

## 2. Sulphur bacteria :-

Acid released during the metabolism of Sulphur bacteria affects the durability of concrete surface. (These acids result in formation of compounds whose volume is much larger in comparison to volume of original constituents. Thereby resulting in development of cracks on concrete surface.)

Catabolism - breakdown of nutrients in lighter ones

Anabolism - utilisation of broken nutrients

Metabolism - combination of above two

Diseases caused by different types of micro-organisms

Bacteria - Typhoid, Cholera, Dysentery

Virus - Hepatitis, (Jaundice), Polio, *Influenza*

Protozoa - Amoebic Dysentery

Malaria

TCO  
T.M.S

## Treatment of water:

i) The treatment of water is dependant on quality of raw water and desired standard of treated water.

ii) There are various methods of water treatment as

- Screening
- Aeration
- Co-ogulation & Flocculation
- Sedimentation
- Filtration
- Dis-infection
- Softening
- Fluoridisation or defluoridisation.
- desalination & defluoridisation.

Friday  
19<sup>th</sup> July 2013

i) If the water is extracted from surface & is provided storage or extracted from ground having turbidity less than 10 NTU the sample is free from taste odour and dissolved gases, in this case dis-infection is sufficient for the treatment of water.

ii) If surface water contains excessive iron, dissolved CO<sub>2</sub> and odourous gases then treatments given are aeration, screening, co-ogulation & flocculation, sedimentation, filtration and dis-infection.

Note:

(i) If slow sand filters are used for filtrations, it is never preceded by co-ogulation-flocculation.

(ii) In order to remove bod taste, odow, dissolved gases and minerals, excess chlorine from water activated carbon in powder form can be used. (due to its strong adsorption properties, it removes all impurities from water).

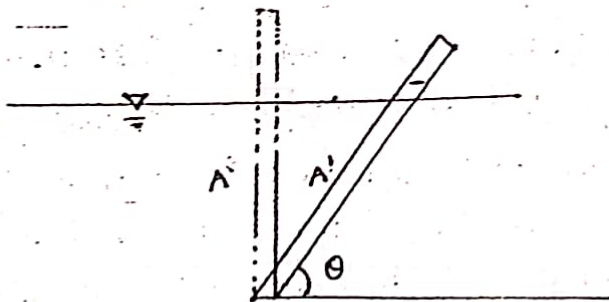
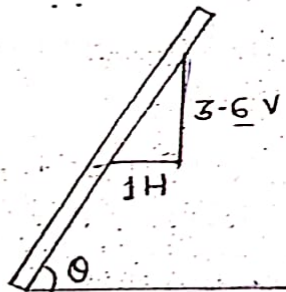
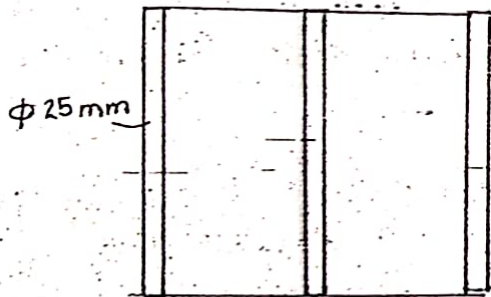
It is recommended to use activated carbon during the process of co-agulation, flocculation, as it increases the efficiency of treatment, and reduces load over the filter, It increases efficiency of co-agulation, flocculation process

### Screening :-

- i) Screening is done to remove the heavier suspended impurities from the water like plants, trees, stones, animals etc.
- ii) Screening is generally adopted with the help of screens of two types...
  - Coarse screens (Bar screens) Trash rack)
  - Fine screens

### Coarse screen/Trash rack :-

It is in the form of bars of dia. 25 mm and spacing of 20-100 mm - is maintained between the bars. These screens are generally placed at an inclination of 3.6 V : 1 H



$$\cos \theta = \frac{A}{A'}$$

$$A' = A \sec \theta$$

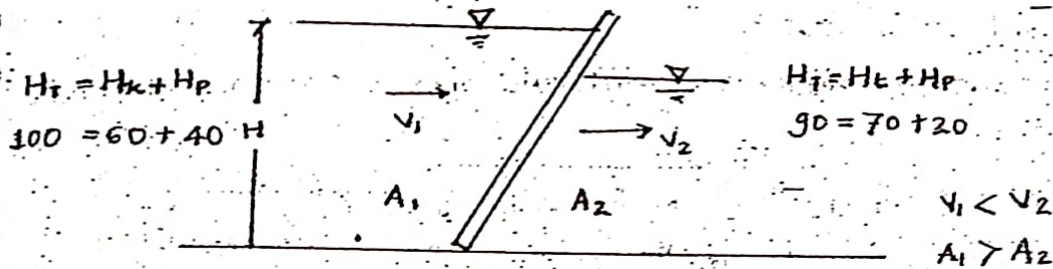
$$\therefore A' > A$$

$$Q = AV = A'V'$$

$$V' < V$$

$$\therefore A' > A$$

- (i) Screens are always placed at inclination, as it increases the efficiency of removal of impurities over a screen by reducing the velocity of flow giving more opportunity is provided to suspended impurities to retain over screen.
- (ii) Inclination helps in better cleaning of screens by racking mechanism.



Head loss in water through screen

$$H_L = \frac{k}{2g} (v_2^2 - v_1^2)$$

$$H_L = 0.0729 (v_2^2 - v_1^2)$$

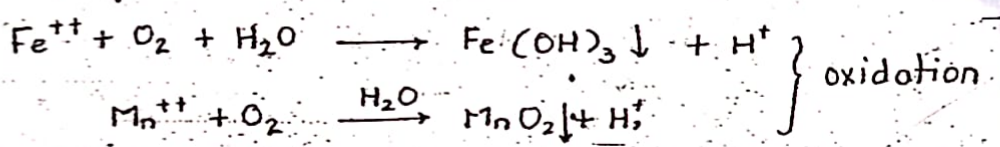
Fine screens:

Fine screens are generally in the form of wire mesh of opening size 10 mm. Fine screen is generally avoided in the normal treatment of water, as it gets frequently clogged requiring its frequent cleaning. Thereby increasing its operational cost of treatment.

It is recommended to use coarse screen instead of fine screens and remove the finer suspended impurities in following sedimentation and filtration process.

2. Aeration :- <sup>→ undesirable</sup> oxid<sup>n</sup> of <sup>Gases like CO<sub>2</sub> and H<sub>2</sub>S</sup> <sup>organic matter</sup>

- i) In aeration water is brought in intimate contact with air so as to remove undesirable gases like CO<sub>2</sub> and H<sub>2</sub>S from water. It is also done to add oxygen in the water to carry out the oxidation of undesirable substances like organic matter and oils.
- ii) It is also done to remove volatile liquids like phenol and humic acid from water.
- iii) It also removes dissolved minerals like Iron and Manganese in water.

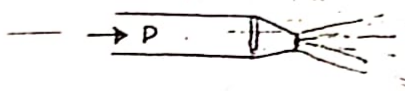


This process also increases acidity of water due to H<sup>+</sup> ions.

Methods of aeration

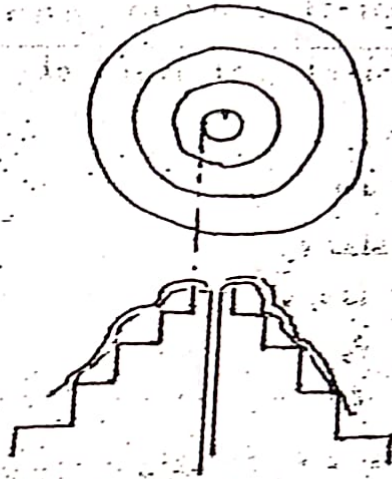
- i) Aeration is generally done for the water which is devoided of O<sub>2</sub> e.g. Groundwater.
- ii) There are several methods to do aeration as:
  - Spray nozzle
  - Cascade aeration method
  - Tray tower method
  - Diffused air method

1. Spray nozzle method :-



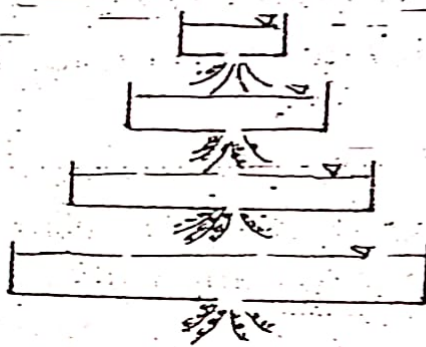
Efficiency of this method in removal of CO<sub>2</sub> is 90% and in removal of H<sub>2</sub>S is 99%. principle is pressurised water to increase surface area of water.

## 2. Coscode Aerator :-



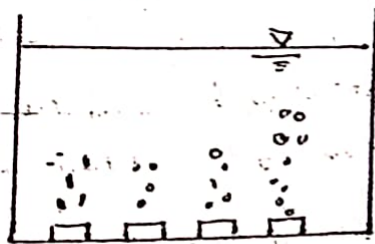
Efficiency of this method in removing  $\text{CO}_2$  is 20 to 45% and in removing  $\text{H}_2\text{S}$  is 35% :-

## 3. Spray tower method :-



This is the best method for removal of  $\text{CO}_2$ . It is also used for removal of Fe and Mn from water. The precipitate of Manganese oxide is generally formed at pH of 9, hence the external alkaline agent like Potassium permanganate  $\text{KMnO}_4$  is added to facilitate the formation of ppt. of  $\text{MnO}_2$ .

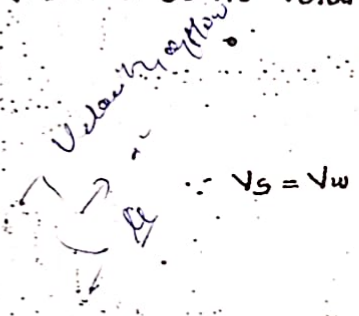
## 4. Diffused air :-



3. Sedimentation:

specific gravity  $G = \frac{\text{wt. of solid in given volume}}{\text{wt. of std fluid of same volume}}$

$$G = \frac{W_s}{W_w} = \frac{W_s/V_s}{W_w/V_w}$$



$$G = \frac{V_s}{V_w}$$

For inorganic solids,  $G = 2.6 \text{ to } 2.9$  (2.65) ✓

For organic solids,  $G = 1 \text{ to } 2$  (1.2) ✓

In the designing of sedimentation tank, factors which opposes setting of particles, are given due consideration:

(i) Velocity of flow ( ~~$V_f$~~  - turbulence)

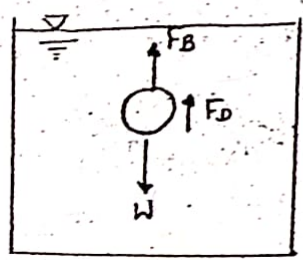
The velocity of flow is reduced to increase tendency of settlement of SS in water.

(ii) Viscosity of water:

In designing, viscosity of water can be reduced in order to increase tendency of settlement.

(iii) Size of particle:

$$V = \frac{W_w(G-1)}{18\mu}$$



$$F_D = \frac{1}{2} C_D \cdot \rho \cdot V_r^2 \cdot A = \frac{1}{2} C_D \cdot \rho \cdot A \cdot (V_p - V_w)^2$$

Force causing downward moment =  $W - F_B$   
 Force resisting =  $F_D$

As particle moves downwards it accelerates.

At certain point,  $F_D = W - F_B$ , Thus acceleration becomes zero

and particle moves with constant velocity called as settling velocity of particle.

$$\frac{W}{V_s} = S$$

$$W - F_B = F_D$$

$$V_s \gamma_s - \gamma_w \cdot V_w = \frac{1}{2} C_D \cdot S \cdot A V_s^2$$

$$V_s = V_w$$

$$V_s (\gamma_s - \gamma_w) = \frac{1}{2} C_D \cdot S \left( \frac{\pi d^2}{4} \right) V_s^2$$

$$\frac{4}{3} \pi \left( \frac{d}{2} \right)^3 (\gamma_s - \gamma_w) = \frac{1}{2} C_D \cdot S \left( \frac{\pi d^2}{4} \right) V_s^2$$

$$\frac{4}{3} \pi \left( \frac{d}{2} \right)^3 (\gamma_s - \gamma_w) g_s = \frac{1}{2} C_D \cdot S \left( \frac{\pi d^2}{4} \right) V_s^2$$

$$\frac{4(\gamma_s - \gamma_w) g_s}{3 C_D}$$

$$C_D \cdot \gamma_w = g_s$$

$$V_s = \sqrt{\frac{4/3 (\gamma_s - \gamma_w) g_s d}{C_D}}$$

Turbulent

For laminar flow ( $Re < 1$  and  $d < 0.1 \text{ mm}$ )

$$V_s = \frac{(\gamma_s - \gamma_w) d^2}{18 \mu}$$

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

$$Re = \frac{S V d}{\mu}$$

$$V_s = \sqrt{\frac{4/3 (\gamma_s - \gamma_w) g_s d}{24 \cdot S V d}}$$

$$= \sqrt{\frac{(\gamma_s - \gamma_w) \gamma_w d^2 V_s}{18 \mu}}$$

$$V_s = \frac{(\gamma_s - \gamma_w) \gamma_w d^2}{18 \mu}$$

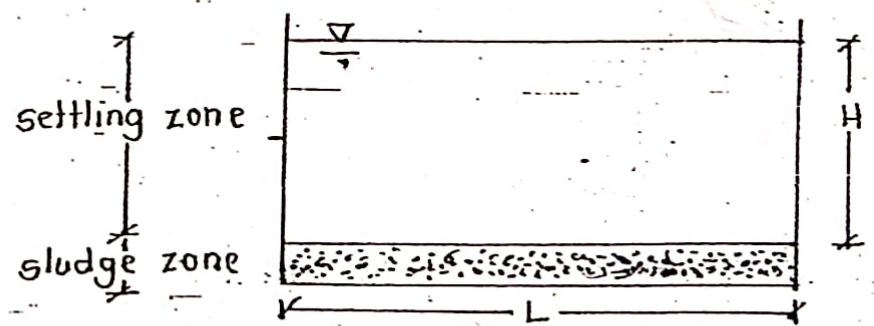
$$\frac{(\gamma_s - \gamma_w) \gamma_w d^2}{18 \mu}$$

$$\begin{aligned} \gamma_w &= 9.8 \text{ kN/m}^3 \\ \mu &= \text{N}\cdot\text{s/m}^2 \\ \nu &= \text{m}^2/\text{sec} \end{aligned}$$

Types of sedimentation tanks:

1. Quiescent type / Fixed type (Fill and draw type tank)

$$V_s = \frac{(\gamma_s - \gamma_w) d^2}{18 \mu}$$



longitudinal section



(i) In this type of tank flow of water is completely stopped. The detention time of this tank is 24 hours.

