Coagulation Added Sedimentation.

Effeciency of P.S. is compatively reduced if water consist of fine suspended particles. Hence in such case cogolation added sedimentation is adopted. to increase the efficiency and it is carried out in three stages:-

(I) Coagulation. (fast fixing)
(ii) Flocculation (slow mixing)
(iii) Sedimentation.

Coagulation

→ Virtually all surface water sources contain perceptible turbidity.

→ Particles common to surface water varies in size from 10mm to 10^{-6} mm.

→

Particle size(mm)	Settling velocity
10	3m/4sec
0.0000001	3m/million year.

→ Particles having size smaller than 50 μm in diameter cannot be expected to be removed by plain sedimentation.

→ Agglomeration (to combine) of these particles into group is done for their settlement.

→ Certain properties of colloids prevent their agglomeration.

Colloidal Stability

→ Colloidal suspension that do not agglomerate naturally are called stable.

→ This stabilily is due to their <u>very small size.</u>

→ <u>Surface phenonmena</u> predominate over mannmom phenomena.

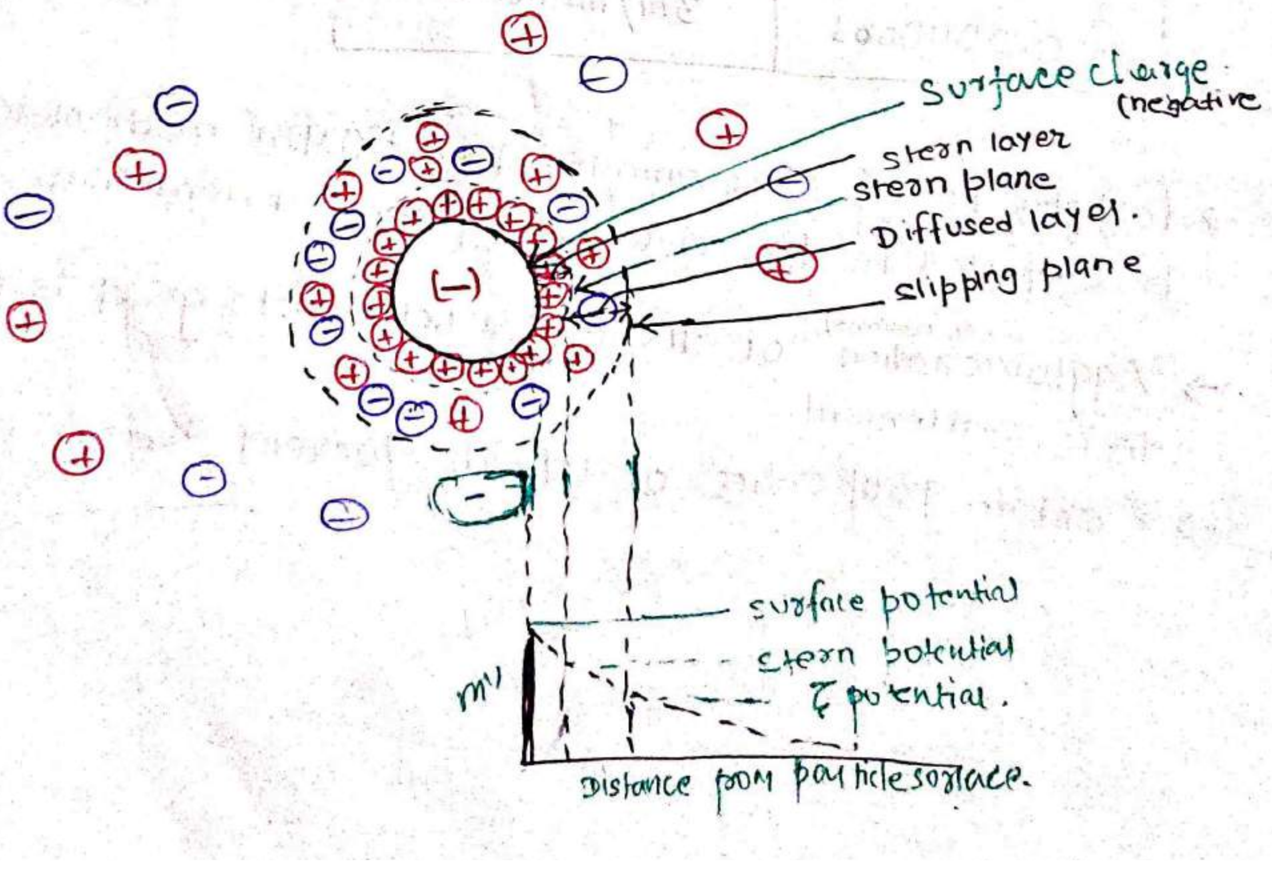
$$d\downarrow \rightarrow SSA\uparrow\uparrow\uparrow \rightarrow G\downarrow$$
$$d\uparrow \rightarrow SSSA\downarrow \rightarrow G\uparrow$$

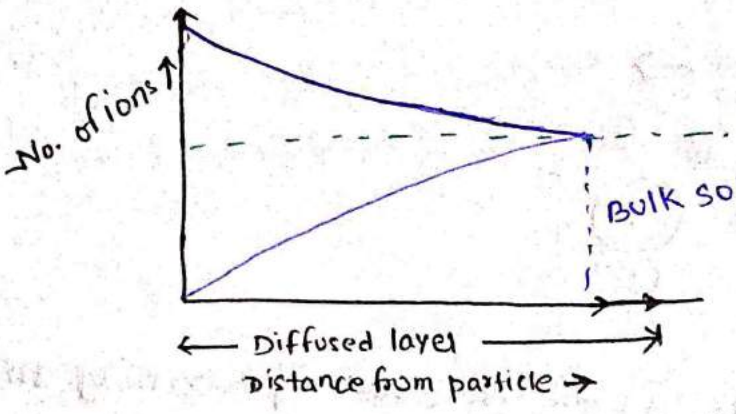
→ The most important surface phenonmenon is the accumulation of electrical charges at particle surface.

→ (i) Molecular arrangement within the particle, (ii) loss of atoms due to abrasion of surface result in surface charge.

→ In most surface water, colloidal surface are negatively charged.

→ Ions contained in the water near the colloid will be effected by charged surface.





→ Positive counter ions first attached to negatively charged particle, forming a rigid layer called STERN LAYER/BOUND.

→ The particle continues to attract more counter ions but now these (counter$^+$ ions) are being repelled by other counter-ions in the vicinity and STERN LAYER itself.

→ The second region is called Diffuse layer.

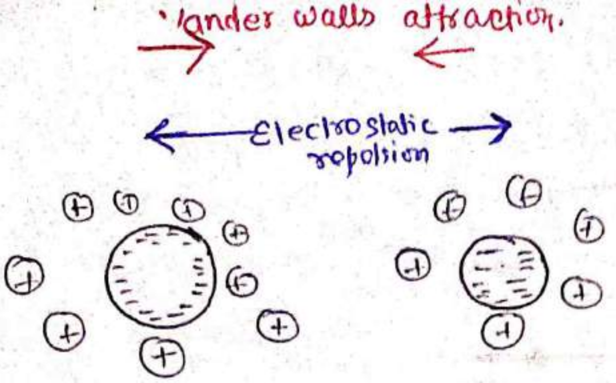
→ Together both layers are termed as the double layer.

→ Electrical potential between the surface of particle and any point in the suspending liquid is called Surface potential.

→ Electrical potential where stern layer meets Diffuse layer ie, Stern plane is called Stern potential

→ Electrical potential where diffuse layer meets the surrounding liquid i.e, slip plane is called Zeta Potential.

→ The higher the value of Zeta Potential, the more stable the suspension.



→ When two colloids come in close proximity there are two forces acting on them.

→ The ELECTROSTATIC/ ZETA POTENTIAL created by interaction of counter ions surrounding each colloid, which cause Repulsion.

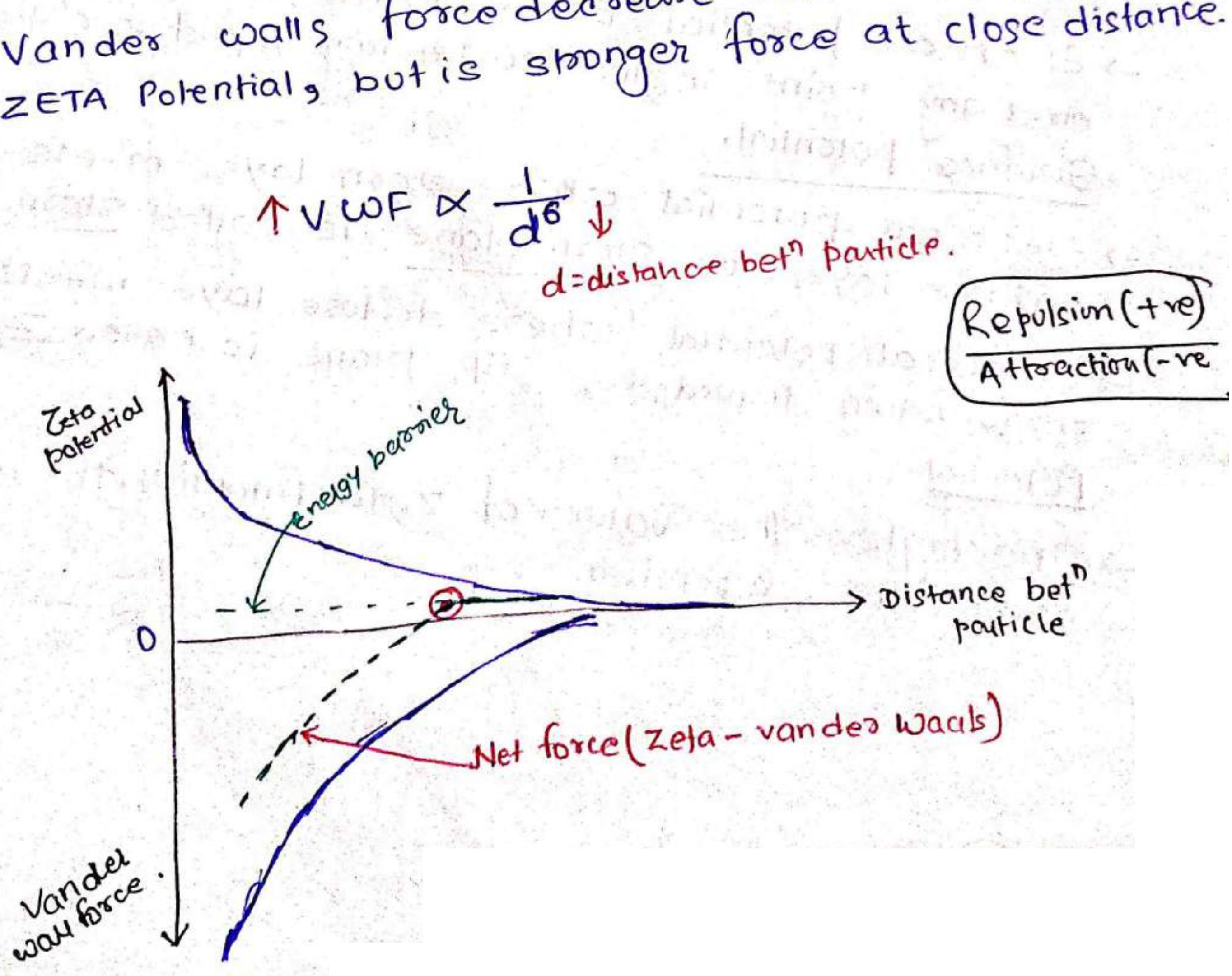
→ The second force, an ATTRACTIVE force called VANDER WALLS FORCE, which support contraction

→ Both zeta Potential and VANDER WALLS force decrease with increase in distance between colloids.

→ Vander walls force decrease more rapidly than ZETA Potential, but is stronger force at close distance.

$$\uparrow VWF \propto \frac{1}{d^6} \downarrow$$

d=distance betn particle.



$F_{net} = Z + (-v) = Z - V$

If d more, $Z > V$, $f_{net} = (+)ve$ (repulsion)

If d less, $V > Z$, $f_{net} = (-)ve$ (attraction)

→ The net force is Repulsive at greater distances and becomes attractive only after passing through maximum net repulsive force.

→ This is called Energy Barrier or THRESOLD Energy.

→ Once the force become attractive, contact between the particle take place.

→ Energy barrier must be overcome before the agglomeration of particle can occur.

→ Brownian movement or mechanical agitation of water may import enough momentum to larger particles to move them across energy barrier.

Coagulation theory:-

The exact method by which coagulation is accomplised cannot be determine, four mechanism are thought to occour.

(i) Ionic layer compression.
(ii) Absorption and charge neutralisation.
(iii) Sweep coagulation.
(iv) Interparticle bridging.

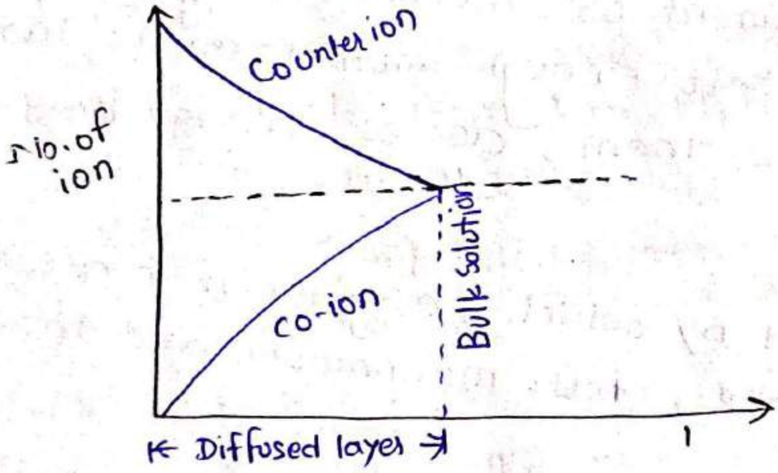
1. Ionic layer Compression. - Ions quantity

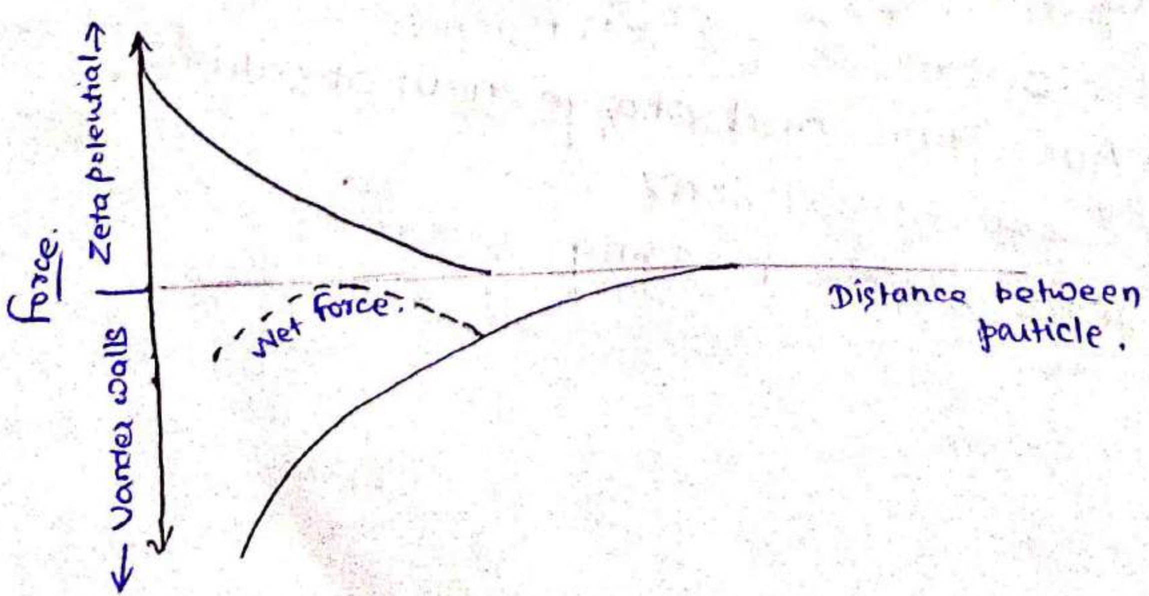
→ The quantity of ions in water surrounding the colloid has effect on the decay of electrostatic potential.

