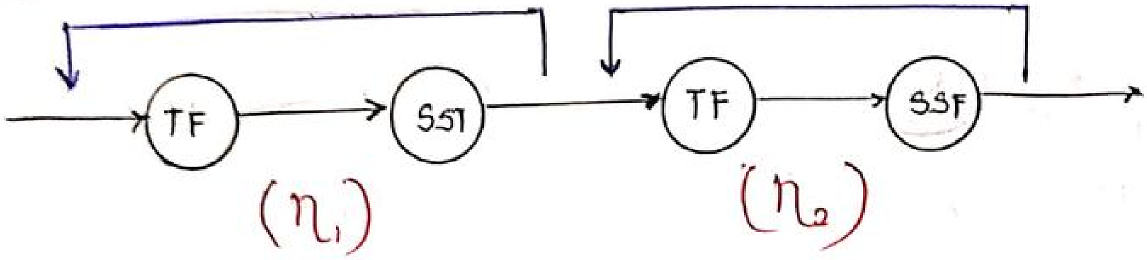
⟨ii⟩ Two Stage HRTF

$$\boxed{\eta_1 = \frac{100}{1 + 0.44\sqrt{\frac{W_1}{V_1F_1}}}}$$

$$\boxed{\eta_2 = \frac{100}{1 + \frac{0.44}{(1-\eta_1)}\sqrt{\frac{W_2}{V_2F_2}}}}$$

where,

$\eta_1 \rightarrow$ in decimal



$$\boxed{\eta_0 = \eta_1 + (1-\eta_1)\eta_2}$$

$W_1 \rightarrow$ kg of BOD entering 1st Filter/day

$W_2 \rightarrow$ kg of BOD entering 2nd Filter/day

$V_1 \rightarrow$ Volume of 1st Filter

$V_2 \rightarrow$ Volume of 2nd Filter

Pg. No. 127 (WB)

Q.46) $\eta = 82\%$

$V = 1365\ m^3$

$R = 1.5$

$W = \_\_\_\_\_\_\ kg/d$

$$F = \frac{1+1.5}{(1+0.1(1.5))^2} = 1.89$$

$$82 = \frac{100}{1 + 0.44\sqrt{\dfrac{W_1}{1365\,m^3 \times 1.89}}}$$

$\therefore W_1 = 642\ kg/d$

Pg. No. 127 (WB)

Q. 49>

$Q = 4.5 \times 10^6 \; l/d$

$BOD = 180 \; mg/l$

$OLR = 150 \; g/m^3/d$

$HLR = 2500 \; l/m^2/d$

$H = ?$

$$2500 \; l/m^2/d = \frac{4.5 \times 10^6 \; l/d}{S.A}$$

$$S.A = 1800 \, m^2$$

$$150 \; g/m^3/d = \frac{0.18 \; g/l \times 4.5 \times 10^6 \; l/d}{V}$$

$$\therefore 5400 \; m^3$$

$$H = \frac{V}{SA} = \frac{5400 \; m^3}{1800 \; m^2} = 3 \; m$$

Pg. No. 128 (WB)

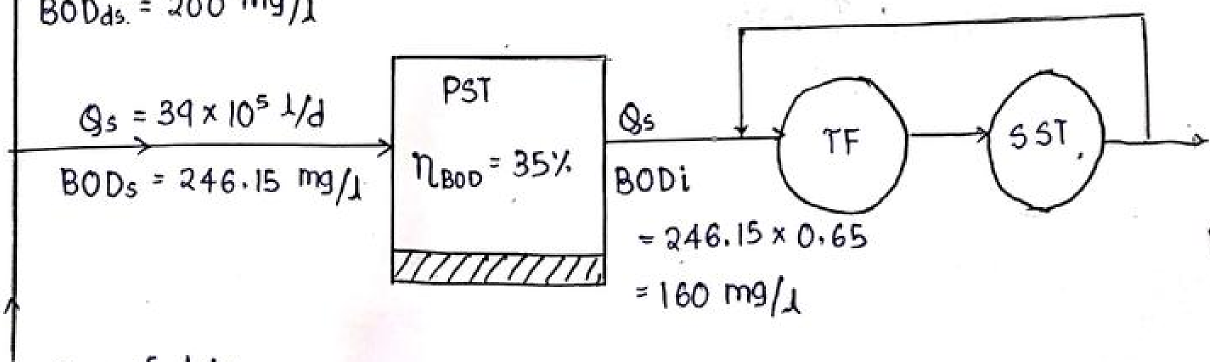
Q. 56>

$Q_{ds} = 30000 \times 120 \; l/d$
$= 36 \times 10^5 \; l/d$

$BOD_{ds} = 200 \; mg/l$

$Q_s = 39 \times 10^5 \; l/d$

$BOD_s = 246.15 \; mg/l$



Q_s

BOD_i

$= 246.15 \times 0.65$

$= 160 \; mg/l$

$Q_{is} = 3 \times 10^5 \; l/d$

$BOD_{is} = 800 \; mg/l$

$$OLR = 10000 \ kg/ha\text{-}m/d$$
$$= 1 \ kg/m^3/d$$

$$1 \ kg/m^3/d = \frac{160 \ mg/l \times 39 \times 10^5 \ l/d}{Volume \times 10^6 \ mg/kg}$$

$$\text{Volume of Filter} = 624 \ m^3$$

$$HLR = 170 \times 10^6 \ l/ha/d$$
$$= 170 \times 10^2 \ l/m^2/d$$

$$170 \times 10^2 \ l/m^2/d = \frac{39 \times 10^5 \ l/d + 39 \times 10^5 \ l/d}{\text{Surface Area}}$$

$$\text{Surface Area} = 458.82 \ m^2$$

$$\frac{\pi D^2}{4} = 458.82 \ m^2$$

$$\therefore D = 24.17 \ m$$

$$H = \frac{624 \ m^3}{458.82 \ m^2} = 1.36 \ m$$

$\because$ Dia. is less than 60 m & Height is between 0.9 - 2.5m

$\therefore$ Design is Ok.

$$\eta_{TF} = \frac{100}{1 + 0.44\sqrt{\frac{OLR}{F}}}$$

$$F = \frac{1+1}{(1+0.1)^2} = 1.653$$

$$\eta_{TF} = \frac{100}{1 + 0.44\sqrt{\frac{1 \ kg/m^3/d}{1.653}}} = 74.5 \%$$

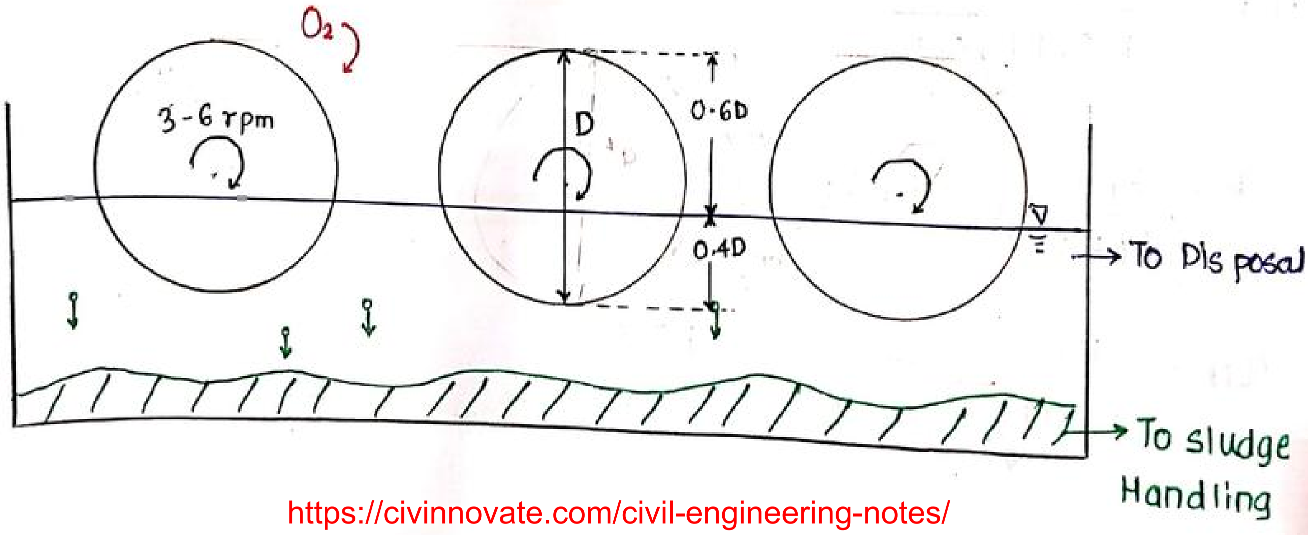
$$74.5\% = \frac{BOD_i - BOD_e}{BOD_i} \times 100$$

$$74.5\% = \frac{160\ mg/l - BOD_e}{160\ mg/l} \times 100$$

$$BOD_e = 40.8\ mg/l$$

Rotating Biological Contactor (RBC)

- In RBC, Aerobic bacteria are predominant.
- It is the film which is moving at 3 to 6 rpm & water is Stationary.
- RBC Discs are immersed upto 40% of their diameter & the micro organisms get organic matter when immersed & get oxygen when exposed.
- As the growth of Bio Film increases & becomes excessive, it gets Sheared off.
- SST is not required because the provision for sludge settlement is provided in the Tank itself.
- It handles relatively less quantity of water as it is operated as fill & Draw Type Tank.
- It is not used for commercial treatment in India.

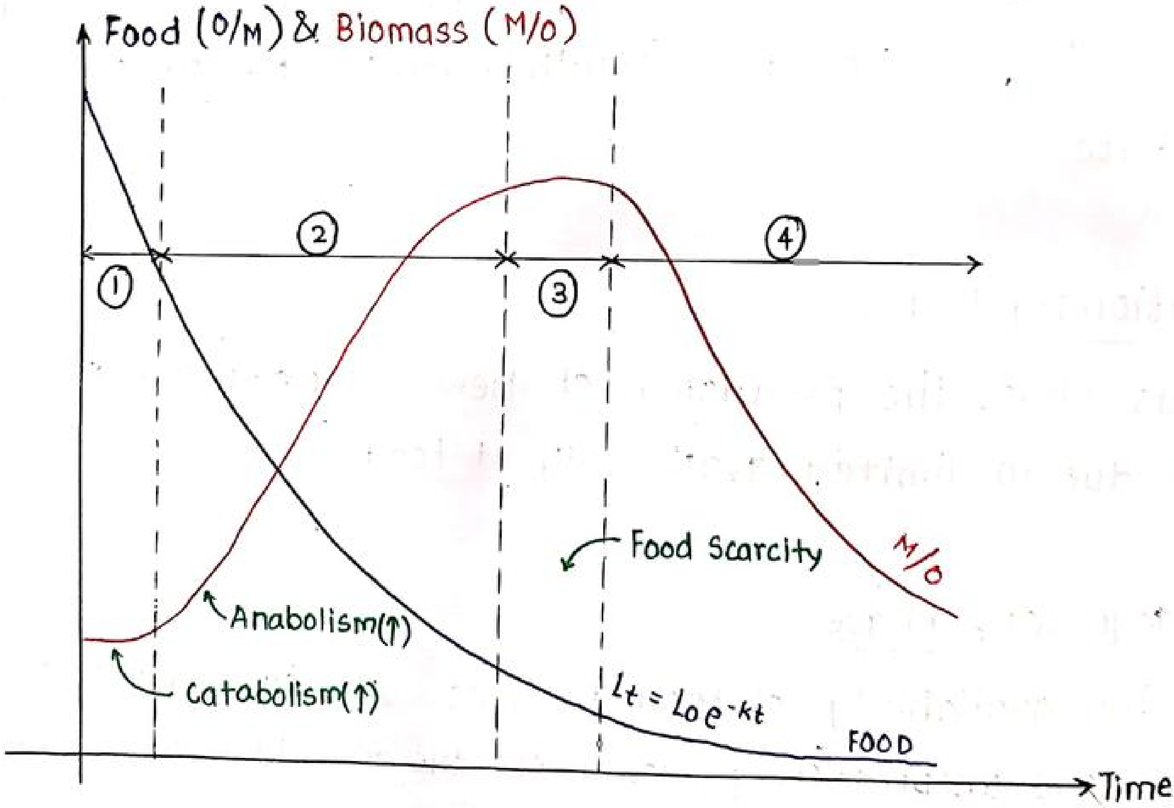


Bio Towers / Super HRTFs

With the invention of modular synthetic media of low weight & High Porosity, it is possible to make vertical arrangement of such media upto several meters of Height. Such materials can be high density Polyethylene, Geosynthetic etc. Such arrangement is called as a Bio Tower which has a higher discharge passing capacity & Higher efficiency than a High Trickling Filter.

Suspended Growth Unit

System Biology



① Lag Phase

② Log Growth Phase

③ Stationary Phase

④ Endogeneous Phase

① Lag Phase

The micro organism in the system 1st become acclimatised to their environment (pH & Temperature). & to the food provided. During this initial phase catabolism dominates over anabolism & thus the micro organism growth is very less, this Phase is called as lag Phase.

The lag phase will become negligible if micro organism is already accustom to a similar environment & to a similar food.

② Log Growth Phase

In this Phase Anabolism dominates due to sufficient availability of food. The micro organism multiply by cell division at very high Rate.

③ Stationary Phase

In this phase the production of new micro organism is roughly offset due to limited availability of food.

④ Endogeneous Phase

When the availability of food becomes ~~severia~~ severely limited, endogeneous respiration predominates due to which the bio-mass conc. decreases significantly.

NOTE:

① $\frac{\text{Food}}{\text{Biomass}} \uparrow\uparrow \rightarrow$ Food remains undecomposed

② $\frac{\text{Food}}{\text{Biomass}} \downarrow\downarrow \rightarrow$ Endogeneous respiration Dominates

* ACTIVATED SLUDGE PROCESS (ASP)

- ASP is an Aerobic Suspended Growth type biological Process that uses active micro organism kept in suspension to decompose & stabilized suspended & dissolved organic Matter.
- In ASP, a part of settled sludge in SST is returned to the Aeration Tank & oxygen is supplied externally to maintain Aerobic condition.
- The retured Sludge from SST is biologically active to the food provided & Hence this process is called as activated Sludge Process

Terminology

1. Return Activated Sludge (RAS)

The Sludge which contains active microorganism & is return to Aeration Tank to ~~kepth~~ keep enough microorganism in system is called as

2. Waste Activated Sludge (WAS)

The Sludge which gets wasted from the system & is taken for Sludge handling is called as

3. Mixed Liquor Suspended Solids (MLSS)

It represents the Total quantity of suspended solid present in the influent of the Aeration Tank.