(ii) This tank is also known as fill and draw type tank as water is filled in tank, kept for 24 hours, and drawn out from it after it is sufficiently clarified.

(iii) Cleaning time of the tank is 6 to 12 hours.

(iv) At a time minimum of 3 tanks are required. 2 operational and 1 standby.

(v) This tank is designed for Maximum daily demand which is equal to 1.8 times average demand.

$$Q_d = Q_{\text{max-daily}}$$

$$= 1.8 Q_{\text{avg-daily}}$$

$$= 1.8 (\text{Population} \times \text{Avg-per capita demand})$$

$$\text{volume of tank, } V = Q_d \cdot t_d$$

Max. daily demand  $\Rightarrow$  1.8 times avg. daily demand

2. Continuous flow type tank:

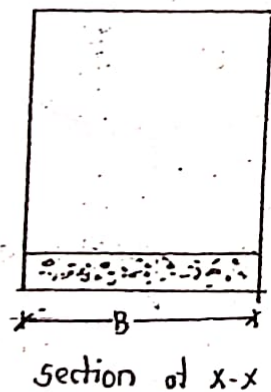
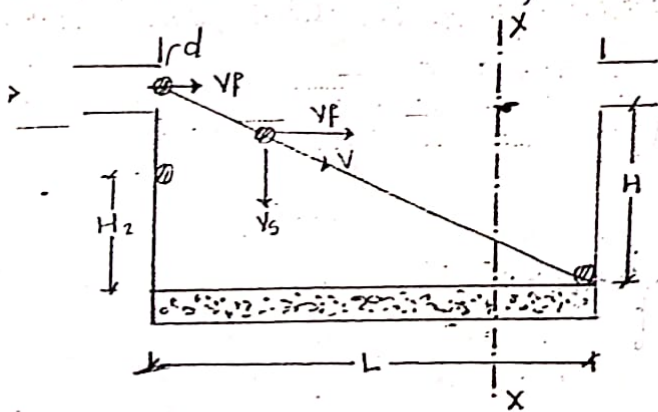
This tank is of two types.

a) Horizontal flow type (square or rectangular)

b) Vertical flow type (circular)

a) Horizontal flow-type tank:

(i) In this type of tank water continuously flows in the horizontal direction.



Assumptions made in designing of continuous flow type tank

- (i) The conc. of particles of all sizes at all points of vertical c/s at inlet is same.
- (ii) A particle is assumed to be removed from the water in the tank, if it reaches the bottom of the setting zone or top of the sludge zone.

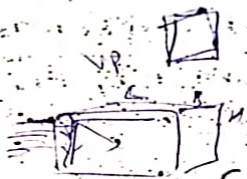
Design of tank:

$$t_d = \frac{L}{v_f} \quad \text{and} \quad t_d = \frac{H}{v_s} \quad (\text{for design particle } d)$$

$$A_c v_f = Q_D$$

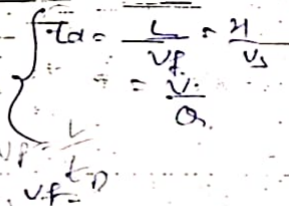
$A_c$  - c/s area

$$v_f = \frac{Q_D}{\text{cross sectional area}}$$



$$\text{c/s area of tank } A_c = \frac{Q_D}{v_f}$$

$$t_d = \frac{L}{v_f}$$



$$A_s v_s = Q_D$$

$A_s$  - plan area

$$v_s = \frac{Q_D}{\text{plan area}}$$

$$\therefore \text{plan area of tank } A_s = \frac{Q_D}{v_s}$$

$$\text{volume} = Q_D \cdot t_d$$

The settling velocity is also known as surface overflow rate (SOR) and it determines efficiency of tank.

- The particle for which sediment tank is designed ( $d$ ) is 100% removed in tank.

$$t_{d_1} = \frac{H}{v_{s_1}} \quad \text{and} \quad d_1 > d \quad (\text{size more than design particle})$$

$$t_{d_1} < t_d$$

$$\therefore v_{s_1} > v_s$$

The particles of size greater than concerned size of particle are also 100% removed in sedimentation tank.

For particles of size less than concerned size of particle: ( $d_2$ )

$$d_2 < d$$

$$t_{d_2} = \frac{H}{v_{s_2}}$$

$$v_{s_2} < v_s$$

$$t_{d_2} > t_d$$

$$L_2 = v_f \cdot t_{d_2}$$

$$L_2 > L$$

The particles of size smaller than designed particle are partially removed in the tank. All the particles of this size present in the tank from 0 to  $H_2$  will be removed in the tank and all remaining particles from  $H_2$  to  $H$  will be carried by the treated water out of the tank (not removed)

$$\% \text{ removal of particle smaller in size than size of concerned size of particle} = \frac{\left(\frac{W}{H}\right) A_2}{W} \times 100 = \frac{H_2}{H} \times 100$$

$$L_D = \frac{H_2}{v_{s_2}} = \frac{H}{v_s}$$

$$\frac{H_2}{H} = \frac{v_{s_2}}{v_s}$$

Saturday  
20<sup>th</sup> July 2015

size (mm)	weight (gm)	
1	100	} 100%
0.8	150	
0.7	50	
0.5	200	70%
0.4	500	50%

$$\eta = \frac{\left\{ (300) \times \frac{100}{100} + 200 \times \frac{70}{100} + 500 \times \frac{50}{100} \right\} \times 100}{1000}$$

$$= 69\%$$

For the given particle size distribution calculate efficiency of tank  
 the total solid in suspended form is 1000 gm and overflow  
 rate in tank is 0.35 mm/sec.

Size (mm)	0.1	0.2	0.3	0.4	0.5	0.6
quantity (%)	10	20	15	5	30	20
settling velocity ( $V_s$ - mm/sec)	0.2	0.25	0.3	0.35	0.4	0.5

overflow rate = settling velocity of design particle.

$$V_s = 0.35 \text{ mm/sec} \quad (\text{for design particle})$$

Particles of size 0.4, 0.5 & 0.6 mm will be completely removed. (100%)

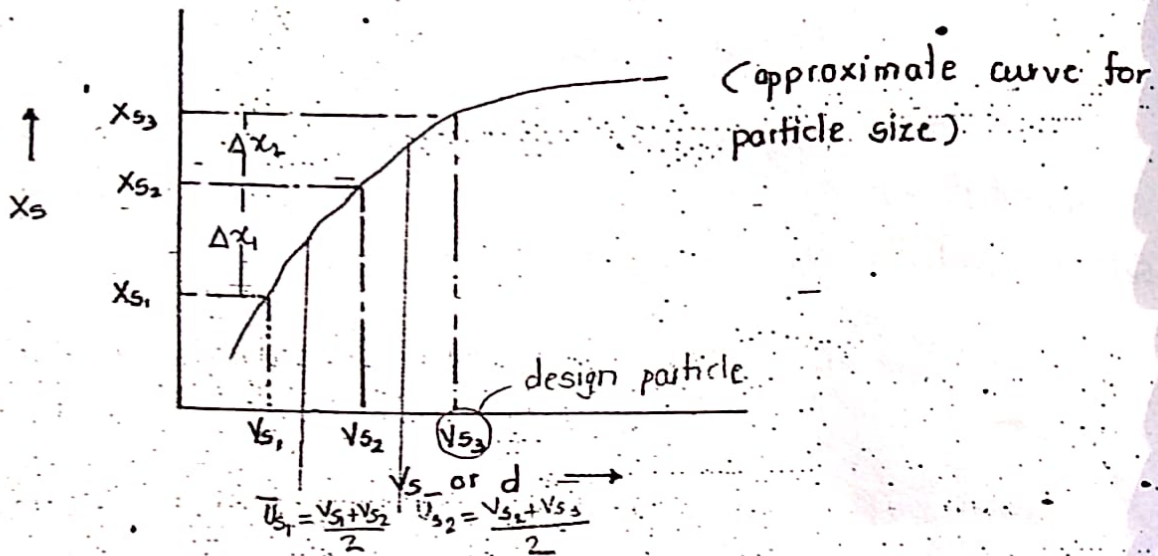
$$\% \text{ removal of } 0.3 \text{ mm size particle} = \frac{0.3}{0.35} \times 100 = 85.71\%$$

$$\frac{0.2}{0.35} \times 100 = 71.4\%$$

$$\frac{0.1}{0.35} \times 100 = 57.14\%$$

$$\begin{aligned} \text{Efficiency of tank } (\eta) &= \left\{ \left( \frac{1000 \times 55}{1000} \right) \times \frac{100}{100} + \left( \frac{1000 \times 15}{100} \right) \times \frac{85.71}{100} \right. \\ &\quad \left. + \left( \frac{1000 \times 20}{100} \right) \times \frac{71.4}{100} + \left( \frac{1000 \times 10}{100} \right) \times \frac{57.14}{100} \right\} \frac{100}{1000} \\ &= 88.14\% \end{aligned}$$

If the particles of all the sizes are present in water then efficiency of water tank is computed with the help of parameter  $x_s$ , which represents % of particles having settling velocity less than stated velocity (settling velocity of design particles) or % of particles size less than design particles.



→ Efficiency ( $\eta$ ) =  $\frac{(100 - X_s) \times 100}{100} + \sum_{i=1}^n \left( \frac{\bar{U}_{s_i}}{V_s} \right) \Delta x_i$

(% of particles more than  $d$ )   ( % of particles less than  $d$  )

$\bar{U}_{s_i} = f(x)$

$\eta = (100 - X_s) + \int_{x=0}^{X_s} \frac{U_s}{V_s} \cdot dx$

2. Determine the overall removal obtained from sedimentation tank given from data:

- overflow rate = 32.6 m/day
- sp. gravity = 1.2
- dynamic viscosity = 1.027 centi poise.
- density of water = 0.997 gm/cm<sup>3</sup>

Particle size (mm)	0.1	0.08	0.07	0.06	0.04	0.02	0.01
wt. fraction greater in size	10%	15%	40%	70%	93%	99%	100%
wt. fraction less in size	90%	85%	60%	30%	7%	1%	0%

overflow rate = 32.6 m/day

=  $\frac{32.6 \times 10^6 \text{ mm}}{1 \times 24 \times 3600}$

= 0.38 mm/sec

Settling velocities of particles in  $N/m^3$ .

$$U_s = \frac{(G-1)\gamma_w \cdot d^2}{18\mu} \quad \text{Ns/m}^2 \quad \left( \begin{array}{l} \gamma_w \text{ in } N/m^3 \\ \mu \text{ poise to SI} \\ \text{centi} \end{array} \right)$$

$$= \frac{(1.2-1)(0.997 \times 10^{-3} \times 10^5) \times d^2 \times 9.81 \times 10^{-6}}{18 \times 1.027 \times 10^{-2} \times 10^{-1}} \times 10^3$$

$$U_s = 105.8 \cdot d^2 \text{ mm/sec}$$

$$U_{s1} = 105.8 \times (0.1)^2$$

$$= 1.058 \text{ mm/sec}$$

particle size d (mm)	0.1	0.08	0.07	0.06	0.04	0.02	0.01
settling velocity (mm/sec)	1.058	0.67	0.51	0.38	0.16	0.04	0.01
				$U_{s3}$	$U_{s2}$	$U_{s1}$	

Designing of Tank is done for analysis of 0.06 mm tank particle

$$X_s = 30\%$$

$$\eta = (100 - X_s) + \sum_{i=1}^n \frac{U_{s1}}{U_s} \Delta X_i$$

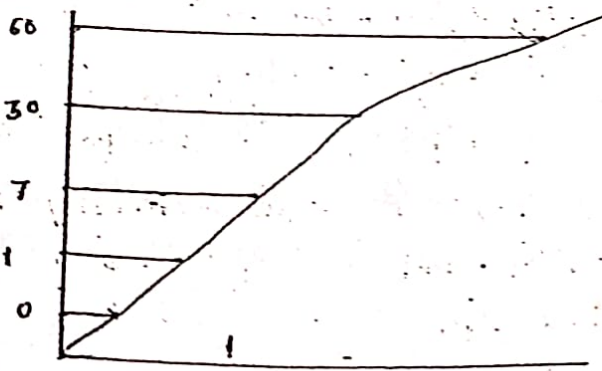
$$\Delta X_1 = (1-0) = 1\% \quad \bar{U}_{s1} = \frac{0.04 + 0.01}{2} = 0.025 \text{ mm/sec}$$

$$\Delta X_2 = (7-1) = 6\% \quad \bar{U}_{s2} = \frac{0.04 + 0.16}{2} = 0.1 \text{ mm/sec}$$

$$\Delta X_3 = (30-7) = 23\% \quad \bar{U}_{s3} = \frac{0.38 + 0.16}{2} = 0.27 \text{ mm/sec}$$

$$\eta = (100 - 30) + \left(\frac{0.025}{0.38}\right) \times 1 + \left(\frac{0.1}{0.38}\right) \times 6 + \left(\frac{0.27}{0.38}\right) \times 23$$

$$= 87.9\%$$



Design data for sedimentation tank:  $12-18 \text{ m}^3/\text{day}$

(i) overflow rate (SOR)

overflow rate is in the range of  $12,000-18,000 \text{ lit}/\text{m}^2/\text{day}$  for plain sedimentation. ( $12-18 \text{ m}^3/\text{m}^2/\text{day}$ )

It is in the range of  $24,000-30,000 \text{ lit}/\text{m}^2/\text{day}$  for the coagulation aided sedimentation ( $24-30 \text{ m}^3/\text{m}^2/\text{day}$ )

$$\text{Plan area of tank} = \frac{\text{design discharge}}{\text{overflow rate}}$$

(ii) Detention time ( $t_D$ )

It is in the range of 4-8 hours for plain sedimentation and 2-4 hours for coagulation aided sedimentation. (C+F+S)

$$\text{volume of tank} = \text{design discharge} \times \text{detention time}$$

$$\text{Depth of the tank} = \frac{\text{volume of tank}}{\text{plan area}}$$

(iii) Velocity of flow ( $V_f$ ):-

It is in the range of  $0.15$  to  $0.9 \text{ m}/\text{min}$ . Normally it is taken to be  $0.3 \text{ m}/\text{min}$ .

$$\text{Length of tank} = \text{velocity of flow} \times \text{detention time}$$

(iv) Width of tank is generally in range of  $10-12 \text{ m}$  and length to width ratio is kept to be 4:1 i.e.

$$L/B = 4:1$$

(v) Horizontal flow sedimentation tanks are designed for max. daily demand equal to 1.8 times of avg. daily demand.