→ A high ionic concentration compresses the double layer (layers composed predominantly of counter ions) towards the surface of colloid.

→ If this layer is sufficiently compressed, then the VAN DER WALLS force will be predominant across the entire area of influence.

→ Net force will be attractive and no energy barriers will exist.





eg
→ for example when a turbid stream flows into the ocean.
→ the ionic content of the water increases drastically and coagulation and setting occurs.
→ Eventually, deposits (deltas) are formed.

→ Adsorption and Charge Neutralisation.

→ Here nature, rather than the quantity of the ions is of prime importance

→ If alum (aluminium sulphate) is used as a cogulant

→ Al (+3) cations immedialely react with water to form a variety of aquometallic ions and hydrogen ion.

$$Al(+3) + H_2O \longrightarrow AlOH(+2) + H(+)$$

$$Al(+3) + 2H_2O \longrightarrow Al(OH)_2(+) + 2H(+)$$

$$7Al(+3) + 17H_2O \longrightarrow Al_7(OH)_{17}(+4) + 17H(+)$$

$$Al(+3) + 3H_2O \longrightarrow Al(OH)_3 + 3H^+$$

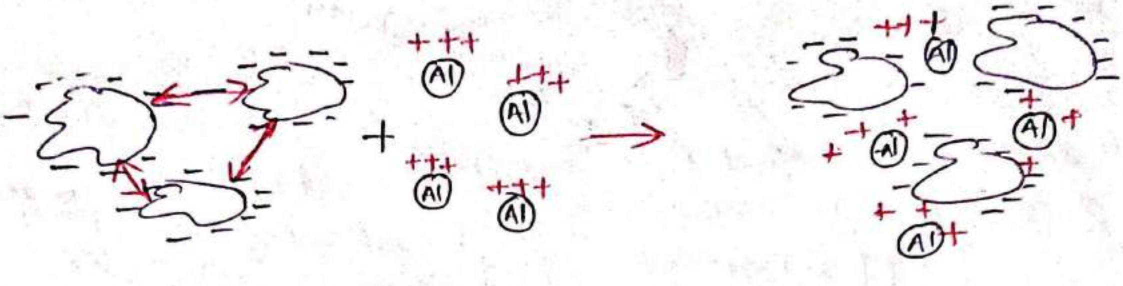
→ These aquometallic ions formed, becomes part of the ionic cloud surrounding the colloid.

→ They are adsorbed onto the surface of the colloid where they neutralise the surface charge.

→ Once the surface charge has been neutralized, the ionic cloud dissipates and the electrostatic potential disappear so that contact occurs freely.

Note: In this overdosing with coagulant can result in restabilizing of the suspension.

If excess aquometallic ions are formed and adsorbed the charges on the particles become reversed and the ionic clouds reform, with negative ions being the counter ions.

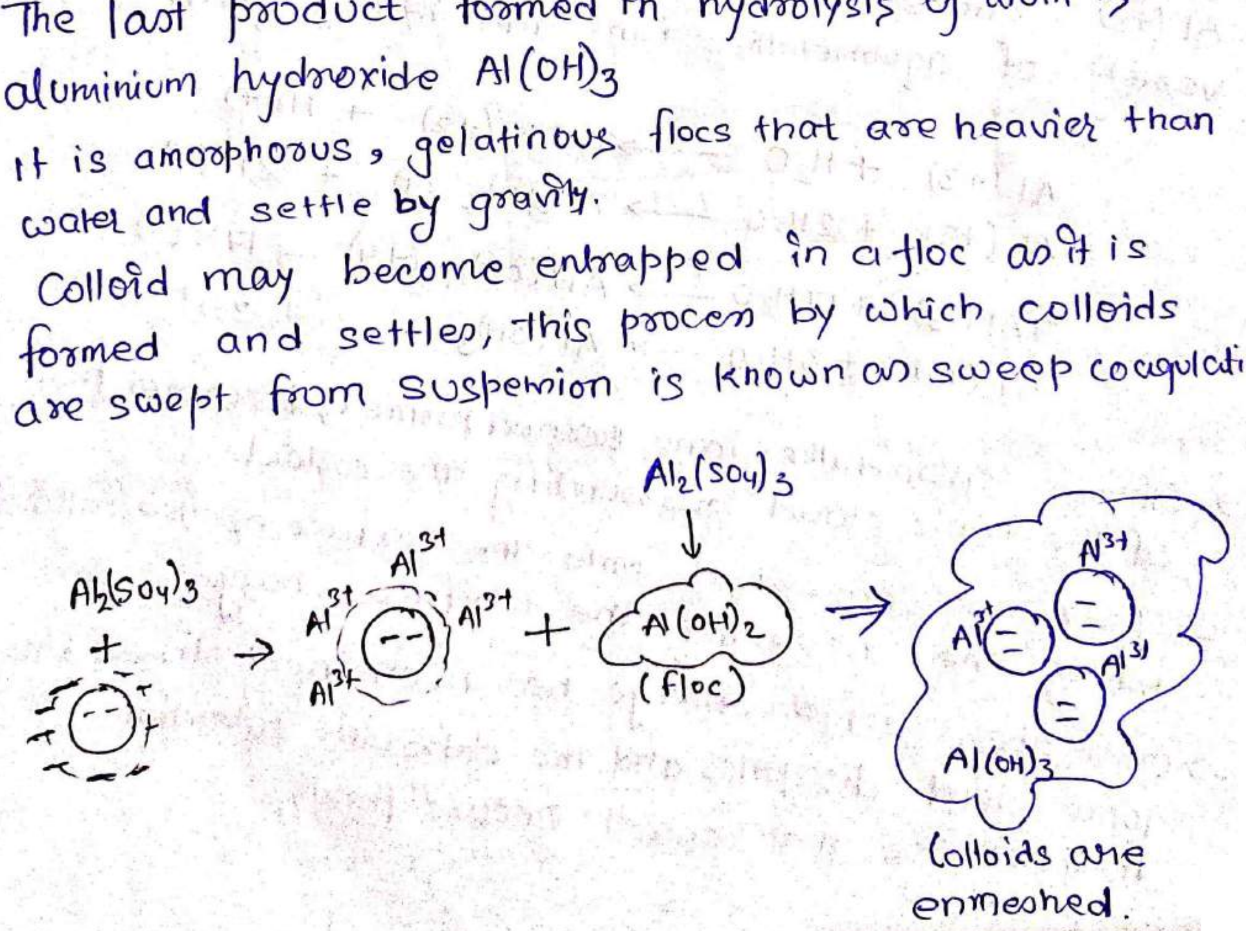


Sweep Cogulation

→ The last product formed in hydrolysis of alum is aluminium hydroxide $Al(OH)_3$

→ It is amorphous, gelatinous flocs that are heavier than water and settle by gravity.

→ Colloid may become entrapped in a floc as it is formed and settles, this process by which colloids are swept from suspension is known as sweep coagulation



Interparticle Bridging.

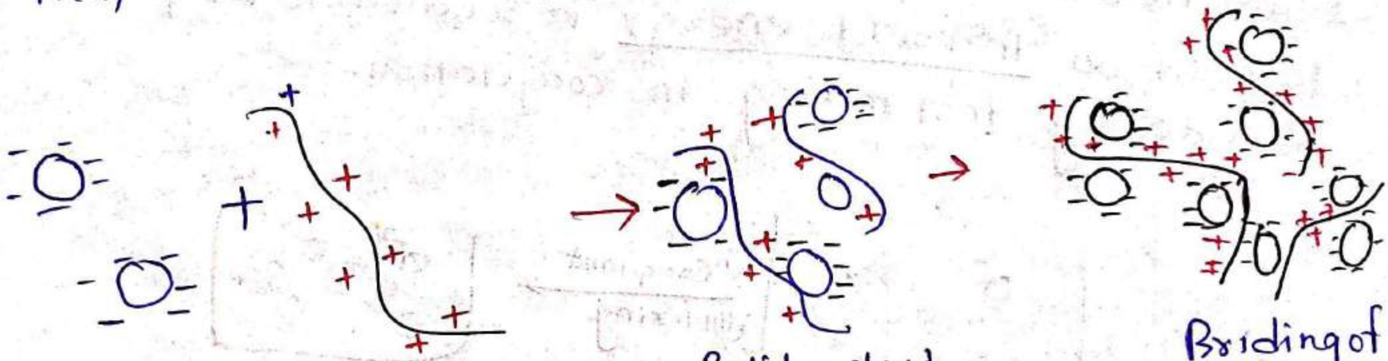
Large molecules may be formed when aluminia or ferric salt dissociate in water. eg. $Al_7(OH)_{17}$

→ synthetic polymers may also used here, which may be linear or branched and are highly surface reactive.

→ Several colloids may be attached to one polymer and several polymer-colloid may become enmeshed and settle.

→ In addition to adsorption forces, charges on polymer may assist in the coagulation process.

→ Metallic polymers formed by addition of aluminium or ferric salts are positively charged, while synthetic polymers may carry positive or negative charges or may be neutral.



Suspended particles negatively charged

Polymer (positively charged) "flocculant".

Particles start to "flock" and second layer of flocculation starts

Briding of particles to form a "flock".

Coagulation Added Sedimentation.

I coagulation (i) (ii)

$$\frac{\eta(I)}{SA = I} > \frac{\eta\, II}{SA = II}$$

(II)

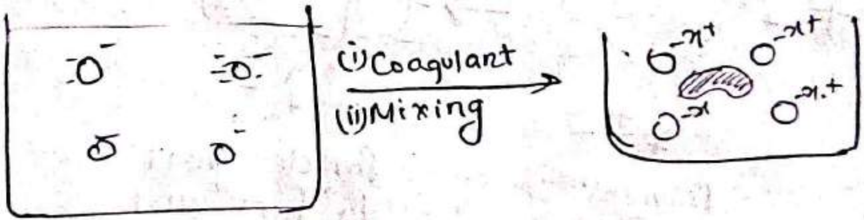
$$\frac{\eta(I)}{SA\, I} = \eta\, II$$

$$SA\, I < SA(II)$$

Coagulation

It is the process in which we add certain chemicals termed as coagulant which neutralise the charge over suspended particles and forms a sticky ppt.

→ To neutralise the charge certain minimum energy termed as thresold energy is required to be provided. by inducing fast mixing in coagulation.



*Different types of coagulant which may be used:-

1.) Alum - $Al_2(SO_4)_3 . 18H_2O$ (Hydrated Aluminium Sulphate)

Alum reacts with alkalinity present in water and result in formation of sticky geletinous precipitate of aluminium hydroxide which attract fine suspended impurities over its surface, thus grow in size and gets easily settled in following sedimentation process.

$$Al_2(SO_4)_3 \cdot 18H_2O + 3Ca(HCO_3)_2 \rightarrow \underset{\text{sticky gelatious}}{\overset{\text{Aluminium hydroxide}}{2Al(OH)_3\downarrow}} + 3CaSO_4 + 6CO_2 + 18H_2O$$

$$\left.\begin{array}{l} Al^{3+} + H_2O \rightarrow Al(OH)^{2+} \\ Al^{3+} + 2H_2O \rightarrow Al(OH)_2^{+} \\ Al^{3+} + 3H_2O \rightarrow Al(OH)_3\downarrow \\ Al^{3+} + 17H_2O \rightarrow Al_7(OH)_{17}^{4+} \end{array}\right\} \text{Charge Neutralisation.}$$

Note:- For sweep coagulation to work water should have high alkalinity & high turbidity.

→ By addition of alum, alkalinity and pH of water decreases but turbidity and permanent hardness increases.

→ If alkalinity is not present in water naturally, it is induced artificially by lime or soda ash.

$$Al_2(SO_4)_3 \cdot 18H_2O + \underset{\text{(lime)}}{3Ca(OH)_2} \rightarrow 2Al(OH)_3\downarrow + 3CaSO_4 + 18H_2O.$$

$$Al_2(SO_4)_3 \cdot 18H_2O + \underset{\text{(soda ash)}}{3Na_2CO_3} \rightarrow 2Al(OH)_3 + 3Na_2SO_4 + 3CO_2 + 12H_2O$$

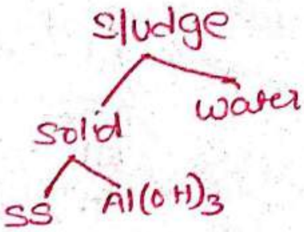
→ Alum works in pH rang of 6.5 - 8.5

→ Alum removes taste, colour, odour and microorganism from water.

→ Does of alum depend upon turbidity and colour of the water. and it is determine using Jar test.

→ Normal does of Alum is 10-30 mg/lit.

→Large quantity of sludge is formed when alum is used as coagulant. , ~~dry weight~~ which is difficult to dewater and dry weight of which can be found as follows:-



2×162

$$Al_2(SO_4)_3 \cdot 18H_2O + 3\,Ca(HCO_3)_2 \longrightarrow 2Al(OH)_3$$

1 mole	3 mole	2 mole.
666gm	486gm	156gm
1gm	$\frac{486}{666} = 0.72$ $\parallel$ 0.45gm as $CaCO_3$	0.234gm

1 gm eq^v of $Ca(HCO_3)$ = 1 gmeq. of $CaCO_3$

gm eq. of $(Ca(HCO_3)_2 = \frac{\text{given weight}}{\text{equivalent weight}}$

$= \frac{0.72}{162/2}$ = gm. equ of $CaCO_3$

Concentration of $CaCO_3 = \frac{0.72}{162/2} \times 50 = 0.45$gm

2) Copperas ($FeSO_4 \cdot 7H_2O$) [Hydrated ferous Sulphate]

→ It also reacts with alkalinity present in water & result in the formation of sticky gelatinous precipitate of $Fe(HCO_3)_2$ which attracts fine suspended impurities over its surface and gets easily settled in following sedimentation tank.

~~$FeSO_4 \cdot 7H_2O + Ca(HCO_3)_2 \rightarrow Fe(HCO_3)_2 + CaSO_4 + H_2O$~~

$\downarrow O_2$

~~$Fe(OH)_3 \downarrow$ ferric hydroxide~~

~~(stick)~~.

→ If lime is added earlier than Copperas.

$$FeSO_4 \cdot 7H_2O + Ca(OH)_2 \longrightarrow \underset{\text{sticky gel}}{\boxed{Fe(OH)_3 \downarrow}} + CaSO_4 + 7H_2O$$

→ If copperas is added earlier than lime.

$$FeSO_4 \cdot 7H_2O + Ca(HCO_3)_2 \longrightarrow Fe(HCO_3)_2 + CaSO_4 + H_2O$$

$$Fe(HCO_3)_2 + 2Ca(OH)_2 \longrightarrow Fe(OH)_2 + 2CaCO_3 + 2H_2O$$

$$4Fe(OH)_2 + O_2 \xrightarrow{H_2O} \boxed{4Fe(OH)_3 \downarrow}$$

→ It is not used for treatment of <u>colour water</u> because it itself impart colour in water.

