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ENVIRONMENTAL ENGINEERING VOLUME I



BY JASPAL SIR

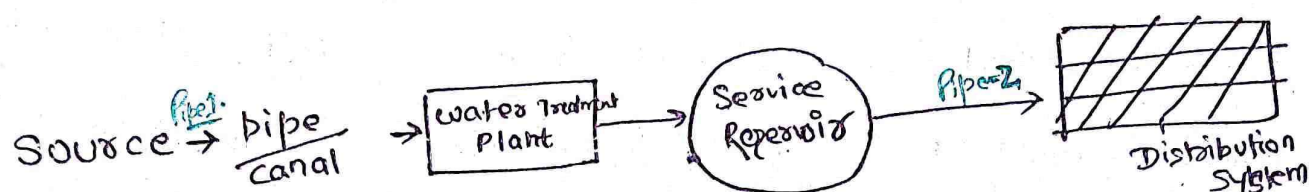
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Water Demand

Raw water: The water which is derived from nature.

river-sedimentation.
rain-aeration.
ground water-filtration



- (i) Water Demand
- (ii) Source of Water
- *** (iii) Quality of Water
- *** (iv) Treatment of Water
- (v) Pipes.
- (vi) Service reservoir
- (vii) Distribution System.

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Water Demand/draft:

(i) Annual draft = V

(ii) Annual avg. daily draft = $\frac{V}{365}$

(iii) Annual avg. per capita daily draft = $\frac{V}{365 \cdot P} = \frac{\text{present population.}}{365 \cdot P}$

Total Demand

l/c/d

✓ Discharge of water

(i) Domestic Water demand = 135-225 (200) (5-6)% flushing system.

(ii) Industrial water demand = 50-450

(iii) Institution water demand = 20-50 l/c/d

(iv) Water for public use = (5-6)% of Total demand = (10-20)

(v) Fire demand = 1 or empirical. Is code = $100\sqrt{P}$ (KL) ← (KL)

thousand
Volume me aaega - India

Kuchling method = $Q = 3182\sqrt{P}$ l/min

P = Population in thousand

Freeman method = $Q = 1136\left(\frac{P}{10} + 10\right)$ l/min

P = Pop. in thousand

National Board of fire underwriters method = $Q = 4637\sqrt{P}(1 - 0.01\sqrt{P})$ l/min

Buston = $Q = 5663\sqrt{P}$ l/min

~~Losses and~~

(vi) Losses and theft - (10-15)% of total demand.

Total demand = 250-350 (335) l/c/d

Factors Influencing demand of water:-

(i) Size of the city.

(ii) Extent of industrialisation.

(iii) Type of sewerage system. (flush system)

(iv) Standard of living / gentry.

(v) Climatic condition. (depend on location of community, latitude)
higher latitude - less temp.
lower latitude - high temp.

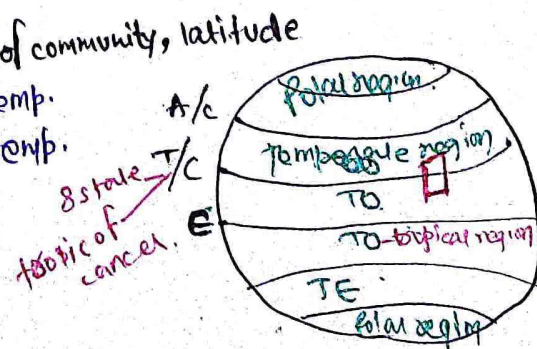
(vi) Cost of water :- *

(vii) Quality of water

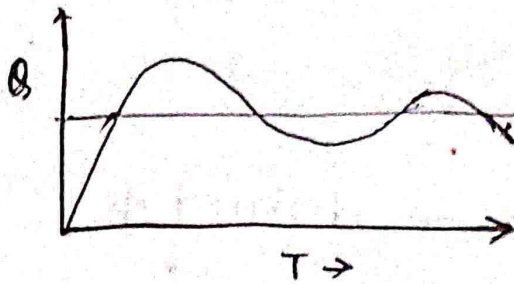
(viii) Pressure in distribution system.

(ix) Type of distribution system. (continuous and ~~intermittent~~)

Demand remain constant but consumption may vary



Fluctuation $FI = \frac{\text{Max. } P}{\text{Avg } P} = \frac{\text{Avg } P}{\text{Min } P}$



Maximum daily demand = $1.8 \times \text{Annual average daily demand} = \frac{1.8 \times 365}{265}$

Maximum hourly demand of max^m day = $1.5 \times \text{Avg. hourly demand of max^m day}$
 $= 1.5 \times \frac{\text{Max^m daily demand}}{24}$

$= 1.5 \times 1.8 \times \frac{\text{Annual average daily demand}}{24}$

Max^m hourly demand of max^m day = $2.7 \times \text{Annual average hourly demand}$

Goodrich eqn:-

percentage fluctuation $\frac{\text{Max^m demand}}{\text{Avg demand}} \times 100$ (in percentage)
 $= 180 t^{-0.2}$ (t=days)

t = time (days) ≥ 1 days and less than 1 year.

Time	Fluctuation
hourly	2.7
daily	1.80
weekly	1.48
Monthly	1.28
Yearly	1

Bigger the sample
lesser the standard
deviation

Population	Peak factor
< 50,000	3
50,000 - 2,00,000	2.5
> 2,00,000	2

} avg 2.7

We know that, raw water scheme designed for maximum hourly design.

(i) Q_{MH} max hourly

(ii) $Q_{MH} + Q_{FD}$ fire demand (on including fire demand cost increased)

(iii) $Q_{MD} + Q_{FD}$ [Coincident demand / draft]

* Design should be done.

(i) Q_{MH}
(ii) $Q_{MD} + Q_{FD}$ } max

* Design life and design discharge:-

Component	Design life (yrs)	Design Discharge.
Source	50	Q_{MD}
Pipe-I	30	Q_{MD}
Pump Canal	15	Q_{MD}^*
WTP	15	Q_{MD}
Service Reservoir	15	Volume
Pipe-II	30	Q_{MH} $Q_{MD} + Q_{FD}$ } max
distribution system	30	Q_{MH} $Q_{MD} + Q_{FD}$ } max

Pump & Canal - is not used for 24 hrs.

$Q = 2 \times 10^5$, AAD = 300 l/c/d.

(i) find Important type of demand.

(ii) design capacity of all components of RWS.

$Q_{MD} = \text{Maximum daily demand} = 1.8 \text{ AAD} = 1.8 \times 300 \times 10^{-6} = 108 \text{ MLD}$

~~$Q_{MH} = \text{Max}^m \text{ hourly demand} = 2.7 \times 300 \times 10^{-6} = 810 \text{ l/c/d}$~~

~~fixed demand =~~

$Q_{MH} = \text{Max}^m \text{ hourly demand} = 2.7 \left(\frac{Q}{24} \right)$
 $= 2.7 \times 300 \times 10^{-6} \times \frac{10^{-6}}{24}$
 $= 6.75 \text{ MLH}$
 $= 6.75 \times 24 \text{ MLD} = 162 \text{ MLD}$

or, $Q_{MH} = 1.5 Q_{MD} = 1.5 \times 108 = 162 \text{ MLD}$

Fixed demand

As per NBFWM.

$Q_{FD} = 4637 \sqrt{P} (1 - 0.01 \sqrt{P})$
 $= 4637 \sqrt{200} (1 - 0.01 \sqrt{200})$
 $= 56303 \text{ l/min}$
 $= 56303 \times 10^{-6} \times 60 \times 24 \text{ MLD}$
 $= 81.07 \text{ MLD}$

Coincident demand.

$Q_{CD} = Q_{MD} + Q_{FD} \}$ max
 $= Q_{MH}$

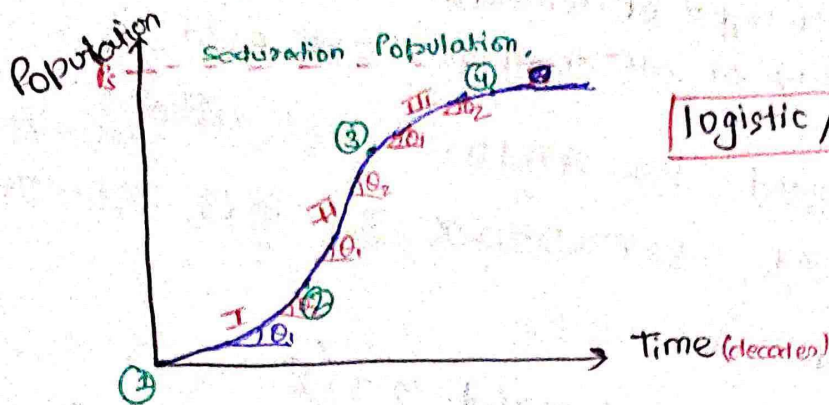
$Q_{MD} + Q_{FD} = 108 + 81 = 189 \text{ MLD} \}$ max
 $Q_{MH} = 162 \text{ MLD}$

$Q_{CD} = 189 \text{ MLD}$

Pipe-II and distribution system for $Q_{CD} = 189 \text{ MLD}$.