4. Mixed Liquor Volatile Suspended Solids (MLVSS)

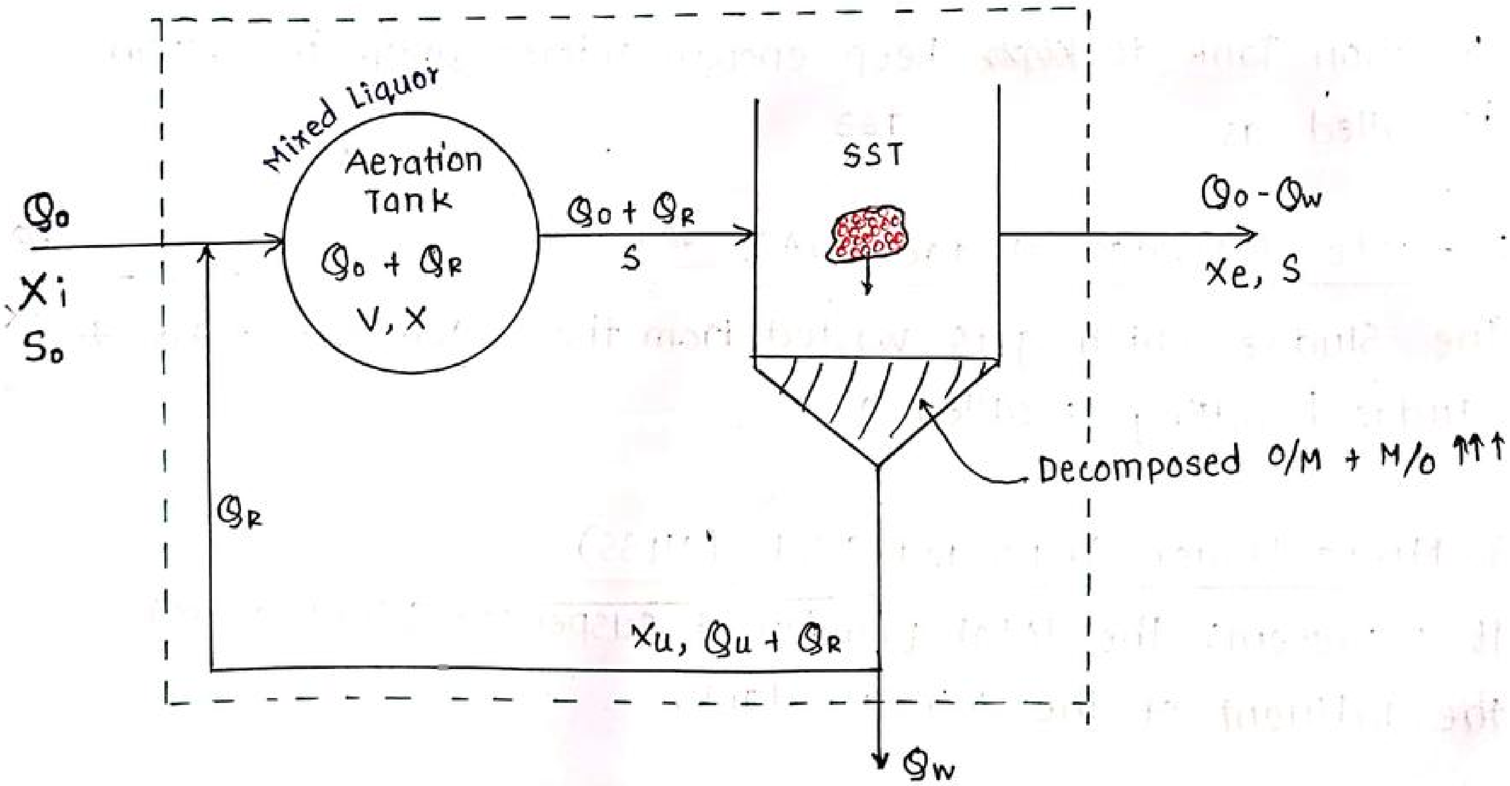
It is a part of MLSS which is actively consuming the incoming food & consists of Active micro organism in suspension. Experimentally when ASP attains equilibrium, MLVSS is found to be 80% of MLSS

$$\boxed{MLVSS \simeq 0.8 \times MLSS}$$

Assumptions

1) All the reactions occurs in Aeration Tank only, this implies that the BOD changes in the Aeration Tank only.

2) The influent & effluent Biomass concentrations are negligible.



$Q_R \rightarrow$ Return Activated Sludge Discharge

$Q_w \rightarrow$ Waste Activated Sludge Discharge

$V \rightarrow$ Volume of Aeration Tank

MLSS or MLVSS $\longrightarrow$ X (mg/l)

$X_u \rightarrow$ underflow SS concⁿ (mg/l)

$X_i \rightarrow$ influent SS concⁿ (mg/l)

$X_e \rightarrow$ effluent SS concⁿ (mg/l)

$S_o \rightarrow$ influent BOD (mg/l)

$S \rightarrow$ effluent BOD (mg/l)

[1] Hydraulic Retention Time (HRT/θ)

- Theoretical avg. time for which incoming sewage stays in the Aeration Tank.

$$\boxed{HRT = \frac{V}{Q_o}}$$

[2] Organic Loading Rate (OLR)

kg of BOD entering the Aeration Tank /m³/d

$$\boxed{OLR = \frac{Q_o S_o}{V}} \quad kg/m^3/d$$

[3] F/M Ratio

F : Food or O/M

M : Microorganism or Biomass

$$\frac{F}{M} = \frac{\text{kg of BOD entering the A.T per day}}{\text{kg of Biomass present in the A.T}}$$

$$\boxed{\frac{F}{M} = \frac{Q_o S_o}{VX}} \quad kg/d$$

Std. unit
↓
day^{-1}

$\frac{F}{M}$ → Primary Factor governing BOD removal

$\frac{F}{M}$ ↑↑↑ → O/M will remain undecomposed in a particular HRT

$\frac{F}{M}$ ↓↓↓ → M/O will undergo endogeneous metabolism

[4] Mean Cell Residence Time / Sludge Age (θ_c)

$$\theta_c = \frac{\text{Qty. of biomass present in A.T}}{\text{Rate of wastage of Biomass from the system}}$$

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

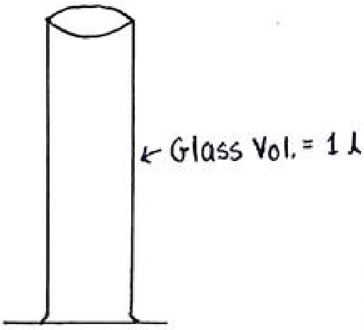
$$\theta_c \rightarrow \frac{l \times mg/l}{l/d \times mg/l}$$

As per Assumption 2

$$\boxed{\theta_c = \frac{VX}{Q_w X_u}}$$

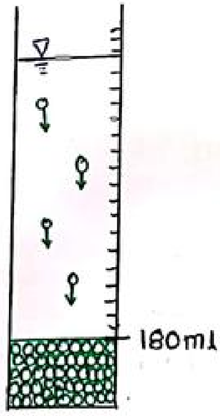
[5] Sludge Volume Index

- SVI is used to indicate Physical State of the Sludge produced in SST of ASP.
- SVI is defined as the volume occupied in ml by 1 gm of MLSS after settling for 30 minutes. The unit of SVI is ml/gm.
- Sludge Settleability & Sludge flowability is determined by SVI.
- As per GOI Manual, SVI should be between 80 to 150 ml/gm.
- If SVI is very high, it denotes the sludge with poor settling characteristics.
- If SVI is very less, it denotes the sludge with poor flowing Characteristics.



SVI Jar

Example



$X = 2500 \ mg/l$

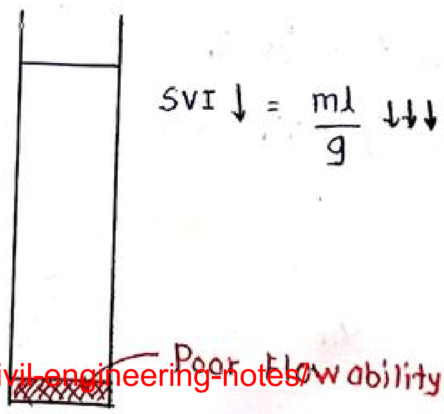
$= 2.5 \ g/l$

$SVI = \dfrac{180 \ ml}{2.5 g}$

$= 72 \ ml/g$

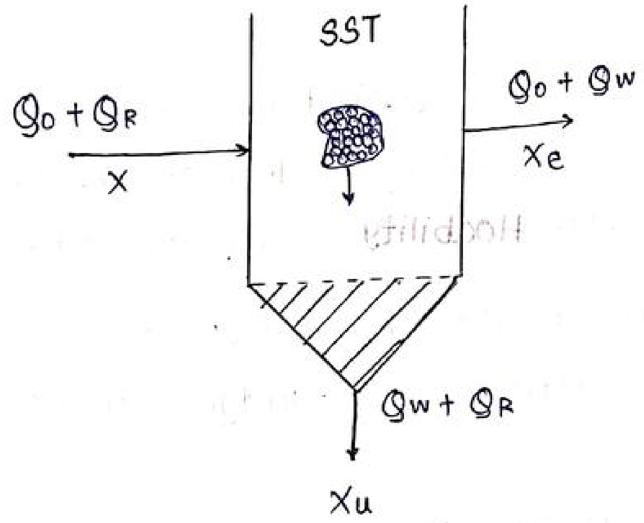
As per GOI Manual, SVI = 80 - 150 ml/g

$SVI \uparrow = \dfrac{ml}{g} \uparrow\uparrow\uparrow$



$SVI \downarrow = \dfrac{ml}{g} \downarrow\downarrow\downarrow$

[6] Recirculation Ratio ($R = Q_R/Q_D$)



Biomass In = Biomass Out [For SST]

$$\boxed{(Q_D + Q_R)X = (Q_D - Q_W)X_e + (Q_W + Q_R)X_u} \quad \text{——— (1)}$$

Find Q_R

Find $R = \dfrac{Q_R}{Q_0}$

As per Assumption 2, $X_e \simeq 0$

$$\boxed{(Q_D + Q_R)X = (Q_W + Q_R)X_u} \quad \text{——— (2)}$$

Special Case,

$$Q_W \simeq 0$$

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

$$Q_0 X + Q_R X = Q_R X_u$$

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

$$\boxed{\frac{Q_R}{Q_0} = \frac{X}{X_u - X} = R} \quad \text{——— (3)}$$

Pg. No. 127 (WB)

Q.45) $Q_0 = 30 \times 10^6 \ l/d$, $X = 3200 \ mg/l$

$S_0 = 260 \ mg/l$

$S = 20 \ mg/l$

$\frac{F}{M} = 0.21 \ d^{-1}$

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

$$0.21 \ d^{-1} = \frac{30 \times 10^6 \ l/d \times 260 \ mg/l}{V \times 3200 \ mg/l}$$

$$\therefore V = 11.6 \times 10^6 \ l$$

$$HRT = \frac{V}{Q_0} = \frac{11.6 \times 10^6 \ l}{30 \times 10^6 \ l/d} = 0.386 \ d \times 24 \ h/d$$

$$= \underline{\underline{9.28 \ hr.}}$$

Pg. No. 127 (WB)

Q.47) $Q_0 = 36000 \ m^3/day$

$S_0 = 250 \ mg/l$

$X = 2500 \ mg/l$

$X_u = 9700 \ mg/l$

$HRT = 8h$

$$HRT = \frac{V}{Q_0}$$

$$8h = \frac{V}{36000 \ m^3/day} = \frac{V}{\frac{36000}{24} \ m^3/h}$$

$$\therefore V = 12000 \ m^3$$

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

$$= \frac{36000\ m^3/day \times 250\ mg/l}{12000\ m^3 \times 2500\ mg/l}$$

$$= 0.3\ d^{-1}$$

Pg. No. 127 (WB)

Q.48) $Q_0 = 36000\ m^3/d$

$S_0 = 250\ mg/l$

$X = 2500\ mg/l$

$X_u = 9700\ mg/l$

$HRT = 8h$

$Q_w = 200\ m^3/d$

$X_e = 30\ mg/l$

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

$$= \frac{12000\ m^3 \times 2500\ mg/l}{(200\ m^3/d \times 9700\ mg/l) + (3600 - 200)\frac{m^3}{d} \times 30\ mg/l}$$

$$= 9.95\ days$$

Pg. No. 126 (WB)

Q.39)

$$\theta_c = \frac{VX}{Q_w X_u + \cancel{(Q_0 - Q_w) X_e} \rightarrow 0}$$

$$HRT = \frac{V}{Q_0}$$

$$= \frac{1250\ m^3 \times 3000\ mg/l}{50\ m^3/d \times 10000\ mg/l}$$

$$V = \frac{15000\ m^3/d \times 2 hr}{24}$$

$$= 7.5\ d$$

$$= 1250$$

Pg. No. 125 (WB)

Q.37, Q.38 >

$Q_0 = 500\ m^3/h$ $\quad V = 4000\ m^3$

$S_0 = 150\ mg/l$ $\quad X = 2000\ mg/l$

$S = 10\ mg/l$

$\theta_c = 240\ h$

$$\frac{F}{M} = \frac{Q_0 S_0}{VX} = \frac{500\ m^3/h \times 150\ mg/l}{4000\ m^3 \times 2000\ mg/l} = 9.375 \times 10^{-3}\ /h$$

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

$$= 0.225\ d^{-1}$$

$$\theta_c = \frac{VX}{Q_w X_u}$$

$$Q_w X_u = \frac{4000\ m^3 \times 2000\ mg/l \times 10^3\ l/m^3}{10\ d \times 10^6\ mg/kg}$$

$$= 800\ kg/d$$

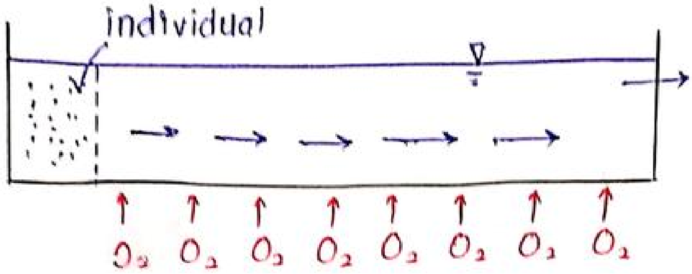
Operation of ASP

Their are following 2 Types of ASP

[1] Plug Flow ASP

- In this ASP, water moves progressively through the Aeration Tank essentially ~~mixed~~ unmixed with the rest of the Tank Content.
- In this Tanks, oxygen is blown from the bottom through Diffusor. Plug Flow Tanks have long & narrow configuration with L/B ratio b/w 5 to 50.
- The Amount of oxygen equivalent of organic matter present at any time 't' is given by

$$\boxed{L_t = L_0 e^{-kt}}$$



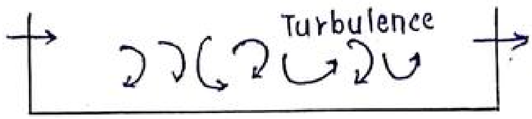
[2] Completely Mixed ASP

In this ASP, Complete mixing of waste water takes place with already present content in the Tank by Mechanical Aeration devices.

The amount of oxygen equivalent of organic matter at any time 't' is given as follows:

$$\boxed{L_t = \frac{L_0}{1+kt}}$$

$k \rightarrow$ deoxygenation const. (base 'e')



Units based on Plug Flow ASP

1) Conventional Aeration Tank

Influent Waste Water & Recycled Sludge enter the head of the Tank. Rate of Aeration is uniform through the length of Tank.

2) Tapered Aeration Tank

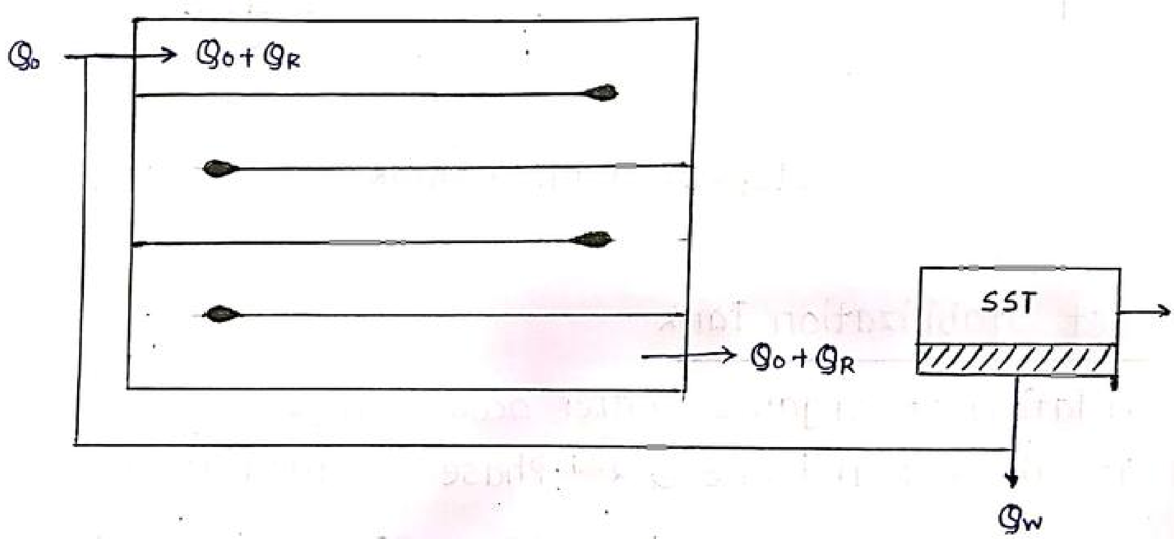
- Influent waste water is entered at the head of the Tank & the oxygen is supplied at a high rate in initial zone & is reduce towards the end zone.
- This is better than conventional Aeration Tank because, the conventional Aeration leads to either oxygen deficiency in the initial zone or wastefull Application in the End Zone.