→ Its normal doses is 10-30 mg/lit but it is comparatively cheaper than alum.

→ It works in pH range of ≥ 8.5. hence it is always used in conjucation with lime.

(iii) Chlorinated Copperas [$Fe_2(SO_4)_3 + FeCl_3$]

→ If we added chlorine in copperas we get chlorinated copperas.

$$FeSO_4 \cdot 7H_2O + Cl_2 \longrightarrow Fe_2(SO_4)_3 + FeCl_3$$
PH: 3.5 - 7 , PH > 8.5

$$FeCl_3 + Ca(OH)_2 \longrightarrow Fe(OH)_3 \downarrow + CaCl_2$$
PH: 3.5 - 6.5 , PH > 8.5

$$Fe_2(SO_4)_3 + Ca(OH)_2 \rightarrow Fe(OH)_3 \downarrow + CaSO_4$$
PH: 4 - 7 , PH > 9

Q A coagulation sedimentation plant is treating 40 MLD of raw water having 100 mg/L suspended solid in it 30 mg/lit alum is added to carry out coagulation.
Compute:
① Annual Quantity of alum in 85% pure CaO needed for treatment of this water.
II Q. Volume of sludge produced at the bottom of Sedimentation tank in a day. Assume efficiency of sedimentation tank 80%, $G_{slude} = 1.2$, solid content = 2%.
III change in Ionic character of water in a day. Assume raw water has alkalinity of 4 mg/lit as $CaCO_3$.

1 ton = 1000 kg
1 kg = 1000 gm
1 gm = 1000 mg

I Quantity of alum required $= 30 \times 40 \times 10^6 \times 365 \times 10^{-9}$
= 438 tons/year

1 gm of alum reacts with 0.45 gm of alkalinity as $CaCO_3$

30 mg/lit " " $= 0.45 \times 30$
= 13.5 mg/lit as $CaCO_3$

Additional alkalinity required = 13.5 - 4 mg/l as $CaCO_3$
= 9.5 mg/lit as $CaCO_3$

lime hydrated lime

$$\underset{1\,mole}{CaO} \xrightarrow{H_2O} \underset{1\,mole}{Ca(OH)_2} \longrightarrow \underset{1\,mole.}{CaCO_3}$$

1 mole of $CaCO_3$ is induced by 1 mole of 100% CaO.

100 gm ———————— 56 gm

9.5 mg/lit of $CaCO_3$ alkalinity is introduced by $= \frac{56}{100} \times 9.5$

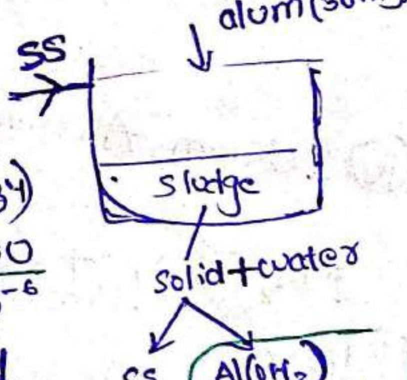
$= 5.32$ mg/lit of 100% CaO.

9.5 mg/lit alkalinity is introduced by $= \frac{5.32}{0.85} = 6.26$ mg/l of 85% pure CaO.

Quantity of 85% pure CaO $= 6.26 \times 40 \times 10^6 \times 365 \times 10^{-9}$

$= 91.396$ tons/year

III Volume of sludge



wt. of solid settle to form sludge $= (100 + 30 \times 0.234) \times 0.80 \times 40 \times 10^6 \times 10^{-6}$

$= 3424.64$ kg/d

Sludge = solid + water.

2 kg solid + 98 kg water = 100 kg of sludge

S = 2%

1 kg of solid $= \frac{100}{2}$ kg of sludge

wt. of sludge formed $= \frac{100}{2} \times 3424.64 = 171232$ kg/day

Volume of sludge formed $= \frac{171232}{1.2 \times 10^3} = 142.7\ m^3/day$

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

1 mole of alum reacts with 0.45 gm of $CaCO_3$ alkalinity.

30 mg/L ———— 0.45×30 = 13.5 mg/lit

Alkalinity of water will be reduced by $= 13.5\times40\times10^{6}\times10^{-6}$
$= 540$ kg/day as $CaCO_3$

1 mole of alum gives 3 mole of $CaSO_4$ hardness as $\overset{3\,mole}{CaCO_3}$

666 gm ———— 300 gm as $CaCO_3$.

1 gm ———— $\frac{300}{666} = 0.45$ gm as $CaCO_3$

Permanent hardness of water will be increased by
$= 0.45\times30\times40\times10^{6}\times10^{-6}$
$= 540$ kg/day.

(iv) Sodium Aluminate $(Na_2Al_2O_4)$

$$Na_2Al_2O_4 + Ca/Mg\begin{cases}Cl^- \\ SO_4^{2-} \\ NO_3^-\end{cases}\text{NCH} \quad \begin{cases}CO_3^{2-} \\ HCO_3^-\end{cases}\text{CH = Alkalinity} \rightarrow Ca/Mg\,Al_2O_4\downarrow + Na\begin{cases}Cl^- \\ SO_4^{2-} \\ NO_3^- \\ CO_3^{2-} \\ HCO_3^-\end{cases}$$

→ It does not require alkalinity to be present in water moreover removes the hardness from the water unlike other coagulant which impart the hardness

→ It removes hardness of water.

→ Its normal does is (10-30)mg/lit but it is costlier than alum.

→ cheapest coagulant is copperas & costliest is sodium aluminate.

→ It is generally used for treatment of boiler feed water.
(zero hardness water in power plant).

→ It works in pH range of . 6-8.5

Note
(i) Iron salts work in large pH range.
(ii) Iron salts are cheaper than aluminimum salt.
(iii) Iron flocs are heavier than aluminium flock.
(iv) Iron flocks are formed earlier than aluminium flock.

→ Alum is generally used for treatment of raw water & Iron salt are used for treatment of sewage.

Drawback
Use of Iron salts causes pitting & tuberculation in pipes.

Pitting → corrosion
tuberculation → Incrustation.
(deposition of solid)

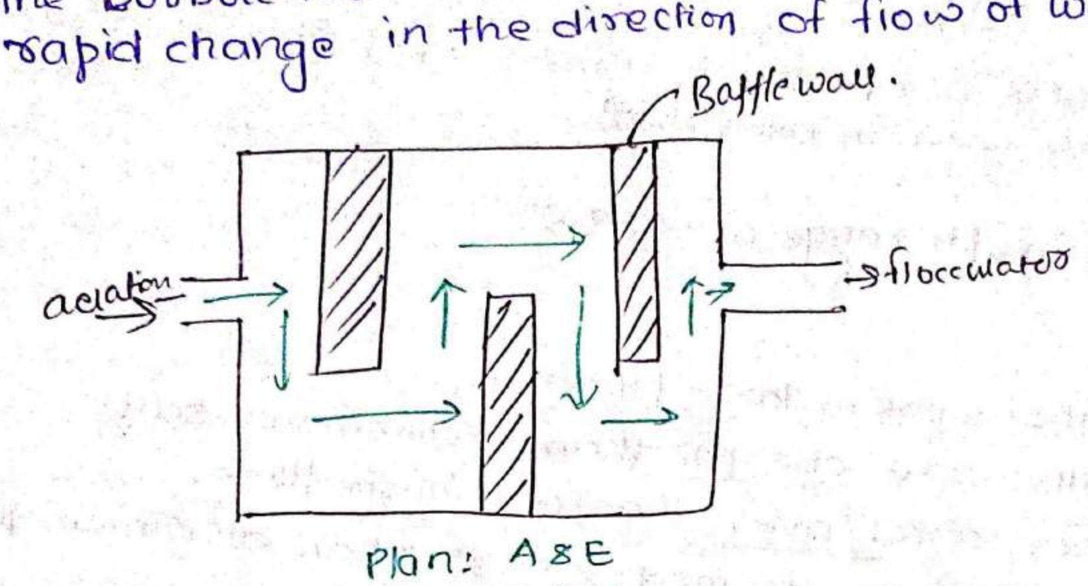
→ To neutralise the charge or for particles to combine minimum energy is required to be given termed as thresold energy. which is provided by ~~[illegible]~~ inducing in the process in any of the following forms. fast mixing

(i) Mixing Basin → Around & End type (horizontal).
→ Over & under type. (water move in vertical plane)

In coagulation we neutralise the particle, and form the sticky particle.

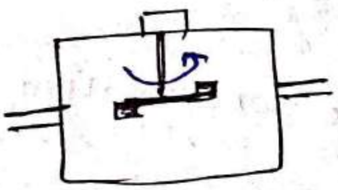
Mixing Basin

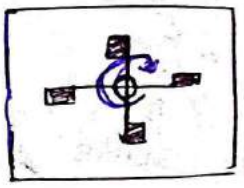
In mixing basin, mixing is induced by generating the turbulence which is achieved by bringing the rapid change in the direction of flow of water.



Plan: A&E
X-X: O&V

(ii) Mechanical Mixer → Horizontal shaft type
→ Vertical shaft type.

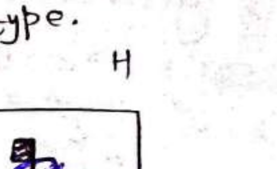




→ In this case intensity of mixing is reported in terms of temporal mean velocity gradient.

→ It(G) signifies the relative velocity between neutralized particles at a particular distance from each other.

$$G = \frac{V_1 - V_2}{x}$$



Unit = m/m/sec
= sec^{-1}

$$G = f(P, \mu, V)$$

$$G \propto \frac{P}{\mu V}$$

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

By dimension analysis.

$$G = \sqrt{\frac{P}{\mu V}} \qquad a = 1/2 \quad b = c = -1/2$$

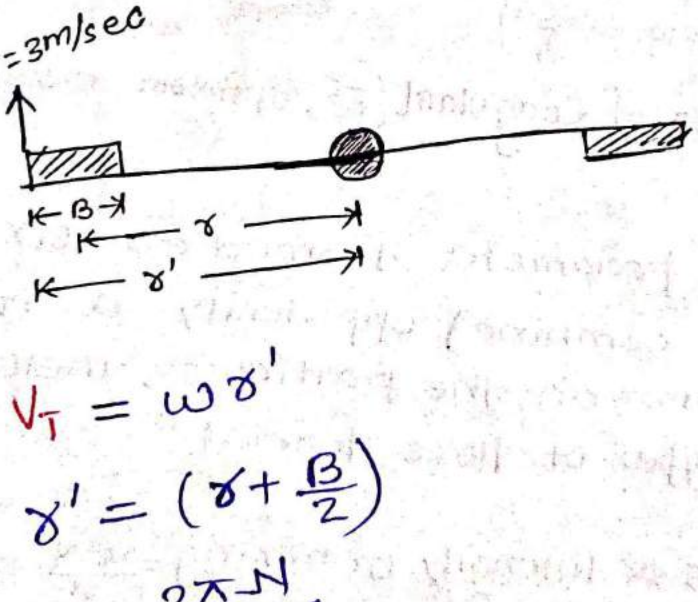
→ Depth to width ratio = 1:1 − 3:1 $= \frac{H}{B \text{ or } D}$

→ Detention time = 30-60 sec.

→ $G > 300\ sec^{-1}$

→ Power supply to shaft = 1-3 watts/(m^3/hr) = 1-3 watt/Disclo

→ Speed of shaft should be velocily at tip of pedle is more than 3m/sec. $= V_T$



$$V_T = \omega r'$$

$$r' = \left(r + \frac{B}{2}\right)$$

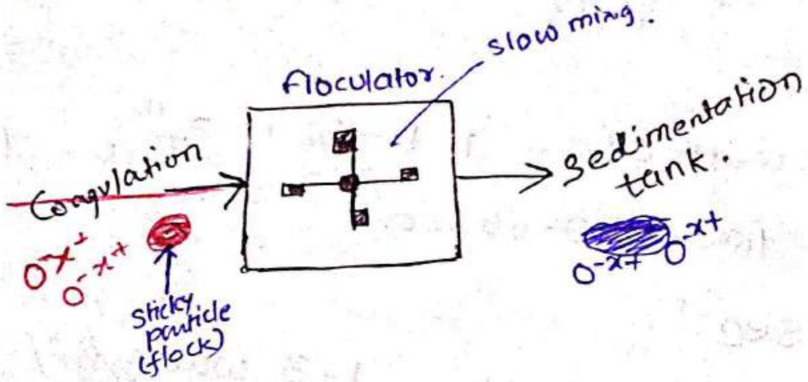
$$\omega = \frac{2\pi N}{60}$$

$$V_T = \frac{2\pi N}{60} \times \left(r + \frac{B}{2}\right)$$

II FLLOCULATION (Slow Mixing)

→ It is the process in which neutralise particle & sticky are brought in contact with each other so as to form a bigger size particle (floc) which can get easily settled in the following sedimentation tank.

→ In order to increase the opportunities of particles to agglomerate (to combine), slow mixing is also induced in this process.



Rate of floculation depend upon following parameter:-

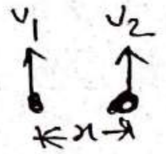
(i) Turbidity. ↑ ROF ↑

(ii) Mixing ↑↓ (optimum range) ($G = 10-75\ sec^{-1}$) ROF ↑

(iii) Type and does of Coagulant (Fe, optimom does) ROF ↑

In flloculation a parameter termed as $G \times t$ represent the conjugation (to combine) oppertunity as it signifyies no. of collison between the particles, thereby helps in identifying the types of flocs formed.

G - TMVS ∝ Intensity of mixing $= \frac{v_1 - v_2}{x}$



t - time of contact.

$Gt = \text{Constant}$ $\qquad G = \frac{(V_1 - V_2)}{x}$ — Constant

(i) G↑ , t↓ = Dense, small =  = Small dense flocks

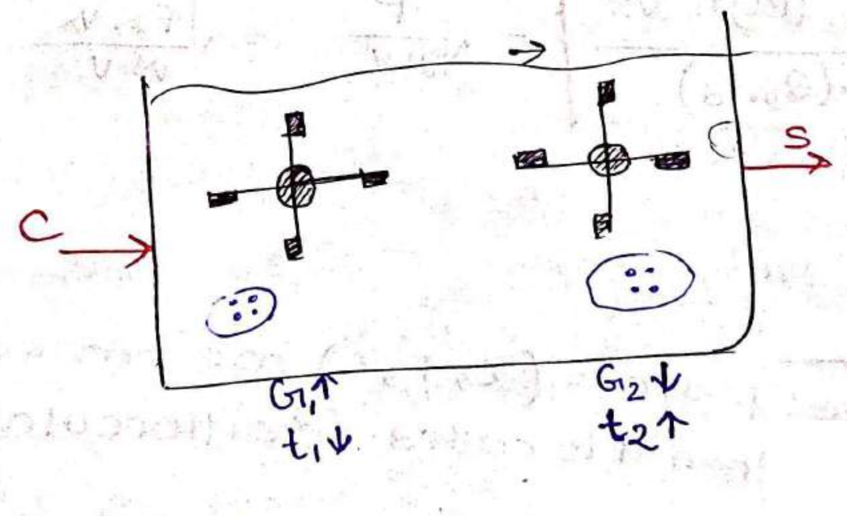
(ii) G↓ , t↑ = Light, Large =  = large light flock

→ As dense and large floc gets easily settled in sedimentation tank it is advantageous to vary the value of G along the length of flocculation tank.