Source, Pipes, Pump, WTP, $Q_{MD} = 108 \text{ MLD}$

* Population forecasting.



$$y = f(x)$$

$$P = f(t)$$

⊕ $\frac{dy}{dx} = \frac{dP}{dt}$ (↑) 'In a young city rate of growth of population increase
($\theta_2 > \theta_1$)

⊖ In old city rate of development stabilises $\frac{dP}{dt} \approx c$
 $\theta_2 \sim \theta_1$

III Very old city - Population is increasing but its rate is decreasing.
 $\theta_2 < \theta_1$
 $\frac{dP}{dt}$ (↓) $\propto (P_s - P) \downarrow$

Part III

Population of any community depend upon:-

- (a) Growth rate.
- (b) Death rate
- (c) Migration rate $\sim (+, -)$
- (d) Increase due to annexation - [forcefully taking land of other country] eg. china taking land of Hongkong.

DO YOU KNOW?

Pandemic - when it is in every place of world.
Epidemic - when it was only in within (local area)

* Methods of Population forecasting:-

(i) Arithmetic Increase Method.

→ In arithmetic Method we considered rate of growth of population is constant. $\frac{dP}{dt} = \text{constant}$

→ In this method, we consider the average increasing of population over the present population.

eq. pop ⁿ -	1970 25	1980 50	1990 100	2000 175	2010 200	2020 200 + 43.75 = 243.75	2030 243.75 + 43.75 = 287.5 ≈ 288
increase, $\bar{x}$ =	25	50	75	25			

$$\text{average } \bar{x} = \frac{25 + 50 + 75 + 25}{4} = 43.75 / \text{decade.}$$

$$t = t_0$$

$$P = P_0$$

$$t = t_1$$

$$P_1 = P_0 + \bar{x}$$

$$t = t_2$$

$$P_2 = P_1 + \bar{x} = P_0 + 2\bar{x}$$

$$t = t_n$$

$$P_n = P_{n-1} + \bar{x} = P_0 + (n-1)\bar{x} + \bar{x} = P_0 + n\bar{x}$$

(ii) Geometric Increase Method / compounding / Uniform Increase method

→ In this method, percentage rate of growth is constant.

Year → 1970	1980	1990	2000	2010	2020	2030
Population → 25	50	100	175	200	$= P_0(1+r)^t$ $= 314.4$	$= 200 \times (1+0.572)^2$ $= 494.23$ ≈ 495
	$\frac{50-25}{25} \times 100 = 100\%$	$\frac{100-50}{50} \times 100 = 100\%$	$\frac{175-100}{100} \times 100 = 75\%$	$\frac{200-175}{175} \times 100 = 14.28\%$		

$$r\% = \frac{dP}{dt} \times \frac{1}{P} \times 100 = \text{const.}$$

Using Arithmetic mean method,

$$= \frac{100 + 100 + 75 + 14.28}{4} \% / \text{decade}$$

$$= 72.32 \% / \text{decade}$$

By using Geometry mean method =

$$\frac{4}{\sqrt[4]{100 \times 100 \times 75 \times 14.28}} = 57.2 \% / \text{decade.}$$

$$\begin{aligned}
 t=t_0 & \quad P=P_0 \\
 t=t_1 & \quad P_1 = P_0 + r \cdot P_0 \cdot 1 = P_0 (1+r) \\
 t=t_2 & \quad P_2 = P_1 + r \cdot P_1 \cdot 1 = \cancel{P_0(1+r)} + P_1 (1+r) \\
 & \quad = P_0 (1+r)(1+r) = P_0 (1+r)^2
 \end{aligned}$$

$$\begin{aligned}
 t=t_n & \quad P_n = P_{n-1} + r \cdot P_{n-1} \cdot 1 = P_{n-1} (1+r) \\
 & \quad = P_0 (1+r)^{n-1} (1+r) \\
 & \quad = P_0 (1+r)^n
 \end{aligned}$$

$$g\% = \frac{dP}{dt} \times \frac{1}{P} \times 100 = \text{constant}$$

⇒ To calculate % rate growth (% g)

(i) Arithmetic mean method. - Used when 3 values given

$$\%g = \frac{g_1 + g_2 + g_3 + \dots + g_n}{n}$$

(ii) Geometric mean method. - Used when 3 values given.

$$\%g = \sqrt[n]{g_1 \cdot g_2 \cdot g_3 \cdot \dots \cdot g_n}$$

$$\boxed{AM > GM}$$

(iii) $P_n = P_0 (1+r)^n$ - when two value given.

$$r = \sqrt[n]{\frac{P_n}{P_0}} - 1$$

Our code says, when you are using ~~as~~ geometric increase method you should use geometric mean method.

(iii) Incremental Increase Method. (It requires 4 data)

- In this method we do ~~not~~ assume rate of growth of population constant.
- In this method we consider an average of incremental over the increase in population.

$$t = t_0$$

$$P = P_0$$

$$t = t_1$$

$$P_1 = P_0 + \bar{x} + \bar{y} \cdot 1 = P_0 + \bar{x} + \bar{y}$$

$$t = t_2$$

$$P_2 = P_1 + \bar{x} + 2\bar{y} = P_0 + \bar{x} + \bar{y} + \bar{x} + 2\bar{y} = P_0 + 2\bar{x} + 3\bar{y}$$

$$t = t_3$$

$$P_3 = P_2 + \bar{x} + 3\bar{y} = P_0 + 2\bar{x} + 3\bar{y} + \bar{x} + 3\bar{y} = P_0 + 3\bar{x} + 6\bar{y}$$

$$t = t_n$$

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

Q

Year	1970	1980	1990	2000	2010	2020	2030
Population	30	50	100	130	165		
Increase $\bar{x}$		20	50	30	35		
$\bar{y}$		30	-20	5			

$$\bar{x} = \frac{20 + 50 + 30 + 35}{4} = 33.75 / \text{decade.}$$

$$\bar{y} = \frac{30 + (-20) + 5}{3} = 5 / \text{decade.}$$

$$P_{2020} = P_0 + n\bar{x} + \frac{n(n+1)}{2} \bar{y}$$

$$= 165 + 1 \times 33.75 + \frac{1 \times 2}{2} \times 5$$

$$= 203.75$$

$$P_{2030} = P_0 + n\bar{x} + \frac{n(n+1)}{2} \bar{y}$$

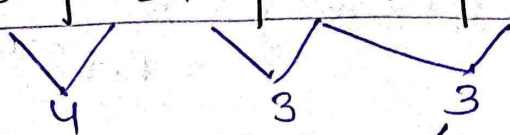
$$= 165 + 2 \times 33.75 + \frac{2 \times 3}{2} \times 5$$

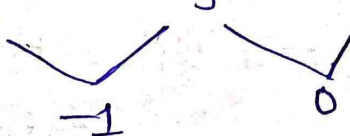
$$= 247.5$$

	Population	City
Arithmetic	- lowest	Very old
Geometric	- Highest	Young city
Incremental	- Intermediate.	Old city.

Q

Year	1970	1980	1990	2000	2030
Population	20	24	27	30	?

$x =$ 

 $y =$ 

Arithmetic method.

$$\bar{x} = \frac{4+3+3}{3} = 3.33$$

$$P_{2030} = 30 + 3 \times 3.33 = 40$$

Geometric method ① First calculate %r

$\frac{24-20}{20} \times 100 = 20\%$

 $\frac{27-24}{24} = 12.5\%$

 $\frac{30-27}{27} = \frac{1}{9} = 11.11\%$

%r is decreasing with time so it is very old city
 So we use arithmetic increase method.

Q Present population = 56000, water demand = 8400 m³/day
 It is expected that after population after 20 years
 = 88000 we have to provide water supply scheme.

Water treatment plant capacity = 12000 m³/day.
 No. of years plant will reach its design capacity. Assume
 geometric growth.

per person = 6.6666 m³/day

no.	Quantity m ³ /day
56000	8400
80000	12000
88000	?

$$r\% = \sqrt[n]{\frac{P_n}{P_0}} - 1$$

$$= \sqrt[20]{\frac{88000}{56000}} - 1$$

$$r\% = 19.52\%$$

$$88000 = 56000(1 + 0.1952)^n$$

Solution

$$P_0 = 56000$$

$$WD = 84000 \text{ m}^3/\text{d}$$

$$APWP = \frac{84000 \times 10^3}{56000} = 150 \text{ l/c/d}$$

$$\text{Design population} = \frac{12000 \times 10^3}{150} = 80000$$

r%

$$P_2 = P_0(1+r)^2$$

$$88000 = 56000(1+r)^2$$

$$1.25 = 1+r$$

$$r = 25\% \text{ or } 0.25$$

$$r\% = 25\% \text{ or } 0.2535\%$$

$$P_n = P_0(1+r)^n$$

$$80000 = 56000(1 + 0.2535)^n$$

$$\frac{10}{7} = (1.2535)^n$$

$$\log\left(\frac{10}{7}\right) = n(\log 1.2535)$$

$$n = 1.578 \text{ decades} = 15.78 \text{ years}$$

IV) Decreasing Rate of Growth Method.