3) Stepped Aeration Tank

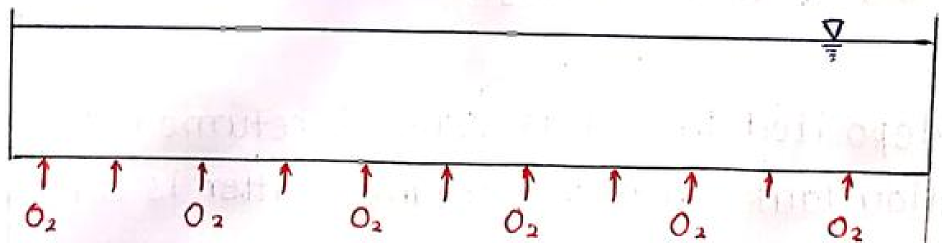
- Sewage is introduced at more than one pt. along the Aeration Tank.
- Recirculated sludge is introduced at the inlet.
- Aeration is done uniformly along the length of the Tank.
- It leads to greater decomposition of organic matter as compared to conventional Aeration.

NOTE: During Design, care has to be taken that endogeneous Phase is not observed in a Tank.

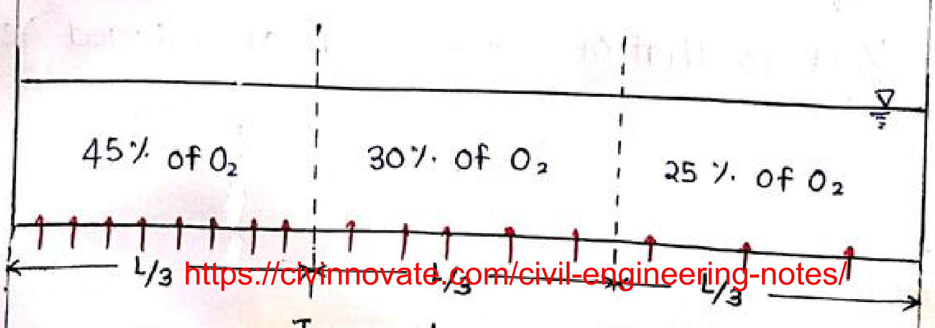
Top View



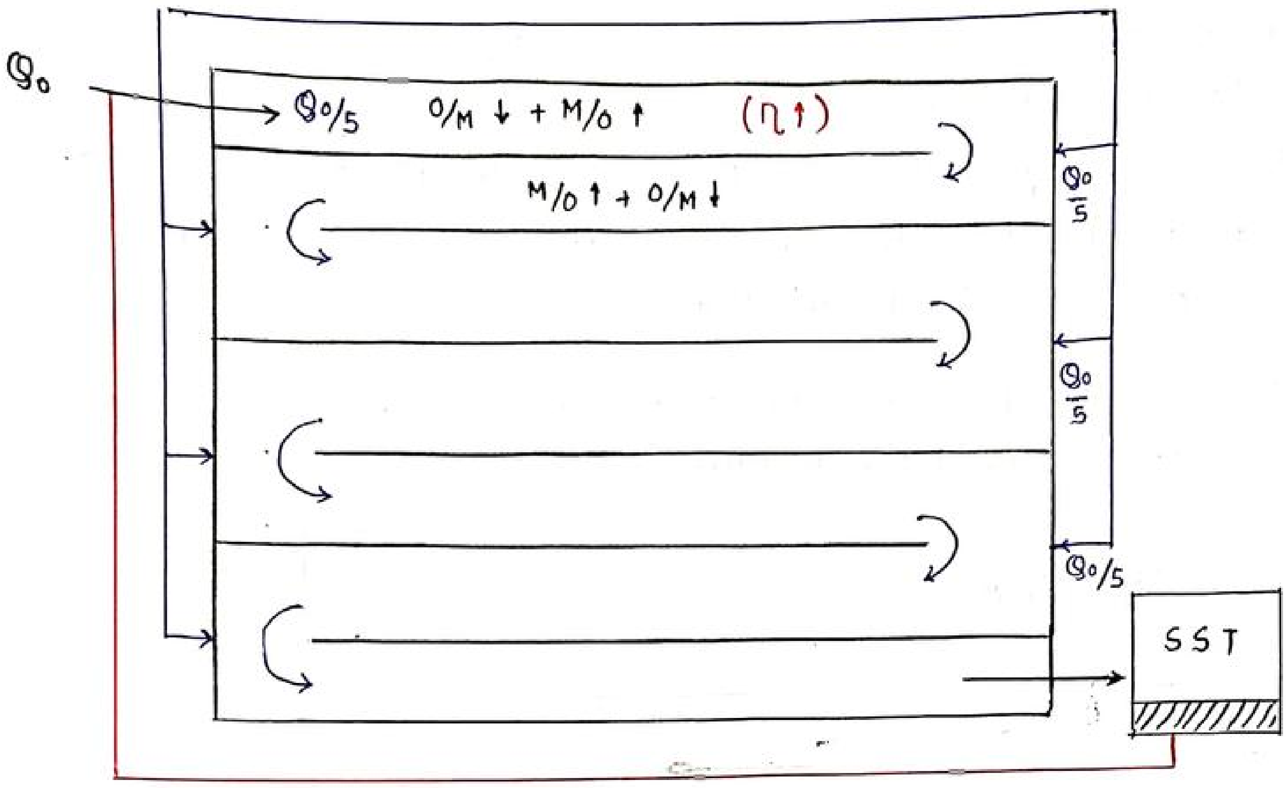
Let L = Total length of Flow



Conventional Aeration



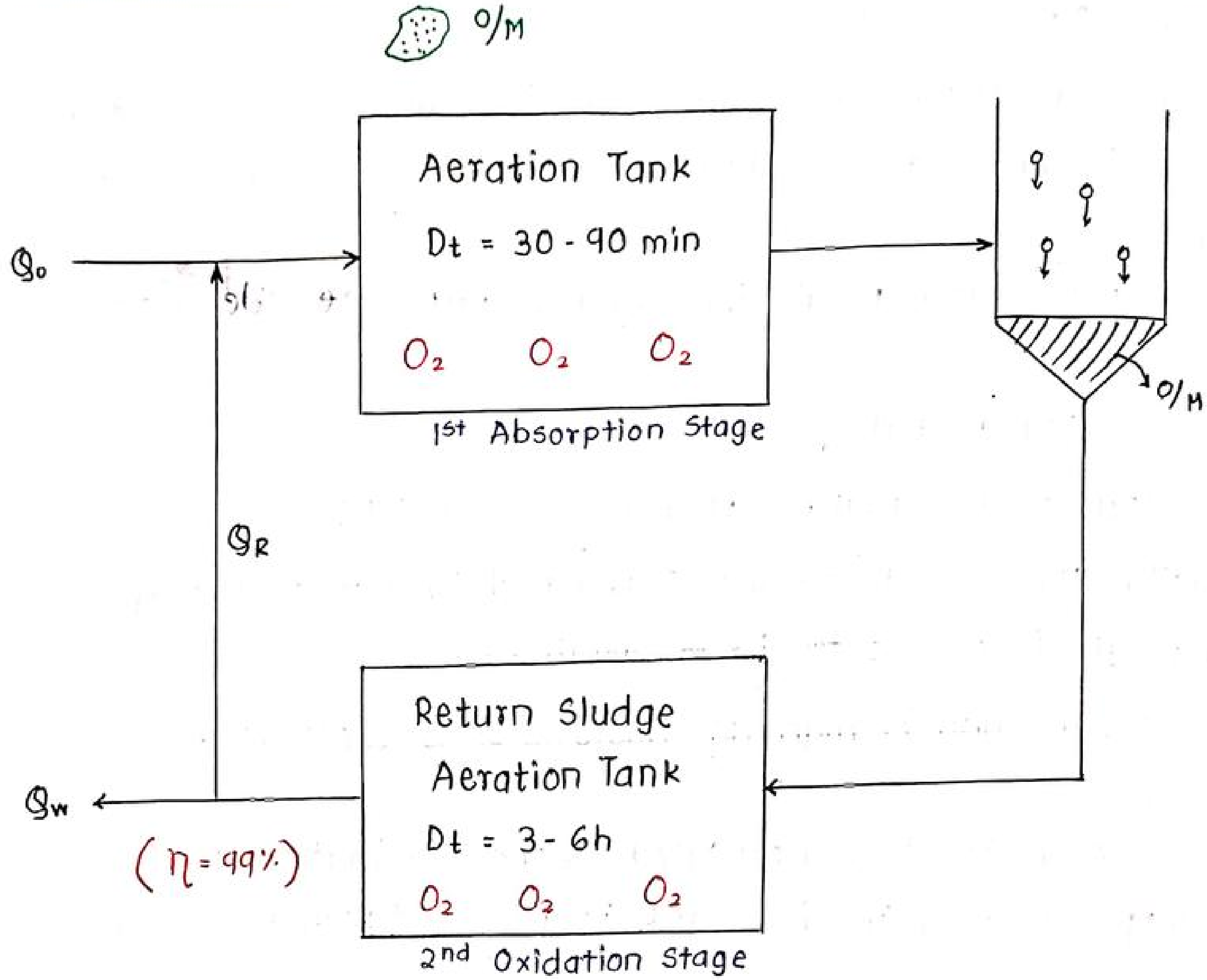
Tapered Aeration



Stepped Aeration Tank

4) Contact Stabilization Tank

- The oxidation of organic matter occurs in 2 Phase, 1st Phase is called as absorption Phase & 2nd Phase is called as oxidation Phase.
- The absorption Phase requires 30 to 90 minutes & during this Phase, suspended & dissolved organic matter starts getting decomposed.
- The sludge deposited in SST is ~~returned~~ returned to a return sludge Aeration Tank where the organic matter is completely oxidised.
- Total volume Requirement of a contact stabilization process is approximately 1/3rd to that of conventional or Tapered Aeration Tank.



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Units based on Completely Mixed ASP

1. Completely Mixed Aeration Tank

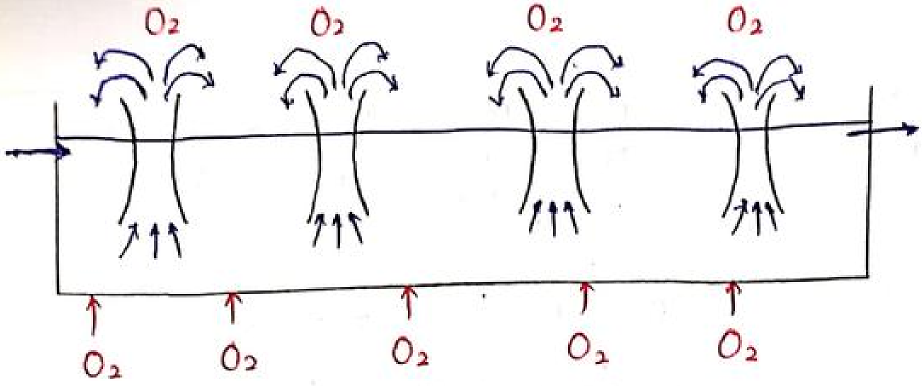
- In this tank, waste water is distributed along with returned Sludge uniformly from one side of the Tank & effluent is collected from other side.
- Rectangular section is divided into smaller section with each Section being installed with a Mechanical Aeration Device.
- With the help of these devices the sewage is sucked from the center & is blown over the surface of the waste water. Thus oxygen is available from the atmosphere. However there is also a provision of supply of oxygen into the system.

2. Extended Aeration Tank

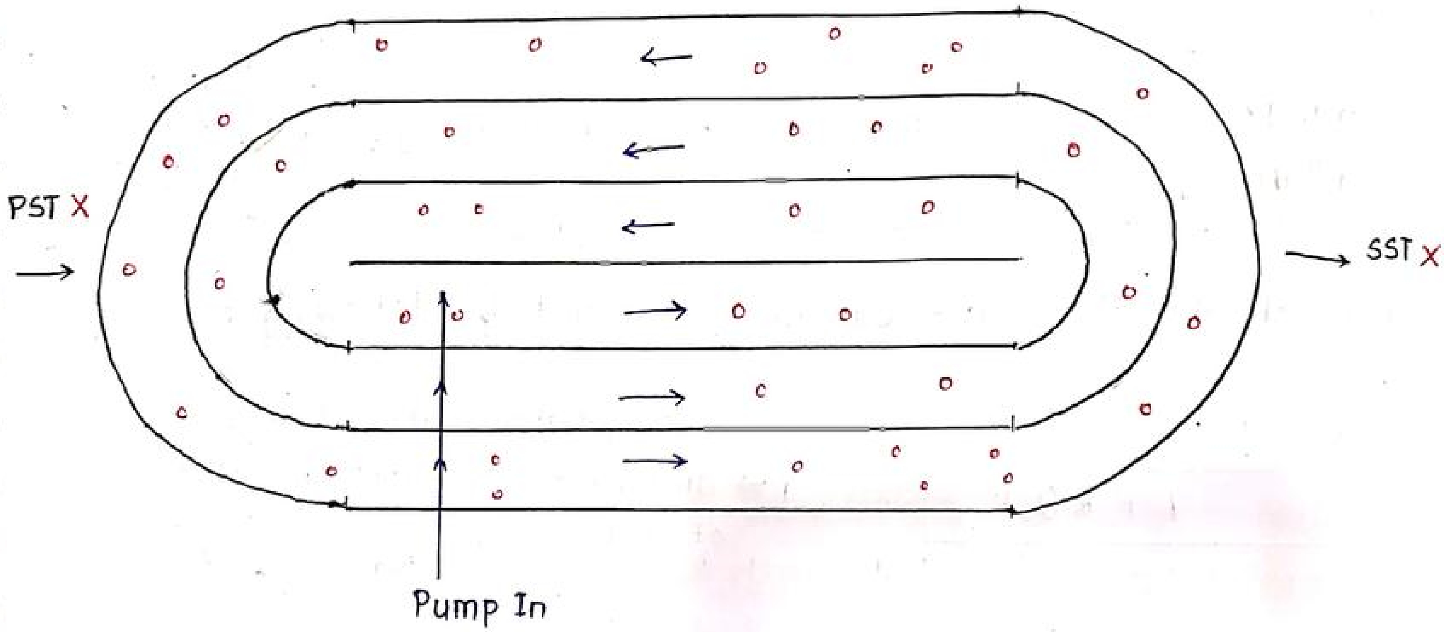
- It is also a completely mixed process in which long Aeration Time is maintained due to which a high efficiency of decomposition is obtained.
- PST is not required as all the organic matter gets decomposed in this Tank.
- It is operated as fill & Draw type Tank.
- It is capable of handling discharges ≤ 4 MLD.
- It usually consists of parallel baffled channel in oval shape.
- This Channels have 1 to 1.5 m depth.
- Power Consumption is high as wastewater is continuosly pumped in the system.
- However extra cost is Compensated by the elimination of PST & SST.
- The Sludge settled in this tank is directly taken for sludge handling.
- The units are called as Oxidation Ditch or Aeration lagoons.

3. Sequential Batch Reactor (SBR)

- It is adopted for small plants (Discharge ≤ 4 MLD) & it involves a single completely Mixed Reactor in which all the stages of ASP occurs.
- It is popular in these Area where there is space restriction (Industry) as it allows the smaller foot Print.