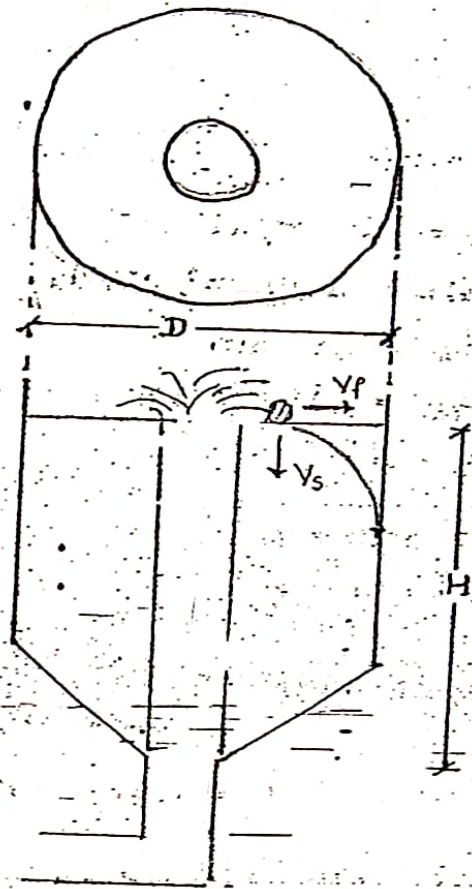
$$Q_D = Q_{\text{max daily}}$$

$$= 1.8 Q_{\text{avg daily}}$$

$$= 1.8 (\text{pop}^n \times \text{Avg. per capita})$$

(vi) If cleaning of sludge is done manually then additional volume for accumulation of sludge is provided by adding  $0.8-1.2 \text{ m}$  of depth over design depth.

b) Vertical flow type tank :-



In vertical flow type tank velocity of flow decreases as the particles moves from centre towards circumference of tank in horizontal direction (but  $v_s$  remains constant during its settlement) Hence the path followed by particle during settling is parabolic. volume of the tank is given by

$$V = D^2 (0.011 D + 0.785 H)$$

$$V = Q_D \cdot t_D$$

∴ plan area of tank  $\left(\frac{\pi D^2}{4}\right) = \frac{Q_D}{\text{overflow rate}}$

$$V = D^2 (0.011 D + 0.785 H)$$

$$V = D^2 (0.011 D + 0.785 H)$$



2. Determine the surface area for settling tank for  $0.5 \text{ m}^3/\text{s}$  flow using design overflow rate as  $32.5 \text{ m}^3/\text{m}^2/\text{day}$ . Also find the depth of tank if detention time is 95 min. Assume  $L/B = 2:1$  to  $L/B = 5:1$  and length of tank should not exceed 100 m.

Data:-

$$Q_D = 0.5 \text{ m}^3/\text{sec}$$

$$\text{overflow rate} = 32.5 \text{ m}^3/\text{m}^2/\text{day}$$

$$t_D = 95 \text{ min}$$

$$L/B = 2:1 \text{ to } 5:1$$

$$L < 100 \text{ m}$$

To find:

$$\text{plan area} = ?$$

$$\text{tank depth } d = ?$$

$$\begin{aligned} \text{plan area of tank} &= \frac{0.5 \text{ m}^3/\text{sec}}{32.5 \text{ m}^3/\text{m}^2/\text{day} \times 86400} \\ &= 1329.2 \text{ m}^2 \end{aligned}$$

$$\begin{aligned} \text{volume of tank} &= Q_D \times t_D \\ &= 0.5 \text{ m}^3/\text{sec} \times 95 \times 60 \text{ sec} \\ &= 2850 \text{ m}^3 \end{aligned}$$

$$\begin{aligned} \text{Depth of tank} &= \frac{\text{volume of tank}}{\text{plan area of tank}} \\ &= \frac{2850}{1329.2} \\ &= 2.14 \text{ m} \end{aligned}$$

$$(L/B) \times (B) = 1329.2$$

$$B = 18.2 \text{ m}$$

$$L = 72.9 \text{ m}$$

$$\text{Assume } L/B = 4$$

$$< 100 \text{ m OK}$$

Q. In continuous flow settling tank 3.5 deep and 65 m long. Flow velocity of water is observed as 1.22 cm/sec. What size of particle of sp. gravity 2.65 may be effectively removed in this tank, if the kinematic viscosity of water is 0.01 cm<sup>2</sup>/sec.

Data:

$$H = 3.5 \text{ m}$$

$$L = 65 \text{ m}$$

$$V_f = 1.22 \text{ cm/sec}$$

$$G = 2.65$$

$$\nu = 0.01 \text{ cm}^2/\text{sec}$$

$$\frac{65}{1.22} = \frac{3.5}{d}$$

To find: dia. of particle.  $d = ?$

3.c

$$\text{Detention time } t_d = \frac{H}{V_s} = \frac{L}{V_f}$$

$$V_s = \frac{H \times V_f}{L}$$

$$= \frac{3.5 \times 1.22 \times 10^3 \text{ mm/sec}}{65}$$

$$= 0.65 \text{ mm/sec}$$

$$t_d = \frac{2.5}{1.22} = \frac{65}{1.22}$$

$$V_s = \frac{(G-1) \gamma_w d^2}{18 \mu}$$

$$= \frac{(2.65-1) \times 9810 \text{ N/m}^3 \times d^2}{18 \times 0.01 \times 10^{-4} \times 10^3}$$

$$0.65 \times 10^{-3} = 0.89 d^2 \times 10^6$$

$$d = 0.027 \text{ mm} < 0.1 \text{ mm}$$

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

$$\mu = \gamma \times g$$

6.718

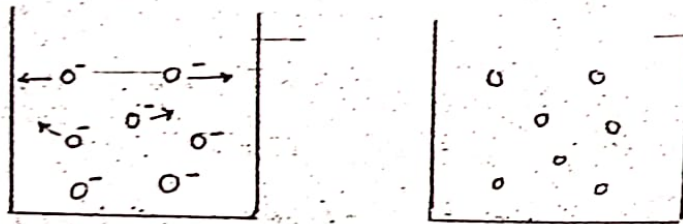
$\frac{V_s}{H}$

#### 4. Coagulation aided sedimentation :-

- (i) The efficiency of sedimentation is very less when water contains very fine SS, hence to improve its efficiency coagulation aided sedimentation is adopted.
- (ii) The entire process of this coagulation is completed in three stages.
  - Coagulation (fast mixing)
  - Flocculation (slow mixing)
  - Sedimentation.

#### Coagulation :-

- (i) It is the process in which certain chemicals known as coagulants are added in the water, so as to neutralise negative protective charge over the particles, in order to increase their contact opportunity in subsequent stages.
- (ii) A certain minimum amount of energy is required for these neutralisation of charge (known as Threshold energy) which is provided in the coagulation process by inducing rapid mixing in the water.

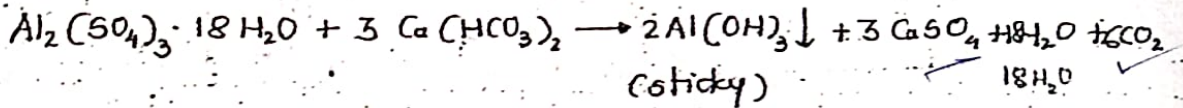


(iii) Different types of coagulants added in treatment of water are

- Alum ( $Al_2(SO_4)_3 \cdot 18H_2O$ ) - Hydrated aluminium sulphate)
- Copperos ( $FeSO_4 \cdot 7H_2O$  - Hydrated ferrous sulphate)
- Chlorinated Copperos ( $Fe_2(SO_4)_3 \cdot FeCl_3$ )
- Sodium Aluminate ( $Na_2Al_2O_4$ )

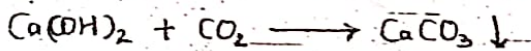
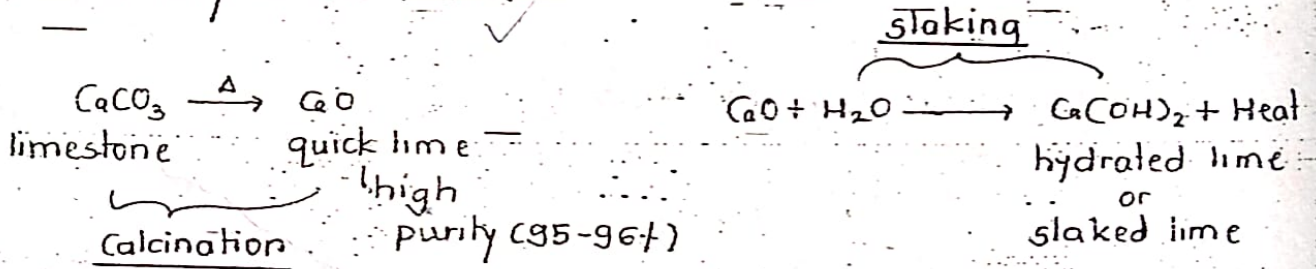
## Alum (Hydrated aluminium sulphate)

(i) Alum reacts with bicarbonate alkalinity present in water to form the sticky gelatinous precipitate of Aluminium hydroxide which attracts the finer suspended particles present in water thus grow in size and is finally removed in the following sedimentation tank.

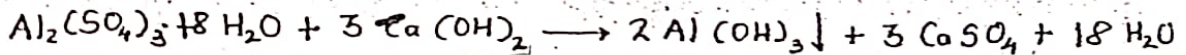


(ii) This process also adds permanent hardness ( $\text{CaSO}_4$ ) in the water and also increasing the acidity of water (due to  $\text{CO}_2$ ) thus making it corrosive.

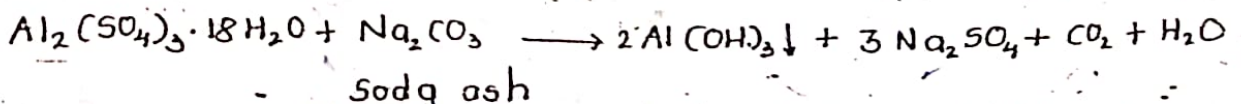
(iii) If alkalinity is not present in water, external alkaline agent like lime and soda ash is added, to induce the alkalinity artificially in the water.



Thus we added solid form of lime i.e. quick lime in the water.

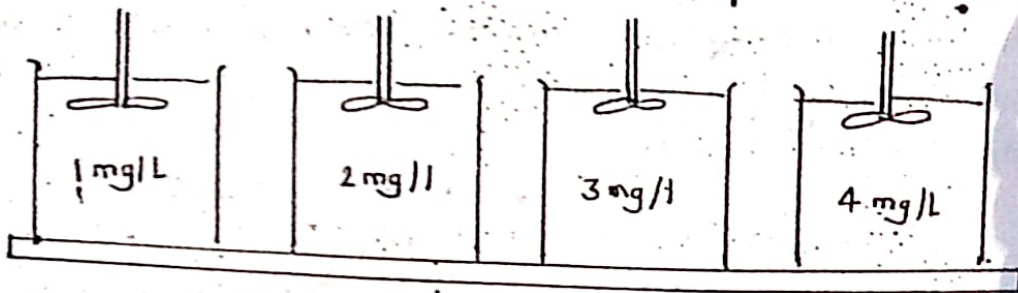


When lime is added acidity is not induced ( $\text{CO}_2$ ) but hardness is induced in water.



When soda ash is added acidity is induced but not hardness.

The dose of alum depends upon turbidity and colour of raw water which is determined with the help of Jar test.

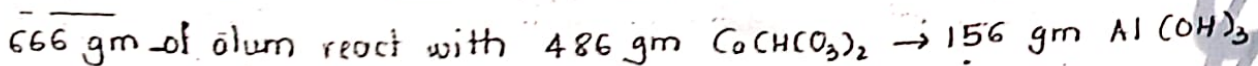
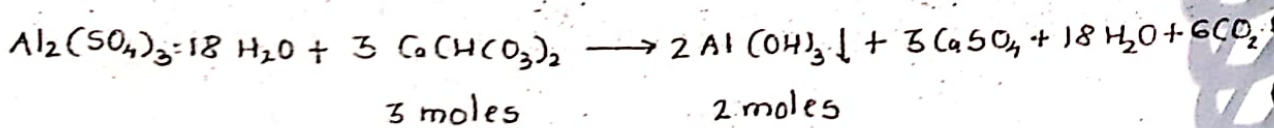


- (i) The normal dose of alum varies between 10-30 mg/l.
- (ii) Alum forms stable flocs and is capable of removing colour and odour from the water.
- (iii) It works in the pH range of 6.5 - 8.5.
- (iv) one mole of alum added in water gives two moles of ppt. of Al(OH)<sub>3</sub>

$$\begin{aligned} \text{Mol. wt. of alum} &= \text{Al}_2(\text{SO}_4)_3 \cdot 18 \text{H}_2\text{O} \\ &= (27 \times 2) + 3(32 + 16 \times 4) + 18(2 \times 1 + 16) \\ &= 666 \text{ gm} \end{aligned}$$

$$\begin{aligned} \text{Mol. wt. of Ca(HCO}_3)_2 &= (40 + 2(1 + 12 + 16 \times 3)) \times 2 \\ &= 486 \text{ gm} \end{aligned}$$

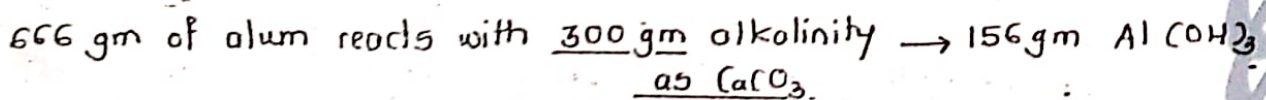
$$\begin{aligned} \text{Mol. wt. of Al(OH)}_3 &= (27 + 3(16 + 1)) \times 2 \\ &= 156 \text{ gm} \end{aligned}$$



1 gm of Alum

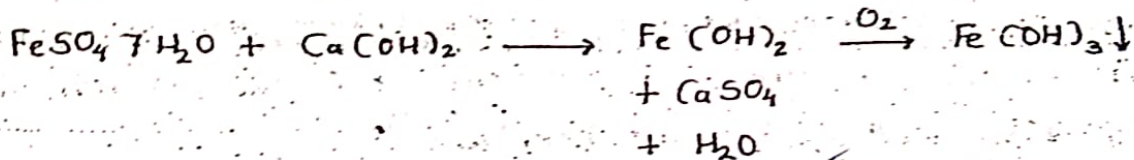
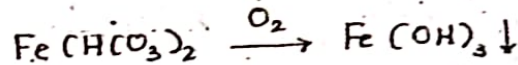
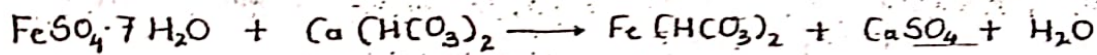
$$\frac{486}{666} = 0.73 \text{ gm} \quad \dots \quad \rightarrow \quad \frac{156}{666} = 0.234 \text{ gm}$$

OR



Copperas ( $\text{FeSO}_4 \cdot 7\text{H}_2\text{O}$ ) - Hydrated ferrous sulphate.