Year	Population (10^5)	% γ	decrease in % γ
1970	50		
1980	60	$20\% = \frac{60-50}{50}$	3.33%
1990	70	16.67%	$16.67 - 14.28 = 2.36\%$
2000	80	14.28%	$14.28 - 18.75 = -4.47$
2010	95	18.75%	
2020	?	$18.75 - 0.416 = 18.334$	
2030	?	$18.33 - 0.416 = 17.918\%$	

avg of decrease in % γ
 $= \frac{3.33 + 2.386 + (-4.47)}{3}$
 $= 0.416\%$

$$P_{2020} = P_{2010} + 18.334\% \times P_{2010}$$

$$= 95 + 18.334\% \text{ of } 95$$

$$= 112.41 \times 10^5$$

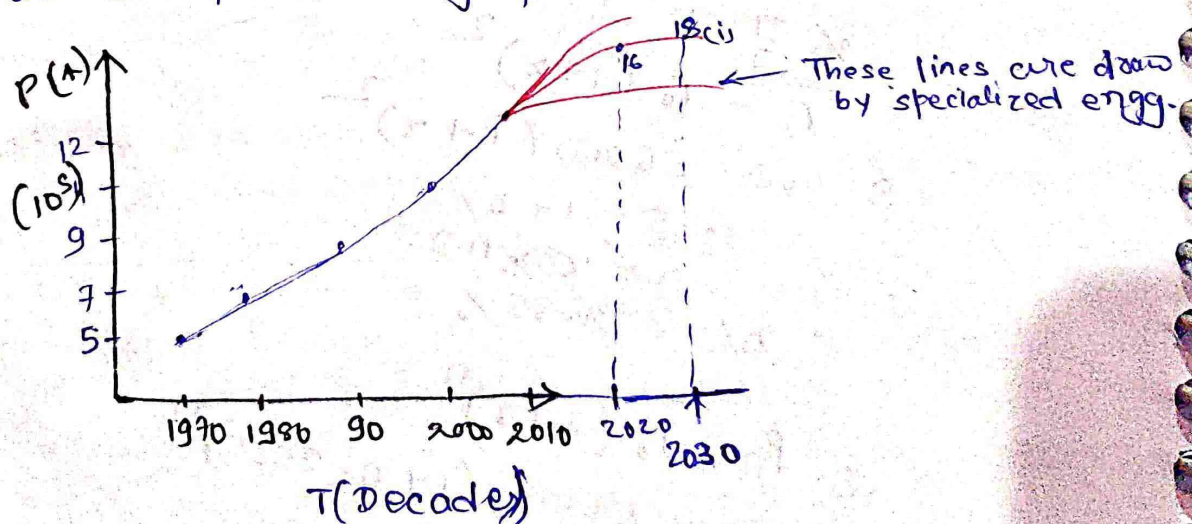
$$P_{2030} = 112.41 + 17.918\% \text{ of } P_{2020}$$

$$= (112.41 + 17.918\% \text{ of } 112.41) \times 10^5$$

$$= 132.62 \times 10^5$$

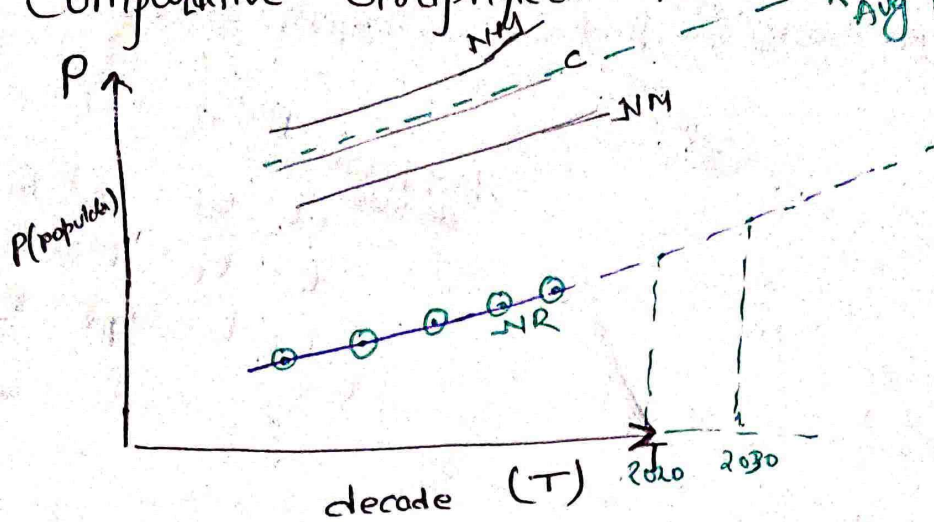
(v) Graphical Method.

In this, we ~~can~~ plot the graph of population for a community.



→ If past data is missing, this method is used.

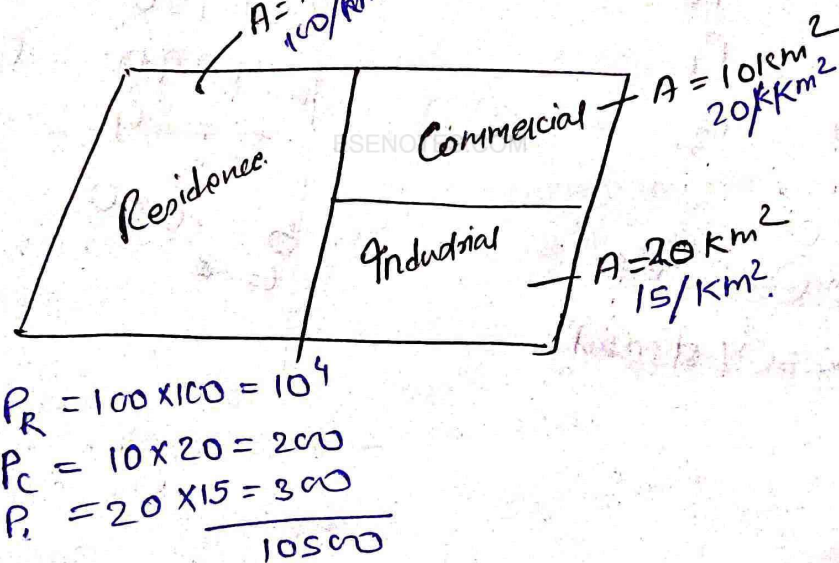
(vi) Comparative Graphical Method



* Comparative graphical method is more accurate as compare to Graphical method.

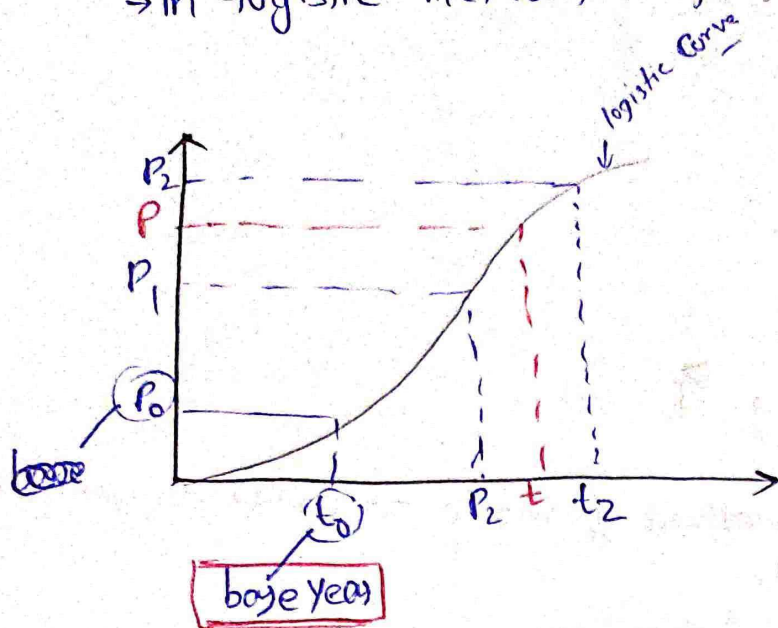
(vii) Master Plan Method.

→ Best method.



(viii) Logistic Method.

→ In logistic method, we find eqn of logistic curve.