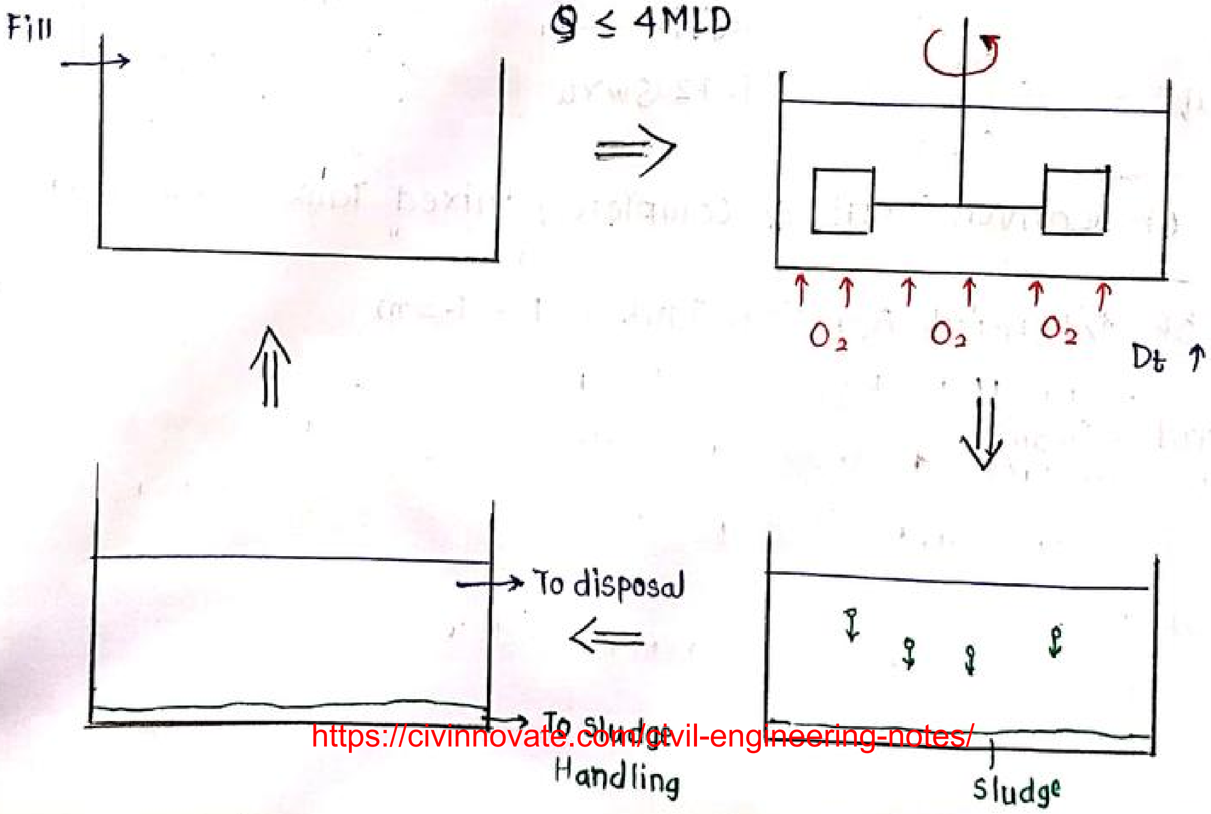
COMPLETELY MIXED AERATION TANK



EXTENDED AERATION TANK

4 Stages

$Q \leq 4 MLD$



Design Data

Type of Aeration Tank	MLSS(x) mg/l	$\frac{MLVSS}{MLSS}$	$\frac{F}{M}$	HRT Q (h)	θ_c (day)	$R = \frac{Q_R}{Q_0}$	η(%)	Y
1) Conventional A.T	1500-3000	0.8	0.3-0.4	4-6	5-8	0.25-0.5	85-95	0.8-1
2) Completely Mixed A.T	3000-4000	0.8	0.3-0.4	4-6	5-8	0.25-0.8	85-95	0.8-1
3) Extended Aerated Tank	3000-5000	0.8	0.1-0.18	12-24	10-15	0.5-1.0	95-99	0.8-1

- Volume of Aeration Tank can be estimated by following expression:

$$\boxed{Vx = \frac{YQ_0(S_0-S)\theta_c}{1 + \theta_c k}}$$

k → deoxygenation const. (base 'e')

Y → amount of O_2 supplied (kg) per kg of BOD removed.

- Amount of O_2 required to be supplied for satisfaction of stage 1/ Carbonaceous BOD

$$\boxed{O_2\ req^r = \frac{Q_0(S_0-S)}{0.68} - 1.42\ Q_w X_u}$$

- Depth of conventional & completely mixed Tank = 2.5 to 4 m
- Depth of extended Aeration Tank = 1 - 1.5m
- Freeboard = 0.3m

Operational Troubles of ASP

1> Sludge Bulking

It refers to sludge having increased volume due to large quantity of water present in it.

It is formed due to the growth of filamentous microorganism (Nitrosomonois, Nitrobactor's] as a result of High sludge age.

This phenomenon increases the handling cost of sludge & also results in Poor quality of effluent. Thus it reduces the effeciency of ASP considerably.

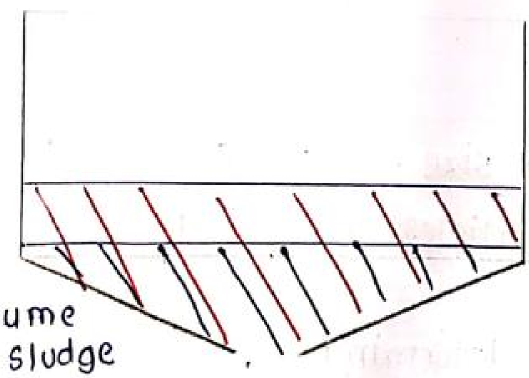
2> Blanket Rising

• ASP is designed for satisfaction of carbonaceous BOD but if some nitrification takes place due to High sludge Age. Oxygen gets consumed to satisfy 2nd Stage BOD or NBOD.

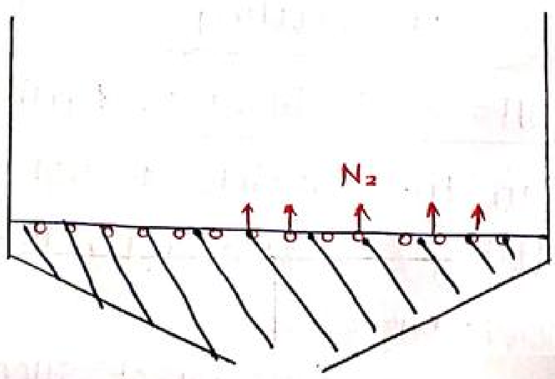
• Nitrification eventually leads to Denitrification in ASP.

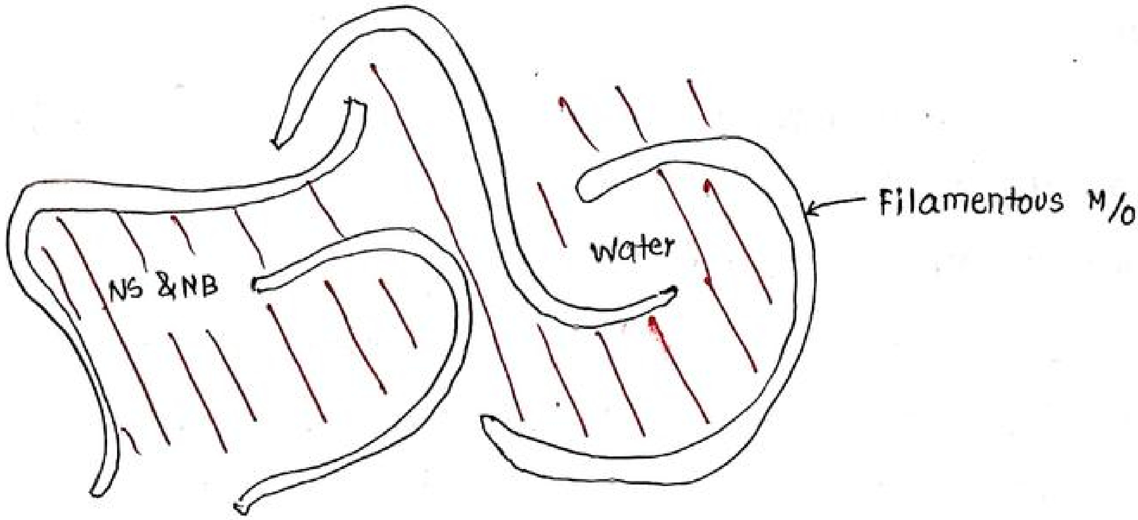
• The phenomenon in which settlability of Solid reduce ~~to~~ due to denitrification as a result of which solid rise to the surface along with nitrogen gas is called as Blanket Rising.

• As per GOI ~~Mano~~ Manual Sludge Bulking & Blanket Rising can be completely avoided if F/M is greater than 0.3 day^{-1}.



Volume of sludge





PST & SST

- PST is designed to remove suspended solids of organic nature as much as possible. The settled sludge in PST is called as Primary or Raw Sludge which is taken for sludge handling.

- SST is designed to remove the bioflocculated solid as much as possible to produce clear effluent.
The sludge settled in SST is called as secondary or biological sludge & it is also taken for sludge Handling.

- In both these tank the concentration of suspended solid is High & thus particles interacts with each other while settling. Therefore Type 1 or discrete particle settling is not observed in this Tanks.

Types of Settling

i) Type 1 OR Discrete Particle Settling

- When the Particle do not change their size, shape, or mass during settling & settle individually, such particles are called as Discrete particles.
- The settling velocity of such particle is determinable.

Eg. (i) Settling in the Sedimentation Tank in water Supply Engineering.
(ii) Settling in Grit Chamber (without appreciable error) due to less Detention Time.

2) Type 2 or Flocculant Settling :

When the particles are closer together such that their velocity field overlap eachother it is referred as Type 2 Settling.
This particles can chemically or Physically combine with each other & ~~Floc~~ flocculate each other.

Eg. Settling of suspended solids in PST, settling in Clariflocculator etc.

3) Type 3 or Zone or Hindered Settling

- In this type of Settling, the particles maintain their Relative position w.r.t eachother & whole mass of particles settle as a single unit.
- This settling results in significant upward displacement of water.
- No Mathematical eqn exists to simulate the behaviour of Type 2 & Type 3 settling & thus the unit based on Type 2 & Type 3 settling are Designed experimentally.

Eg. Settling in Settling zone of SST.

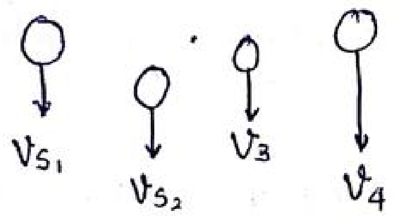
4) Type 4 or Compression Settling

- In the bottom most zones of the sedimentation Tank, the concentration of particle becomes so high that the particles are in physical contact with eachother the lower layers
- Supporting the weight of Upper layer, consequently any further

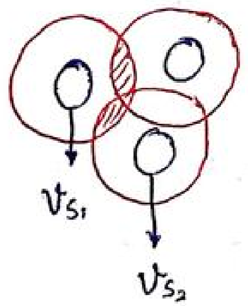
Settling result are due to compression of whole mass of Particle which is accompanied by sequezing out of water from Pores of Solids & is referred as compression settling.

• This type of settling occurs in the deep sludge mass in PST & SST.

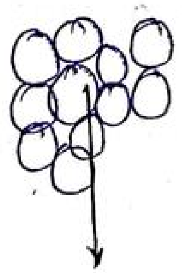
<1> Type 1 or Discrete Particle settling



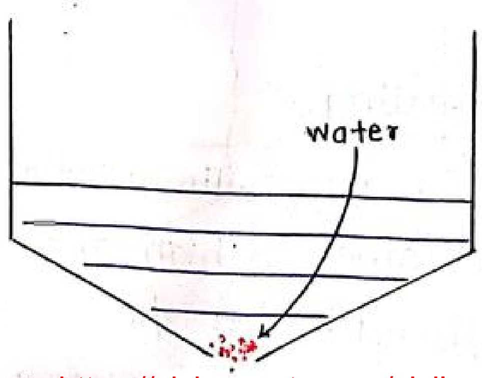
<2> Type 2 or Flocculant Settling



<3> Type 3 or Zone or Hindering Settling



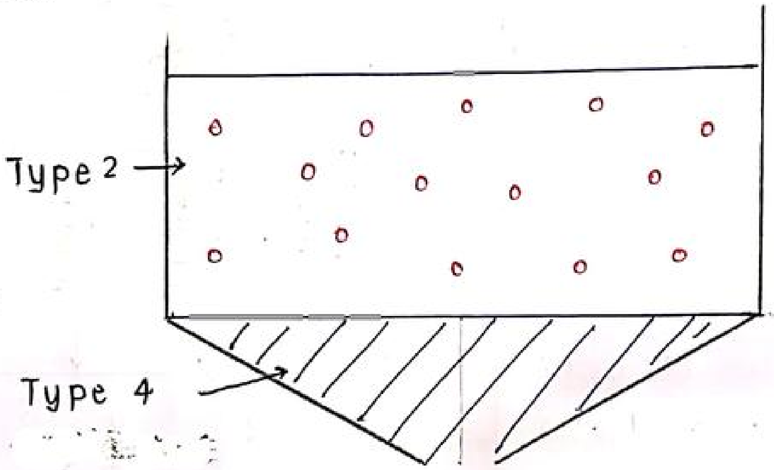
<4> Type 4 or Compression settling



Design of PST & SST

1. The Design of PST is based upon Surface Overflow Rate & Design of SST is based upon Surface Overflow Rate as well as Solid loading Rate.
2. The Detention Time in these Tank varies from 1.5 Hr - 2.5 Hr.
3. The Dia. of these Tank varies from 7.5 m to 12.5 m.
4. The Height of these Tank varies from 6.5 m to 9 m

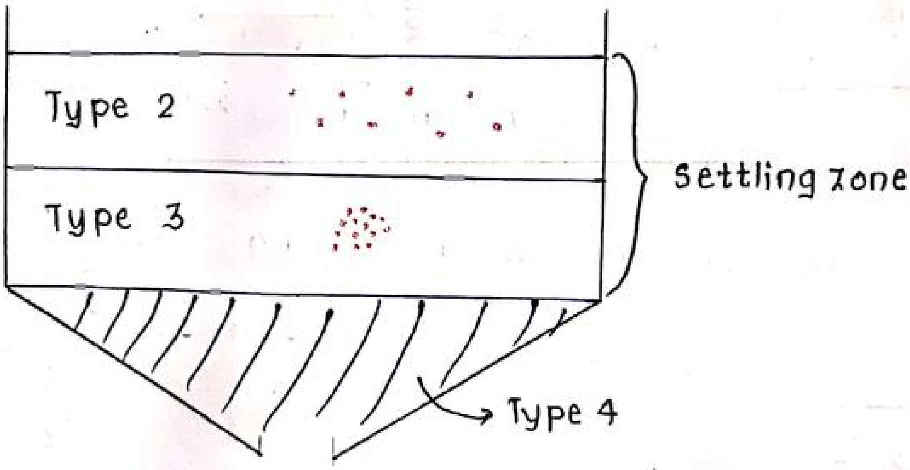
PST



$V_o = \frac{Q}{\pi D^2/4}$, D_t, H,

$V = Q \cdot D_t$

SST

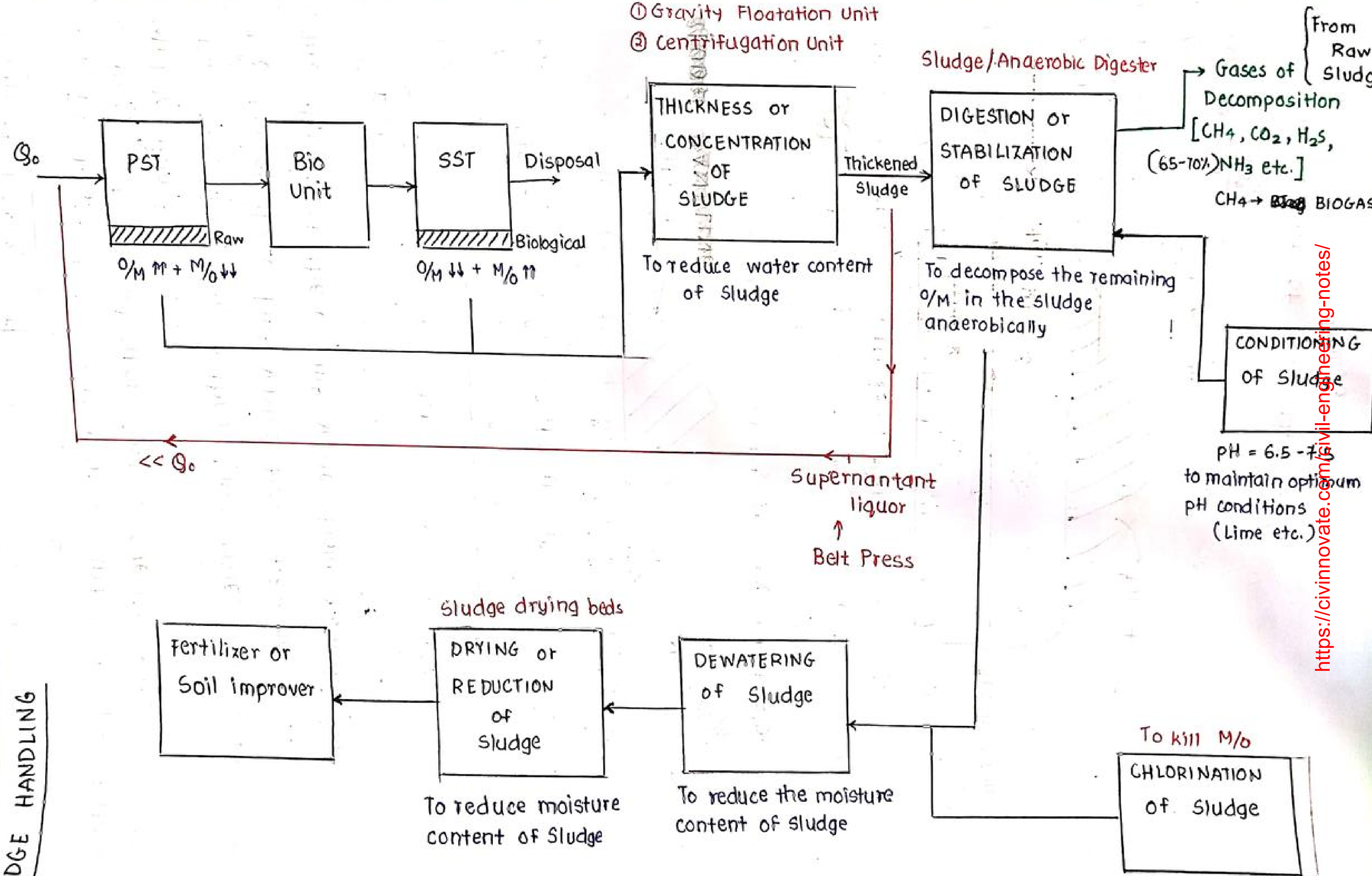


$V_o = \frac{Q}{\pi D^2/4}$

SLR → Amt. of O/M loaded /m³/d

kg/m³/d

SLUDGE HANDLING



Sludge Digestion

In the process of Sludge Digestion, the incoming sludge gets broken down into following 3 forms :