(i) Copperas is effective coagulant when used in conjugation with lime. It also reacts with bi-carbonate alkalinity present in water to form the sticky ppt. of Ferric hydroxide



(ii) This process also adds hardness in the water.

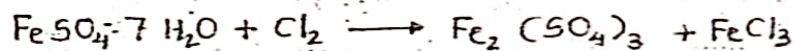
(iii) Copperas is never used for the treatment of coloured water.

(iv) The dose of copperas is same as that of alum, 10-30 mg/l but it is cheaper than alum.

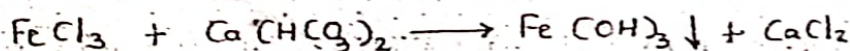
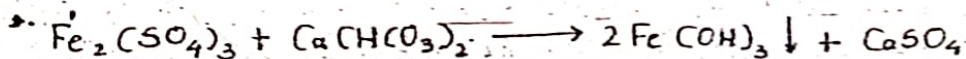
(v) It works in the pH range of 8.5 and above.

Chlorinated copperas ( $\text{Fe}_2(\text{SO}_4)_3 + \text{FeCl}_3$ )

(i) It is formed by the addition of chlorine in copperas.



— (Redox-reaction)



(ii) Chlorinated copperas is an effective coagulant as it can work in wide pH range.

(iii) When Ferric chloride ( $\text{FeCl}_3$ ) is used independently, it works in the pH range of 3.5 to 8.5 and above 8.5.

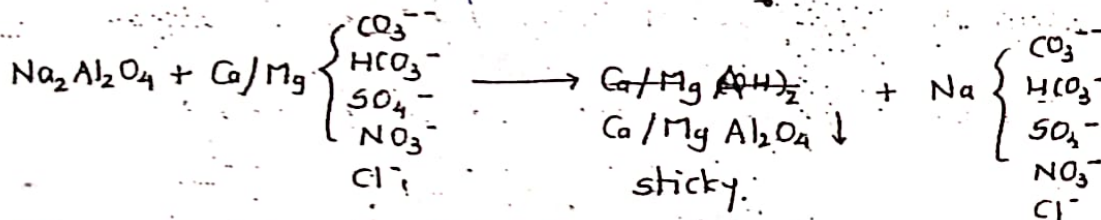
(iv) When Ferric sulphate  $\text{Fe}_2(\text{SO}_4)_3$  is used independently it works in pH range of 4 to 7 and above 9.

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## Sodium aluminate ( $\text{Na}_2\text{Al}_2\text{O}_4$ )

n) Sodium aluminate reacts with Ca & Mg present in water to form sticky ppt of Calcium-Magnesium aluminate.



- (i) This process removes both temporary and permanent hardness in water hence generally used for boiler feed water treatment.
- (ii) Sodium aluminate is costlier than alum.
- (iv) This process does not require the presence of alkalinity in water.
- (v) It works in pH range of 6 to 8.5.

### Note:

(i) Iron flocs are much heavier than aluminium floc. Thus time required for formation of Iron flocs is less in comparison to Aluminium flocs.

(ii) Iron salt works in wide pH range.

(iii) Iron salts are cheaper than Aluminium salts.

(iv) Alum is generally used for the treatment of raw water whereas Iron salts are used for treating sewage.

(v) Iron salts leads to the growth of Iron bacteria in water which causes pitting and tuberculation.

(vi) Strict supervision is required in the handling of Iron salts and its quality gets degraded with increase in storage time.

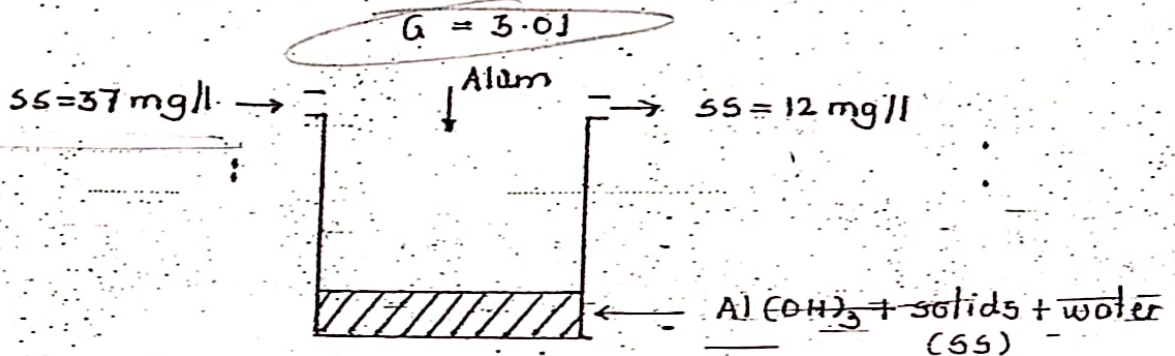
(vii) The disadvantage of Iron coagulant due to iron bacteria is overcome in sewage treatment as concrete pipes are used in sewer constructions.

Q. A coagulation treatment plant with flow of  $0.5 \text{ m}^3/\text{sec}$  is dosing alum at  $23 \text{ mg/l}$ . No other chemical is being added. The raw water SS concentration is  $37 \text{ mg/l}$  and effluent solid concentration is  $12 \text{ mg/l}$ . The solid content in sludge is  $1\%$  and specific gravity of the sludge solids is  $3.01$ . What volume of sludge must be disposed per day from this plant.

Data:  $Q = 0.5 \text{ m}^3/\text{sec}$

Alum dose =  $23 \text{ mg/l}$

$G = 3.01$



$$\text{settled SS in sludge} = 37 - 12 = 25 \text{ mg/l}$$

$$\begin{aligned} \text{wt. of SS settled in tank in one day} &= (25 \times 0.5 \text{ m}^3/\text{sec}) \times 10^3 \times 86400 \times 10^{-6} \\ &= 10800 \text{ kg/day} \end{aligned}$$

1 gm of alum gives 0.23 gm ppt of  $\text{Al}(\text{OH})_3$   
 $23 \text{ mg}$  of alum gives  $0.234 \times 0.23 = 0.538 \text{ mg/l}$  of  $\text{Al}(\text{OH})_3$

$$\begin{aligned} \text{wt. of ppt. formed in tank} &= 0.538 \text{ mg/l} \times (0.5 \times 10^3 \times 86400 \times 10^{-6}) \\ &= 232.4 \text{ kg/day} \end{aligned}$$

\* Assuming 100% ppt. is settled in tank.

$$\begin{aligned} \text{wt. of solids in the sludge} &= 1080 + 232.4 \\ &= 1312.4 \text{ kg/day} \end{aligned}$$

$$\begin{aligned} 1 \text{ kg of solids} + 99 \text{ kg water} &= 100 \text{ kg of wet sludge} \\ 1312.4 \text{ kg solids} + 99 \times 1312.4 \text{ kg} &= 129927.6 \text{ kg wet sludge} \\ &= 129927.6 \quad 131240 \end{aligned}$$

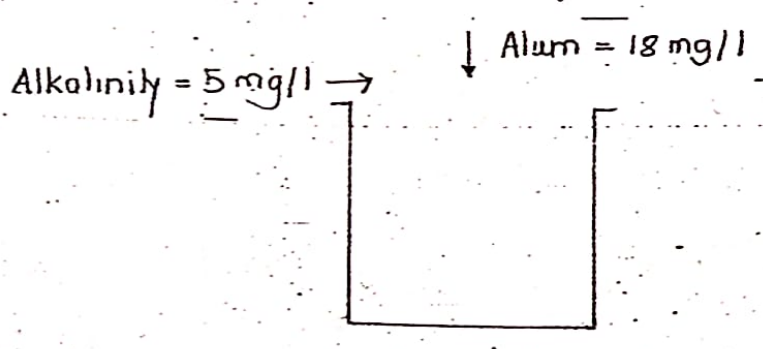


$$\begin{aligned}
 \text{volume of sludge} &= \text{volume of solids} + \text{volume of water} \\
 &= \frac{1312.4}{3.01 \times 1000} + \frac{129927.6}{1000} \\
 &= 0.436 + 129.9 \\
 &= 130.35 \text{ m}^3/\text{day}
 \end{aligned}$$

$$G = \frac{V_s}{V_w}$$

Q. A coagulation sedimentation plant clarifies 40 mld of water. The quantity of filter alum required at the plant is 18 mg/l. If the raw water is having an alkalinity equivalent to 5 mg/l as CaCO<sub>3</sub>, determine the quantity of filter alum and quick lime required which is 85% pure per year by the plant.

Data: Q = 40 mld  
Alkalinity = 5 mg/l



$$\begin{aligned}
 \text{Quantity of filter alum reqd. per year} &= 18 \text{ mg/l} \times 40 \times 10^6 \times 365 \times 10^{-6} \\
 &= 262.8 \text{ tonnes/year}
 \end{aligned}$$

1 mole of alum reacts with 3 moles alkalinity as CaCO<sub>3</sub>

666 gm	—	300 gm	—
1 gm	— = —	0.45 gm	—
18 mg/l		0.45 × 18 = 8.1 mg/l	

$$\begin{aligned}
 \text{Additional alkalinity to be added as CaCO}_3 &= 8.1 \text{ mg/l} - 5 \text{ mg/l} \\
 &= 3.1 \text{ mg/l}
 \end{aligned}$$

1 mole of CaCO<sub>3</sub> alkalinity is formed by 1 mole 100% pure lime

100 gm	—	56 gm	—
--------	---	-------	---

1 gm of  $\text{CaCO}_3$  alkalinity is formed by adding  $\frac{56}{100}$  mg 100% pure lime

$$3.1 \text{ mg} \longrightarrow 0.56 \times 3.1 = 1.736 \text{ mg}$$

3.1 mg of  $\text{CaCO}_3$  alkalinity with 85% pure lime is formed by adding

$$\frac{1.736}{0.85} = 2.04 \text{ mg/l}$$

$$\begin{aligned} \text{Quantity of 85\% pure lime in one year} &= 2.04 \times 40 \times 10^6 \times 365 \times 10^{-3} \times 10^{-3} \\ &= 29.7 \text{ tonne/year.} \end{aligned}$$

Sunday  
21<sup>st</sup> July 2013

### 5. Coagulation:

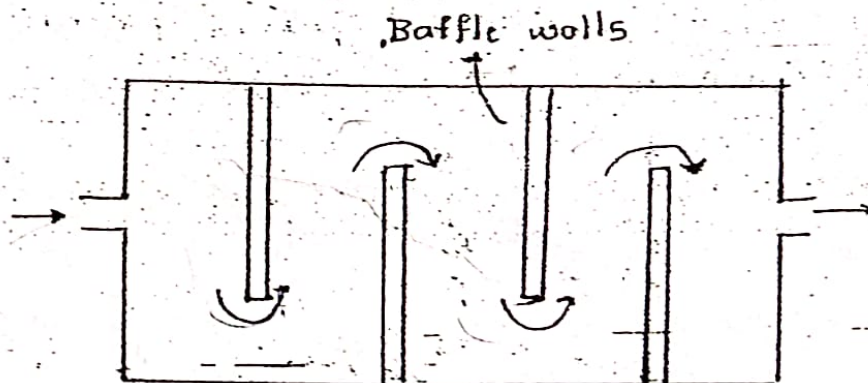
(i) In coagulation mixing of water can be induced either by the use of mixing basins or mechanical mixers.

Mixing basins:

(a) Round and end type

(b) Over and under type

(a) Round and end type mixing basin:



plan

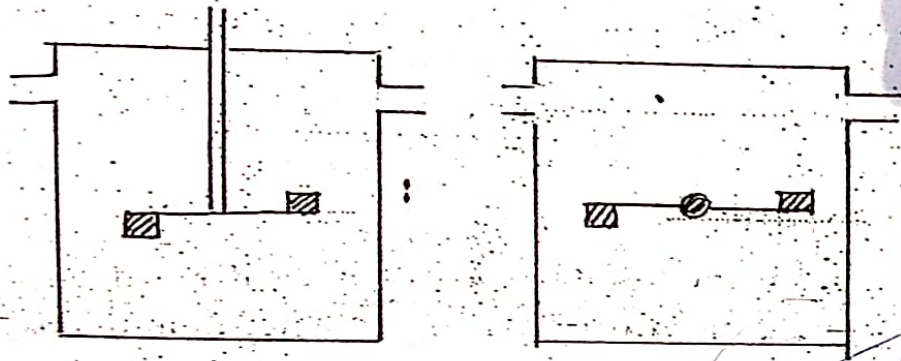
(c/s for over and under type)

In mixing basins, mixing is induced by creating the turbulence in water and this turbulence is created by rapid change in direction of flow of water.

In round end type movement of water is in horizontal plane while in over and under type it is in the vertical plane.