$P(x, y) \rightarrow (t, P)$
 → last known decade $t = t_0$ $P = P_0$
 $t = t_1$ $P = P_1$
 $t = t_2 = 2t_1$ $P = P_2$

eg. 1950 P_0
 1970 P_1
 2010 P_2

$t_0 = 1950$
 $t_1 = 2 \text{ decade} = 1970$
 $t_2 = 4 \text{ decade} = 1990$
 $t_2 = 2t_1$

1950 P_0
 1970 P_1
 1990 P_2
 $t_2 = 2t_1$
 2030
 $t = 80$

Equation of logistic curve,

$$\ln\left(\frac{P_s - P}{P}\right) - \ln\left(\frac{P_s - P_0}{P_0}\right) = -K P_s t$$

P_s = Saturation population
 P_0 = base year population
 P = Population at any time
 K = constant depend upon P_0, P, P_s

$P_s, P, K = ?$

$y = f(x)$
 $y = f(t)$

$$t = t_1$$

$$\ln \left(\frac{P_s - P_1}{P_1} \right) - \ln \left(\frac{P_s - P_0}{P_0} \right) = -k P_s t_1 \quad \text{---(ii)}$$

$$\ln \left(\frac{P_s - P_2}{P_2} \right) - \ln \left(\frac{P_s - P_0}{P_0} \right) = -k P_s t_2 \quad \text{---(iii)}$$

$$\boxed{\ln^{-1} = e}$$

$$\text{or, } P = \frac{P_s}{1 + m \ln^{-1}(nt)}$$

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

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

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

Year	Population
1991	80000 = $P_0 = 8$
$t_1 = 10 \text{ years}$ 2001	250000 = $P_1 = 25$
$t_2 = 20 \text{ years}$ 2011	480000 = $P_2 = 48$
2031	

$$P_0 = 25000$$

$$P_1 = 30000$$

$$P_2 = 37000$$

$$t = 0$$

$$t_1 = 10 \text{ years}$$

$$t_2 = 20 \text{ years}$$

$$P_s = \text{saturation population} = \frac{2 P_0 P_1 P_2 - P_1^2 (P_0 + P_2)}{P_0 P_2 - P_1^2}$$

$$= \frac{2 \times 80000 \times 250000 \times 480000 - (250000)^2 (80000 + 480000)}{80000 \times 480000 - (250000)^2}$$

$$= \frac{2 \times 8 \times 25 \times 48 - (25)^2 (8 + 48)}{8 \times 48 - (25)^2}$$

$$= 655602$$

$$\text{(ii) } P = \frac{P_s}{1 + m \ln^{-1}(nt)}$$

$$\text{(i) } m = \frac{P_s - P_0}{P_0}$$

$$= \frac{655602 - 80000}{80000}$$

$$= 7.195$$

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

$$= \frac{1}{10} \ln \left[\frac{80000(655602 - 250000)}{25000(655602 - 80000)} \right]$$

Years

$$= \cancel{0.9} - 0.149 = \overset{\text{Sir tale}}{-1.49}$$

$$\Rightarrow P = \frac{655602}{1 + 7.195 \ln^{-1}(-1.49 \times t)}$$

$$t = 4 \text{ decades}$$

$$P = \frac{655602}{1 + 7.195 \ln^{-1}(-1.49 \times 4)}$$

$$P = \frac{655602}{1 + 7.195 e^{-5.96}}$$

$$P = 643654$$

$$P = \frac{P_s}{1 + m \ln^{-1}(nt)}$$

$$\begin{aligned} \ln^{-1} t &= x \\ t &= \ln x \\ x &= e^t \end{aligned}$$

$$\begin{aligned} \ln^{-1}(-1.996) \\ = e^{-5.96} \end{aligned}$$

SOURCE OF WATER

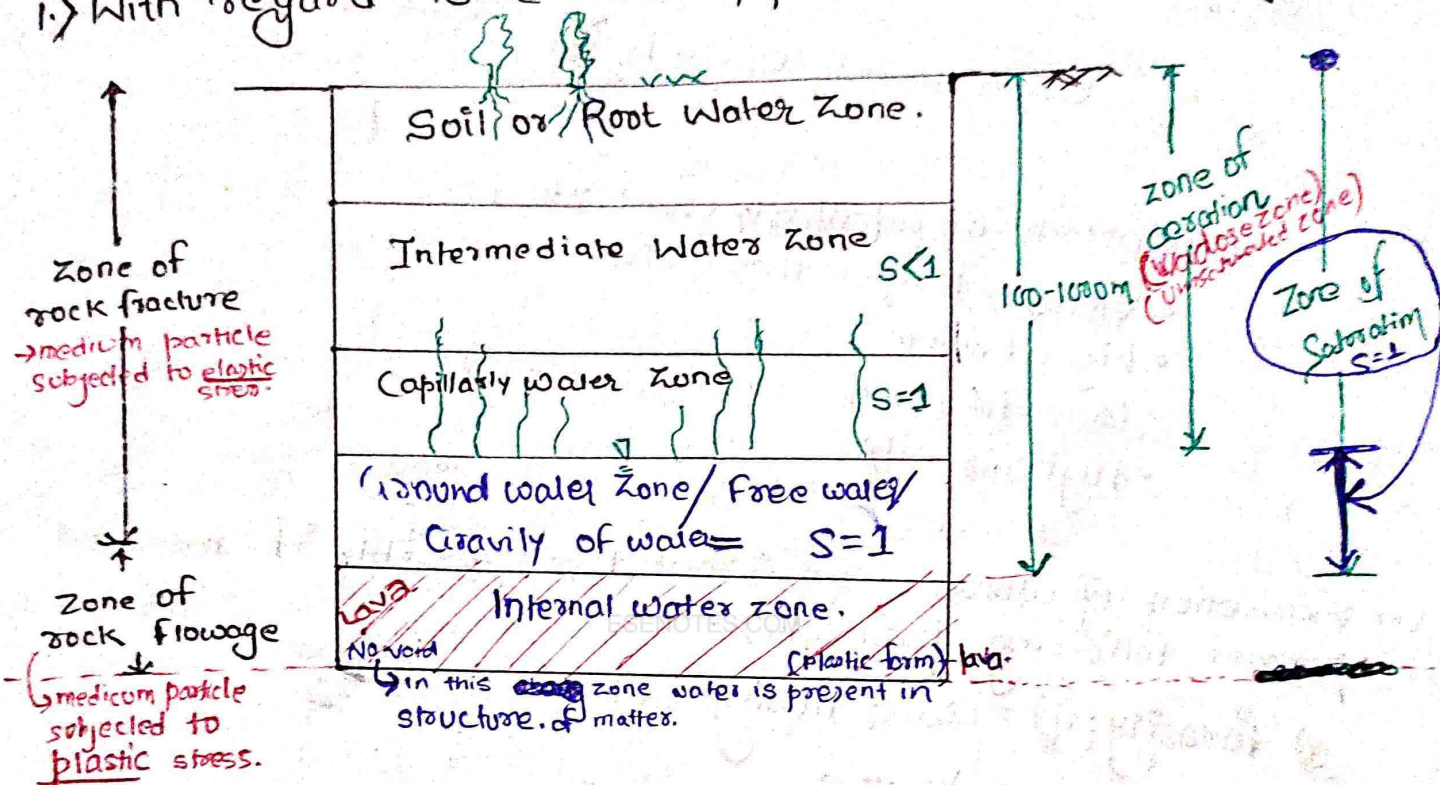
marshy - water table

∇ water table

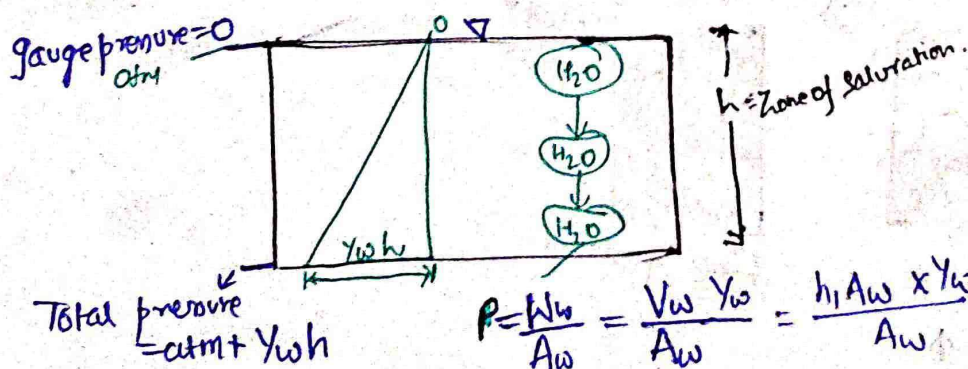
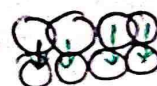
g.L.

SOURCE $\rightarrow$ Surface
 $\rightarrow$ Subsurface (ground water) - soil
 $\rightarrow$ Rain/precipitation.