→ small and dense floc formed in the initial section of tank (G↑ , t↓) combine with large & light flocs formed in later section of the tank (G↓ , t↑) & result in the formation of large & dense-floc. Hence,

$G_{inlet} = 2 G_{outlet}$

temporal M ✓



Design data

Depth = 3-3.5m

Detention time = 10-30 min

$G = (10-75) \text{sec}^{-1}$

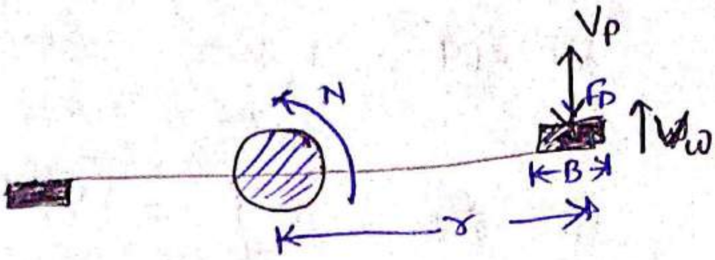
Total Area of Pedals = (10-25)% of the tank area.

Gt = 20000 - 60000 for alum

1 - 1.5 lakh for Iron salt.

Calculation of "G". (Temporal mean velocity gradient)

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



Volume, $V = Q_D \cdot t_d$

Power, $P = F_D \cdot V_r$

$V_r = V_P - V_\omega$

$V_P = \omega r$

$\omega = \frac{2\pi N}{60}$

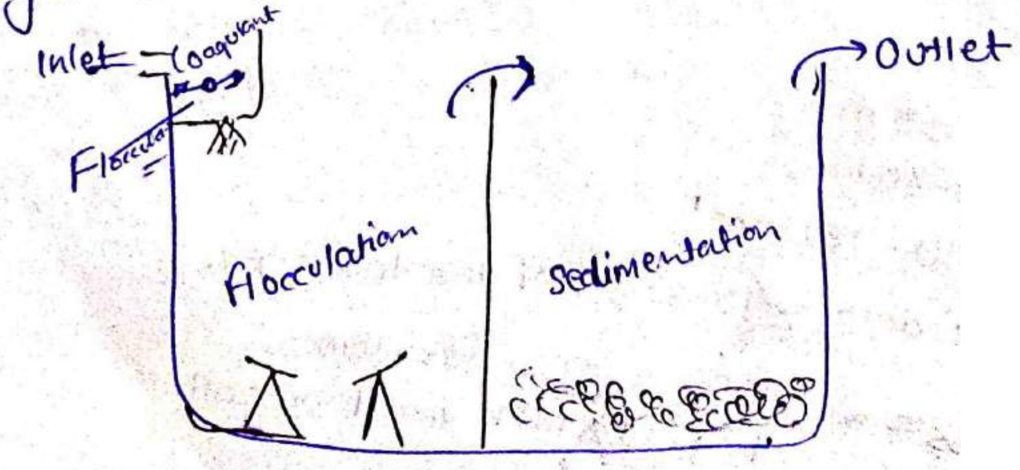
$F_D = \frac{1}{2} C_D \rho_\omega A_T \cdot V_r^2$

$A_T = (BL)n$, $C_D \cong 1.8$

$$\boxed{G = \sqrt{\frac{\frac{1}{2} C_D \cdot \rho_\omega \cdot (BL)n \; V_r^3}{\mu \cdot (Q_D \cdot t_d)}}} = \sqrt{\frac{P}{\mu V}} = \sqrt{\frac{F_D \cdot V_r}{\mu V}}$$

Clariflocculator

If all these process (C, F, S) are carried out in single tank, then it is called clariflocculator



Q A flash fixer is treating 2m³ of water @ mixing rate of 300 sec⁻¹ at the temp of 20°c. where viscosity = 0.01 poise. Compute the power requirement of the mixer for treating the per unit volume of water.

$\mu = 0.01$ poise
$= 0.01 \times 10^{-1} \frac{N s}{m^2}$

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

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

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

$$= (300)^2 \times 0.01 \times 10^{-1} \times 1 = 90 \text{ watt}$$

Q. A fllocculation tank 30m long, 12m wide, and 4m deep is treating 10MLD of water it is provided with 4 shaft at the mid depth of the tank supporting 2 pedal each one opposite to each other. The center line of the pedal is 1.8m from the center of shaft. Size of pedal is 0.3m X12m. velocity of water is $\frac{1}{4}$ the velocity of pedal

Assume, N=30

compute

(i) Detention time / time of flocculation.
(ii) Power requirement of tank.
(iii) Intensity of mixing(G)

(i) Detention time = $\frac{V}{Q}$

$$= \frac{30 \times 12 \times 4}{10 \times 10^6 \times 10^{-3}} = 0.144 \text{ day} = 3.456 \text{ Hour.}$$

(ii) Power requirement = $F_D . V_r$.

$$= \frac{1}{2} C_D . \rho_w . A_T . V_r^3 .$$

$C_D = 1.8$
$\rho_w = 1000 kg/m^3$

$$= \frac{1}{2} \times 1.8 \times 1000 \times (0.3 \times 12) \times \underline{8} \times 4.23^3$$

$$= 1961.806 \text{ KW}$$

$$= 1961.806 \text{ KW}$$

$$V_r = (V_p - V_w)$$

$$= \frac{3}{4} V_p$$

$$V_p = \omega r$$

$$= \frac{2\pi \times 30}{60} \times 1.8 = 5.654 \text{ m/sec.}$$

$$V_r = \frac{3}{4} V_p = \frac{3}{4} \times 5.654 = 4.241 \text{ m/sec.}$$

(iii) $G = \sqrt{\frac{P}{\mu V}}$

$$= \sqrt{\frac{1961806}{0.01 \times 10^{-1} \times (30 \times 12 \times 4)}}$$

$$G = 1167 \text{ sec}^{-1}$$

IV Filteration

→ filteration is provided for removal of fine suspended particle and floc from water.

→ It also removes organic matter, micro-organism, dissolve solid from water.

→ filteration is carried out in the units termed as filter. broadly classified into:→

(i) Gravity filter. –[→ Slow sand filter / → Rapid sand filter.]

(ii) Pressure filter.

→ Gravity filters are those in which head required by the water to flow through filter medium is provided by the height of water itself. above the filter medium whereas

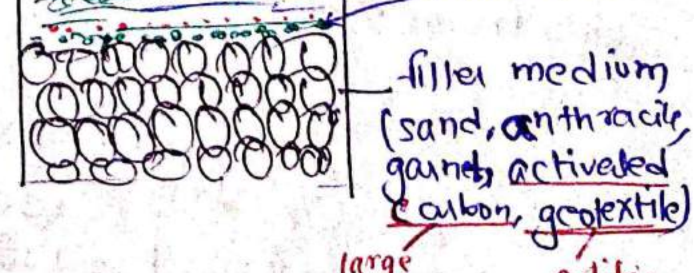
→ in case of pressure filter the head is provided by the external application of pressure over the water over the filter medium.

Mechanism of filtration.

When water passes through the filter medium removal of impurities takes place by flowing mechanism:-

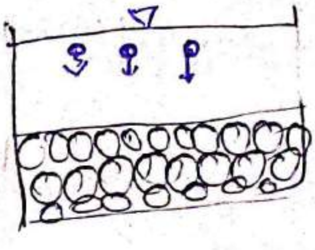
(i) √Mechanical straining.

→ When the water passess through the filter medium, suspended solid of size greater than the size of void of filter medium are strained (removed) out from it resulting in the formation of layer of entrapped imupities over the filter medium (mat layer).



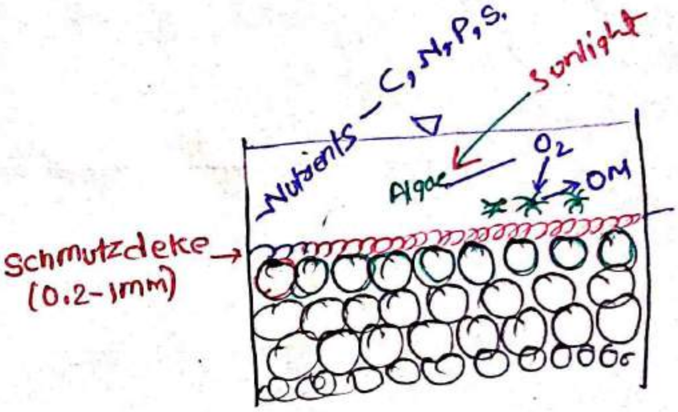
→ Suspended solid of size smaller than the size of voids of filter medium are retained over this layer (mat layer) of entrapped impurities as the size of voids of this layer is less than the size of the voids of medium.

(ii) Sedimentation



Due to the removal of turbulence from the water over the filter medium, solid get settled over the water free from suspended solids.

(iii) Biological Changes (OM & MO)



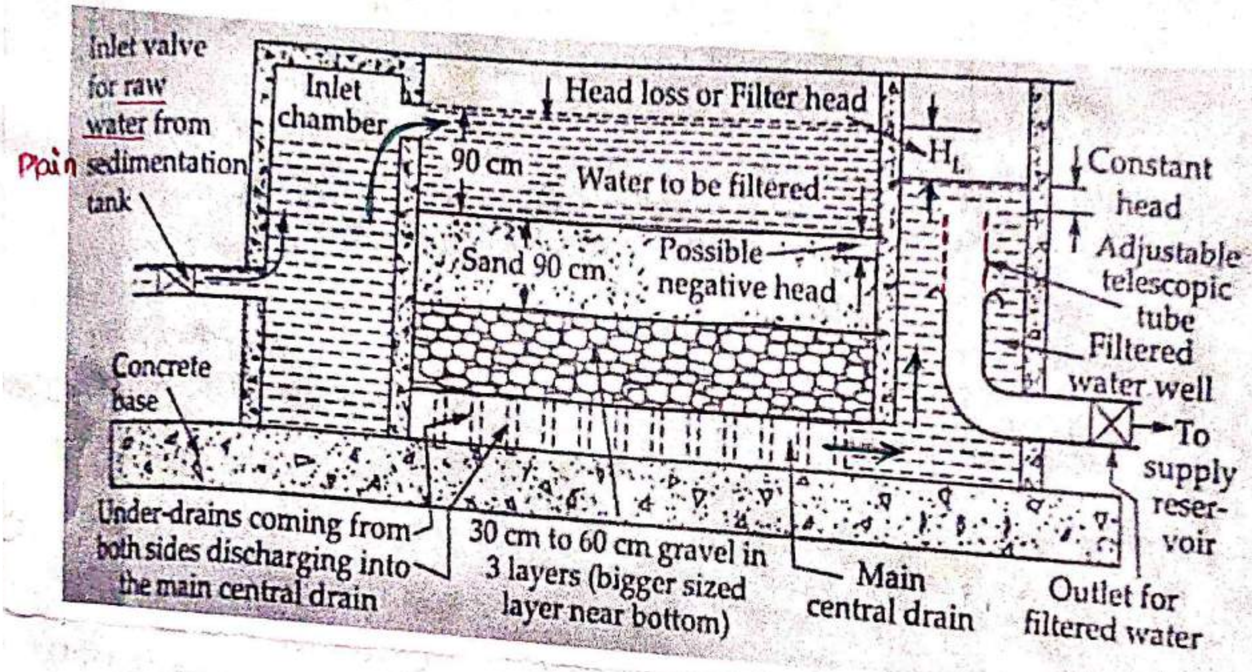
For the first few days when the water passes through the filter medium the upper layer of filter medium get coated with sticky deposit of partial decomposition of OM (organic matter) along with nutrients like C, N, P, S, Al, Fe, Mn which promotes the growth of algae that release oxygen during photosynthesis which is further utilised by the micro-organism to carry out decomposition of organic matter. (A thin layer in which entire activity takes place is referred as schmutzdeke). Over a period of time due to removal of organic matter by the micro-organism endogenous respiration takes place which also reduce the microbial load over the filter.

* Efficiency of the filter with regard to the removal of organic matter and micro-organism depend upon the growth of schmutzdeke.

(iv) Electrolytic Changes (DS).

Impurities present in the water and the medium particles are oppositiely charged. hence because of their interaction, neutralisation of these impurities takes place which are further removed from the filter during its cleaning process.

Slow Sand Filter



- → It utilise water from source or from plain sedimentation (settled water) only. as in case of coagulation aided sedimentation, effluent leads to frequent chocking of filter media require its frequent clean which in turn increase operational cost
- → Gravel layer is provided only to support the sand medium.
- → Depth of tank is 2.5 – 3.5 m.
- → Plan area required is 100 – 200 m².
- → Filter medium used sand, anthracite, garnet, activated carbon, geotextile.

→ D_{10} (Effective size of grain] = 0.2 - 0.3mm.

Sand 75μ – 10% $\overset{D_{10}}{|}$ 90% → 4.75mm
$\boxed{0.2-0.3mm}$

→ As D_{10} is comparatively less, efficiency is complelely more and rale of filtration is slow, so it is known as Slow sand filter. [$\eta\uparrow$, ROF$\downarrow$]

→ Co-efficient of uniformity of the medium particle is 3-5.

$$C_u = \frac{D_{60}}{D_{10}} = 3\text{-}5$$ (Poorly graded)

* Well graded medium = all size particle.
Medium or poorley graded medium = Same size particle.

D_{10} - It is the size below which 10% particles are finner.
D_{60} - It is the " " 60% particles are finner.

$C_u = 1$ (Poorley graded)
$C_u = 5$

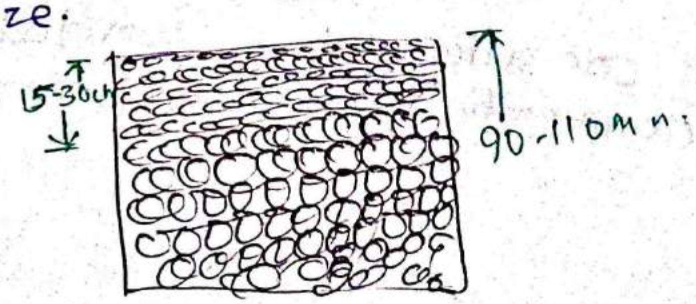
$D_{10} = 0.2mm$.
10% pastille are finner than 0.2mm.

$D_{60} = 1mm$
60% particles are less than 1mm

→ Depth of sand = 90 - 110 cm.

→ Depth of water over sand medium is kept same as that of sand medium.

→ Top 15-30cm of sand layer is of fine variety and rest of uniform size.



→ Freshly cleaned filter has a head loss of 10-15cm which goes on increasing during filtration hence height of telescopic tube is adjusted in accordance with the head loss to maintain constant discharge.

cleaning

→ When head loss become 0.7 to 0.8 times the depth of sand, cleaning is done.

→ for each cleaning top 1.5 to 3cm of sand is removed and remaining surface is scrapped by rack.

→ Filter is washed with clarified water & again loaded with raw/settled water but filter water is not used for next 24-36hr as sufficient time provided for regrowth of schmutzdecke that is removed during cleaning & for maturity of filter.

→ Frequency of cleaning is 1-to 3month.

→ This cleaning is termed as surface cleaning.

→ when 10cm top layer of sand is removed it is refilled.

→ ROF = 2400 - 4800 $\underbrace{l/m^2/d}_{\text{Discharge/unit area}}$

$$\boxed{\underset{(L\times B)}{\text{Plan area}} = \frac{Q_D}{ROF}}$$

eg $Q_D = 1.8MD = 1.8 \times 300 \times 30\times10^5 = 1.6\times10^9\ l/d$

$PA = \frac{Q}{ROF} = \frac{1.6\times10^9}{4800} = 337500m^2 = 0.337Km^2$
$= (1Km \times 337m)$

→ These filter are suitable to be provided for small village & communities having low water demand is less.

→ Minimum no. of slow sand filler required for operation 1
for standy by 1.

→ No. of fillers required depend upon area of these filter.