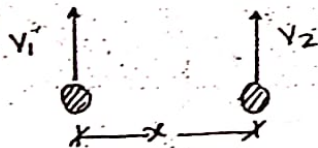
Mechanical mixers :

- (a) Horizontal shaft type
- (b) Vertical shaft type



(i) In mechanical mixers, mixing is induced in the water by vigorous agitation in water with the help of movements of blade attached over the shaft.

(ii) The intensity of mixing in these mixers depends upon the parameter  $G$  (Temporal means velocity gradient) which signifies the relative velocity between two particles situated at particular distance.



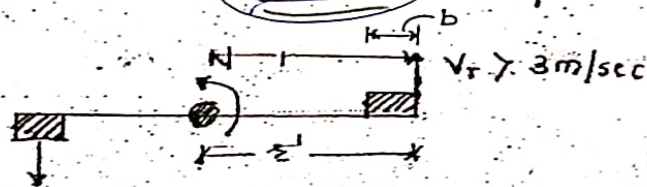
$$G = \frac{v_1 - v_2}{x} \quad / \text{sec}$$

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

Where,

- $P$  - power (Watts)
- $\mu$  - dynamic viscosity ( $\text{Ns}/\text{m}^2$ )
- $V$  - volume of water ( $\text{m}^3$ )

- (ii) The detention time in these mixers is normally kept to be 30-60 sec.
- (iv) Circular or square tanks are used in this case having depth to width ratio from 1:1 to 3:1. ( $H/B$  or  $H/D$ )
- (v) The power required for the mixing is 0.1 to 3 Watts per unit discharge in  $m^3/hr$ .
- (vi) The ratio of impeller diameter to tank diameter is in the range of 0.2 to 0.4.
- (vii) The speed of the shaft should be such that tangential velocity of greater than 3 m/sec is imparted at the tip of the blade.



$$V_t = \omega r$$

$$\omega = \frac{2\pi N}{60} \text{ rad/sec}$$

$$r = r + b/2$$

### Flocculation :

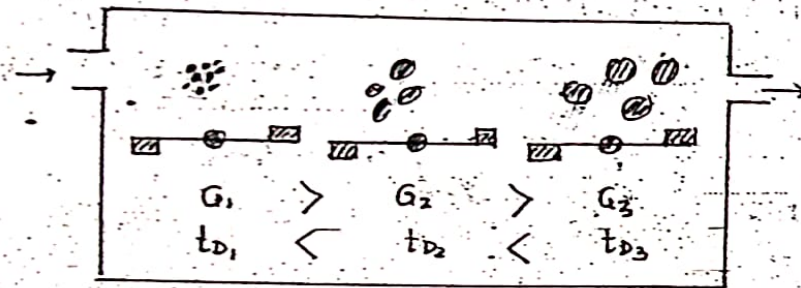
- (i) In flocculation neutralised suspended particles are provided with sufficient contact opportunity so as to agglomerate (to combine) and grow in size and get finally removed in the following sedimentation process.
- (ii) In order to promote the contact between the neutralised particle so as to form flocs greater in size than the original size of the particle, slow mixing is induced in water.
- (iii) The rate of flocculation depends upon the following factors.
  - a) Turbidity (more turbidity more is the flocculation)
  - b) Type and dose of coagulant (Iron coagulant will form heavy flocs in less time)
  - c)  $G$  (Temporal mean velocity gradient) - depends on power of shaft.
- (iv)  $G \cdot t_D$  is a parameter which determines the conjugation opportunity between the particles. or no of collisions i.e. no of times particles may come in contact with each other.

If  $G$  is more and  $t_D$  is less, small dense flocs will be formed.

$$G = \frac{v_1 - v_2}{x} \quad (\text{less } x)$$

If  $G$  is less and  $t_D$  is more, larger but light flocs will be formed.

Density is governed by  $G$  and size of floc is governed by detention time of particles.



(i) Since large dense flocs can be easily removed in the sedimentation tank, it is advantageous to vary the value of  $G$  over the length of flocculation tank.

(ii) Small dense flocs formed in the initial section of flocculation tank (more  $G$  and less  $t_D$ ) combines with the large light flocs formed in the subsequent sections of tank (less  $G$  and more  $t_D$ ) resulting in the formation of large dense flocs.

(iii)  $G$  at inlet is normally kept to be twice the  $G$  at outlet.

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

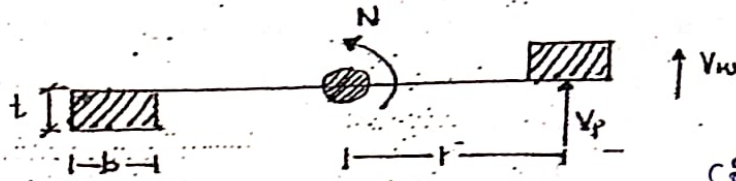
If At inlet  $G = G$ ,  $P = P$

& At outlet  $G = G/2$ ,  $P = P/4$

If At inlet  $G = G$ ,  $P = P$

& At outlet  $G = G$ ,  $P = P$

# Calculation of temporal mean velocity gradient (G)



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

Detention time  $\rightarrow$  30-60 sec

Flocculation -  
10-30 min

$$P = F_D \cdot V_r$$

velocity of flow

$\rightarrow$  2 m/s

2-8 m/s

where,

$F_D$  - drag force

$V_r$  - relative velocity of wall

$$V_r = V_p - V_w$$

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

projected area of plates on which drag force will act

$$A_T = n(b \cdot t)$$

for  $n$  blades

$$P = \left( \frac{1}{2} C_D \cdot \rho_w \cdot A_T \cdot V_r^2 \right) \cdot V_r$$

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

( $C_D = 1.8$  for this case)

Fin  
400

$$G = \sqrt{\frac{\frac{1}{2} \times 0.8 \times \rho_w \cdot A_T \cdot V_r^3}{\mu V}}$$

- (i) The height of the tank is normally taken to be 3-3.5 m
- (ii) Detention time for flocculation is 10-30 min.
- (iii) The area of the plates is approximately 10-25% of total area of tank. (plan area)
- (iv) The velocity of flow is in the range of 0.2-0.8 m/sec.
- (v) Temporal mean velocity gradient (G) is in the range of 10-75 1/sec.
- (vi)  $G \cdot t_D$  for alum is 20,000 - 60,000 and 1-1.5 lacs for Iron flocs.

## 5. Filtration:-

- (i) Filtration is most often used as limiting step to remove the flocs and finer undegraded particles.
- (ii) It also removes organic matter, micro organisms and the dissolved minerals from the water.
- (iii) Filtration is carried out with the help of filters which are generally of two types, - Gravity filters and pressure filters.

### Gravity filters:-

These are the type of filters in which head required by the water to pass through filter medium is provided by the height of the water itself over the filter medium.

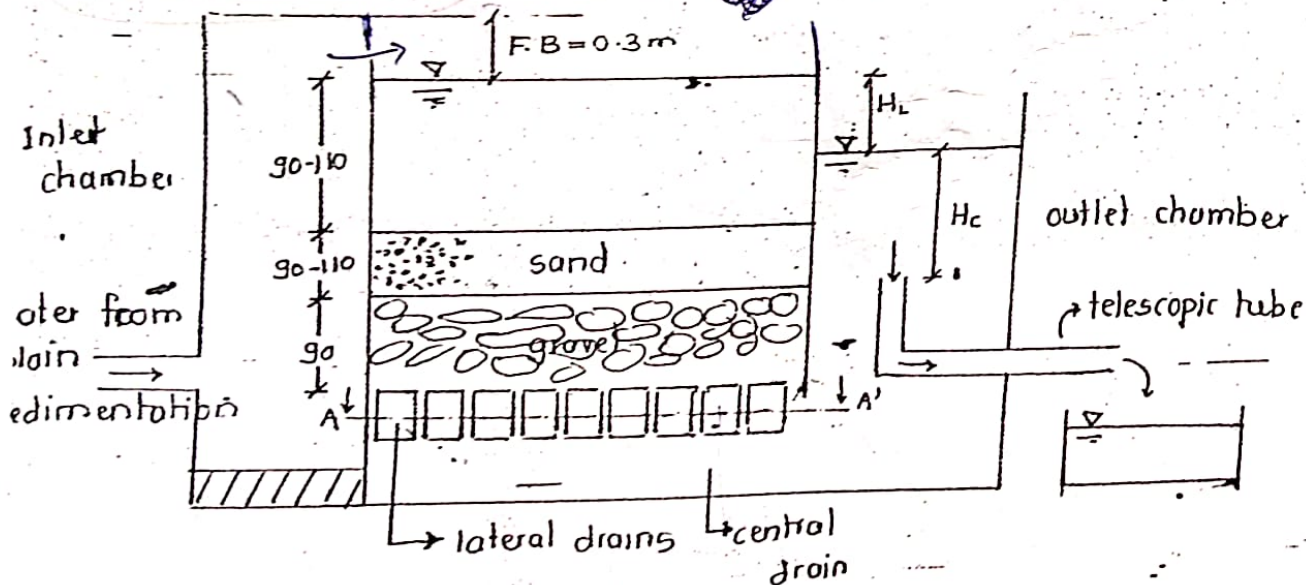
These filters are of two types

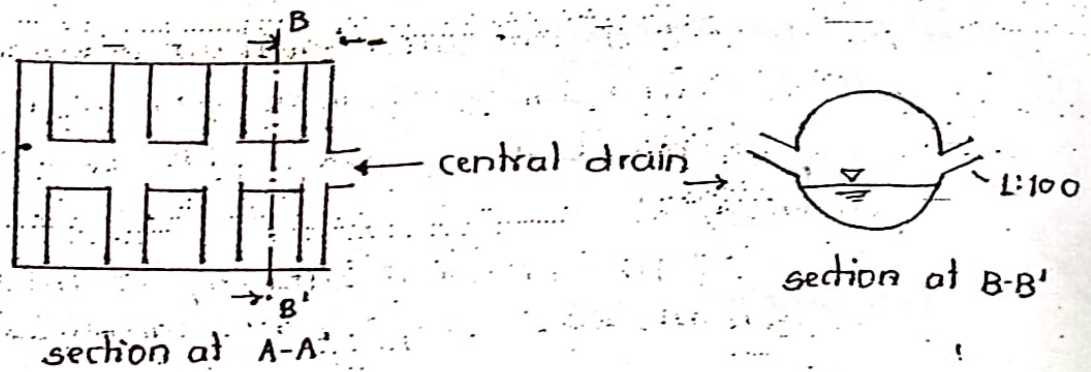
- Slow sand filters
- Rapid sand filters.

### Pressure filters:-

These are the type of filters in which head required by the water to pass through filter medium is provided artificially by external application of pressure over the water.

#### 1. Slow sand filters:-





### Theory of filtration :-

(i) During filtration of the water by the means of any type of filter following processes takes place.

#### (a) Mechanical straining:

During filtration, the particles greater in size than the size of filter pores are strained out from water and forms a layer of settled impurities over the filter medium. The size of the voids in these layer of settled impurities is less in comparison to the size of voids in filter medium which results in the removal of those particles also whose size is smaller than the size of voids of filter medium.

#### (b) Sedimentation:

Since turbulence is removed from water during filtration suspended particles present in water settles over filter medium.

#### (c) Biological changes:

For the first few days, upper layer of filter medium gets coated with the sticky deposits of the partially decomposed organic matter along with nutrients like Iron, manganese, Aluminium, Sulphur, Nitrogen, Phosphorous, etc. which promotes the growth of algae over the filter medium.

This algae carries out photosynthesis reaction in the presence of sunlight due to which  $O_2$  is released in water that is being utilised by the micro organisms to carry out the oxidation of organic matter retained over filter medium.

Over a period of time due to scarcity of organic matter, endogenous respiration starts leading to the reduced bacterial and organic load over filter.



The layer over the filter medium containing nutrients,  $O_2$ , organic matter, micro organisms and algae in which the above mentioned process takes place is known as the Schmutzdecke.

The biological efficiency of filter is dependent on the Schmutzdecke's layer. (more thickness more efficiency.)

(d) Electrolytic changes: -

The impurities present in the water and the particles present in filter medium carries opposite charge; hence due to their interaction, neutralisation of the impurities present in water takes place leading to the change in the chemical parameters of water.

(i) In slow sand filters, gravel layer is provided only to support the sand layer.

(ii) Slow sand filters utilises effluent from plain sedimentation only. (large flocs may clog the voids)

(iii) Depth of tank is in the range of 2.5-3.5 m (overall depth). plan area required is 100-2000 m<sup>2</sup> for each unit.

(iv) The medium of filter used can be sand, anthrasite or plastic medium (geotextiles).

(v) The depth of sand layer is 90-110 cm. The depth of water over the sand layer is same as the depth of sand layer.

(vi) The depth of gravel medium is 60-90 cm.

(vii) The effective size of sand particles in filter medium ( $D_{10}$ ) is in the range of 0.2 to 0.3 mm.