1. With regard to extraction/presence of water in ground.

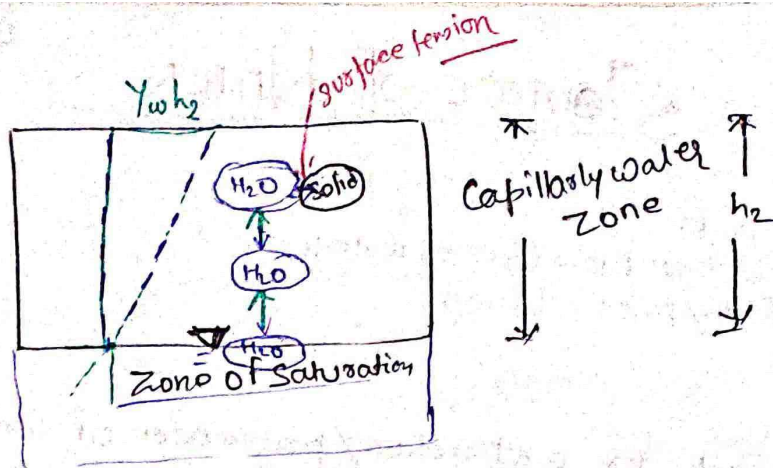


* Soil transfer its weight through soil grains not water.



$$P = \frac{W_w}{A_w} = \frac{V_w \gamma_w}{A_w} = \frac{h_1 A_w \times \gamma_w}{A_w} = \gamma_w h$$

$\rightarrow$ hydrostatic pressure (compression)
 $\rightarrow$ Pressure head = h



-ve	+ve = tension
+ve	-ve = Compression

$$\text{Surface tension} = W_w = h_2 \gamma_w$$

Every property of capillary water :-

- Viscosity is higher than water
- higher boiling
- lower freezing
- higher density

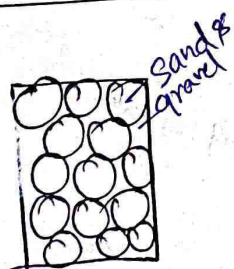
(ii) Extraction of water from Ground is possible if medium possess following properties:-

Porosity (n) = water holding capacity & voids.

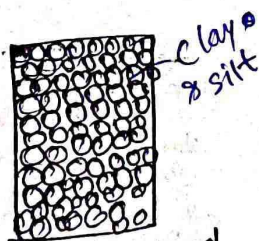
$$= \frac{V_v}{V} \times 100$$

$$n \propto V_v \propto \frac{1}{d}$$

d = size of particle.



Coarse



Fine grained

Size of voids $\rightarrow \uparrow$
 Volume of void $\rightarrow \uparrow$
 no. of voids $\rightarrow \downarrow$

$\downarrow$
 $\downarrow$
 $\uparrow$
 $\uparrow$

Total volume of void = $n \times V_v$

Ground Water Development.

With regards to the existence of water below the Earth crust. It is divided into following zone.

(i) Zone of rock flowage:-

- It is the zone in which medium particles are being subjected to permanent deformation due to the application of plastic stress over them. **NO VOIDS.**
- Water in this zone is present only inside the structure of medium particles thereby it is termed as internal water zone. Structural water.
- Thickness of this zone extends upto several kms. below the earth crust.
- This zone is no. of egg use. as water can't be derived by it by any means.

(ii) Zone of Rock Fracture!

- It is the zone in which medium particles are being subjected to elastic stresses hence water is available in the voids of this zone extraction of water which depends upon the properties of the medium.
- Thickness of this zone is the range of 100-1000m below the earth crust.
 - This zone is further divided into:-
 - (a) Zone of saturation & zone of aeration.

Ground / free / gravity of water zone:-

Ground → because it found in ground.

free → free to move.

gravity → only weight of water act.

Note:-

(ii) If we have choice to extract water from capillary ground then our first choice would be ground water zone because in this water is free but in capillary zone water is held by soil so more energy is required to extract water from this zone.

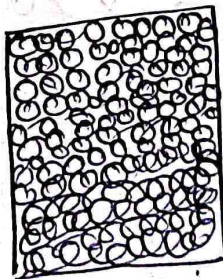
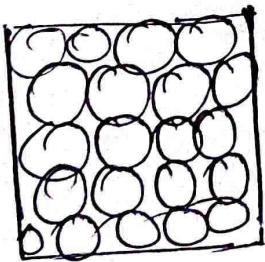
(ii) If we have the choice in ^{between} ~~blue~~ intermediate water zone and capillary zone then our first choice will be capillary because there more water is available whereas in intermediate zone voids are ~~poor~~ probability filled with water so here will be lack of water.

⑪ Permeability/Hydraulic Conductivity (K)
→ Permeability is the ability of the medium to permit the flow of voids fluid through its interconnecting voids it may be defined as per Darcy's law.

⑫ Permeability/Hydraulic Conductivity (K)
→ Permeability is the ability of the medium to permit the flow of voids fluid through its interconnecting voids it may be defined as per Darcy's law.

$$K = f(\text{friction}) \propto \frac{1}{f}$$

* friction ~~depends~~^{act} upon ^{on} contact area.
but in soil it act on surface area.



droplet
particle
surface area

no. of particle

Total surface area = $n \cdot SA$

Permeability

* Water is only extracted from coarse site, fine sand.

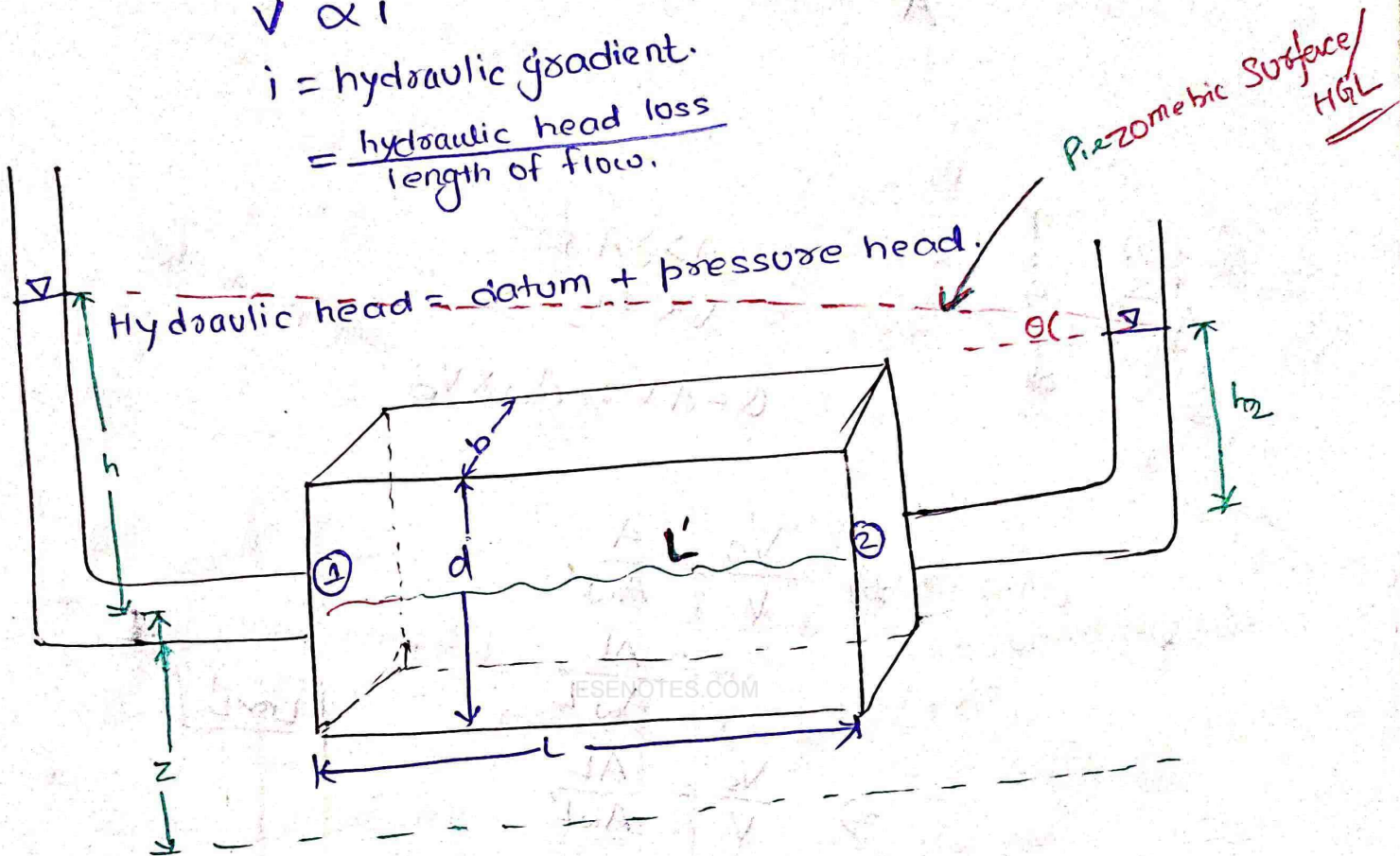
Darcy's law

Assumption: - $S=1$ and laminar flow.