Area(m^2)	No. of fillers
<20	2 (1+1)
20-249	3 (2+1)
250-649	4 (3+1)
650-1200	5 (4+1)
>1200	6 (5+1)

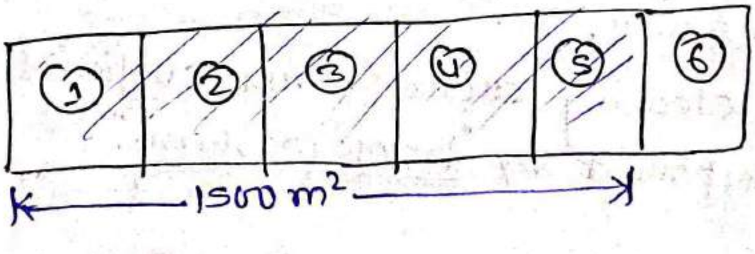
eg

$SA = 1500 m^2$

$N = 6 (5+1)$

$SA_1 = \frac{1500}{5} = 300 m^2$ ~~250 m^2~~

$SA_T = 1800 m^2$ ($1500 + 300$) m^2.

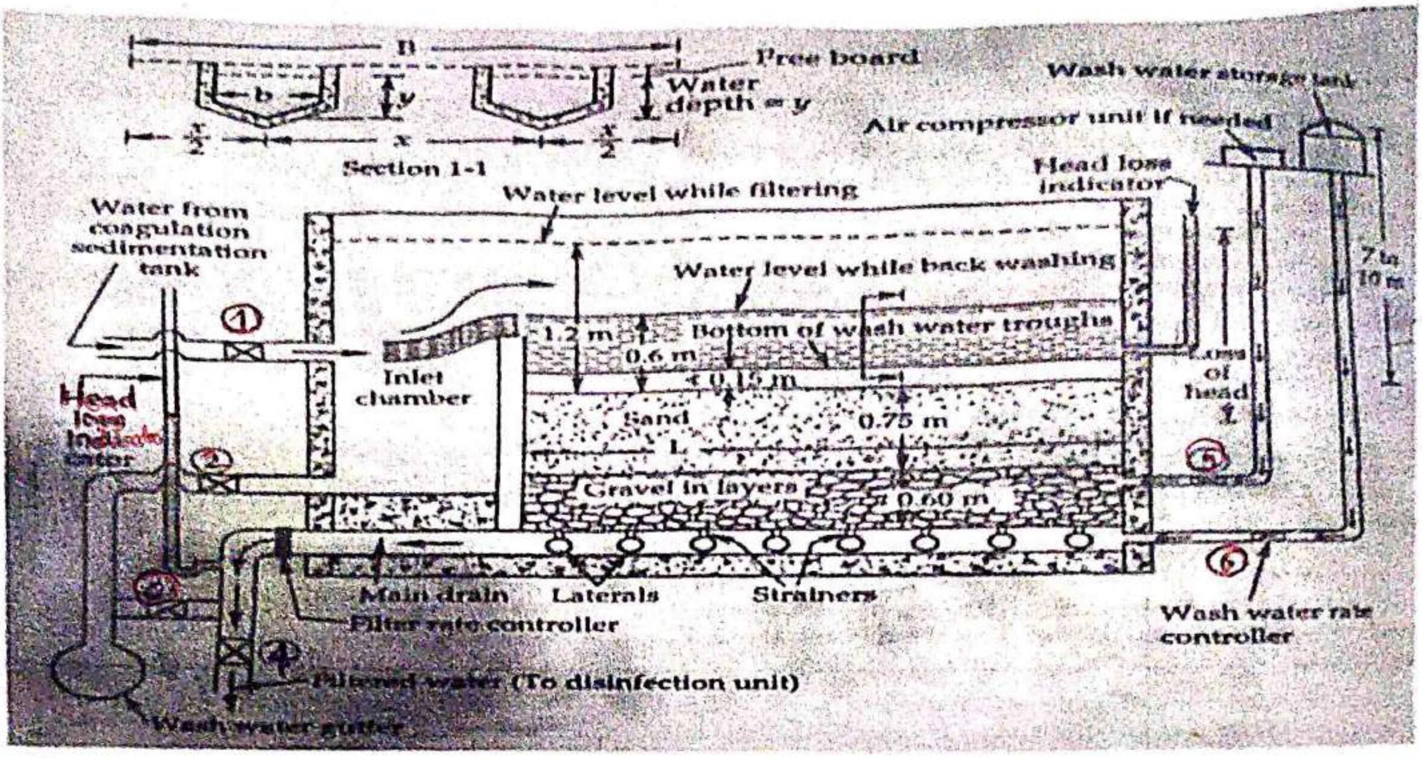


→ Effeciency of slow sand filter for removal of organic matter and MO is 98-99%.

→ Effluent from slow sand filler contain lesser nutrient (C, N, P, S) hence offer lesser opportunity for growth of algae in distribution system.

→ This filter not used if turbidity of water is more than 50NTU as it result in frequent chocking of filter

Rapid Sand Filter.



→ working of RSF is same as SSF.

→ As size of medium particle is comparatively bigger & medium is comparatively poorly graded than slow sand filter, impurities are able to penerate in lower parts of filter medium.

→ Hence surface cleaning alone is not sufficient in this case & is being accompanied by "back washing".

↳ During operation valve no. 1 & 4 are opened wherein settled water is fed into the filter through valve no. 1 and filtered water is collected through valve no. 4.

→ During backwashing volve no. 1 & 4 are closed. Value no. 2, 5, & 6 are opened. as a result of which compressed air and backwash water forces its way ~~to~~ up through the filter medium resulting in its incresed porosity that in turn increased the opportunity of entraped impurities to get washed away along with backwater into [illegible] wash trough from where it is further disposed into washwater gutter through value no. 2.

→ Once backwashing is completed, value no. 2, 5 & 6 are closed and filter is again loaded with settled water through valve no. 1 but filtered water is not used and is disposed in wash water gutter through valve no. 3. as sufficient time is provided for the regrowth of schemutzdecke and the removal of left over entrapped impurities.

→ After sufficient time, valve no. 3 is closed and filtered water is collected through value no. 4.

→ The entire-process of backwashing is completed within 15-30 minutes.

→ The washing period is normally 24-42 hrs.

→ The amount of water required for backwashing is 2-5% of water required to be treated by filter in a day.

→ Velocity during back-washing is in range of 15-90cm/min

(45cm/min) [This range is in the basis of fact the back wash velocity must be less than settling velocity of smallest sand particle.

→ ROF → 3000 - 6000l/m²/hr. (≃ 30SSF)

→ Rate of application of wash water 6-16 times than that of rate of filtration.

→ The flow of water during filtration is in laminar and is in transition during backwashing.

→ In rapid sand ~~san~~ filter better gradation of gravel layer is required because distribution of back wash water is critical function which takes place through gravel layer only.

→ Area of tank - 10-80m².

→ Minimum no. of unit is required 2(1+1) & addinational

→ Minimum no. of unit can be found. $\boxed{N = 1.22\sqrt{Q}}$ Q(MLD)

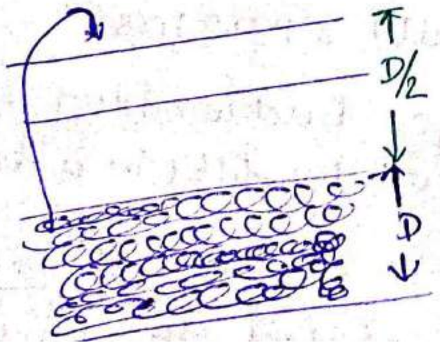
→ Sand layer depth is 60-90 cm.

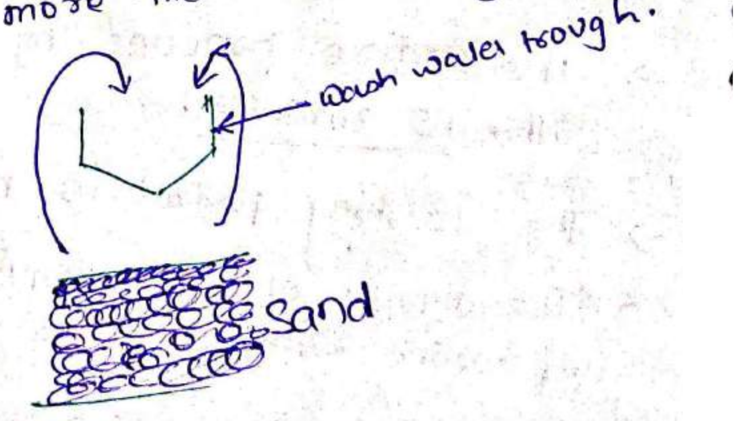
→ D_{10} of sand = 0.35-0.55m

→ C_u of sand = 1.2-1.6

→ Bacteriological efficiency is 80-90%.

→ If D is the depth of filter medium (sand), then top of the wash water trough must be placed atleast at a distance of $\frac{D}{2}$ from the top surface of filter medium. As filter medium is not likely to expand by more than 50% during backwashing.





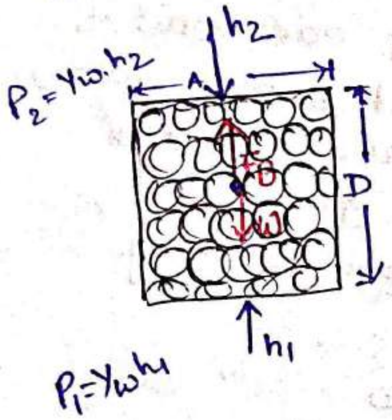
→ To hydraulically expand the porous bed, loss of head through it is equal to bouyant weight of particle forming the bed.

→ Head loss required to expand the bed.

$$H_L = D(1-\eta)(G-1)$$

→ Head loss through expanded bed = Head loss through unexpanded bed as there is no change in bouyant weight of particle during expansion.

→



$$\Sigma F_y = 0 \quad [\text{for medium to be in equilibrium}]$$

$$P_1 A + F_B - P_2 A - W = 0$$

$$P_1 A - P_2 A = W - F_B$$

$$\boxed{\text{loss of Energy} = \text{Boyant weigth} = B_w}$$

$$\gamma_w h_1 A - \gamma_w h_2 A = V_s \gamma_s - V_s \gamma_w$$

$$\gamma_w A (h_1 - h_2) = \underline{V_s} \gamma_w \left(\frac{\gamma_s}{\gamma_w} - 1 \right)$$

$$\gamma_w . A . H_L = \frac{V}{1+e} \gamma_w (G-1)$$

$$\gamma_w . A . H_L = \underline{V}(1-n) \gamma_w (G-1)$$

$$\cancel{\gamma_w} \cancel{A} . H_L = \cancel{A} \times D (1-\eta) \cancel{\gamma_w} (G-1)$$

$$\boxed{H_L = D(1-\eta)(G-1)}$$

Head loss through expand bed = Head loss through unexpanded bed.

$$D'(1-\eta')\underline{(G-1)} = D(1-n)\underline{G-1)}$$

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

Porosity of expanded bed is experimentally found to be function (V_S, V_B).

$$\eta' = f(V_B, V_S)$$

$$\eta' = \left(\frac{V_B}{V_S}\right)^{0.22}$$

Q A rapid sand filter is to treate water for 275000 people with water demand of 200 l/c/d. Rate of filtration = 15 $m^3/m^2/hr$ Determine the no. of fillers required for treating this water. The available surface area of each unit is 10×4 m^2. Also compute upflow velocity and head loss to expand the bed to 0.66 m from its original depth of 0.6 m. The porosity of bed 0.5, G = 2.5, d = 0.6 mm, C_D = 5.02. Flow is in transition during back washing.

(i) Design Discharge through filter = $Q_D = Q_{MD}$

$= ~~Q~~\ 1.8 \times Q_{AD}$

$= 1.8 \times 275000 \times 200$

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

Assuming 5% of back wash water is used in 30 minutes

Net design discharge through filter $= Q_D + 5\%\ Q_D$

$= 1.05 \times 99 \times 10^6$

$= 103.95 \times 10^6$ l/d

Net effective design discharge through filter.

$$Q_{ND}' = \frac{103.95 \times 10^6\ l}{23.5\ hr.}$$

$$\text{Plan area} = \frac{Q_{ND}'}{ROF} = \frac{103.95 \times 10^6 \times 10^{-3}}{23.5 \times 15}$$

Plan area = 294.8 ~ 295 m^2

$PA_I = 10 \times 4 = 40 m^2$.

$$N = \frac{PA}{PA_I} = \frac{295}{40} = 7.375 = 8 + ① \text{— Standby Unit}$$

$$= 9$$

(ii) $h_L = D(1-n)(G-1)$

$h_L = 0.6(1-0.5)(2.5-1)$

$= 0.45 m$

(iii) $h_L' = h_L$

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

$0.66(1-n') = 0.6(1-0.5)$

$n' = 0.545$

$$V_S = \sqrt{\frac{4}{3}\frac{(G-1)gd}{C_d}}$$

$$= \sqrt{\frac{4}{3}\frac{(2.5-1) \times 9.81 \times 0.6 \times 10^{-3}}{5.02}}$$

$= 0.048 m/s$

$$\eta' = \left(\frac{V_B}{V_S}\right)^{0.22}$$

$$0.545 = \left(\frac{V_B}{V_S}\right)^{0.22}$$

~~$V_B =$~~ [illegible]