(viii) Coefficient of uniformity ( $C_u$ ) for the filter medium is 5.

$$C_u = \frac{D_{60}}{D_{10}} = 5$$

(ix) Top 15-30 cm of sand is of finer variety in comparison to the remaining sand. (so as to remove more finer impurities efficiently)

- (x) Design life of filters is approximately 10-15 years.
- (xi) In freshly cleaned filters, head loss of 10-15 cm is observed which goes on increasing with the operation of filter, hence, telescopic tube is adjusted in accordance with head loss so as to obtain constant discharge from the filter.
- (xii) The cleaning of the filter should be done when head loss becomes 0.7-0.8 times depth of filter medium.
- (xiii) During cleaning of filter top layer of the medium is scabbled and 1.5-3 cm of sand layer is removed from the filter medium.
- (xiv) The frequency of cleaning is 1-3 months.
- (xv) Rate of filtration is 2400-4800 lit/m<sup>2</sup>/day.  $\frac{100 \text{ m}^3}{\text{sec}}$

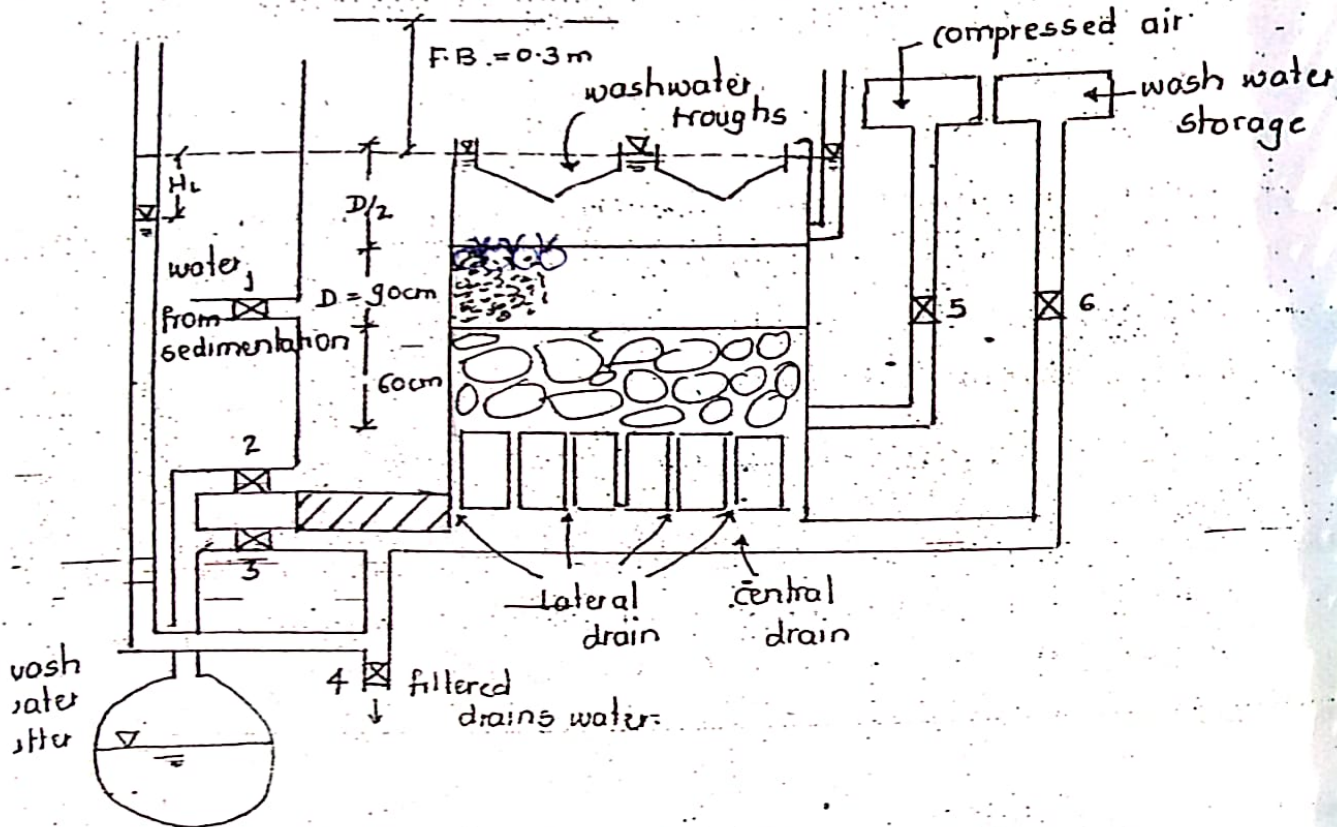
$$\text{plan area of filter} = \frac{\text{design discharge}}{\text{rate of filtration}}$$

- (xvi) After cleaning of filter it is washed with clarified water and again loaded with effluent from plain sedimentation but the filtered water is not used for next 24 to 36 hours.
- (xvii) Slow sand filters are generally adopted for villages and small towns.
- (xviii) The efficiency of this filter in removing micro organisms is 98-99%.
- (xix) The effluent of this filter are sufficiently clarified and free from nutrients like Carbon, Nitrogen and Phosphorous which have reduced possibilities of aftergrowth of organisms in distribution system.
- (xx) Slow sand filters are not used for water of turbidity greater than 50 NTU.
- (xxi) The number of filters required for treatment depends on the area of treat filter.

Area (m <sup>2</sup> ) —	No. of filters
< 20	2 (1+1)
20-249	3 (2+1)
250-649	4 (3+1)
649-1200	5 (4+1)
> 1200	6 (5+1)

Monday  
22<sup>nd</sup> July 2013

## 2. Rapid sand filters :



- (i) The operation of rapid sand filter is exactly same as that of slow sand filter. During operation of filter valve no. 1 and 4 are open.
- (ii) In case of RSF, the size of the particles of sand is more in comparison to that in SSF. Hence the size of the voids is comparatively more, which results in deeper penetration of the impurities in filter medium. Hence in this case surface washing alone is not sufficient, and is accompanied by backwashing.
- (iii) In backwashing valve no. 5, 6 and 2 are opened and valve 1 & 4 are closed.
- (iv) During this process, compressed air and pressurised water is passed through the medium and in backward direction (upward direction) resulting in the increase in porosity of filter medium. Thereby increasing opportunity of entrapped

impurities to get washed away with flowing pressurised water that is finally collected in wash water trough and disposed out of filter through valve no. 2 into wash water gutter.

(v) After backwashing is completed valve no. 2, 5 and 6 are closed and valve no. 1 and 3 are opened by which filter is again loaded with settled water (from sedimentation tank) but is not used for further treatment after filtration (sufficient time is provided for growth of Schmutzde's layer which is removed in cleaning process). After the growth of layer, valve no. 3 is closed and filtered water is started collecting from valve no. 4.

(vi) The entire process of backwashing is completed in 15-30 min.

(vii) Frequency of cleaning is 24-48 hours.

100-2000 m<sup>3</sup>

(viii) Quantity of water required for backwashing is 2-5% of the discharge required to be filtered by the filter in one day.

(ix) Area required by this filter is 10-80 m<sup>2</sup>.

(x) The rate of backwash is in the range of 15-90 cm/min. and normally it is taken to be 45 cm/min. (Backwash velocity).

The above limit is on the basis of the fact that the backwash velocity should be such that it does not carry the smallest particle of filter medium to be retained in it along with the washwater. But should be sufficient to remove the entrapped impurities.

(xi) The effective size of the sand particles is in the range of 0.35-0.55 mm. and coefficient of uniformity ( $C_u$ ) is 1.2-1.6.

(xii) The depth of the sand layer is approximately 90 cm and that of gravel medium is 60 cm.

(xiii) Minimum no. of units to be present at any time in treatment unit are 2 (one operational, one standby).

The no. of units can be calculated by following empirical relation.

$$N = 1.22 \sqrt{Q} \quad ; \quad Q \text{ is in MLD}$$

(cxv) In rapid sand filters better gradation of gravel medium is ensured as distribution of washwater takes place through the gravel medium, as compared to slow sand filter.

(cxvi) The rate of filtration is  $3000 - 6000 \text{ Lit/m}^2/\text{hour}$ .

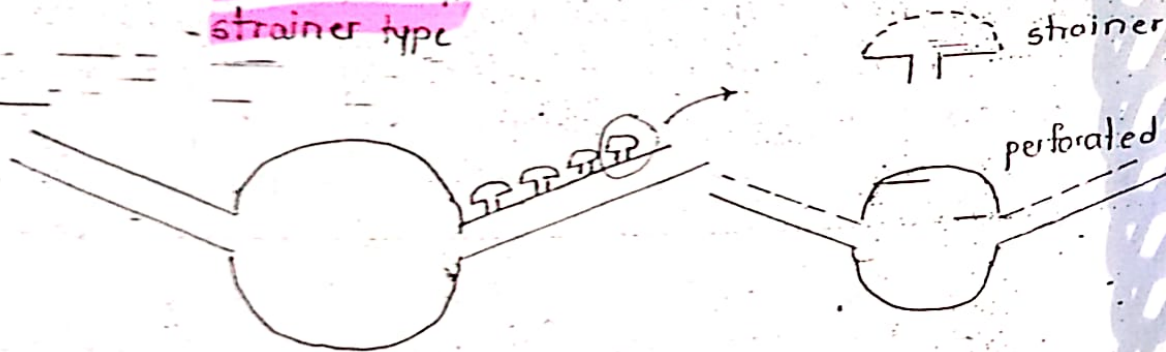
(cxvii) During the filtration flow is assumed to be laminar and during backwashing it is in transition. It can be converted into turbulent if proper gradation of gravel is not provided.

(cxviii) The UD system should be capable of carrying backwash discharge effectively. (Q)

(cxix) The underdrainage system designed is lateral and manifold system.

(cxix) Lateral drains provided in system is of two types

- perforation type
- strainer type



(cxix) If perforation type lateral drain is used, washing is termed as high velocity wash as in this case compressed air is not used. and if strainer type laterals are used, washing is termed slow velocity wash as compressed air is also used.

(cxix) The size of the perforation in the laterals is taken to be either 6 mm or 13 mm.

(cxix) If 6 mm perforations are adopted then spacing between perforations is taken to be 7.5 cm c/c and if 13 mm perforations are adopted, spacing is 20 cm c/c.

(cxix) The total c/s area of all the perforations is 0.2% of the filter area. (plan area)

(cxix) The spacing between laterals is 15-30 cm c/c and c/s area of each lateral is 2 or 4 times c/s area of perforation in it.

$$\text{c/s area of lateral } \left( \frac{\pi d^2}{4} \right) = 2-4 \cdot (\text{c/s area of perforations})$$

$$\text{No. of laterals} = 2 \left( \frac{L}{S} + 1 \right)$$

$$\text{Area of perforation} = \frac{\text{Filter area}}{\text{No. of laterals}}$$

(xxi) Factor 2 is used when size of perforation assumed is 15 mm and factor 4 is used when size of perforation is 6 mm.

(xxii) Cross section area of central drain is twice the cross section area of all laterals present in the drain.

$$\text{C/S area of central drain} = \left( \frac{\pi D_c^2}{4} \right)$$

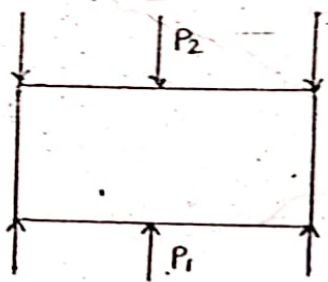
(xxiii) Length of each lateral to dia. of each lateral should not be more than 60.

$$\text{Length of lateral} = (B - D_c) / 2 \quad \text{Capprox.}$$

(xxiv) The velocity of flow through central drain is limited to 1.8-2.4 m/sec.

(xxv) If the thickness of filter medium is D, then the top of the washwater trough is kept at least at a distance of D/2 from the top of filter medium as during backwashing medium is likely to expand by 50%.

(xxvi) To hydraulically expand the filter bed, head loss through the filter must be equal to the buoyant weight of particles forming the filter medium.



$$P_1 = \gamma_w \cdot h_1$$

$$P_2 = \gamma_w \cdot h_2$$

$$P_1 - P_2 = \gamma_w (h_1 - h_2)$$

$$= \gamma_w H_L$$



$$\text{Buoyant wt. of particle} = W_B = W - F_B$$

$$= V_s \gamma_s - V_w \gamma_w$$

$$= V_s \gamma_s - V_s \gamma_w$$

$$= V_s (\gamma_s - \gamma_w)$$

$$W_B = V_s \gamma_w (G - 1)$$

Buoyant wt. of all the particles.

$$W_B = n \cdot V_s \cdot \gamma_w (G-1)$$

$$= V_s \cdot \gamma_w (G-1)$$

$V_s$  - total volume of solids.

porosity of solids =  $\frac{V_v}{V}$

$$1 - n = 1 - \frac{V_v}{V}$$

$$= \frac{V - V_v}{V}$$

$$1 - n = \frac{V_s}{V}$$

$$V_s = V (1 - n)$$

$$W_B = V (1 - n) \cdot \gamma_w (G-1)$$

$$= (A \cdot D) \cdot (1 - n) \cdot \gamma_w \cdot (G-1)$$

To convert pressure head loss into weight

$$(P_1 - P_2) A = W_B$$

$$\gamma_w \cdot H_L \cdot A = (A \cdot D) \cdot (1 - n) \cdot \gamma_w \cdot (G-1)$$

$$H_L = D (1 - n) (G-1)$$

(xxx) Head loss through the filter medium in the expanded state is equal to head loss through medium in unexpanded state.

for expanded medium -  $D' > D$  &  $(1 - n') < (1 - n)$

As there is no change in the buoyant weight of particles

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

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

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

(xxxii) The porosity of filter medium in expanded state is experimentally found to be function of porosity of filter backwash velocity and settling velocity of medium particles

$$\eta' = f(V_B, V_s) \quad = \text{empirical}$$

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

$$\text{Backwash velocity } (V_b) = \frac{\text{Backwash discharge } (Q_{bw})}{\text{plan area } (A)}$$

$$\text{Settling velocity } (V_s) = \frac{(G-1)\gamma_w \cdot d^2}{18\mu} \quad (\text{for laminar flow only}) - \text{NOT applicable here.}$$

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

$$C_D = 5 \text{ to } 6$$

Q. A rapid sand filter proposed for a water supply treatment plant of town having population of 75,000. Avg. water supply in the town is 150 lpcd. Rate of filtration is 100 l/m<sup>2</sup>/min. Find the size and no. of filter bed required. Design the lateral and manifold under drainage system. Compute the washwater discharge required if rate of washing is 45 cm/min.

Data: ——— population = 75,000 ✓  
 $R_f = 100 \text{ lit/m}^2/\text{min}$   
 Avg. demand = 150 lpcd.

$$\begin{aligned} \text{Design discharge } (Q_d) &= 1.8 \times Q_{\text{avg. daily}} \\ &= 1.8 \times (75,000 \times 150 \times 10^{-3}) \\ &= 20250 \text{ m}^3/\text{day} \end{aligned}$$

Assuming backwashing for 30 min in filter and 5% of  $Q_d$  is required for backwashing.