1> Gases of Decomposition

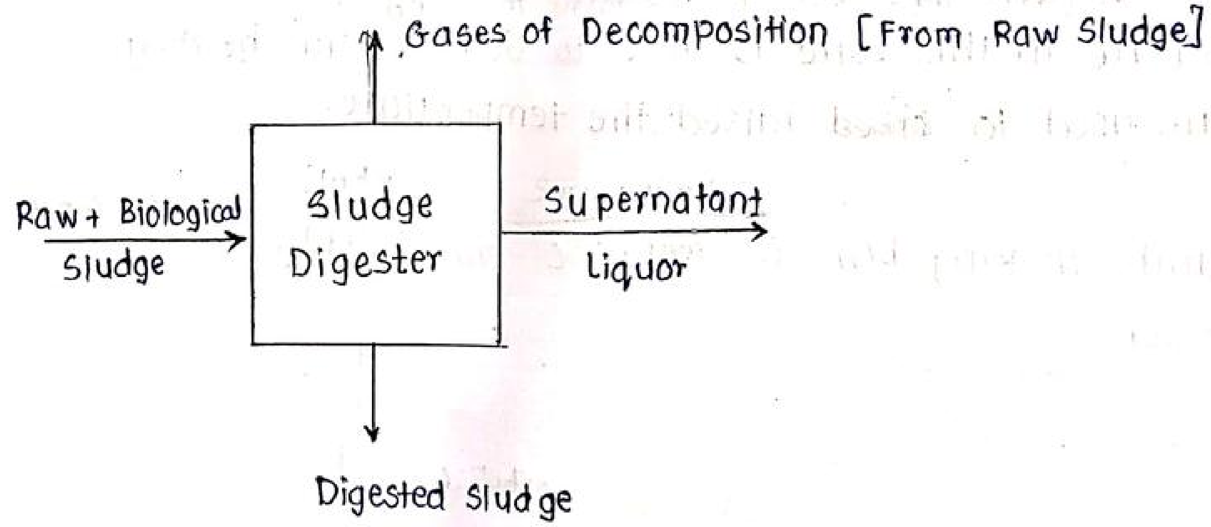
- Gases like Methane (CH_4) & Carbon dioxide is produced along with traces of other gases like : H_2S, NH_3 etc.
- Methane has a high calorific value & is use as a fuel

2> Supernatant liquor

- It is a viscous liquid of High Turbidity which comes to the top during the Detention Time of the sludge in the digester.

3> Digested Sludge

It is a stable & decomposed mass having reduced volume usually, the volume of digested Sludge is found to be 1/3rd the volume of undigested Sludge.



Factors Affecting Sludge Digestion

1) pH

- To oxidise the organic unstable acid produced during Anaerobic deformation, Methane formers are essential. the Methane formers requires an optimum pH of 6.5 to 7.5.
- If the Methane Formers do not operate, it leads to further accumulation of acid.
- The acidic conditions are avoided by conditioning the sludge with lime.

2) Temperature

Following zones of Digestion are define w.r.t Temperature.

(a) Zone of Mesophilic Digestion

- It is a medium temperature zone in which digestion is brought about by common mesophilic microorganism.
- The temperature in this zone is 25°c to 45°c which is easily available in most parts of India.

(b) Zone of Thermophilic Digestion

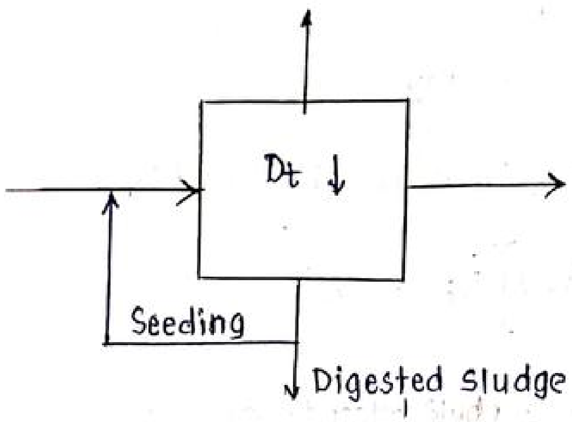
It is a high temperature zone in which digestion is brought about by heat loving Thermophilic micro organism.

The temperature in this zone is 40°c to 60°c & thus heating elements are used to ~~raised~~ raised the temperature.

NOTE: The units working b/w 40 to 50°c have higher operational cost but lesser

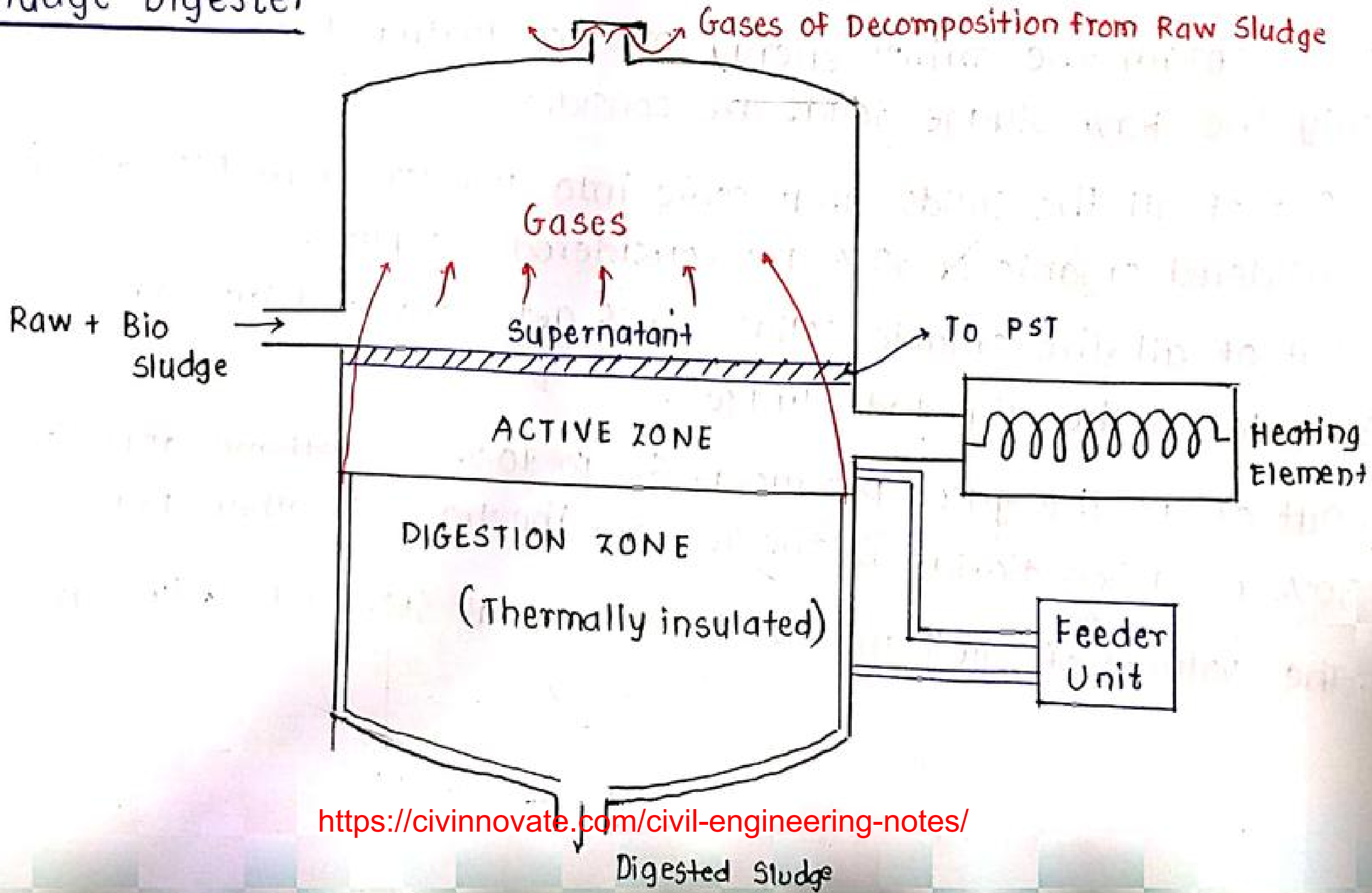
3> Seeding with Digested Sludge

- The sludge digestion Tank can be seeded with digested Sludge to ensure sufficient quantity of microorganism in the system.
- If the provision for recirculation of sludge is provided, it is called as high rate sludge ~~digester~~ digester & if no provision, it is called as standard Rate Sludge Digester.



High Rate Sludge Digester

Sludge Digester



1) Dia. = 6 - 18 m

2) Height = 6 - 12 m

3) Dt = 15 - 30 days (depends on Temp)

4) Volume of the Tank is found out by 'FAIR ET AL' formula

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

$V_1 \rightarrow$ Vol. of Sludge fed/day

$V_2 \rightarrow$ Vol. of Slude withdrawn/day

5) If Monsoon or winter storage is considered

$$\boxed{V = \left[V_1 - \frac{2}{3}(V_1 - V_2) \right] Dt + V_2 T}$$

$T \rightarrow$ No. of days of monsoon or winter or both

Min^m Energy Production in Sludge Digestion

1. To obtain the min^m energy produce during sludge Digestion, only the Raw Sludge Solids are considered.

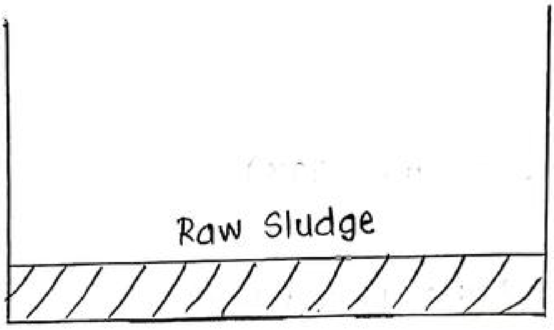
2. Out of all the Solids that come into digestor from PST, 70% are considered organic & 30% are considered inorganic.

3. Out of all the organic solids, 65% gets converted into gases & 35% forms the digested Sludge.

4. Out of all the gases produced, 65 to 70% is methane, approximately 30% is carbon dioxide & the rest are the traces of other gases.

5. The volume of methane is 0.9 m^3/kg at Std. Temp. & Pressure.

6. The calorific value of methane is 8600 kcal/m^3 or 36000 kJ/m^3.



Solids + water
(100 kg)

30% Inorganic
(30 kg)

70% Organic
(70 kg)

35% wt - Digested Sludge
$(0.35 \times 70) = 24.5$ kg

65% wt. - Gases
$(0.65 \times 70) = 45.5$ kg

65% (wt.) - CH_4 ($45.5 \text{ kg} \times 0.65 = 29.57$ kg)

≃ 30% - CO_2

Traces of other gases

STP (0°c, 1 atm)

$V_{CH_4} = 0.9$ m^3/kg

$[\therefore V_{CH_4} = 29.57 \times 0.9 \text{ m}^3 = 26.61 \text{ m}^3]$

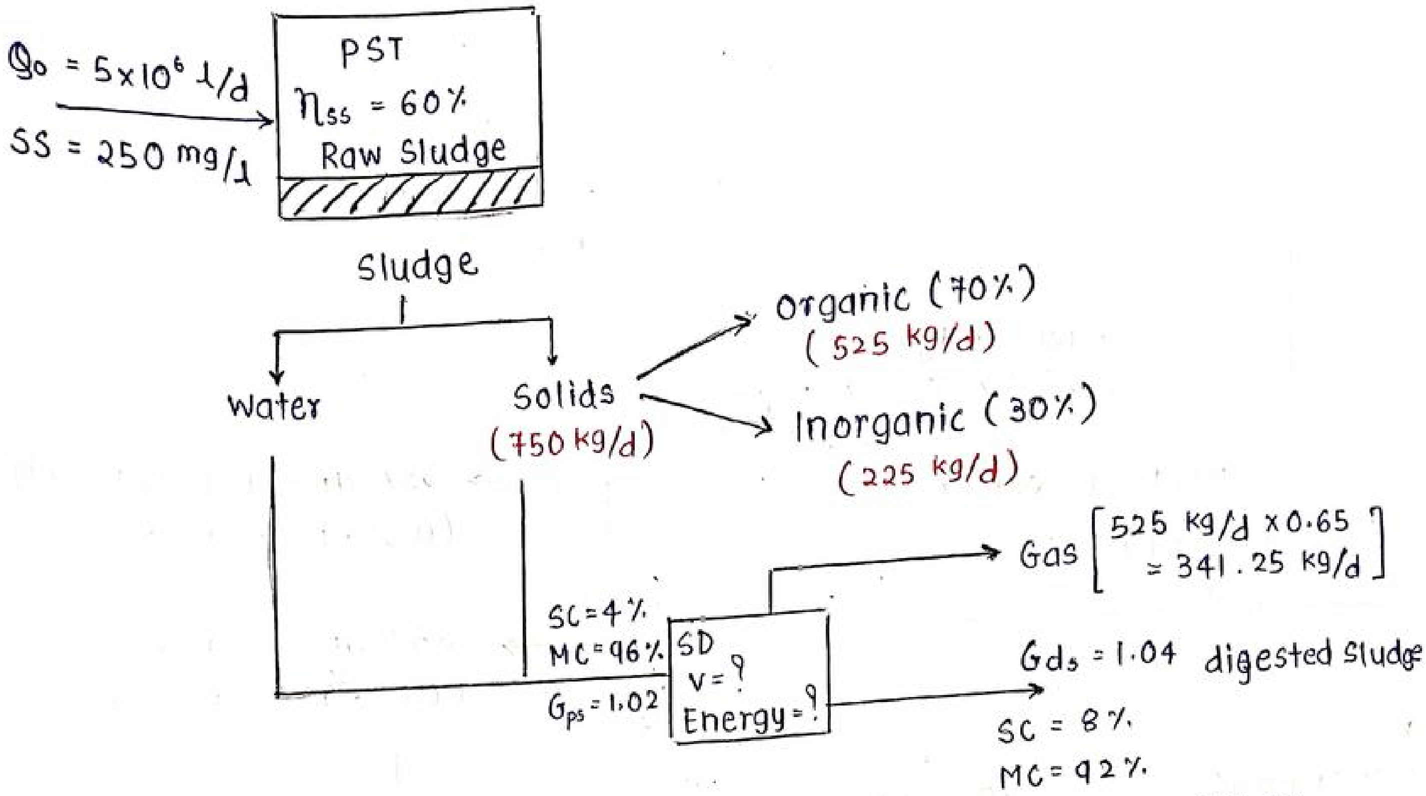
Energy = 8600 k cal/m^3
= 36000 kJ/m^3

1 Cal. = 4.184 J

$\therefore$ Energy = 8600 k cal./m^3 $\times$ 26.61 m^3 = 2.28×10^5 k cal.
= 36000 kJ/m^3 $\times$ 26.61 m^3 = 9.57×10^5 kJ