$V_B = 3.04 \times 10^{-3} m/sec.$

~~$\log 0.545 = 0.22[\log V_B - \log V_S]$~~

$$0.545 = \left(\frac{V_B}{0.048}\right)^{0.22}$$

$$0.545 \times 0.048^{0.22} = V_B^{0.22}$$

$$V_B = \sqrt[0.22]{0.545 \times 0.048^{0.22}}$$

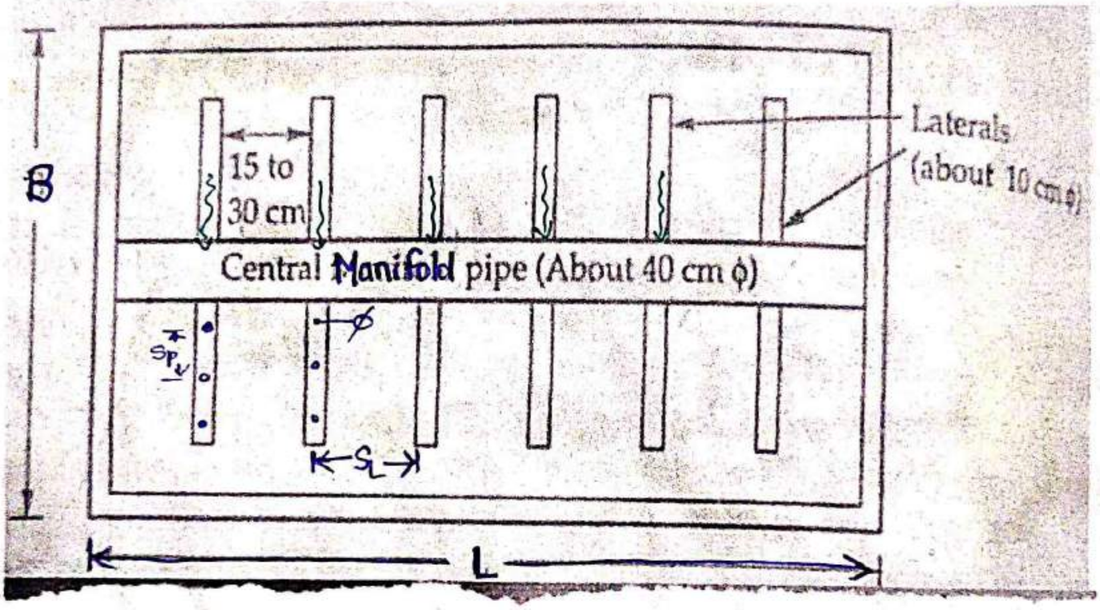
$= 3.04 \times 10^{-3} m/sec.$

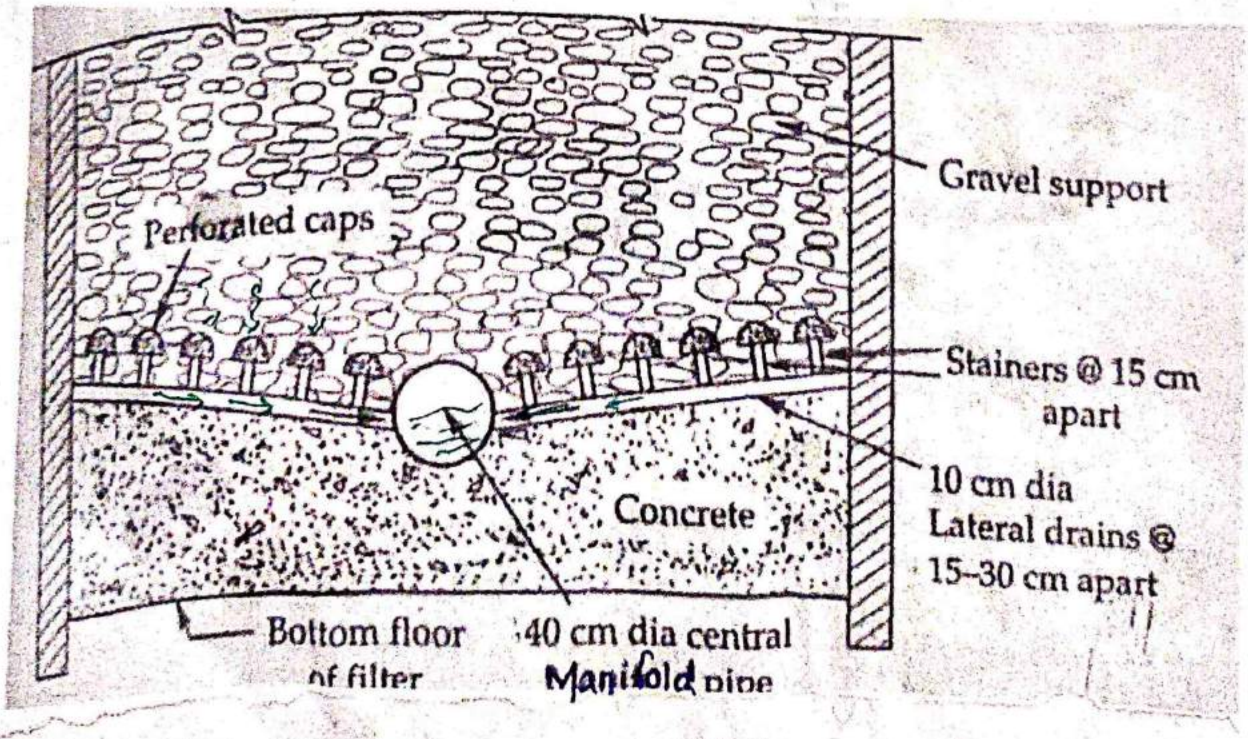
→ When water passes through filter medium loss in its head takes place to overcome the resistence offered by medium particles. Initial head loss through the filter is given by "Roes Equation".

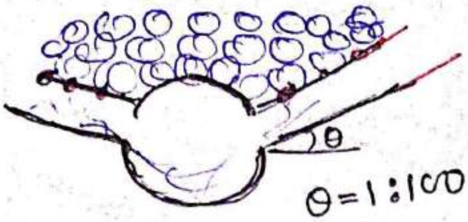
$$\boxed{H_L = \frac{1.067\, V^2 \cdot D}{\phi \cdot g \cdot \eta^4} \cdot \frac{C_D}{d}}$$

ϕ = shape factor.

Design of Under-Drainage System.







→ Lateral andmanifold system of under drain ~~po~~ provided. in RSF which is design to carry back wash discharge.

→ Dia of perforation = 6 to 13 mm.

→ spacing between perforation = 7.5 to 20 cm. c/c.

→ spacing between the laterals. is 15 to 30 cm.

→ No. of laterals, $N = \left(\frac{L}{S_L}\right) \times 2$, (clear spacing).

If spacing center to center = $\left(\frac{L}{S_L} + 0.5 + 0.5\right)^2$

→ Total cross section area of perforation = 0.2% of filter area.

→ Cross-section area of one lateral = 2/4 times of x-x area of perforation in it.

Factor 2 is used when 13mm peroration adopted.
Factor 4 is used when 6mm perforation adopted.

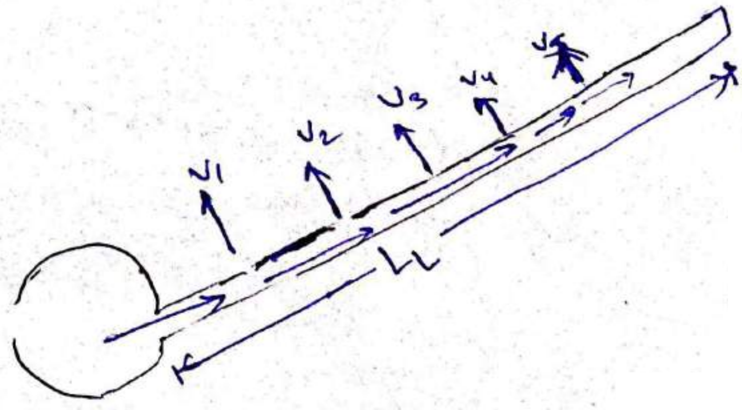
$\left(\frac{\pi}{4} \times D_L^2\right)$ = 2/4 times of x-x area of perforation in it.

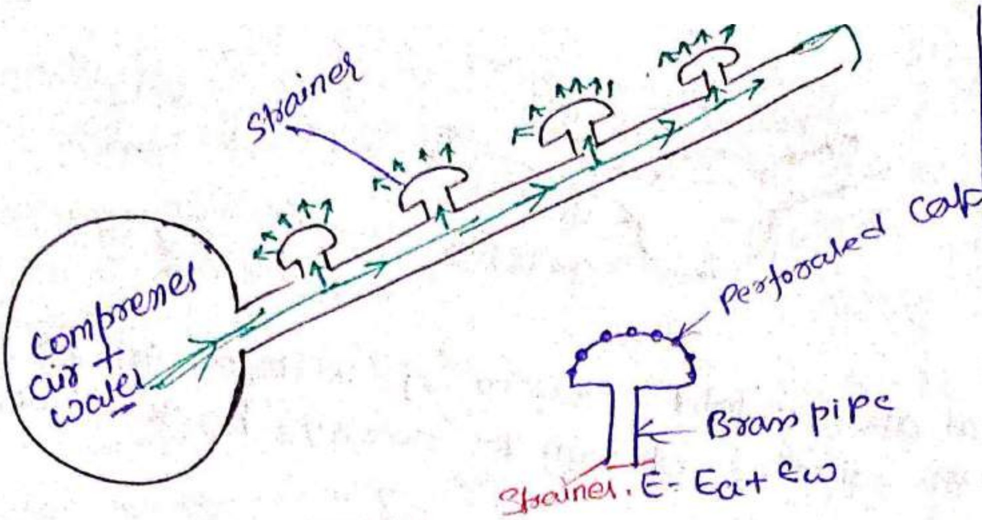
→ X-X area of Manifold = Cross section of all laterals.

$\frac{\pi}{4} D_M^2 = (x - x_L) \cdot N$.

Check

$\frac{\text{length of lateral}}{\text{dia of lateral}} \not> 60$





$E = E_a + E_w$
Slow velocity wash

$E = E_w$
high velocity wash

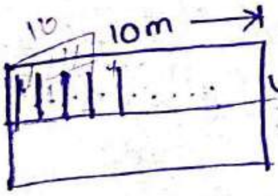
→ High velocity wash is done when we ~~use~~ donot use strainer

→ slow velocity wash is done when we use strainer.

Q. Same Question Design lateral and Manifold.

Area = 10×4
let, dia of perforation = 13mm
let, spacing of perforation = 20cm c/c.
let, spacing between lateral = 20 cm c/c.

$$\text{No. of lateral} = \left(\frac{L}{S_L}\right)\times 2 =$$
$$= \frac{10\times10^2}{20}\times 2 = 100$$



Cross section area of perforation area = 0.2% of C.A

$$= \frac{0.2\times10\times4}{100}$$
$$= 0.08\,m^2$$
$$= 0.08\times10^4$$
$$= 800\,cm^2$$

Cross-sectional area of one lateral = 2 × total cross section area in one lateral

$$\frac{\pi D_L^2}{4} = 2\times\frac{800}{100}$$

$$D_L = 4.5\,cm$$

Crossection area of Manifold =

$$\frac{\pi D_M^2}{4} = 2 \times 16 \times 100$$

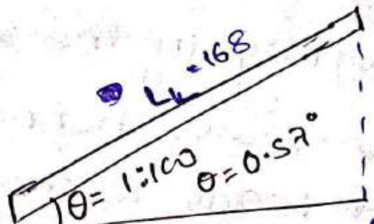
$$D_m^2 = 63.8 \text{ cm}$$

$$D_m = 64 \text{ cm}$$

Check

$$\frac{\text{length of lateral}}{\text{dia of lateral}} = \frac{168.00}{4.5} = 37.33 \ngtr 60$$

OK



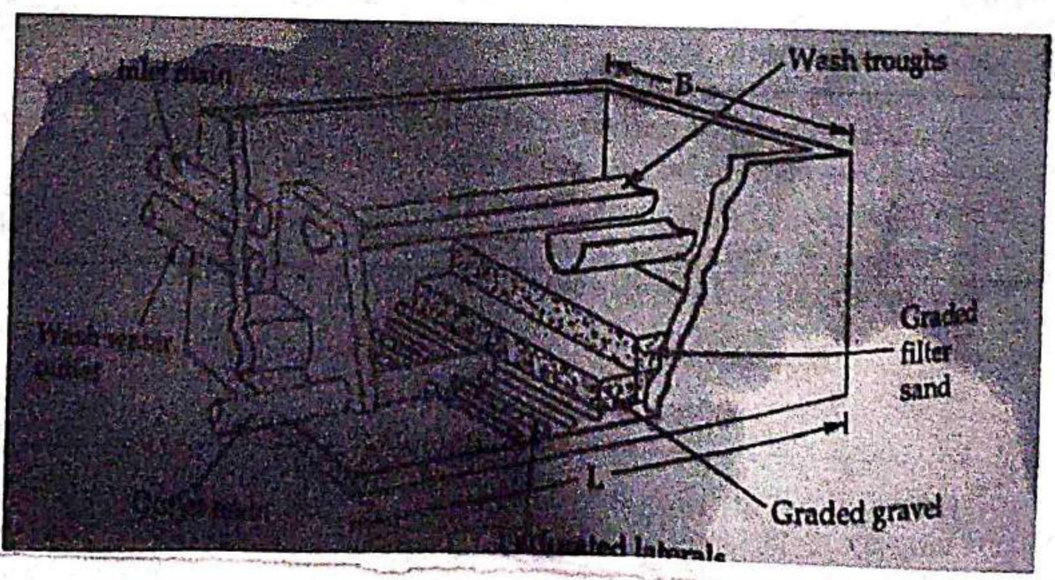
$$\left(\frac{B - D_m}{2}\right) = \frac{400 - 64}{2} = 168 \text{ cm}$$

$$\cos\theta = \frac{168}{L_L}$$

$$L_L = \frac{B - D_M}{2} \times \frac{1}{\cos\theta}$$

$$= \frac{168}{\cos 0.57}$$

$$= 168.0083$$

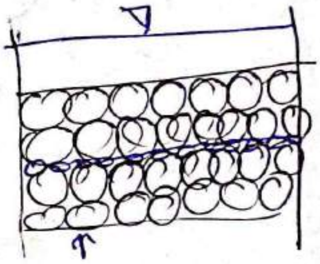


Operational Problem of RSF

These are the problem developed during operation of filter:

(i) Air Binding

→ Initial loss of head through freshly cleaned filter is in range of 15-30cm which goes on increasing as more and more impurities are entrapped in the void of filter medium. a stage comes during operation of filter when resistance offered by medium particles along with entrapped impurities exceed the static head of water. At this point, the bottom layer of the filter medium starts acting like a vaccume resulting in the release of dissolved gases from the water bubbles of which rises to the surface and leads to the binding of medium particles from the bottom thereby seriously effecting the operation of the filter.

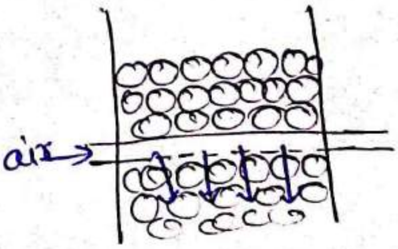


→ To avoid Air Binding proper cleaning of filter should be done when head loss becomes 2.5-3.5m or when negative head becomes 1.2m.

→ Avoid increase in temperature during filtration.

→ Removed the algae from source from where water is derived.

→ Pumping in air.

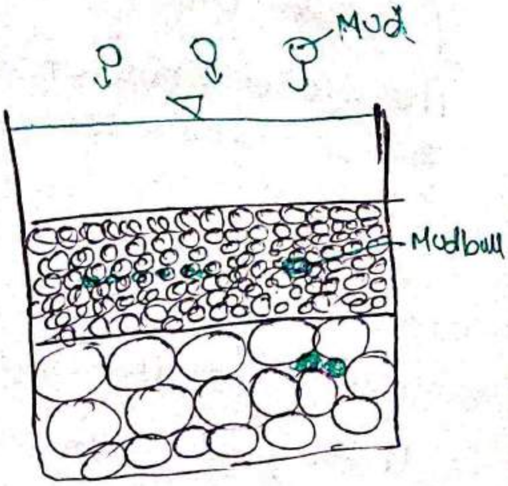


→ It only takes place in RSF because, in SSF, impurities are entrapped in upper layer only.

(ii) Mudball formation.

→ Role of gravel layer is to uniformly distribule the backwash water to filter medium (sand).

→ It is not found in SSF because no backwashing is done in that case, no effect ~~of~~ if mud particle is in gravel layer.



→ Mud from atmosphere enters into the filter medium and over a period of time, it sinks down to the bottom layer of the medium where it combine with entrapped impurities and leads to the formation of mudballs that if enter into the gravel layer cause turblence around them during backwashing leading to removal of medium particles along with the backwash water out from the filter.

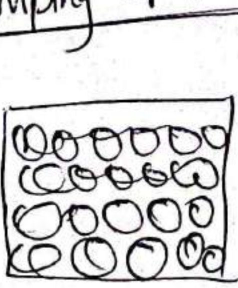
→ To avoid mud ball formation compress air scouring for 3-5 min is done during backwashing along with surface racking.

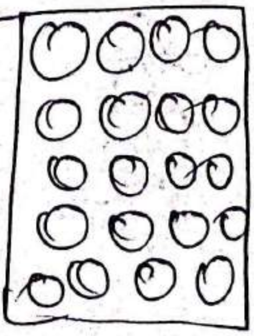
(iii) Cracking of the filter.

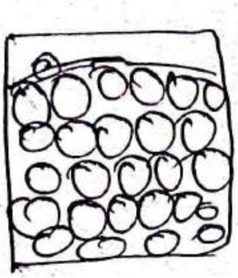
Due to alternate wetting & drying of medium particles, shrinkage cracks are developed over the surface of Medium, size of which goes on increasing due to constant application of water over the medium surface.