Thus effective discharge filtered by filter is

$$\begin{aligned} Q_{\text{eff}} &= \frac{20250}{23.5 \text{ hours}} \times 24 \checkmark \\ &= 20680.85 \text{ m}^3/\text{day} \end{aligned}$$

considering backwash quantity.

$$\begin{aligned} Q'_{\text{eff}} &= 20680.85 \times 1.05 \\ &= 21714.89 \text{ m}^3/\text{day} \end{aligned}$$



$$\text{plan area of filter } A = \frac{21714.89 \text{ m}^3/\text{day} \times 10^3}{100 \text{ l/m}^2/\text{min} \times 24 \times 60}$$

$$= 150.79 \text{ m}^2$$

$$\text{No. of filters. - } N = 1.22 \sqrt{Q} \quad Q \text{ in MLD}$$

$$= 1.22 \sqrt{21714.89 \times 10^3 \times 10^{-6}}$$

$$= 5.6 \approx 6 \text{ units}$$

No. of filters are (6 operational + 1 standby)

$$\text{Area of each filter} = \frac{150.79}{6}$$

$$= 25.13 \text{ m}^2 \approx 25 \text{ sq.m.}$$

Assuming filter to be square

$$B = 5 \text{ m}$$

Assuming size of perforations = 6 mm

spacing between perforations = 7.5 cm c/c  
laterals = 25 cm

$$\text{c/s area of all the perforations} = 0.2 \% \text{ of filter area}$$

$$= \frac{0.2}{100} \times 25$$

$$= 0.05 \text{ m}^2$$

$$\text{No. of laterals (N)} = 2 \left( \frac{L}{S} + 1 \right)$$

$$= 2 \left( \frac{500}{25} + 1 \right)$$

$$= 42$$

$$\text{c/s area of each lateral } \left( \frac{\pi d^2 l}{4} \right) = 4 \times \text{c/s. area of perforation}$$

$$= 4 \times \frac{0.05}{42} \times 10^4$$

$$d_l = 7.78 \text{ cm}$$

$$\text{c.s. area of central drain } \left( \frac{\pi D_c^2}{4} \right) = 2 \times \text{c.s. of all laterals}$$

$$= 2 \times \left( \frac{0.05}{42} \right) \times 42 \times 10^4 \times 4$$

$$D_c = 71.36 \text{ cm}$$

check:

$$\frac{\text{Length of lateral}}{\text{dia. of lateral}} = \frac{(B - D_c)/2}{d_l}$$

$$= \frac{(500 - 71.36)/2}{7.78}$$

$$= 27.54 \rightarrow 60 \text{ OK.}$$

$$\text{Backwash discharge } (Q_{BW}) = V_{B/W} \times \text{plan area}$$

$$= 45 \text{ cm/min} \times 25 \text{ m}^2 \times 10^{-2} \times 24 \times 60$$

$$= 16200 \text{ m}^3/\text{day} \quad (\text{for one filter})$$

Q. A rapid sand filter is to be provided in the water treatment plant for population of 2,75,000. Water demand is 200 lpcd and rate of filtration is  $15 \text{ m}^3/\text{m}^2/\text{hour}$ . Allow 5% of the filtered water for storage to meet the backwash requirement. Each backwash period is 30 min. Determine the no. of filters required for allowing one standby, if available surface area is  $10 \times 4 \text{ m}^2$  for each filter. Also calculate upflow velocity and the head loss required to expand the bed to 0.66 m from the original depth of 0.60 m and porosity of 0.5. Drag coeff. for flow is 5.02. Kinematic viscosity of water is  $0.10136 \times 10^{-5} \text{ m}^2/\text{sec}$ . Backflow is in transition. Sp. gravity of particles forming filter medium is 2.5 and effective size of particles is 0.6 mm.

Data

$$\text{population} = 2,75,000$$

$$R_f = 15 \text{ m}^3/\text{m}^2/\text{hour}$$

$$\text{Design discharge } Q_D = 1.8 \times 2,75,000 \times 200 \times \frac{24}{23.5} \times 1.05 \times 10^{-3}$$

$$= 106161.7 \text{ m}^3/\text{day}$$

(considering backwash time and backwash discharge)

$$\text{plan area required (A)} = \frac{Q_D}{R_i}$$

$$= \frac{106161.7 \text{ m}^3/\text{day}}{15 \text{ m}^3/\text{m}^2/\text{hour} \times 24}$$

$$= 294.89 \text{ m}^2$$

$$\text{No. of filters (N)} = \frac{294.89}{(10 \times 4)}$$

$$= 7.37 \approx 8$$

Total filters required - (8 operational + 1 standby) = 9

$$\text{Head loss } (h_f) = D (1 - \eta) (G - 1)$$

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

$$= 45 \text{ cm}$$

$$\text{porosity of expanded state } (\eta') = \left( \frac{V_B}{V_S} \right)^{0.22}$$

$$V_B = \eta'^{1/0.22} V_S$$

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

$$\frac{0.6}{0.66} = \frac{(1 - \eta')}{(1 - 0.5)}$$

$$\eta' = 0.545$$

$$V_S = \sqrt{\frac{4/3 (G - 1) g \cdot d^2}{C_D}}$$

$$= \sqrt{\frac{4/3 (2.5 - 1) \times 9.8 \times 0.6 \times 10^{-3}}{5.02}}$$

$$= 0.048 \text{ m/sec}$$

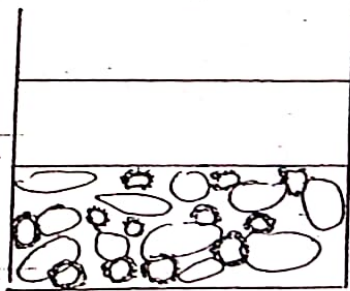
$$\begin{aligned}
 V_B &= (0.545)^{1/0.22} \times (0.048) \\
 &= 3.04 \times 10^{-3} \text{ m/sec} \\
 &= 3.04 \text{ mm/sec}
 \end{aligned}$$

### Operational troubles associated with RSF:

#### 1. Air binding:

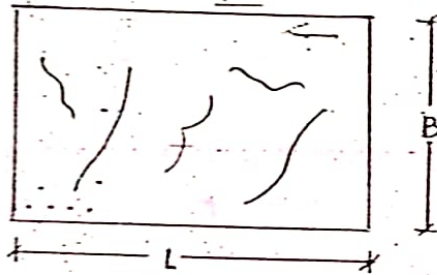
- (i) The initial head loss in the filter is 15-30 cm and this goes on increasing with the operation of filter as more and more impurities are entrapped in the voids of the filter medium.
- (ii) A stage comes when the frictional resistance offered by the medium particles exceeds static head of water. Due to which bottom layers of sand medium starts acting like a vacuum, resulting in the release of dissolved gases present in the water.
- (iii) The bubbles of these gases rises towards surface and gets stick to the medium particles thereby reducing effective area of flow through the medium. The process is known as air binding that seriously affects efficiency of filtration.
- (iv) To avoid it cleaning of filter should be done when head loss becomes equal to 2.5-3.5 m and negative head equal to 1.2 m.
- (v) Inverted head manometer is used for measurement of negative head.

#### 2. Mud ball formation:



- (i) Mud from atmosphere enters into filter medium and gets deposited over particles of medium. Over period of time due to inadequate washing it settles down in bottom layers of medium where it grows in size by combining with the impurities entrapped in the voids of medium, leading to the formation of mudballs.
- (ii) Once these mudballs enter into gravel layer, it starts interfering with the distribution of backwash water, thereby seriously affecting efficiency of filter.
- (iii) To avoid it, adequate surface washing along with the use of Potassium hydroxide (KOH) is adopted.

### 3. Cracking of filter:-



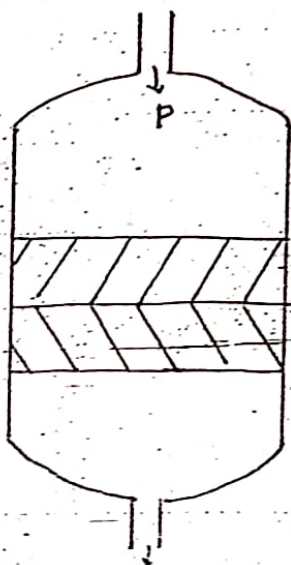
- (i) Due to alternate wetting & drying of filter medium, the medium particles are subjected to shrinkage stresses that leads to the development of shrinkage crack over the surface.
- (ii) As the surface of medium is subjected to constant application of water pressure, cracks once developed increase in size resulting in deeper penetration of impurities in filter medium - thereby reducing efficiency of filter.

Thursday

25<sup>th</sup> July 2013

Pressure filters:

- (i) Pressure filters, the head required by water to pass through filter medium is provided by external application of pressure.
- (ii) This unit is exactly similar to Rapid Sand Filter with the only difference that the entire process takes place in a closed container.



sand

gravel

100-200 m/s  
2400-4800 m/s  
6000-15000 m/s  
Slowly  
Rapid  
Pressure

(iii) In pressure filters water is directly fed to the filter unit without employing plain sedimentation or coagulation-aided sedimentation.

(iv) The dia. of tank is in the range of 1.5-3 m. Depth of tank is 3-3.5 m. Rate of filtration is 6000-15000 lit/m<sup>2</sup>/hr, which is almost double of Rapid sand filter.

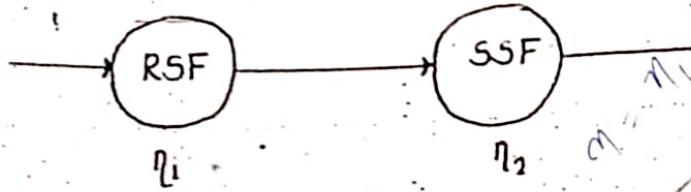
(v) Filters are generally used for the treatment of industrial water or water used in swimming pools.

The container is used to avoid the dissipation of the pressure.

Swimming pool  
Industrial water

HINDUSTAN PHOTOSTAT  
All Study Material Available  
A-16a, Lado Sarai, New Delhi  
Phone: 9911234514

before slow sand filter. The process of double filtration and RSF used is known as Roughing filters.



$R_{f2}$  - role of filtration in SSF is more than conventional SSF. (size of sand is large)

overall efficiency  $(\eta) = \eta_1 + (1 - \eta_1)\eta_2$

$\eta_1 + (1 - \eta_1)\eta_2$

### 6. Disinfection:-

(i) It is the process of removal of disease causing micro-organisms from the water.

Note:

The process of removal of all the micro organisms is called sterilisation. The process of removal of only disease causing microbes is called disinfection.

(i) During disinfection, it is assumed that disinfectant destroys microorganism by any of the following mechanism.

1. by inactivating the enzymes of the micro-organisms required to perform metabolic activities.
2. by damaging the cell walls of the micro-organisms.
3. by changing the nature of their self protoplasm.
4. by altering the permeability of their cells.

(iii) Disinfection of water can be carried out either physically or chemically.

#### Physical disinfection

- Boiling ✓
- UV rays ✓

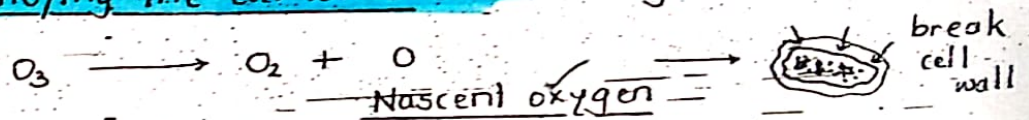
#### Chemical disinfection

- Acids and alkalies ( $pH < 5$ ,  $pH > 11$ )
- Metallic ions ( $Ag^+$ ,  $Hg^+$ )
- Oxidising agents ( $Cl_2$ ,  $Br_2$ ,  $I_2$ ,  $O_3$  &  $KMnO_4$ )

Minor methods:

#### 1. Treatment with ozone ( $O_3$ )

(i) Ozone is strong oxidising agent which carries out the disinfection of water by destroying the cell walls of micro-organisms.



(ii) It also removes organic matter from the water by oxidation. but if organic matter is present in water, efficiency of ozone in disinfection will be reduced.

(iii) Ozone is highly unstable hence nothing remains in the system till it reaches the distribution system. Hence it does not safeguard water against future recontamination.

(iv) Ozone adds pleasant taste to water when used as disinfectant.

(v) Normal dose of ozone is 2-3 ppm. ✓

Note:-

(i) Ozone treatment is used in India only in Chandigarh city due to small area covered by distribution system & availability of capital.

(ii) Boiling, UV rays are practically difficult and uneconomic processes of disinfection for water supplies, but can be carried out at households, industries.



## 2. Treatment with Potassium permanganate ( $KMnO_4$ ):

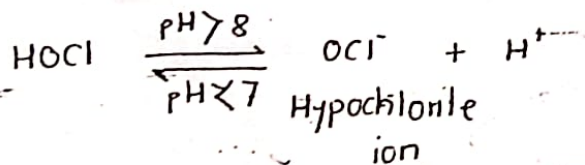
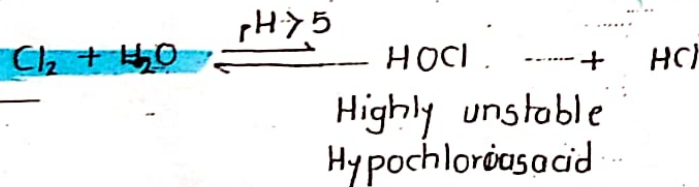
- (i) It is one of the most popular disinfectant used for the treatment of well water supplies which consists of lesser impurities.
- (ii) Potassium permanganate also removes organic matter apart from micro-organism from water.
- (iii) When  $KMnO_4$  is added in the water it induces pink colour in it. If this pink colour disappears, it signifies the presence of the micro organisms and organic matter in water.
- (iv)  $KMnO_4$  is added further in the water, upto an extent pink colour stands in it, signifying the complete removal of organic matter and micro organisms from it.
- (v) The water is not used for next 48 hours till pink colour subsidize. (psychological effect is induced due to colour)
- (vi) Normal dose of  $KMnO_4$  is generally 1-2 mg/l having contact period of 4-6 hours.

More is the reactivity of disinfecting reagent, less is the contact period required.

- (vii)  $KMnO_4$  removes 98% of bacteria and 100% of bacteria causing cholera.