$$V \propto i$$

i = hydraulic gradient.

$$= \frac{\text{hydraulic head loss}}{\text{length of flow.}}$$



$$i = \frac{(z+h_1) - (z+h_2)}{L'} = \frac{h_1 - h_2}{L'}$$

$(L' > L)$

HGL Hydraulic gradient line,

$$\tan \theta = \frac{h_1}{L}$$

Assume $L' = L$

$$i = \frac{h_1}{L} \approx \frac{h_1}{L}$$

$$V = Ki$$

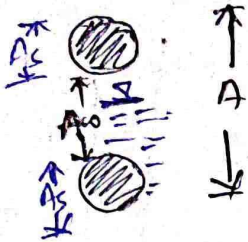
$$Q = VA$$

$$Q = k i A$$

$$\text{if } i = 1$$

$$Q = k A$$

$$k = \frac{Q}{A} \text{ (m/sec)} = \frac{\text{m}^3}{\text{sec} \cdot \text{m}^2}$$



$$A \gg A_w$$

$$V \ll V_s$$

$$Q = A V = A_w \times V_s \text{ (seepage velocity)}$$

$$\Rightarrow \frac{V_s}{V} = \frac{A}{A_w}$$

$$= \frac{A L}{A_w L}$$

$$L \propto L'$$

$$\Rightarrow \frac{V_s}{V} = \frac{A L}{A_w L'}$$

$$\Rightarrow \frac{V_s}{V} = \frac{V}{V_w}$$

$$= \frac{V}{V_v} \cdot \frac{V_w}{V_w}$$

$$\frac{V_s}{V} = \frac{1}{\left(\frac{V_v}{V}\right) \left(\frac{V_w}{V_w}\right)}$$

$$V_s = \frac{V}{\eta_s}$$

$$\Rightarrow \text{if } s = 1$$

$$V_s = \frac{V}{\eta}$$

$$0 < s \leq 1$$

$$0 < \eta < 1$$

$$\therefore V_s > V$$

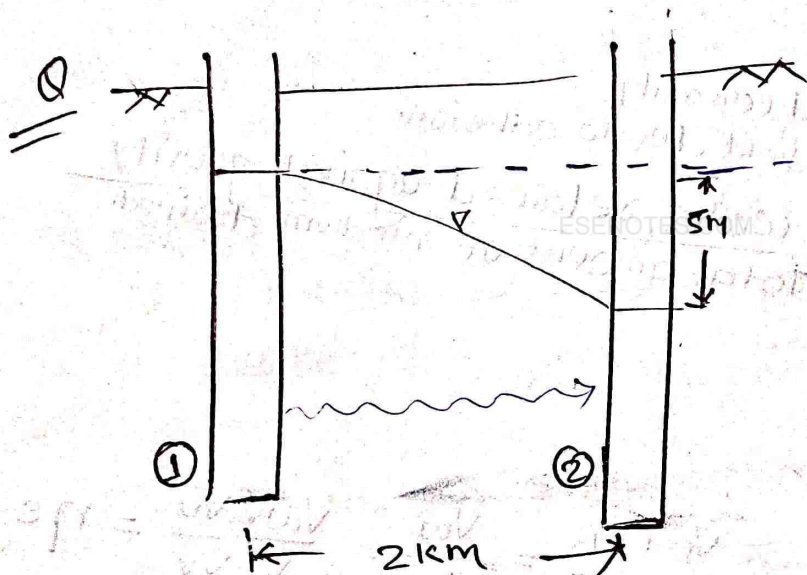
Transmissibility

→ Rate of flow through medium per unit width. ~~on~~ under unit hydraulic gradient.

$$T = \frac{Q}{b} = \underline{m^3/m/sec}$$

$$K = \frac{Q}{bd}$$

$$K = \frac{T}{d} \Rightarrow \boxed{T = Kd} = m^3/m/sec.$$



$$\begin{aligned} \eta &= 0.35 \\ K &= 300 \text{ m/day} \\ t &= ? \end{aligned}$$

$$\text{time of travel} = \frac{\text{distance} \approx 2 \text{ km}}{V_s}$$

$$= \frac{2000 \text{ m}}{\frac{V}{\eta}}$$

$$= \frac{2000 \times \eta}{V}$$

$$= \frac{2000 \times 0.35}{Ki}$$

$$= \frac{2000 \times 0.35}{300 \times \frac{5}{2000}}$$

$$= 933.3 \text{ days} \\ = 2.55 \text{ years.}$$

$$i = \frac{h_L}{L} \approx 2000 \text{ m}$$

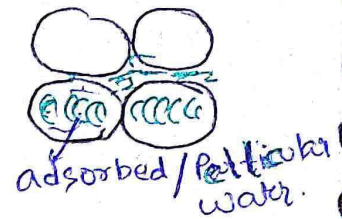
(ii) Specific Yield (Y_s)

(specific - per unit quantity (m or v))

→ Even if the medium is porous and permeable, we can't withdraw all water. because certain amount of water will be retained over surface of ^{medium} particle because of physical force of attraction between water molecule and particle. and this water retained inside the medium particle this water is called adsorbed water.

(iii) Specific Yield (Y_s)

Amount of water drained by gravity.
 $Y_s = \frac{\text{Amount of water drained by gravity}}{\text{Amount of medium drained}}$



$$Y_s = \frac{V_w}{V} \times 100.$$

Specific Retention / field capacity.
 → The water retained in field due to adhesion.

$$Y_R = \frac{\text{Amount of water retained against gravity}}{\text{Total amount of medium drained}}$$

$$= \frac{V_R}{V} \times 100.$$

$$Y_s + Y_R = \frac{V_D}{V} + \frac{V_R}{V} = \frac{V_D + V_R}{V} = \frac{V_w}{V} = \frac{V_w}{V} \cdot \frac{V_v}{V_v} = \eta_s$$

$$Y_s + Y_R = \eta_s$$

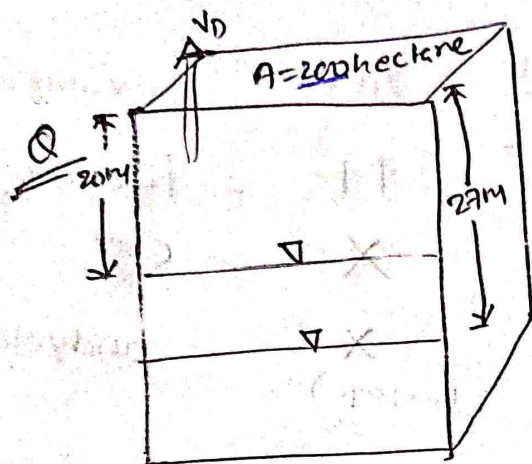
if $S = 1$

$$Y_s + Y_R = \eta$$

$$Y_s = f[d] \quad [d \uparrow \rightarrow SSA \downarrow \rightarrow f \downarrow \rightarrow Y_s \uparrow]$$

$$Y_R = f[d] \quad [d \uparrow \rightarrow SSA \downarrow \rightarrow Y_R \downarrow]$$

Clay, silt $\rightarrow$ retention $\uparrow$ Yield $\downarrow$
 Sand, gravel $\rightarrow$ retention $\downarrow$ Yield $\uparrow$



change in ground water storage due to pumping
 $\eta = 0.4$, $Y_R = 0.15$.

Nature of change = decreasing.

V_D

$$Y_s = \frac{V_D}{V}$$

$$V_D = Y_s V$$

$$V_D = (\eta - Y_R) \times V$$

$$= (0.4 - 0.15) \times 7 \times 200$$

$$= 350 \text{ hectare-meter.}$$

$$Y_s + Y_R = \eta$$

$$Y_s = (0.4 - Y_R)$$

~~350~~

$$\eta = 0.4$$

If due to rain 500 hectare-m

$$300 \text{ hac-m} \rightarrow 7 \text{ m} \uparrow$$

$$50 \text{ hac-m} \rightarrow 1 \text{ m} \uparrow$$

$$500 \text{ hac-m} \rightarrow 10 \text{ m} \uparrow$$

$$\text{New water table} = 27 - 10 = 17 \text{ m from top.}$$

$$\eta = 0.4 = \frac{V_D}{V} = \frac{V_w}{V}$$

$$= V_w = \eta \times V$$

$$= 0.4 \times 200 \times 7$$

$$= 560 \text{ hectare-m}$$

DIFFERENT TYPES OF GEOLOGICAL FORMATION

1) Aquifer: Aquifer are those geological formation which are highly porous and permeable. hence, they give high yield.
eg. fine sand, coarse sand.

	Porosity (n)	Permeability (k)	Yield	Example
1) AQUIFER	↑↑	↑↑	↑↑	fine sand, coarse silt.
2) AQUICLUDE	↑↑↑	↓↓↓	X	clay
3) AQUITARD	↑↑ ↑↑	↓↓	X (seepage)	- sandy clay
4) AQUIFUSE	X (voids zero)	X	X	Granite stone

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Types of Aquifer :-

- Unconfined Aquifer / Non-artesian / water table aquifer.
- Confined aquifer / Artesian.
- Perched aquifer.