→ These problem not found in new filter, it will develop later.

(iv) Bumping of filter Bed







Pressure filters

These are types of filter in which energy required by water to overcome the resistance of medium provided by application of pressure.

→ These filter are same as RSF with a difference, here treatment is carried out in closed container and it utilises raw coagulated water and flocculation is done inside the filter itself.

→ These are less efficient as compared to RSF but offers higher rate of filtration of 6000 l/m²/hr

→ Dia of filter = 1.5 − 3m

→ Depth − 3.5 − 8.5m

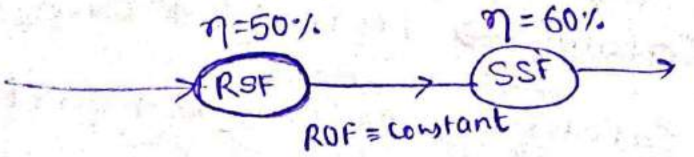
→ They are suitable for treatment of limited quantity of water. eg. water in swiming pool or industrial water.

→ Their size is comparatively much smaller than gravity filter hence are more flexible.

→ There cleaning is also done by back washing.

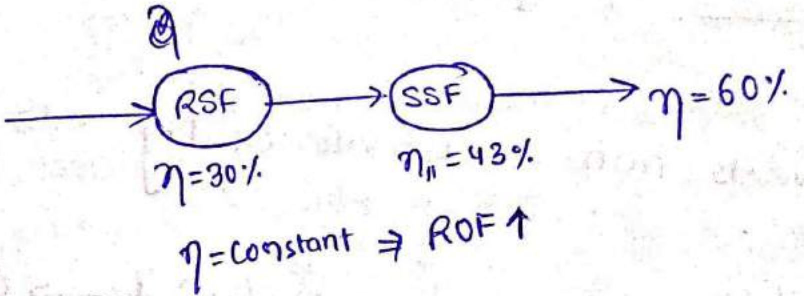
Roughning filter and Double filteration.

→ In order to increase the ROF of SSF without compromising with its efficiency rapid sand filter is used before SSF, the process is termed as Double dual media/multi media / mixed media filteration. & rapid sand filter used is termed as roughning filter.



$\eta = 80\%$

$$\boxed{\eta_0 = \eta_I + (1-\eta_I)\,\eta_{II}}$$



IV Diatomaceous Earth filter/ Diatomite filter.

→ These filters are used to remove chlorine resistant dysentery cysts, or larve (egg) of Schistusomiasis.

→ Diatomaceous earth or diatomite is a naturally occuring soft silicious sedimentary rock that is easily crumbled into a fine white to off-white powder.

Softening

→ It is the process of removal of hardness from water.
→ It can be achieved by any of the following method:-

Lime Soda Method

(a) Lime removes all the carbonates hardness from the water.

$$\left.\begin{array}{l} Ca(HCO_3)_2 + \underset{\text{lime}}{Ca(OH)_2} \longrightarrow 2\,CaCO_3\downarrow \\ MgCO_3 + Ca(OH)_2 \rightarrow Mg(OH)_2\downarrow + CaCO_3\downarrow \\ \underline{Mg(HCO_3)_2} + \underbrace{Ca(OH)_2}_{1} \rightarrow Mg(OH)_2\downarrow + \underline{Ca(HCO_3)_2} \xrightarrow{Ca(OH)_2} 2CaCO_3 \end{array}\right\}\alpha$$

(b) Lime converts non-carbonate of Mg into NCH of Ca (β)

$$Mg\begin{cases} Cl^- \\ SO_4^- \\ NO_3^- \end{cases} + Ca(OH)_2 \longrightarrow Mg(OH)_2\downarrow \longrightarrow Ca\left.\begin{cases} Cl^- \\ SO_4^{2-} \\ NO_3^- \end{cases}\right\}\beta$$

(c) Lime also react with carbon-dioxide to get wasted out as $CaCO_3$ ppt.

$$Ca(OH)_2 + CO_2 \longrightarrow CaCO_3\downarrow \;\}\gamma$$

(d) NCH of calcium either originally present or converted from magnesium is removed by soda Ash. (δ)

$$Ca\begin{cases} Cl^- \\ SO_4^- \\ NO_3^- \end{cases} + \underset{\text{(soda ash)}}{Na_2CO_3} \longrightarrow CaCO_3\downarrow + Na\left.\begin{cases} Cl^- \\ SO_4^- \\ NO_3^- \end{cases}\right\} d$$

→ Precipitate of calcium carbonate is formed at pH greater than 9 to 9.5. Precipitate of $Mg(OH)_2$ is formed at pH>11, hence pH of water is not is above range it is first increased by addition of <u>base</u>. <u>like lime</u>.

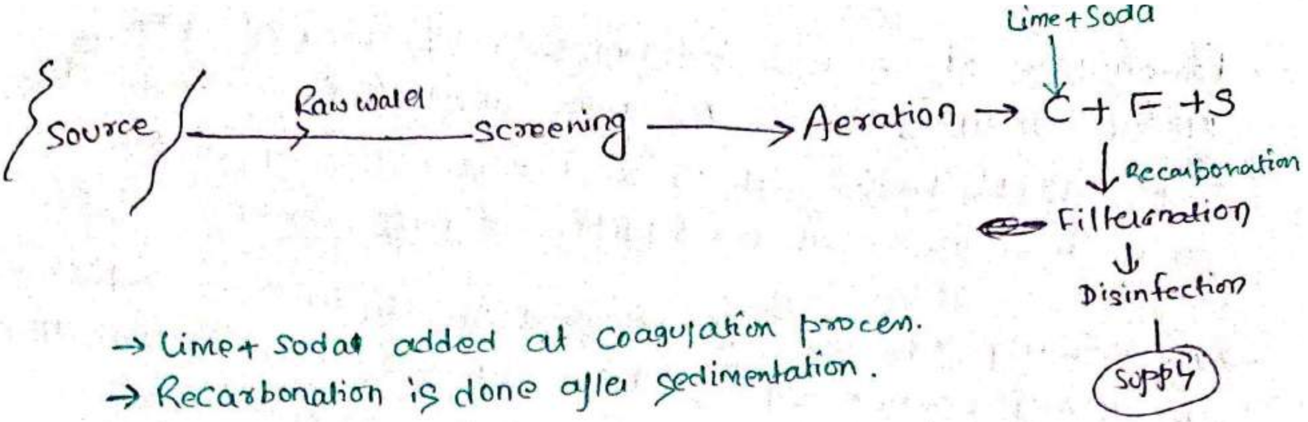
→ Availability of lime for removal of hardness is reduced (Y) in the presence of CO_2. hence if CO_2 is more than 10 ppm then it is first removed by aeration.

→ This method also helps in removing micro-organism and avoiding <u>corrossion</u> in pipe.
└ Catalyst - oxygen, acidity.

→ Complete removal of hardness by chemical precipitate can't be achieved under normal treatment condition. as up to 40 mg/lit of $CaCO_3\downarrow$ & 10mg/lit of $Mg(OH)_2$ is formed which if allow very slowly & remain in softened water would lead to incrustation to enter into distribution system. precipitate are again being made soluble in pipes. hence these in the process termed as in water by passing CO_2 in it. which takes place at pH ≃ 9.5. <u>recarbonation</u>.

$$CaCO_3\downarrow + CO_2 \xrightarrow{H_2O} Ca(HCO_3)_2$$
$$Mg(OH)_2\downarrow + CO_2 \xrightarrow{H_2O} MgCO_3$$
} pH 9.5

→ This method does not gives zero hardness water, but reduces hardness it upto 50mg/lit



→ Lime + soda added at coagulation process.
→ Recarbonation is done after sedimentation.

→ Large quantity of sludge is formed when lime-soda method is used for removal of hardness. dry weight of which can be computed as follows:-

(a) 1mg of Ca hardness removed as $CaCO_3$ forms 1mg of sludge of $CaCO_3$.

(b) 1 mg of Mg hardness removed as $CaCO_3$ form 0.58 gm of sludge of $Mg(OH)_2$

(c) 1 mg of lime added as $CaCO_3$ forms 1mg of sluge as $CaCO_3$.

$Ca^{2+} \rightarrow CaCO_3 \downarrow$	$CaO \xrightarrow{H_2O} Ca(OH)_2 \rightarrow CaCO_3 \downarrow$	$Mg^{2+} \rightarrow Mg(OH)_2 \downarrow$
(1 mole as $CaCO_3$) 1mole	(1mole as $CaCO_3$) 1mole	1mole as $CaCO_3$ removed → 1mole
100gm 100gm	100gm 100gm	100gm → 58gm
1 mg/lit 1mg/lit	1mg/lit 1mg/lit	1mg/lit → 0.58 mg/lit of ppt

Dry sludge (mg/lit) = [Ca^{2+} hardness removed as $CaCO_3$ (mg/lit) + lime added as $CaCO_3$ (mg/lit) + 0.58 mg^{2+} hardness removed as $CaCO_3$]

mole concept

Q. For treatment of 10MLD of given water sample compute annual quantity of lime and soda required also calculate quantity of sludge formed in sedimentation tank provided after it having efficiency of 65%. Raw water suspended solid concentration is 50 mg/lit & metallic flocculating alum used is 20mg/lit. Solid content of sludge, S = 1.5% specific gravity = 2.5.

		ppm			ppm
precipitate →	$CaCO_3$ (ppt)	100	soda-	$CaCl_2$	50
Lime →	$Mg(HCO_3)_2$	150	ppt-	SiO_2	60
(salt) × →	NaCl	-120	Lime	CO_2	10
Lime, Soda →	$MgSO_4$	-110	soda	$CaSO_4$	100.

(1 mole of $Mg(HCO_3)$ react with 2 mole of lime)

miligram/liter (40+16) (CaO)

$$\text{I) Quantity of lime} = \left(\underset{\alpha}{\frac{150}{146}} \times 2 + \underset{b}{\frac{110}{(120)}} \times 1 + \underset{y}{\frac{\overset{CO_2}{10}}{44}} \times 1\right) \times 56 \times 10 \times 10^6 \times 365 \times 10^{-9} \text{ ton/year.}$$

$= 653.82$ tones/year.

$$\text{Quantity of soda} = \left(\frac{\overset{MgSO_4}{110}}{120} \times 1 + \frac{\overset{CaCl}{50}}{111} \times 1 + \frac{100}{136} \times 1\right) \times \overset{\text{soda ash}}{106} \times 10 \times 10^{-6} \times 365 \times 10^{-9}$$

$= 813.42$ tones/year,

Soda ash = Na_2CO_3
$= 23 \times 2 + 12 \times 16 \times 3$
$= 106$

$$\text{II wt of Solid in Sludge} = \left[\left[\frac{\overset{CaCl}{50}}{111/2} + \frac{100}{136/2} + \frac{\overset{\alpha+\beta+y\,(\text{lime})}{179.13}}{56/2} + 0.58\left(\frac{150}{146/2} + \frac{110}{120/2}\right)\right] \times 50\right.$$

miliequivalent $CaCO_3$

$$\left. + \underset{(\text{solid})}{50} + \underset{\text{Due to alum}}{20 \times 0.234} + \underset{(CaCO_3\downarrow)}{100} + \underset{(SiO_2\downarrow)}{60}\right] \times 0.65 \times \underset{\text{MLD}}{10} \times 10^6 \times 10^{-6}$$

$= 4978.6$ kg/day.

$G = \frac{W_s}{W_w}$

$G = \frac{Y_s}{Y_w}$

$G \times Y_w = \frac{W_s}{V_s}$

$V_s = \frac{W_s}{G \times Y_w}$

$G = \frac{\text{kg/day}}{Y_w}$

Volume of solids in sludge $= \frac{4978.6}{2.5 \times 10^3} = 1.99 \text{ m}^3/\text{day}$

$S = 1.5\% \Rightarrow 1.5$ kg of solid + 98.5 kg of water = 100 kg of slu

wt. of water in sludge $= \frac{98.5}{100} \times \frac{4978.6}{1.5} =$
326928 kg/day

Volume of water $= \frac{326928 \text{ kg/day}}{1000}$
$= 326.928 \text{ m}^3/\text{day}$

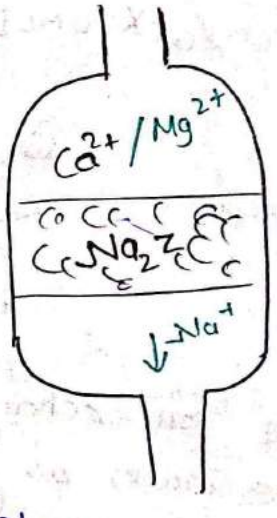
Volume of sludge = Volume of (water + Solid)
$= 326.928 + 1.99$
$= 328.89 \text{ m}^3/\text{day}$

(ii) Zeolite Method (Cation or Base Exchange Method)

- → Zeolite is natural or synthetic cation or base exchange hydrated silicate of aluminium and sodium.
- → The equipment used in this process is same as that of pressure filter.
- → Hard water having multivalent cation is passed through zeolite bed which exchange its univalent sodium ion with multivalent cation causing the hardness in water and get itself converted into exhausted zeolite which is to be again regenerated by passing 5-10% brine (NaCl) solution through it before carrying out the further treatment with it.

$$\underbrace{Na_2O \cdot Al_2O_3 \cdot xSiO_2 \cdot yH_2O}_{Na_2Z}$$

$$Ca^{2+}/Mg^{2+} \begin{cases} Cl^- \\ SO_4^{2-} \\ NO_3^{2-} \\ CO_3^{2-} \\ HCO_3^- \end{cases} + \underline{Na_2Z} \rightarrow \underset{\text{(Exhausted Zeolite)}}{Ca/MgZ} + Na^+ \begin{cases} Cl^{-1} \\ SO_4^{2-} \\ NO_3^- \\ CO_3^{2-} \\ HCO_3^- \end{cases}$$



Regeneration of exhausted Zeolite,

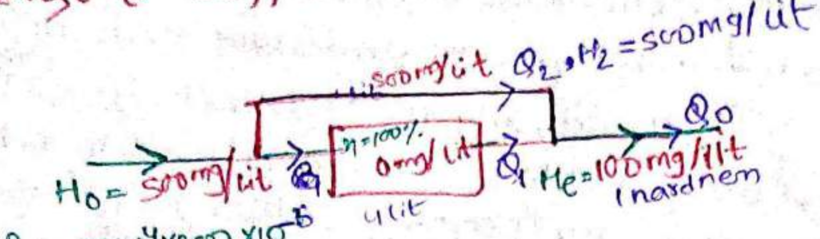
$$\underset{\text{Exhausted Zeolited}}{Ca/MgZ} + \underset{\text{Brine sol}^n\ (5-10)\%}{NaCl} \longrightarrow \underset{\text{Regenerated zeolite.}}{Na_2Z} + Ca/MgCl_2$$

- → Hardness comes in water used for regeneration, will be thrown out.

- → This method gives zero hardness water and does not lead to formation of sludge while removing the hardness.
- → If iron and Magnese are also present in water this process becomes costly as regeneration of iron magnese zeolite does not takes place.

$$Fe/MnZ + NaCl \xrightarrow[\text{no regeneration}]{\times}$$

Q A ground water having hardness of 500mg/lit. is treated with ion exchange method having 100% hardness removal efficiency and is to be supplied in the community of 50000 individuals having water demand of 200 l/c/d if 100mg/lit of hardness is required in treated water to ensure sufficient taste compute the by pass flow rate.