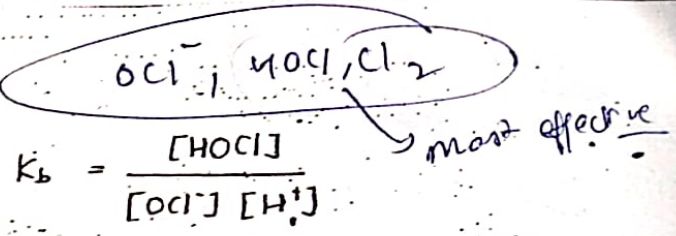
## Major method :-

### Treatment with Chlorine ( $Cl_2$ ):



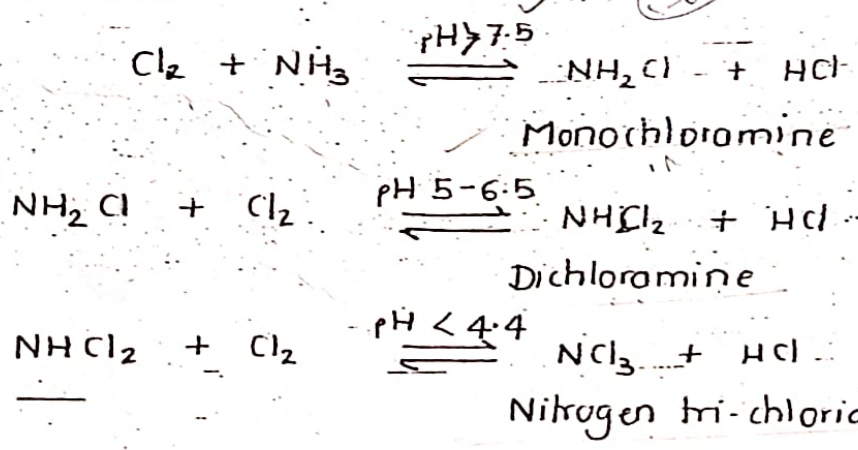
-Rate constants:

$$k_f = \frac{[OCl^-][H^+]}{[HOCl]}$$



$$k_b = \frac{[HOCl]}{[OCl^-][H^+]}$$

- (i) Chlorine reacts with water at pH of greater than 5 to form highly unstable Hypochlorous acid which further dissociates into Hypochlorite ion at pH of greater than 8 and remains in the water as Hypochlorous acid (HOCl) if pH is less than 7.
- (ii) All the above these forms of chlorine (Molecular chlorine -  $Cl_2$ , HOCl and  $OCl^-$ ) known as freely available chlorine is responsible for the disinfection of water.
- (iii) Out of these three forms of chlorine Hypochlorous acid (HOCl) is the most destructive reagent (80 times more effective than hypochlorite ion). ( $OCl^-$ )
- (iv) While carrying out disinfection pH of water is kept slightly less than 7 as Hypochlorous acid will be predominant species present in water at this pH.
- (v) Moreover chlorine reacts with ammonia immediately to form chloramines.



}

M  
M  
M

Chloramines are combined forms of chlorines which are less effective than freely available chlorine as disinfectant. (25 times less effective) but are most stable in comparison to freely available chlorine hence safeguards water from future recontamination.

- (vi) Dichloramine is predominant species present in water at pH of less than 7.
- (vii) The chlorine dose added in the water should be such that, residual of 0.2 mg/l after the contact period of 10 min is left in the water.
- (viii) Chlorine is assumed to inactivate the enzymes of the micro organisms required to carry out the metabolism.

### Forms of Chlorine ( $\text{Cl}_2$ )

#### 1. Free chlorine (solid, liquid or gas):

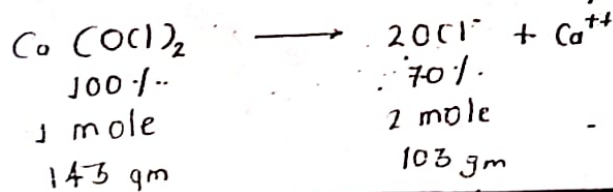
- (i) Chlorine is generally added in liquid form to carry out the disinfection of water.
- (ii) Chlorine is highly temperature susceptible. Chlorine freezes if temperature falls below  $10^\circ\text{C}$  leading to choking of distribution pipe and burns the container in which it is stored above  $92^\circ\text{C}$ .
- (iii) Optimum temperature range to carry out disinfection using chlorine is  $32-48^\circ\text{C}$ .
- (iv) Chlorine does not form sludge while carrying out disinfection nor its quality gets deteriorated with storage.

Note:

Chlorine makes explosive mixtures with Carbon monoxide (CO)

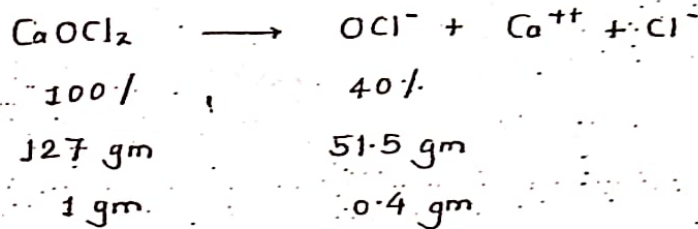
#### 2. Hypochlorides and bleaching powder [ $\text{Ca}(\text{OCl})_2$ , $\text{CaOCl}_2$ ]

- (i) When Hypochlorites and bleaching powder are used as a disinfectant, hypochlorite ion and hypochlorous acid carries out disinfection.
- (ii) For 100% pure Hypochlorite (Calcium hypochlorite) freely available chlorination ability is approximately 70%.



And this also degrades as bleaching powder comes in contact with the moisture present in the atmosphere.

(iii) For 100% pure bleaching powder, freely available chlorination ability is approximately 40%.



Ble  
✓

(iv) This method is generally used for treatment of water in swimming pools and not for treatment of water supplies as it leads to the formation of sludge due to carrying out of disinfection.

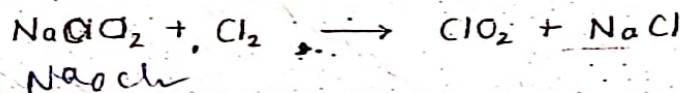
### 3. Chloramines

(i) Chloramine is generally used for treatment of water in which organic compounds like phenols are present.

### 4. Chlorine dioxide (ClO<sub>2</sub>)

phenols

(i) Chlorine dioxide is formed by the addition of Chlorine in the solution of Sodium chlorate.



(ii) Chlorine dioxide is strong oxidising agent which can remove both organic matter and micro organisms from water.

It is highly effective as disinfectant (2.5 times more effective than freely available chlorine) but also highly unstable, hence does not safeguard against future recontamination.

(iii) It can be used for disinfection in water, if phenol is present

stronger is the oxidising agent lesser is the stability.

Practically we go for free chlorine disinfection as it offers both stability and strong oxidation.

NaOCl<sub>2</sub>

2.2

## Effectivity in disinfection:

$\text{ClO}_2 > \text{HOCl} > \text{chloramines} > \text{Hypochlorite ion (OCl}^-)$

## Types of chlorinations :-

- plain chlorination /
- pre chlorination
- post chlorination
- double chlorination
- De-chlorination

### 1. Plain Chlorination :-

When no other treatment except chlorination is given to the water, the treatment is known as plain chlorination. It removes organic matter, micro-organisms and colour from water. It is generally adopted for water having turbidity less than 20-30 ppm. Normal dose is 0.5 mg/l.

### 2. Pre-chlorination :-

- Pre-chlorination is adopted when the conc. of micro-organisms is large in raw water.
- In this method chlorine is added either at the time of coagulation or filtration.
- The normal dose of chlorine should be such that 0.1-0.5 mg/l is reached upto filtration. (If added in coagulation, dose will be more)
- Normal dose is 5-10 mg/l (at coagulation)
- Post-chlorination is always performed after pre-chlorination.

### 3. Post chlorination :-

- When chlorination is the last treatment given to the water, it is termed as post chlorination.
- The normal dose of chlorine should be such that 0.2 mg/l of residual is left in the water after contact period of 10 min.

#### 4. Double chlorination:

When both pre and post chlorination is done simultaneously is known as Double chlorination.

#### 5. Super chlorination:

(i) When excess chlorine is added (5-50 mg/l) while carrying out disinfection of water during epidemic so as to leave 1-2 mg/l of residual chlorine, the process is known as Superchlorination.

(epidemic - mass spread of disease)

(ii) When this excess chlorine is removed from water the process is known as "De-chlorination" and the reagents added in the water to carry out dechlorination are termed as "De-chlorinating agent."

e.g.  $\text{Na}_2\text{S}_2\text{O}_3$  (Sodium thiosulphate)

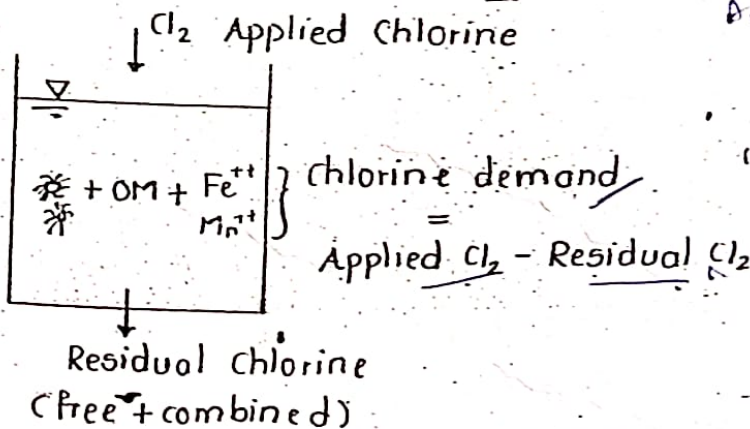
$\text{SO}_2$  (Sulphur dioxide)

Activated carbon

$\text{Na}_2\text{S}_2\text{O}_5$  (Sodium meta bisulphate)

$\text{Na}_2\text{HSO}_3$  (Sodium bisulphide)

#### 6. Breakpoint chlorination:



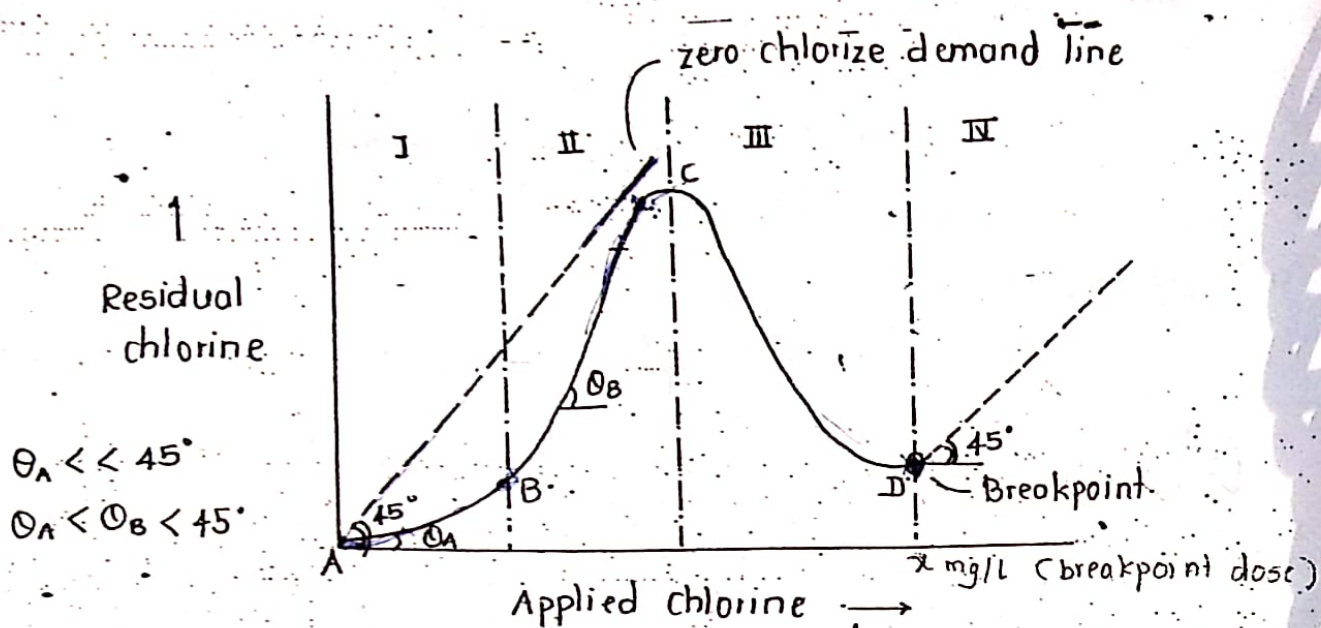
Applied Cl<sub>2</sub> - Residual Cl<sub>2</sub>

(i) Chlorine consumed for the removal of micro organisms, organic matter and Fe<sup>++</sup>, Mn<sup>++</sup>

(i) In the I stage chlorine performs function of removal of micro-organisms and oxidation of minerals present in it.

(ii) During stage 1, residual chlorine appeared in water sample is very less as demand for chlorine in water is very high.

$$\text{O}_A \ll 45$$



- (vi) During stage II, chlorine combines with ammonia & other compounds leading to the formation of chloramines & other combined forms of chlorine which appear as residual chlorine.
- (vii) In stage II, as the demand for disinfection is satisfied, residual chlorine in water increases, but at no point slope is equal to  $45^\circ$  (as some amount of chlorine is still used for disinfection)
- $\theta_A < \theta_B < 45^\circ$
- (viii) At the beginning of III stage (point C), bad smell starts coming out from water sample, indicating oxidation of the organic matter is started leading to increase in chlorine demand of water and decrease in Residual chlorine of water.
- (ix) In stage III, free chlorine breaks the chloramines and the chloro organic compounds into nitrogen compounds. (because of increase in chlorine demand)
- (x) The end of the stage is characterized by removal of organic matter from water which is indicated by bad smell of water. (D)
- (xi) After point D, whatever chlorine is added in the water breaks free from the water and appears as residual chlorine, Hence point D is known as Breakpoint & chlorine dose reqd. is known as "breakpoint dose."

(ix) Theoretically, no chlorine should be added in water after point D, but practically 0.2 mg/l of residual is ensured after point D at 10 min of contact period.

(x) The chlorine demand after point D remains constant.

Q. For the following information, find the breakpoint dose and the chlorine demand at dose of 1.2 mg/l.

Applied $Cl_2$ (mg/l)	Residual $Cl_2$ (mg/l)	
0.2	0.19	} 0.17
0.4	0.36	
0.6	0.50	} 0.14
0.8	0.48	
1.0	0.20	} -0.02
1.2	0.40	
1.4	0.60	} -0.20
1.6	0.80	

Breakpoint dose is 1 mg/l of  $Cl_2$ .

$$\begin{aligned} \text{chlorine demand at } 1.2 \text{ mg/l} &= 1.2 - 0.4 \\ &= 0.8 \text{ mg/l} \end{aligned}$$