② Aquiclude:- These are type of geological formation which are high porous but impermeable hence water can't be drained through it eg. clay.

③ Aquitard: These are type of formation which are porous (but less as compare to aquiclude) and less permeable hence water does not flow through them but instead it seeps. (seepage) eg. sandy clay.

④ Aquifuse:- These are formation which are neither porous nor permeable.
eg. granite rock.

Type of Aquifer:-

(i) Unconfined/Water Table Aquifer/Non-artesian.

- No confining pressure.
- No artificial pressure.
- Top zone of saturation is water table level.
- These are type of saturation aquifer in which G.W.T. marks the upper limit of saturation.
- Water in these aquifer is subjected to pressure not greater than hydrostatic pressure.

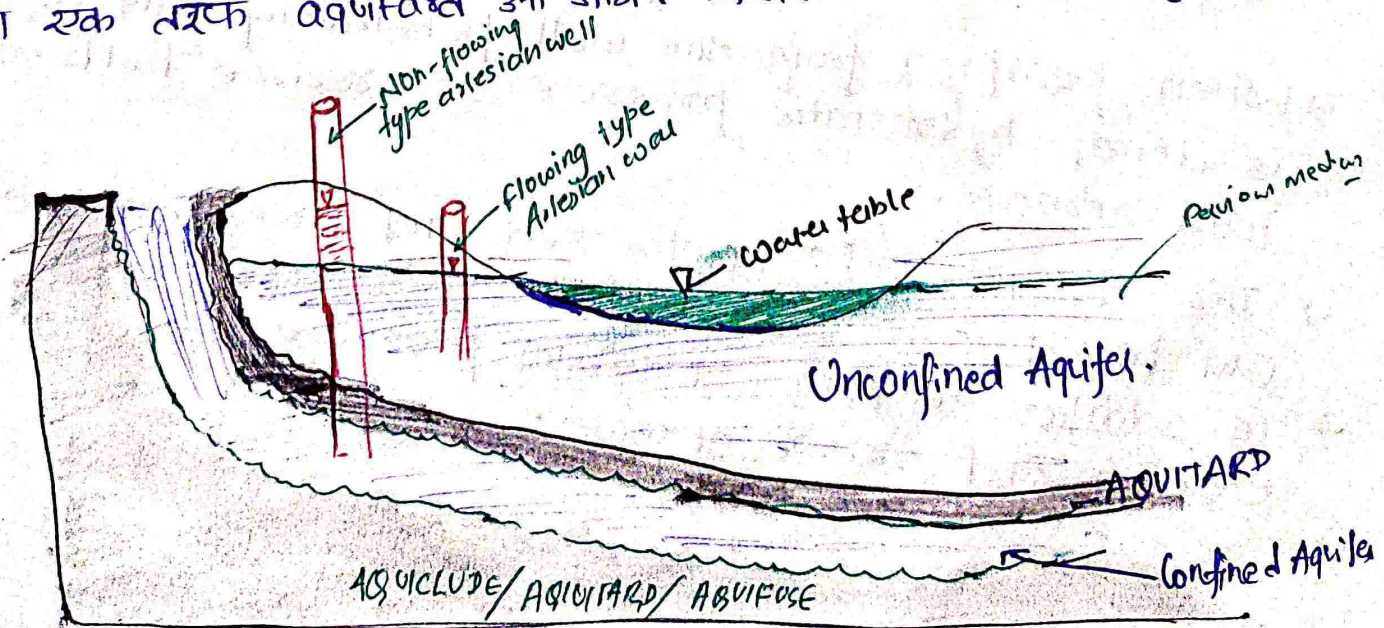
(ii) Confined Artesian Aquifer:

- Confining pressure.
- These are the type of aquifer which are formed when a pervious medium is found to exist in between two impermeable layer.
- Water in these aquifer are subjected to pressure greater than hydrostatic pressure.

(iii) Perched aquifer.

Sometimes an impermeable layer is found to exist in a pervious medium, above the G.W.T. carrying the saturated soil mass that can also be used as source of water and is termed as perched aquifer.

Semi-confined:- Confined aquifer. के दोनों तरफ ~~aquifer~~ या एक तरफ aquitard आ जाये. जिससे water leak हो जाता है।



Storativity / specific storage / co-efficient of storage (S)

~~S~~ $S = \frac{V_w}{V} \text{ (by any means)}$

$$S = \frac{\text{Volume of water drained from medium (By any means)}}{\text{Volume of medium drained (SA} \times \text{drop in water table)}} \quad \text{Pore water head}$$

$$\gamma_s = \frac{V_w}{V} \quad \text{or} \quad \frac{V_w}{V}$$

(i) For unconfined aquifer

$$\gamma_s = S$$

(ii) For confined aquifer.

$$\gamma_s \neq S$$

→ In case of confined aquifer water is drained because of = compression of aquifer + Expansion of water.

→ In confined aquifer, the hydrostatic within the partially supports the overburden, while remaining is supported by the soil structure of aquifer.

→ When pumping from the well penetrating this aquifer is done, hydrostatic pressure is reduced but aquifer load increases.

→ The aquifer gets compressed and forces some water out from it.

→ In addition lowering of water pressure cause small expansion and release of water.

Hence in this case storage coefficient is defined as volume of water release from the aquifer from full aquifer height and unit cross-sectional area, when piezometric surface decline by unity.

Storage of coefficient for confined aquifer:-

$$S = \gamma_w H (\alpha + \eta \beta)$$

$$= \gamma_w H \left\{ \frac{1}{E_s} + \eta \frac{1}{E_w} \right\}$$

γ_w = unit weight of water

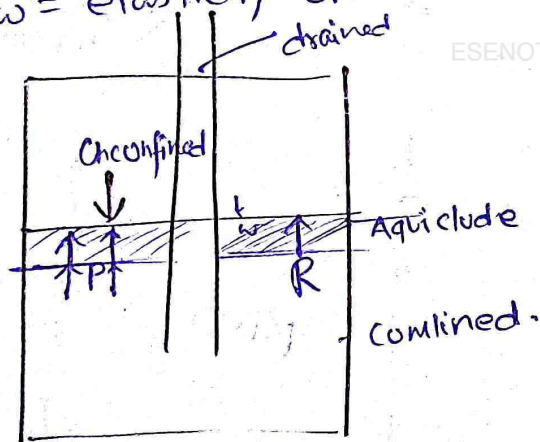
H = confined aquifer thickness m.

α = compressibility of the aquifer = $\frac{1}{E_s}$

E_s = Elasticity of aquifer.