$Q_0 = 5\times10^4\times200\times10^{-6}$
$= 10MLD$

$$Q_1 + Q_2 = Q_0 \Rightarrow Q_1 + Q_2 = 10MLD$$

$$\underline{He.\,Q_0} = H_1\times Q_1 + H_2\times Q_2 \quad \text{—(i)}$$

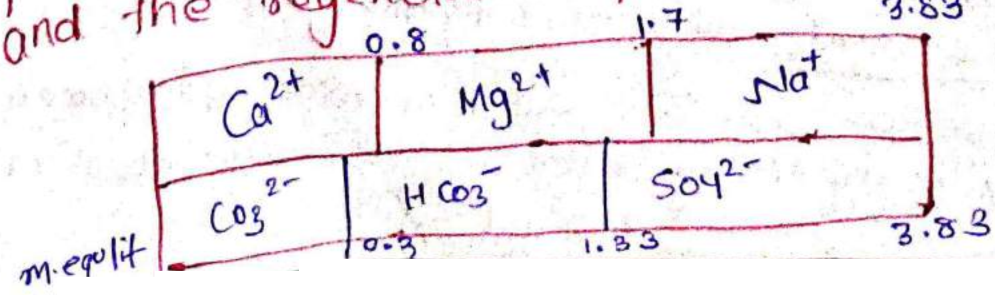
$$100mg/lit\times 10MLD = \overline{0\times Q_1} + 500\,mg/lit\times Q_2$$

$$Q_2 = \frac{100\,mg/lit\times10\,MLD}{500\,mg/lit}$$

$$\boxed{Q_2 = 2MLD}$$

Q An ion exchange softner is to be ~~remo~~ used to treate the water as given below. The medium selected has an obsorptive capacity of 90kg/m³ @ a flow rate of 0.4 m³/min/m².

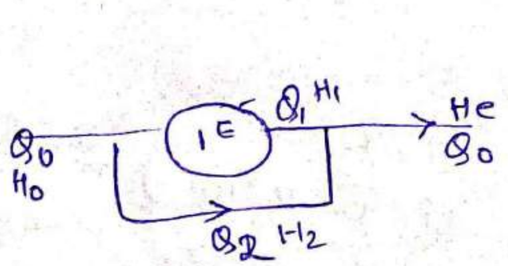
Regeneration is accomplised using 150 kg of sodium chloride per cubic meter of Zeolite in 10% solution Determine the volume of medium required and the physical arrangement for continuous operation in the filter bed also determine the chemical requirement and the regeneration cycle time.



Assume regeneration loading rate 0.64 m³/m²/min. rate of flow 25000 m³/d. and required hardness = 75 mg/lit.

Sol TH = 1.7 × 50 = 85

$H_e \times Q_e = H_1 Q_1 + H_2 Q_2$



$75 \times 25000 = 0 \times Q_1 + 85 \times Q_2$

$Q_2 = 22059\ m^3/d$

water to be treated, Q_1

$Q_e - Q_2 = 25000 - 22059$

$= 2941\ m^3/day$.

$$\text{Total hardness to be removed} = \overset{m^3/day}{2941} \times \overset{mg/lit}{85} \times 10^3 \times 10^{-6}$$

$= 250\ kg/day$.

$$\text{Volume of medium required} = \frac{250}{90} = 2.78\ m^3$$

$$\text{Area of medium require} = \frac{2941\ m^3/day}{0.4\ m^3/min/m^2}$$

$$= \frac{2941}{0.4 \times 24 \times 60}$$

$$= 5.106\ m^2$$

~~depth~~ Provide 5 units operation unit + 1 standby.

$$SA_I = \frac{5.106}{5} = 1.02\ m^2$$

$$\text{Depth of each unit, } H_{A_1} = \frac{Volume}{5 \times SA_I}$$

$$= \frac{2.78}{5 \times 1.02} = 0.545\ m$$

$SA_I = 1.02\,m^2$

$\frac{\pi}{4} \times D^2 = 1.02\,m^2$

$\boxed{D = 1.14\,m}$

II $\left.\begin{array}{l} D = 1.14\,m. \\ H = 0.545\,m \end{array}\right\}$ @6(S+1)

III Quantity of Sodium chloride required $= 150 \times 2.78 = 417$ kg.

Qty of Brine solution $= \frac{100}{10} \times 417$

$= 4170$ kg/day.

Volume of brine Soln $= \frac{4170}{10^3} = 4.17\,m^3/day$

IV Regeneration cycle time $= \frac{4.17}{0.04\,m^3 \times 5.1}$

$= 20.44$ minute.

Disinfection of Water.

→ It is the process of removal of disease causing micro-organism termed as pathogens from water.

→ The process of removal of all types of micro-organism is termed as Sterlization.

→ Disinfection is a type of sterlisation

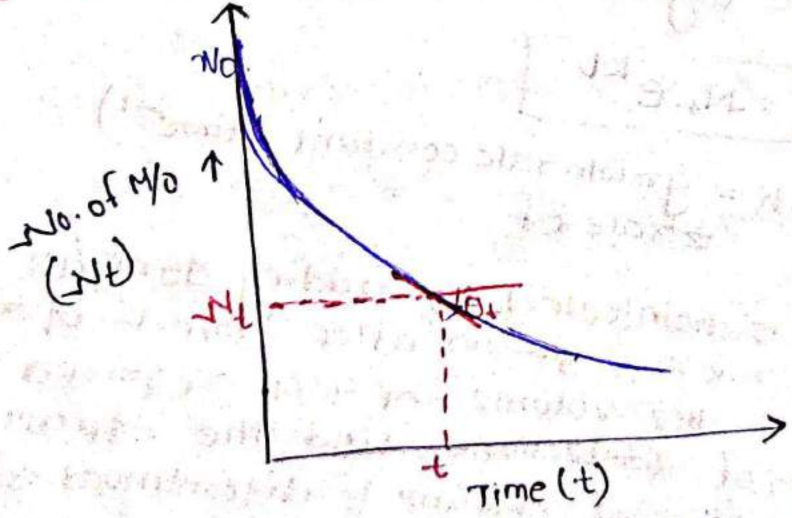
→ It is the most significant treatment of water and is adopted at the final stage in the chain of water purification

→ Disinfection can be done by any of the following method.

Physical
- → UV rays – sterlization.
- → Boilis. – temporary but not in distribution system
  - ↳ costly method

chemical
- → Acids & alkalies
  ($P_H < 3$) ($P_H > 11$)
  Acidic corrosion and irritant toxic
- → Metallic ions. (Ag^+, Hg^+)
- → Oxidising agents (Cl_2, Br_2, I_2, O_3, $KMnO_4$)

Kinetics of Disinfection. (As per chick's law)



$$\frac{dN_t}{dt} \propto -N_t \quad [\theta_x \propto y]$$

↳ jatore of change of variable.

$$\frac{dN_t}{dt} = -KN_t$$ K=disinfection const. (time^{-1})

= It signify rate of disinfection.

~~$$\int_{N_t=0}^{N_t=N_t} \frac{dN_t}{dt} = -K\int_{t=0}^{t=t} dt$$~~

$$\int_{N_t=N_0}^{N_t=N_t} \frac{dN_t}{N_t} = -K\int_{t=0}^{t=t} dt$$

$$\left|\ln N_n\right]_{N_0}^{N_t} = -K\left|t\right|_0^t$$

$$\ln N_t - \ln N_0 = -Kt$$

$$\ln\left(\frac{N_t}{N_0}\right) = -Kt$$

$$\boxed{N_t = N_0 . e^{-Kt}}$$

$$y = f(x)$$

Note → Growth of micro-organism is also observed to be exponential. $\boxed{N_t = N_0 . e^{Kt}}$

Here, K = growth rate constant (time^{-1})
≥ rate of

Q 10 MLD of water is disinfected by adding 15 mg/lit of chlorine. Find the removal of micro-organism after 5 minutes of addition of chlorine. Compute the volume of tank required to achieve 99% efficiency of disinfection. Find the efficiency of same tank if addition of chlorine is discontinued after half operation. Assume disinfection constant $K_D = 0.234\,min^{-1}$, $K_g = 0.465\,min^{-1}$.

(i) $N_t = N_0 \cdot e^{-kt}$

$N_5 = 15 \times e^{-0.234 \times 5}$

$= 4.65$ mg/lit

$\% \text{ Removal} = \dfrac{N_0 - N_t}{N_0}$

$= \dfrac{N_0 - N_5}{N_0} \times 100 = \dfrac{15 - 4.65}{15} \times 100 = 69\%$

(ii) $\% R = \dfrac{N_0 - N_t}{N_0} \times 100$

$0.99 = \dfrac{N_0 - N_0 \cdot e^{-kt}}{N_0}$

$0.99 = 1 - e^{-kt}$

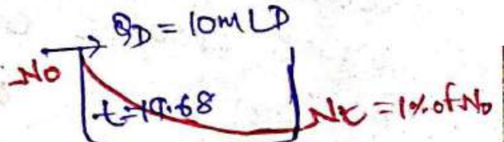
$0.99 = 1 - e^{-0.234 \times t}$

$e^{-0.234 \times t} = 0.01$

$\ln(e^{-0.234 \times t}) = \ln(0.01)$

$-0.234 \times t = -4.605$

$t = \underline{19.68 \text{ minute}}$



(ii) $d_t = 19.68$

$V = Q_D \times d_t$

$= \dfrac{10 \times 10^6 \times 10^{-3}}{24 \times 60 \times 8} \times 19.68$

$= 136.67 \text{ m}^3$

III

t = [illegible]

% removal of micro-organism after half operation; % left in water.

$N_t = N_0 \cdot e^{-kt}$

$= 15 \times e^{-0.234 \times \frac{19.68}{2}}$

$= N_0 \times 0.1$

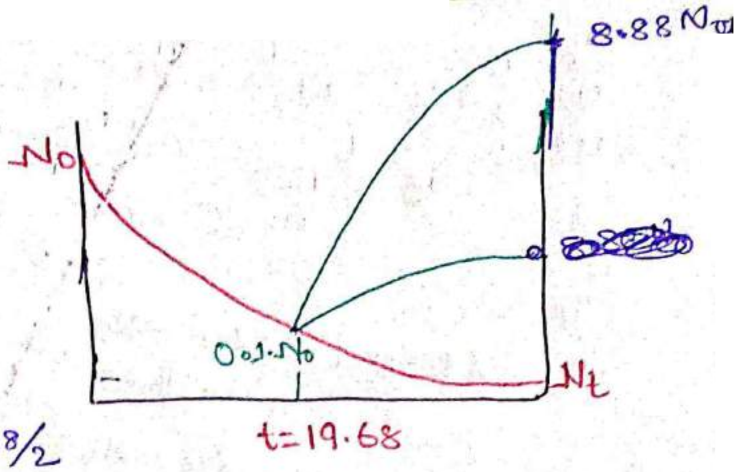
After half operation,
growth.

$N_t' = N_0' \cdot e^{kt}$

$N_t = 0.1\, N_0 \cdot e^{kt}$

$= 0.1 \times N_0 \times e^{0.456 \times 19.68/2}$

$= 8.88 \times N_0$



$$\% \text{ efficiey} = \frac{N_0 - 8.88 N_0}{N_0} \times 100$$

$$= -788\,\%$$

Minor Method of Disinfection.

i) Treatment with ozone. [future

→ It is a strong oxidising reagent which when added in water dissociates into molecular and nacent oxygen. which carries out. Disinfection and oxidation of organic matter.

$O_3 \xrightarrow{H_2S} O_2 + \underset{\text{nacent oxygen}}{O}$

→ Since it is highly reactive it quickly discipates in water, and does not safeguard it aggainst future recontamination.

→ It gives pleasent taste in water unlike other disinfectants but its comparatively costlier.

→ It destroy the cell wall of micro-organism to carry out disinfection.

→ It's normal does varies in range of 2-3 ppm.

→ It is less effective than chlorine in killing bacteria and virus but more effective in killing protozaa.

→ future recontamination chances.

used in chandigarh

(ii) Treatment with $KMnO_4$ - 6

→ It is very popular disinfectant which is used for treatment of well water supplies in villages that are contaminated to lesser extent.

→ It when added in water imparts pink colour, which if disappear indicates the presence of organic matter and micro-organism in water and hence further $KMnO_4$ is added upto an extent pink colour stands in water.

→ It increases the permeability of cell wall of micro-organism to carry out disinfection.

→ It removes 98% of bacteria in general. and 100% of cholera causing bacteria.

→ Its normal does is 1-2mg/lit at contact period of 4-6 hr.

→ $KMnO_4$ is the least oxidising agent out of all cleaning reagent used for least polluted well. ie well.

Note Watson's law.

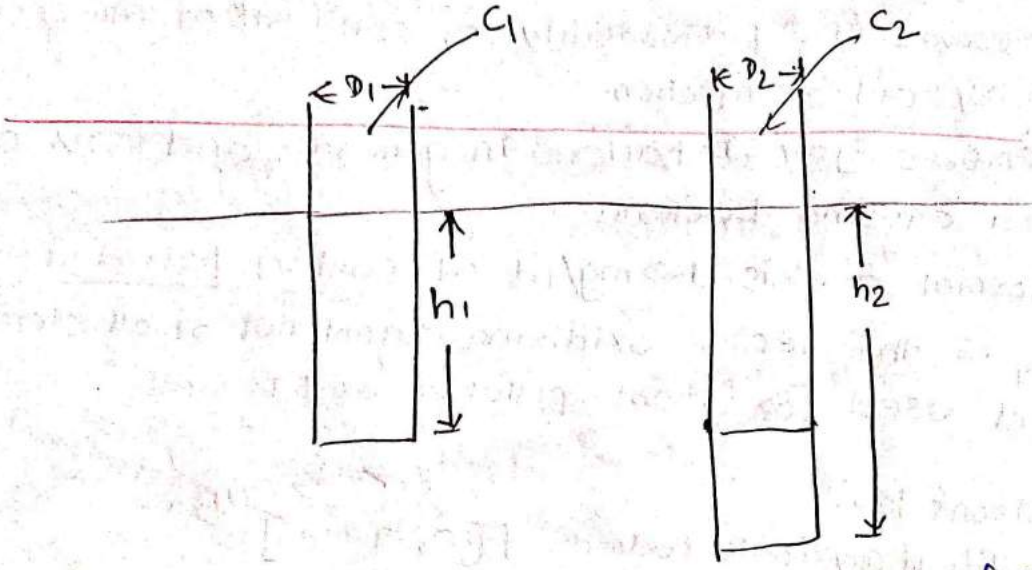
Solubility of any chemical contanct= $f[\overset{\text{concetration}}{C}, \text{Time}]$ — time of contacnt → to solube in water.

for given solubility = $t\,C^n$ = constant

n = dillution co-efficient..

Q GDK

(180) Q 40 kg of KMNO4 of is added in a well 'of dia-5m, & having water standing water depth of 10m in it. Compute the ~~does of KMNO4 is required~~ level at water @ which same disinfection would be achieved by 30kg of KMNO4 added in another well of dia-3m in same medium in half time. Assume n=1



for same degree of disinfection. Solubility of KMnO4 must be same in both the well

$$\Rightarrow t\,C^n = \text{constant}$$

$$t_1 C_1^n = t_2 C_2^n$$

$$t_1\left(\frac{W_1}{V_1}\right) = t_2\left(\frac{W_2}{V_2}\right)$$

$$\cancel{t_1}\times\frac{40}{\cancel{\frac{\pi}{4}}\times5^2\times10} = \frac{\overset{1}{\cancel{t_1}}}{2}\times\frac{30}{\cancel{\frac{\pi}{4}}\times3^2\times h_2}$$

$$h_2 = 10.416\,\underline{m}$$