Pg. No. 127 (WB)

Q.55>



M_{solids} = 750 − 341.25
= 408.75 kg/d

Total qty. of S.S settled/day $= \dfrac{250 \text{ mg/l} \times 0.6 \times 5\times10^6 \text{ l/d}}{10^6 \text{ mg/kg}}$

= 750 kg/d

M_{sludge} produced/day $= 750 \text{ kg/d} \times \dfrac{100}{4} = 18750 \text{ kg/d}$

$$1.02 = \left(\frac{M_{sludge}}{V_{sludge}}\right) \times \frac{1}{\rho_w}$$

$$= \frac{18750 \text{ kg/d}}{V_{sludge} \times 1000 \text{ kg/m}^3}$$

$$\boxed{V_{sludge} = 18.38 \text{ m}^3\text{/d} = V_1}$$

$V_2 \rightarrow$ Volume of digested Sludge withdrawn per day

$M_{solids} = 408.75$ kg/d in digested Sludge

$M_{digested\ sludge} = 408.75 \text{ kg/d} \times \frac{100}{8} = 5109.375 \text{ kg/d}$

$$G_{ds} = 1.04 = \frac{M_{dig.\ Sludge}}{V_{dig.\ Sludge}} \times \frac{1}{\rho_w}$$

$$V_{digested\ sludge} = \frac{5109.375 \text{ kg/d}}{1.04 \times 1000 \text{ kg/m}^3} = 4.91 \text{ m}^3/\text{d} = V_2$$

By 'FAIR ET AL' Formula :....

$$V = \left[18.38 \frac{m^3}{d} - \frac{2}{3}(18.38 - 4.91)\frac{m^3}{d}\right] \times 15d = 141 m^3$$

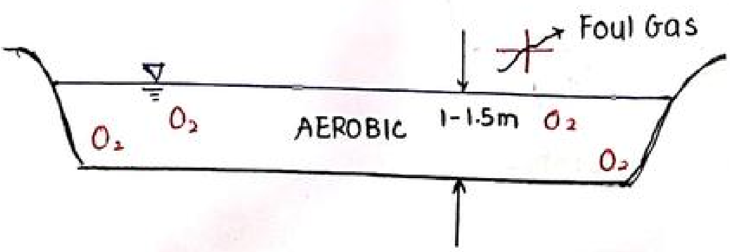
For 100 kg solids present in raw sludge, min^m energy produced $= 2.28 \times 10^5$ kcal.

$\therefore$ For 750 kg/d solids, energy produced $= 2.28 \times 10^5 \text{ kcal} \times 750 \text{ kg/d}$
$= 17.1 \times 10^5$ kcal/d.

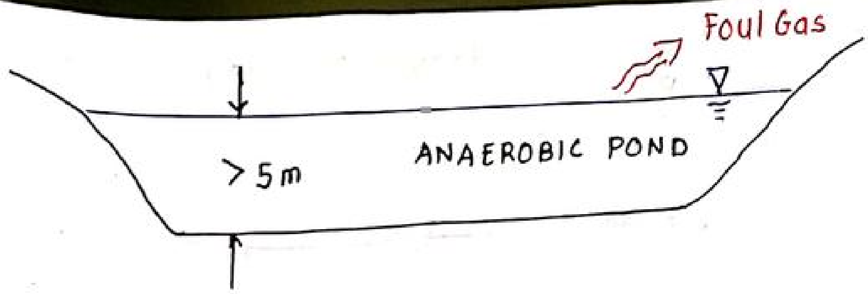
* Waste Stabilization Pond/Oxidation Pond

- Oxidation Ponds are low cost shallow earthern basins which are used to decompose the organic matter present in the waste water.
- Oxidation Ponds are classified on the basis of depth as follows.

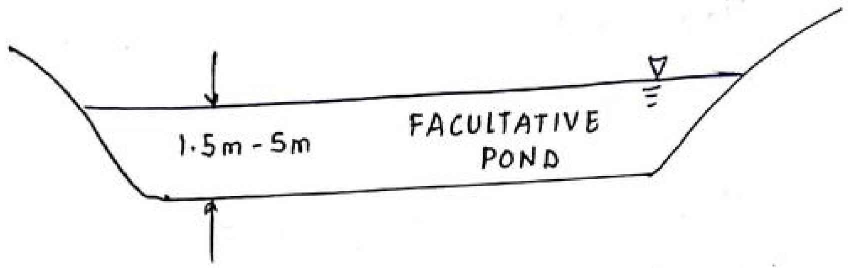
1)



2)

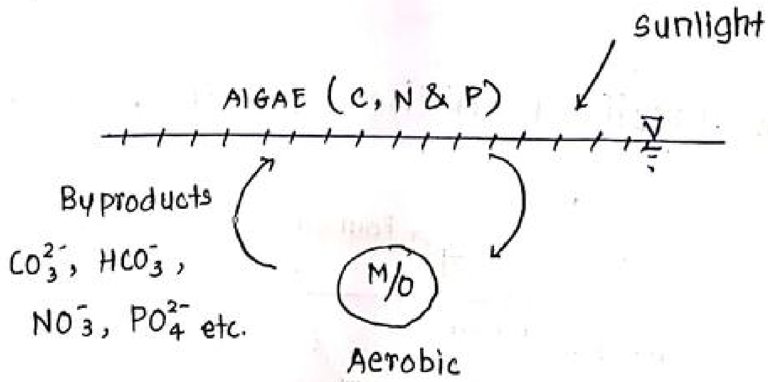


3)



- In Totally Aerobic Pond of Shallow depth, the decomposition is brought about by Aerobic bacteria which flurish in the presence of Oxygen.
- Their exists a mutual relationship b/w Algae & microorganism present in the pond which is referred as Algal Symbiosis.
- Algal Symbiosis is a phenomenon in which Algae by growing in presence of sunlight produces oxygen due to photosythesis & this oxygen is utilised by microorganism to decompose the organic matter present in waste water.
- End Products of decomposition are useful for the growth of Algae.

NOTE : This phenomenon is observed only in the presence of Sunlight



1. $CO_2 + H_2O \rightleftharpoons H_2CO_3$

2. $H_2CO_3 \rightleftharpoons H^+ + HCO_3^-$

3. $HCO_3^- \rightleftharpoons H^+ + CO_3^{2-}$

4. $CO_3^{2-} + H_2O \rightleftharpoons OH^- + HCO_3^-$

- When Algae grows in large numbers, it starts consuming appreciable quantity of bicarbonate due to which accumulation of OH^- ions takes place. This increases the pH of the water present in the Pond (Reaction '4'). Thus when a water body is infested with Algae, its pH is usually observed to be greater than '9'.

Design Data

1) Oxidation Pond is installed inconjunction with screen
2) Depth of Pond is 1 - 1.5m
3) Detention Time is 2 to 6 weeks
4) L/B Ratio is 2 to 4
5) Area of each unit is 0.5 to 1 Ha.
6) It is to be adopted for domestic sewage only.
7) The surface Area of the pond is computed by organic ~~bod~~ loading Rate (OLR) as follows :

Latitude	Max^m OLR (kg of BOD/ha/d)
8°	325
12°	300
16°	275
20°	250
24°	225
28°	200
32°	175
36°	150

8) Freeboard = 0.3 m

9) Detention Time can be found out by following eq^n

$$\boxed{D_t = \frac{1}{K} \ln \left(\frac{BOD_i}{BOD_e} \right)}$$

k → deoxy. const. (base 'e')

BOD_i → influent BOD

BOD_e → effluent BOD

$$\boxed{OLR/SLR = \frac{\text{kg of BOD loaded onto the pond/day}}{\text{S.A of the Pond}}}$$

Pg. No. 128 (WB)

Q.58> Population = 20,000

$q = 150\ l/c/d$

BOD w/w = 150 mg/l

∵ The town doesn't have any source of power, oxidation pond is preferred.

Max^m Daily Supply $= 1.8 \times 150\ l/c/d \times 20000\ c$

$= 5.4 \times 10^6\ l/d$

Assume that 80% of water supplied gets converted to sewage

Max^m Daily Production of Sewage $= 5.4 \times 10^6\ l/d \times 0.8$

$= 4.32 \times 10^6\ l/d$

Let OLR = 150 kg/ha/d

$$150\ kg/ha/d = \frac{150\ mg/l \times 4.32 \times 10^6\ l/d}{10^6\ mg/l \times \text{Surface Area required}}$$

SA = 4.32 ha.

Provide 5 units of $\frac{4.32}{5}$ ha. = 0.864 ha. {∵ Area is b/w 0.5 - 1 ha. it is ok}

$\frac{L}{B} = 3$

$L = 3B$

$3B^2 = 8640\ m^2$

$\therefore B = 53.65\ m$

$\therefore L = 53.65 \times 3$

$= 161\ m$

Assume H = 1.5 m

$V = LBH = 8640\ m^2 \times 1.5\ m = 12960\ m^3$

Detention Time of each unit,

$$D_t = \frac{V}{Q} = \frac{12960}{\left(\frac{4.32\times10^3}{5}\ m^3/d\right)} = 15 \text{ days}$$

~~i.e. Dt is between~~

[∵ Dt is between 2 - 6 weeks, design is OK]

Final Dimensions

Providing 5 units of $L = 161m$, $B = 53.65m$, $H = 1.5m$
& Freeboard $= 0.3m$.

Pg. No. 127 (WB)

Q. 50)

2000 —— 0 $\eta\uparrow$

2000 —— 20 $\eta\downarrow$

BOD_i BOD_e

$$D_t = \frac{1}{k}\ln\left(\frac{BOD_i}{BOD_e}\right)$$

$k_{20°C} = 0.3\,d^{-1}$

$k_{24°C} = 0.3\,d^{-1}(1.047)^{24-20}$

$= 0.36\,d^{-1}$

$$= \frac{1}{0.36d^{-1}}\ln\left[\frac{2000}{20}\right]$$

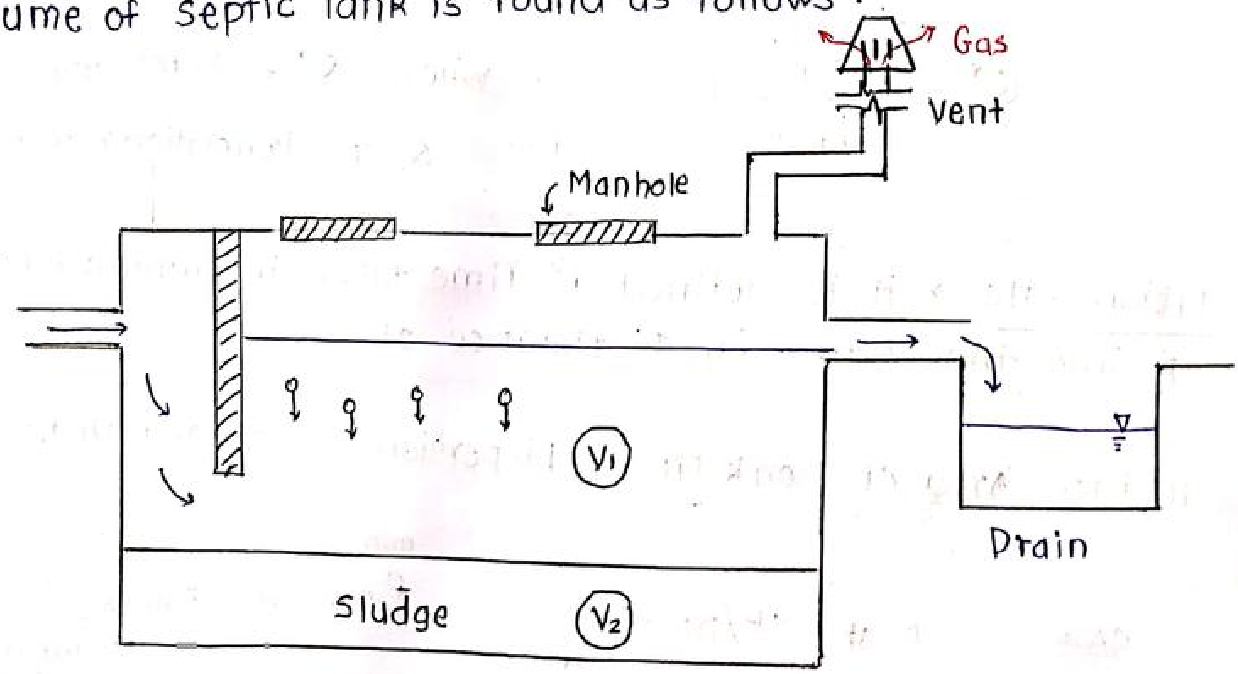
$\therefore D_t = 12.77$ days

SEPTIC TANK

Septic Tank is an Anaerobic unit which works in absence of oxygen. The Raw Sewage is directly fed into these tank in which the solids settle down resulting in the formation of Sludge. Various gases of decomposition are released through a Vent Pipe.

Design Data

1) It is Designed for a Maximum of 300 people
2) The Rate of flow of Sewage is taken as 40-70 l/c/d.
3) The Rate of accumulation of Sludge is 30-70 l/c/yr.
4) The Detention Time required in the settling zone is 12-36 Hrs.
5) The optimum Cleaning interval is 6-12 months.
6) $\frac{L}{B} = 2$ to 4
7) The depth of the Tank is 1m to 2m
8) Free Board = 0.3 m
9) Volume of septic Tank is found as follows :



Volume of ST, $V = V_1 + V_2$

$V_1 \rightarrow$ Volume of Settling zone

$V_2 \rightarrow$ Volume of Sludge Zone

V_1 = Rate of flow of Sewage × Detention Time required

V_2 = Rate of ~~Flow~~ Accumulation of sewage × Cleaning Interval

- The effluent of Septic Tank is ~~disposed~~ disposed in the drains which ultimately delivers water in soak pits or dispersion Trenches

- Soak Pits are deeper as compare to dispersion trenches but have a smaller surface Area. These arrangements are made sufficiently permeable so as to allow easy percolation into the ground water.

- The maximum rate of application of sewage effluents is given by following empirical equation

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

where, $Q^* = l/m^2/day$

& t = Percolation rate in min

Percolation Rate → It is defined as Time taken in minute by water to seep into the ground by a distance of 1 cm.

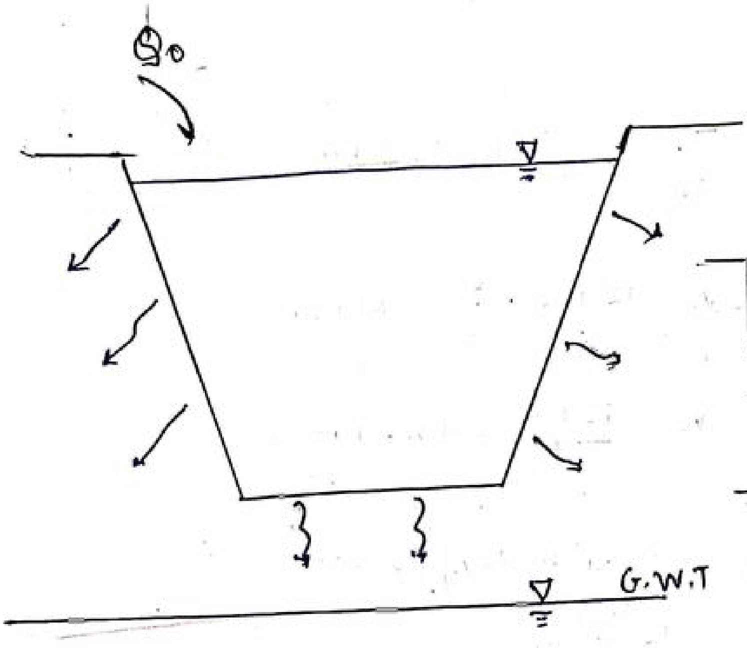
- The surface Area of Soak Pit or Dispersion trench required is given by

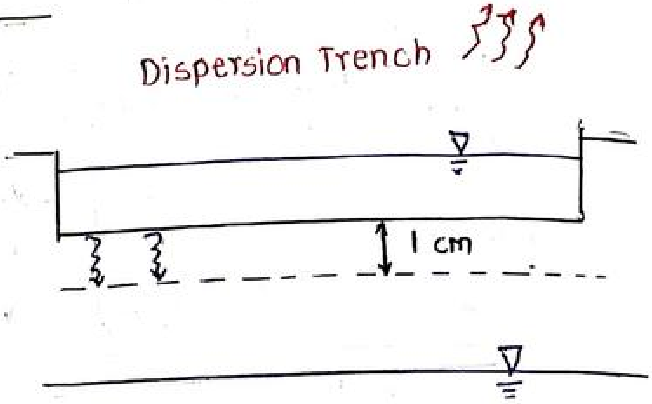
~~SA~~ $$\boxed{SA \text{ of } SP/DT = \frac{Q_0}{Q^*}}$$

min/cm

Percolation Rate $\propto \frac{1}{\text{Infiltration Rate}}$

cm/min

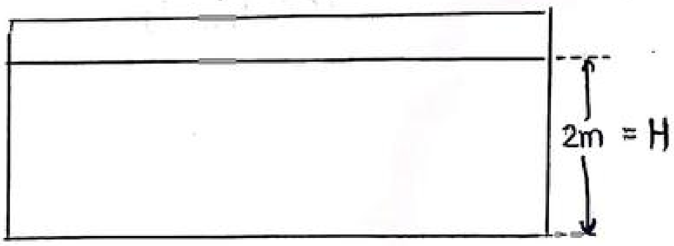




Pg. No. 128 (WB)

Q. 59) $\frac{L}{B} = 2.25$

Liquid depth = 2m



Population = 300

$q = 100$ l/c/d

Max^m Daily Demand = 1.8×100 l/c/d $\times 300$ c
= 54000 l/d

Assuming there is no alternative treatment method available.

∴ Design Discharge into ST = 54000 l/d × 0.8
= 43200 l/d

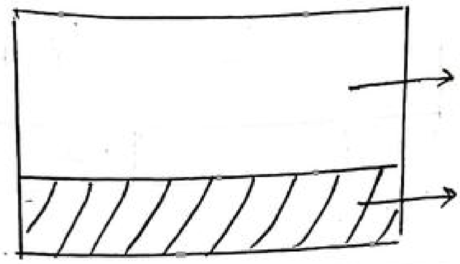
$V = Q D_t$
= 43200 l/d × 3d
= 129600 l
= 129.6 m³

$2.25\,B^2 \times H = 129.6\ m^3$

$2.25\,B^2 \times 2m = 129.6\ m^3$

$\therefore\ B = 5.36m$

$L = 12.07\ m$



$V_1 = 129.6 \times \frac{2}{3} = 86.4\ m^3$

$V_2 = \frac{129.6}{3} = 43.2\ m^3$

Rate of Production of Sludge $= 0.04\ m^3/c/y \times 300\ c$

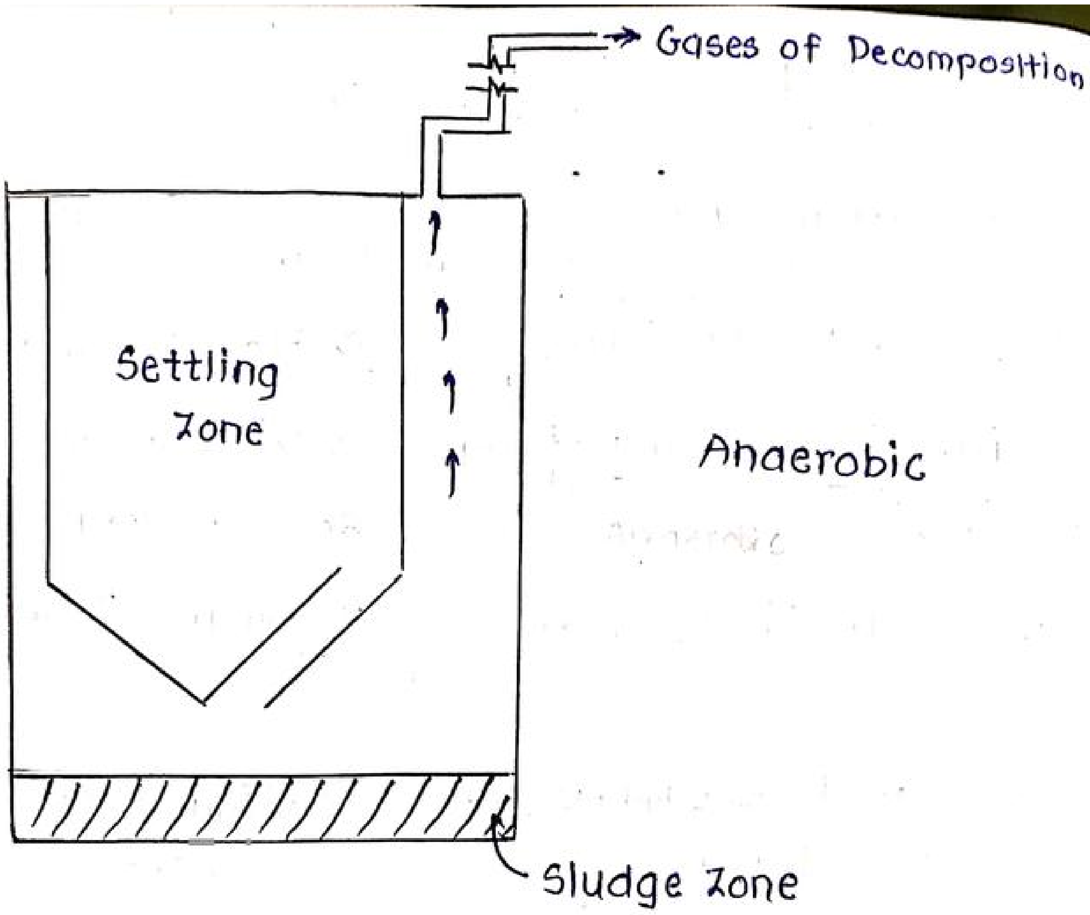
$= 12\ m^3/y$

Cleaning / desludging interval $= \frac{43.2\ m^3}{12\ m^3/y} = 3.6y$

Area of Dispersion Trench $= \frac{43200\ l/d}{100\ l/m^2/d} = 432\ m^2$

IMOFF TANK

- Imoff Tank is an anaerobic unit in which settling & Digestion are carried out in 2 different compartments it is also called as Two Storey Tank in which settling is carried out in the upper storey & the Digestion of Sludge is carried out in the lower storey
- The Gases are collected from the Top & used as fuel.
- It can be used for commercial Treatment.
- It is able to handle fluctuation.
- The Turbulence in the settling zone doesnot effect the sludge zone.


Gases of Decomposition
Settling Zone
Anaerobic
Sludge Zone

9. DESIGN OF SEWERS & SEWERAGE SYSTEM

Distribution System	Collection System
1> Flow is under Pressure	1> Flow is under Gravity
2> Concentration of Suspended Solid is less	2> Concentration of Suspended Solid is very high.
3> Wear & Tear/Abrasion of Pipes is less	3> Wear & Tear of pipes is more.
4> Material of Pipe can be relatively less hard	4> Material of Pipe should be more hard.

* Sequence of Steps for Constructing a Collection System

1> Marking of Alignment
2> Excavating the trenches
3> Bracing the sides of trenches
4> Sealing of Surface
5> Laying of P.C.C
6> Laying of Pipes
7> Joining of Pipes
8> Checking for Leaking → Pressure Test
- Air Test (For Large Pipes)
- Water Test (For small pipes)
  - → Fill the Pipe
  - → Check for leakage

9> Fill the Trenches

* Types of Sewer Systems

1) Sanitary

- It carries the sewage produce from Domestic & Industrial Activites.

2) Storm Water System

- It carries the surface runoff developed during the periods of rainfall. It is usually open & surface runoff enters directly into it.
- If Storm water System is closed, the Rain water can enters the sewer through various opening such as at sides of Pavements, Footpath etc.

3) Combined System

- It consists of a single sewers of larger diameter & carries domestic & industrial sewage along with surface runoff.

- Separate Sanitary & Storm water system is provided in the area of High Rainfall usually more than 150 days.
- A Combined system is usually provided in the areas of less & moderate rainfall. In such area combined system becomes economical because a single sewer system severs dual system.
- Circular sections are mostly adopted as sewer but other section such as Rectangular, square, U-Shaped, Egg Shaped sewer can also be used.
- The Circular section is preferred because

(i) It can be manufactured very easily.
(ii) It is Hydraulically more efficient.
(iii) Can be joined very easily.

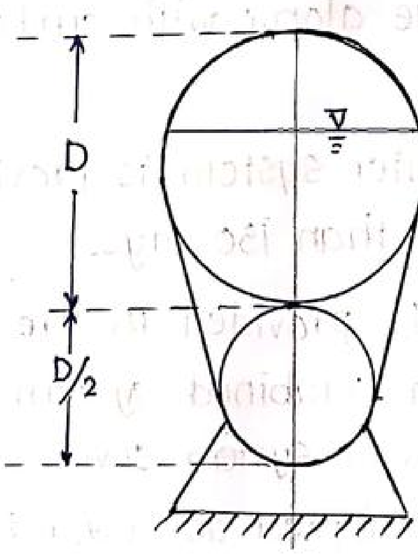
- However in low flow season circular section are not able to generate sufficient velocity due to which the sewer may get chocked. As a remedial measure the sewer are either designed to generate a minimum velocity even at low discharges or Provide a Egg shaped sewer.
- Egg shaped sewer can be used which can generate sufficient velocities at low discharges.
- However, egg shape sewers suffers from following Disadvantages.

(i) It is difficult to manufacture.

(ii) Requires more material.

(iii) Unstable

(iv) A smaller base resists the weight of upper broader section & Thus crushing stresses are high.



* Maximum & Minimum velocity in Sewers

- The velocity of flow in the sewers should be such that neither the suspended solid in the sewage gets deposited nor the sewage material gets scoured.
- The 1st limitation limits the minimum velocity & 2nd limitation limits the maximum velocity.
- The minimum velocity is called as self cleaning velocity which depends on the suspended solid prevents in the sewage.
- In Indian condition, a minimum velocity of 0.8 m/s is sufficient to flush out most of the suspended solid present in sewage.
- Maximum velocity is called Non scouring velocity which depends upon the material of sewer.
- As per G.O.I manual, V_{max} for

$$CI \rightarrow 3.5 - 4.5 \ m/s$$
$$CC/RCC \rightarrow 2.5 - 3.5 \ m/s$$

* Shield's Self Cleansing Velocity

As per shield, a minimum horizontal velocity should be maintain to keep a particle of Dia. 'd' & Sp. Gravity 'G_s' in suspension

$$\boxed{V_{Hmin} = V_{self} = C\sqrt{Kd(G_s - 1)}}$$

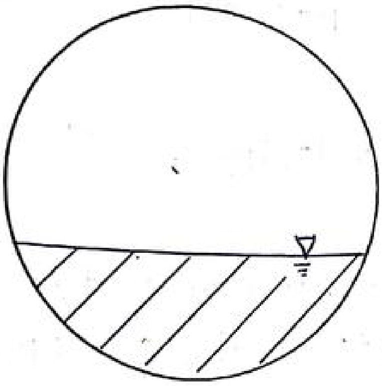
$d \rightarrow$ diameter of Particles

$G_s \rightarrow$ Specific Gravity of Particle

$C \rightarrow$ Chezy's Constant

$K \rightarrow$ Shield's constant (dimensionless)
$\hookrightarrow$ 0.04 - 0.08

* Flow Velocity in sewers



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

$V \rightarrow$ Flow velocity at the section

$n \rightarrow$ Manning's constant

$\hookrightarrow$ Manning's Roughness /

$n \propto f$

$R \rightarrow$ Hydraulic Radius

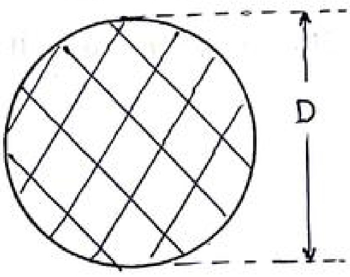
$$R = \frac{\text{wetted Area}}{\text{wetted Perimeter}} = \frac{A}{P}$$

$S \rightarrow$ Slope of TEL

Assuming the Flow is Steady & uniform :-

$S \rightarrow$ Bed slope $\left\{ \text{Slope of TEL} = \text{slope of HGL} = \text{Slope of Bed} \right\}$

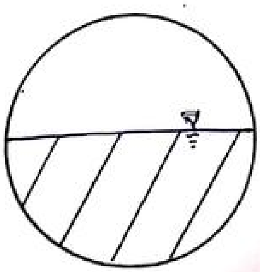
CASE I : Full Flow Condition



$$R = \frac{\pi D^2/4}{\pi D}$$

$$R = D/4$$

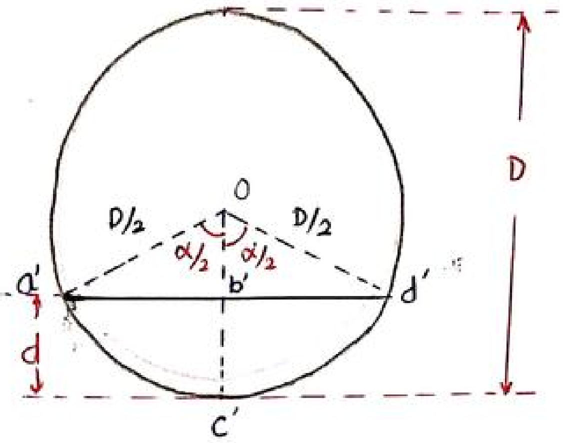
CASE II : Half Flow Condition



$$R = \frac{\pi D^2/8}{\pi D/2} \qquad R = D/4$$

23/11/19

CASE III :



$\frac{d}{D}, \frac{a}{A}, \frac{p}{P}, \frac{r}{R}, \frac{v}{V}, \frac{q}{Q}$ → Actual flow Parameter ; ↘ Full Flow Parameter

(i) $\frac{d}{D}$

$d = oc' - ob'$

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

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

(ii) $\frac{p}{P} = \frac{\alpha}{360}$

(iii) $\frac{a}{A}$

$a = ar(\Delta oa'c'd') - ar(\Delta oa'b'd')$

$\frac{\pi D^2}{4}$ — $360°$

? — α

$\sin 2\alpha = 2\sin\alpha\cos\alpha$

$\sin\alpha = 2\sin\frac{\alpha}{2}\cos\frac{\alpha}{2}$

$ar(\Delta oa'c'd') = \frac{\alpha}{360} \times \frac{\pi D^2}{4}$

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

$ar(\Delta oa'b'd') = 2 \times \frac{1}{2} \times \frac{D}{2}\cos\alpha\ \frac{D}{2}\sin\frac{\alpha}{2}$

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

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

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

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

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

(iv) $\frac{r}{R}$

$$\frac{r}{R} = \frac{a/p}{A/P}$$

$$\frac{r}{R} = \frac{a/A}{p/P}$$

$$\boxed{\frac{r}{R} = \frac{\left[\alpha/360 - \frac{\sin\alpha}{2\pi}\right]}{\alpha/360}}$$

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

If sewer is same, $s = S$ & $n = N$

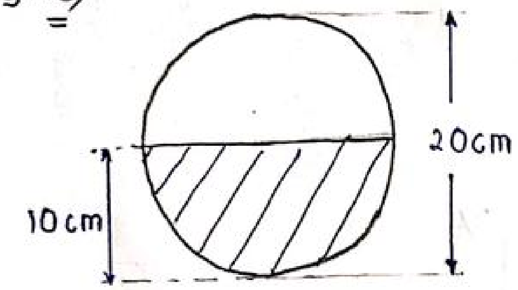
$$\boxed{\frac{v}{V} = \left(\frac{r}{R}\right)^{2/3}}$$

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

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

Pg. No. 119 (WB)

Q. 18>



$S = 0.004$

$q = ?$

$q = av$

$$= \left(\frac{\pi D^2}{8}\right)\left(\frac{1}{n} \times \left(\frac{D}{4}\right)^{2/3} \times S^{1/2}\right)$$

$$= 9.63 \times 10^{-3} \ m^3/s$$

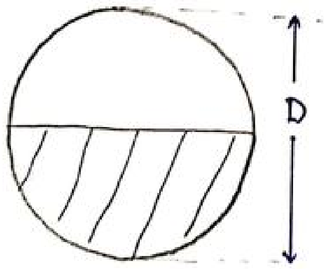
$$= 9.63 \ l/s$$

$$Q_{half\ full} = \frac{Q_{full}}{2}$$

$$v_{half\ full} = v_{full}$$

Pg. No. 119 (WB)

Q. 19>



$q = 0.08 \ m^3/s$

$$S = \frac{5m}{1\ km} = \frac{5m}{1000\ m}$$

$n = 0.013$

$$0.08 = \left(\frac{\pi D^2}{8}\right)\left[\frac{1}{0.013} \times \left(\frac{D}{4}\right)^{2/3} \times \left(\frac{5}{1000}\right)^{1/2}\right]$$

$D = 0.4126$ m

$$v = \frac{1}{0.013} \times \left(\frac{0.4126}{4}\right)^{2/3} \times \left(\frac{5}{1000}\right)^{1/2}$$

$$= 1.196 \text{ m/s}$$

Pg. No. 119 (WB)

Q.20)

$$\frac{d}{D} = \frac{1}{2}\left(1 - \cos\frac{\alpha}{2}\right) = 0.3$$

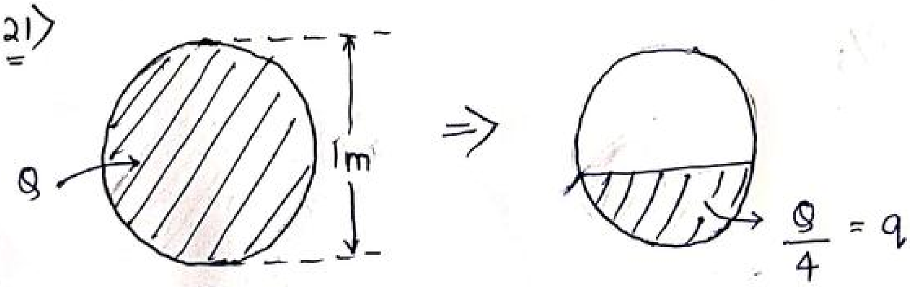
$$\alpha = 132.8^\circ$$

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

$$= 0.195$$

Pg. No. 119 (WB)

Q.21)



$$\frac{q}{Q} = 0.25$$

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

$$\therefore \alpha = 142.84^\circ$$

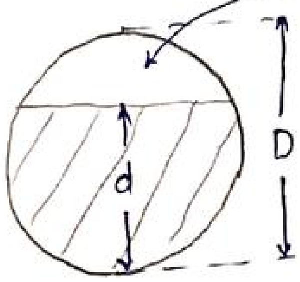
$$\frac{r}{R} = \frac{\left(\frac{142.84}{360} - \frac{\sin(142.84)}{2\pi}\right)}{142.84/360} = 0.7577$$

$$\frac{v}{V} = \left(\frac{r}{R}\right)^{2/3} = (0.7577)^{2/3} = 0.8311$$

Pg. No. 119 (WB)

Q. 22>

To accomodate Fluctuations



$S = \frac{1}{625}$

$N = 0.01$

$P = 36,000$

$q = 135$ l/c/d

$S = \frac{1}{625}$

$Q_{act} = 4\, q_{avg.}$

$N = 0.01$

$\frac{d}{D} = \frac{3}{4} = 0.75$ $\quad$ $V = ?$

Design Discharge $(q) = 4 \times 0.8(36{,}000 \times 135$ l/c/d$)$

$q = 15.552 \times 10^6$ l/d $\quad$ [At 3/4th Depth]

$$Q = A \times v$$

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

$$0.75 = \frac{1}{2}\left(1 - \cos\frac{\alpha}{2}\right)$$

$$1.5 = 1 - \cos\frac{\alpha}{2}$$

$$\cos\frac{\alpha}{2} = 1 - 1.5$$

$$\frac{\alpha}{2} = 120 \qquad \therefore \alpha = 240$$

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

$$\frac{15.552 \times 10^6}{Q} = 0.804 \times 1.13$$

$\cancel{Q = 17.41 \times 10^6 \ m^3/d}$ $Q = 17.052 \times 10^6 \ m^3/d$

$$Q = \frac{17.052 \times 10^6}{86400} \ m^3/s$$

$$Q = 0.197 \ m^3/s$$

$$Q = A \times V$$

$$0.197 = \frac{\pi}{4} D^2 \times \left[\frac{1}{n} \times R^{2/3} \times S^{1/2} \right]$$

$$0.197 = \frac{\pi}{4} \times D^2 \times \frac{1}{0.01} \times \left(\frac{D}{4} \right)^{2/3} \times \left(\frac{1}{625} \right)^{1/2}$$

$$D = 0.5 m$$

$$v = \frac{1}{n} r^{2/3} \times S^{1/2}$$

$$= \frac{1}{0.01} \times r^{2/3} \times \left(\frac{1}{625} \right)^{1/2}$$

$$\frac{r}{R} = \frac{\left(\frac{\alpha}{360} - \frac{\sin\alpha}{2\pi} \right)}{\alpha/360} = 1.206$$

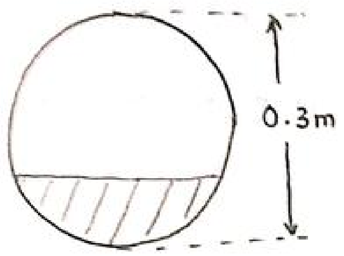
$$\frac{r}{R} = \frac{D}{4} \qquad \therefore R = \frac{0.5}{4} \times 1.206 = 0.15$$

$$v = \frac{1}{0.01} \times (0.15)^{2/3} \times \left(\frac{1}{625} \right)^{1/2}$$

$$\therefore v = 1.13 \ m/s$$

Pg. No. 1107(WB)

Q.2)



$S = \frac{1}{280}$

$q = 1728 \ m^3/d$

$$Q = \frac{\pi D^2}{4} \times \frac{1}{n} \left[\frac{D}{4}\right]^{2/3} S^{1/2}$$

$$= \frac{\pi 0.3^2}{4} \times \frac{1}{0.015} \left(\frac{0.3}{4}\right)^{2/3} \times \sqrt{\frac{1}{280}}$$

$$= 0.05 \ m^3/s$$

$$= 4320 \ m^3/d$$

$$\frac{q}{Q} = \frac{1728}{4320} = 0.4$$

$$\frac{d}{D} = 0.4$$

$$d = 0.4 \times 0.3$$

$$= 0.12 \ m = 120 \ mm$$

$\frac{v}{V} = 0.7$ (From Graph)

$$v = 0.7 \times V$$

$$= 0.7 \times \frac{1}{0.015} \times \left(\frac{0.3}{4}\right)^{2/3} \times \sqrt{\frac{1}{280}}$$

$$= 0.49 \simeq 0.5 \ m/s$$

* SEWER DESIGN

1) 80 % of supplied water gets converted to sewage.

2) Sewers are never designed to Run full in order to accomodate fluctuations.

$\frac{d}{D}$	D
$\frac{d}{D} = 0.5$	$D \leq 400$ mm
$\frac{d}{D} = 0.67 = \frac{2}{3}$	$400\text{ mm} < D \leq 900\text{mm}$
$\frac{d}{D} = 0.75 = \frac{3}{4}$	$D > 900\text{mm}$

3) Sewers are designed to carry maximum hourly flow & are checked for generation V_{self} at minimum hourly flow.

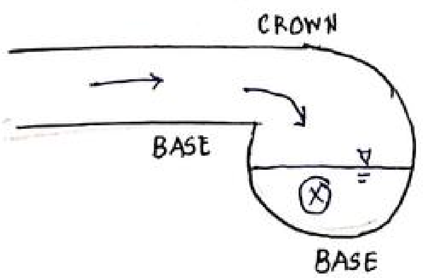
4) Maximum Daily Discharge = 2 × Avg. Daily Discharge

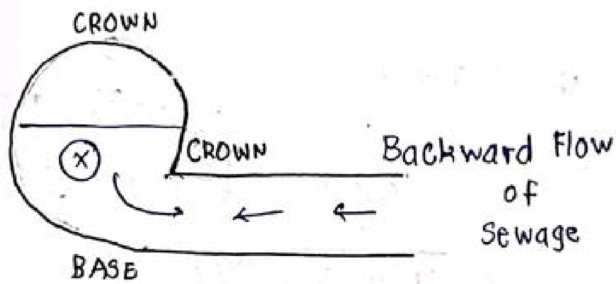
Maximum Hourly Discharge = $1.5 \times 2 \times \left[\frac{\text{Avg. Daily Discharge}}{24}\right]$

Minimum Daily Discharge = $\frac{2}{3}$ × Avg. daily discharge.

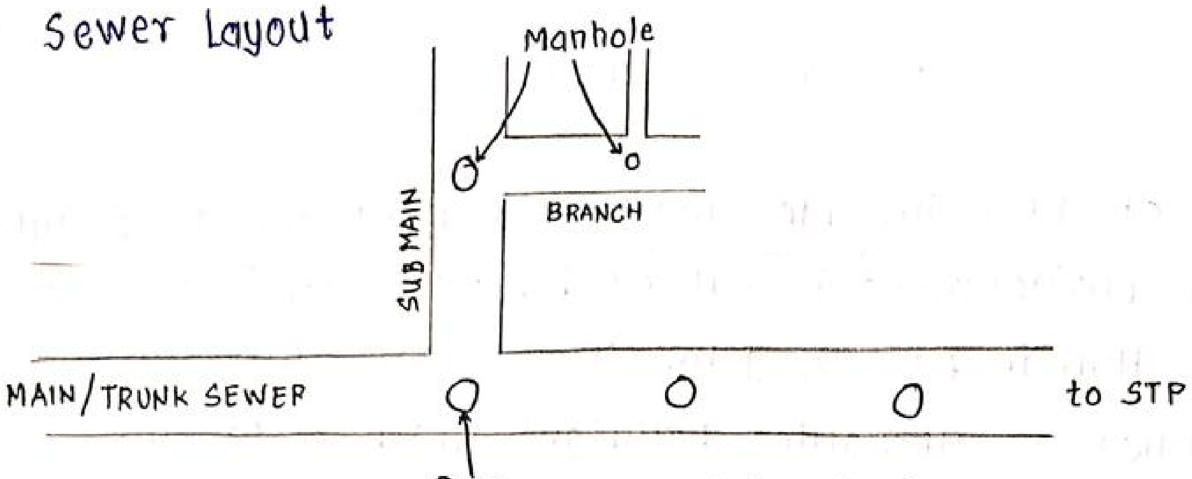
Minimum Hourly Discharge = $\frac{1}{2} \times \frac{2}{3} \times \frac{\text{Avg. daily discharge}}{24}$

5)





6> Sewer Layout



Drop Manhole → large in size
→ C, I & M is facilitated by dropping inside the sewer.

O → Manhole
→ Easy cleaning, inspection & maintainance

12. NOISE POLLUTION

The air act of 1981 includes noise as one of the air pollutants. Sound in the environment is caused by vibrations in the air that reaches Human ears & generate a sense of hearing. Noise is defined as unwanted pollutant which ~~de~~ produce undesirable physiological & psycologinal effect ~~with~~ in the individual by interferring one's social activities like work, Rest, Sleep etc.

NOTE : Noise is subjective & Transiant in Nature

* Sound & Its Measurement

$P_{min} = 20\,\mu Pa = P_{rms_0}$

$I_{min} = 10^{-12}\ W/m^2 = I_0$

Sound level - L

$$\boxed{L = \log_{10}\left(\frac{I}{I_0}\right)} \quad \text{Bel (B)}$$

$$\boxed{L = 10\log_{10}\left(\frac{I}{I_0}\right)} \quad \text{deciBel (dB)}$$

$$I = \frac{p^2}{\rho c}$$

$I \rightarrow$ Intensity of sound (W/m^2)

$p \rightarrow$ root mean square pressure (Pa)

$\rho \rightarrow$ Density of medium

$c \rightarrow$ velocity of sound in the medium

$$L = \log_{10}\left[\frac{P_{rms}^2 / \rho C}{P_{rms0}^2 / \rho C}\right]$$

$$\boxed{L = 2\log_{10}\left[\frac{P_{rms}}{P_{rms0}}\right]}$$ Bel

$$\boxed{L = 20\log_{10}\left[\frac{P_{rms}}{P_{rms0}}\right]}$$ deciBel (dB)

Addition of Sound

i) 50 dB + 50 dB

(1) $50\text{ dB} = 20\log_{10}\left[\frac{I}{10^{-12}}\right]$

$$I = 10^{-7}\ \text{W/m}^2$$

$$I_{total} = I_1 + I_2$$
$$= 2\times 10^{-7}\ \text{W/m}^2$$

$$L = 10\log_{10}\left[\frac{2\times10^{-7}}{10^{-12}}\right]$$

$$L = 53.01\ \text{dB}$$

NOTE :

$x\text{ dB} + x\text{ dB} = (x+3)\text{ dB}$

(ii) $50 = 20\log_{10}\left[\frac{P_{rms}}{20\mu Pa}\right]$

$$P_{rms} = 6324.55\ \mu Pa$$

$$I_{Total} = I_1 + I_2 + \dots\dots\dots$$

$$\frac{P_{rms\,total}}{\rho c} = \frac{P_{rms_1}^{2}}{\rho c} + \frac{P_{rms_2}^{2}}{\rho c} + \cdots\cdots$$

$$\boxed{P_{rms\,total} = \sqrt{P_{rms_1}^{2} + P_{rms_2}^{2} + \cdots}}$$

$$P_{rms\,total} = \sqrt{6324.55^2 + 6324.55^2} = 8944.27 \text{ Pa}$$

$$L = 20 \log_{10}\left(\frac{8944.27}{20}\right)$$

$$= 53.01 \text{ dB}$$

2) 100 dB + 90 dB

(i) $$100 \text{ dB} = 10 \log_{10}\left[\frac{I}{10^{-12}}\right]$$

$$I_{100dB} = 10^{-2} \text{ W/m}^2$$

$$I_{90dB} = 10^{-3} \text{ W/m}^2$$

$$I_{Total} = 10^{-2} + 10^{-3}$$

$$= 0.011 \text{ W/m}^2$$

$$L = 10 \log_{10}\left(\frac{0.011}{10^{-12}}\right)$$

$$= 100.4 \text{ dB}$$

3) 50 dB + 50 dB + 50 dB

$I_{total} = 3 \times 10^{-7}$ W/m²

$$L = 10 \log_{10}\left[\frac{3\times10^{-7}}{10^{-12}}\right]$$

$$= 54.77 \text{ dB}$$

NOTE : x dB + x dB + x dB
$= (x + 4.77)$ dB

Average Sound Levels

$$\bar{L} \text{ or } L_{avg} = 20 \log_{10}\left\{\frac{1}{N}\left(\sum_{n=1}^{n=N} 10^{\,L_n/20}\right)\right\}$$

$L_n \rightarrow n^{th}$ sound level in dB

Equivalent Sound Level (L_{EQ})

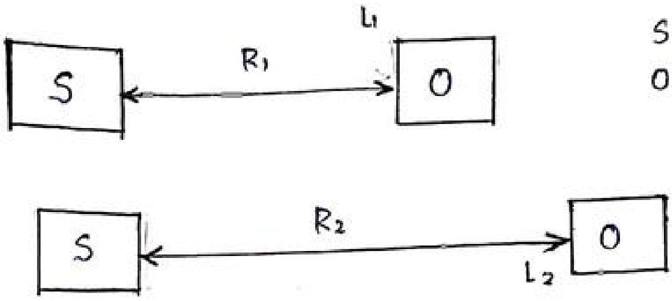
L_{EQ} is defined as the constant sound level which over a given time produces the same energy as is produced by flucctuating sound levels over the same line.

$$L_{EQ} = 10 \log_{10}\left\{\sum_{n=1}^{n=N} 10^{L_n/10} \times L_n\right\}$$

$L_n \rightarrow n^{th}$ sound level in dB

$t_n \rightarrow$ Time duration of n^{th} sample expressed as a fraction of Total sample time.

Variation of Sound levels with distance from the source



S: Source
O: Observer

$$\boxed{L_2 = L_1 - 20\log_{10}\left(\frac{R_2}{R_1}\right)}$$

LN Concept

The value of LN will indicate the sound level that will exceed for N%. of the gauging Time.

Maximum Permissible Noise Levels

Category of Area	Day	Night
Industrial Area	75	70
Commertial Area	65	55
Residential Area	55	45
Silence Zone	50	40

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