β = compressibility of water = $\frac{1}{E_w}$

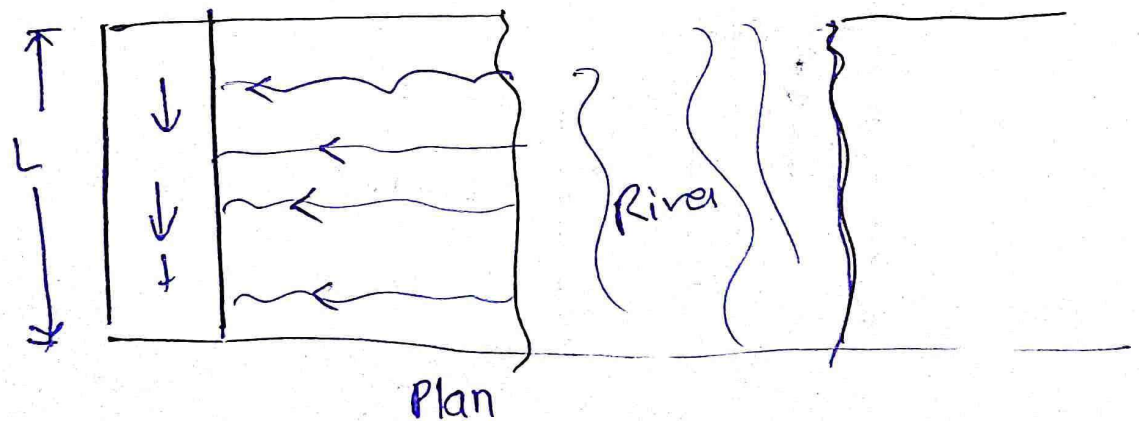
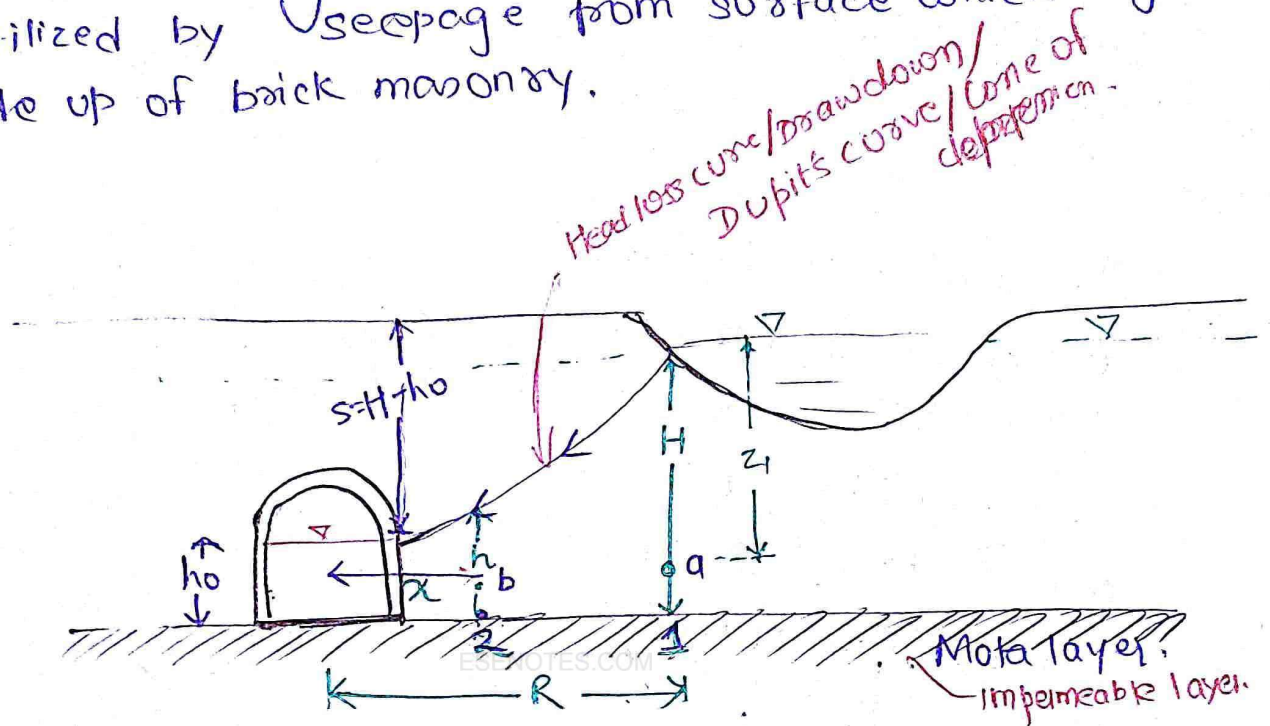
E_w = elasticity of water.



Different forms of extraction of Ground Water:

1) Infiltration Gallery

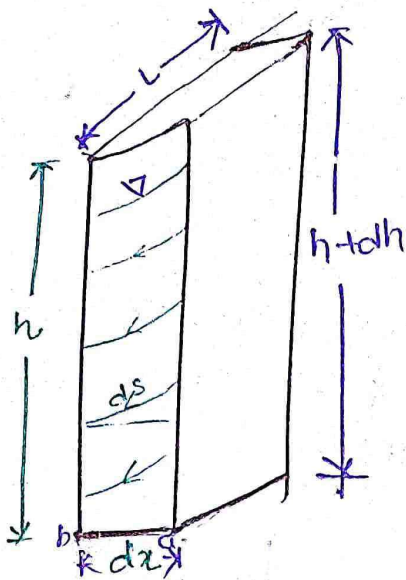
Infiltration galleries are generally horizontal tunnel constructed at shallow depth of 3-6m along the bank of the river in water bearing structure through which the water is utilized by seepage from surface which is generally made up of brick masonry.



H = Height of the water in the river/
or high of original water table.

h_0 = height of water in ~~the~~ ~~river~~ gallery

R = radius of influence $\approx 300\text{m}$.



$$q_x = q_x \times V_x$$

$$= (h \cdot L) \cdot K i_x$$

$$q_x = h L \cdot K \frac{dh}{dx}$$

$ds > dx$
if θ is small, $\sin \theta \approx \tan \theta$

$$ds = dx$$

$$\therefore q_x = h L K \left(\frac{dh}{dx} \right)$$

Assume $dh = ds$
 $\sin \theta = \frac{dh}{ds}$, $\tan \theta = \frac{dh}{dx}$

$\tan \theta = \frac{1}{300}$
 $\theta = 0.19^\circ$

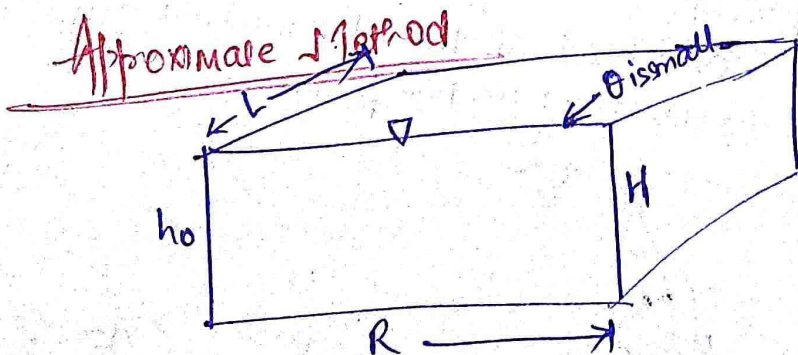
$$Q = \int q_x = \int h L K \frac{dh}{dx}$$

~~$Q = \int_{x=0}^R q_x dx = \int_{h=h_0}^H h L K dh$~~

$$Q = \int_{h=h_0}^H h L K dh$$

$$Q[x]_0^R = L K \left[\frac{h^2}{2} \right]_0^H$$

$$Q = \frac{K L (H^2 - h_0^2)}{2R}$$



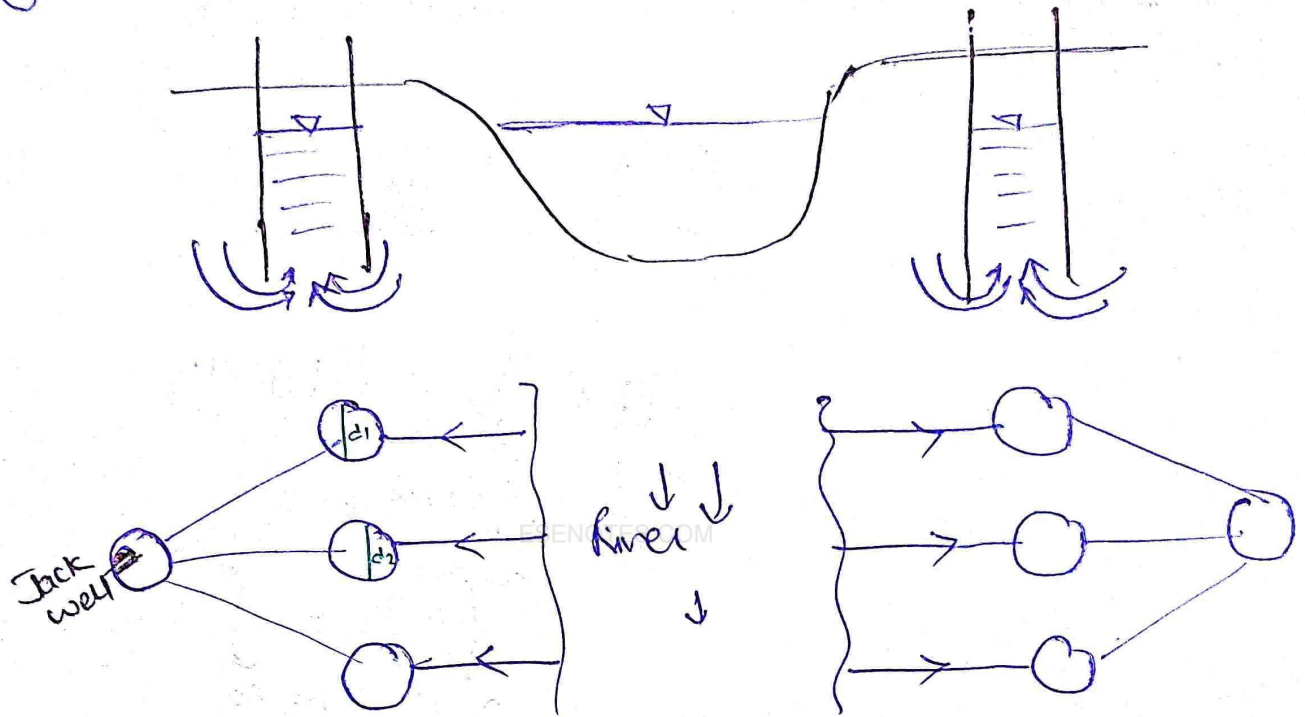
$$Q = A_{avg} \times V_{avg}^{ki}$$

$$= \left(\frac{H L + h_0 L}{2} \right) \frac{K (H - h_0)}{R} = \frac{K L (H^2 - h_0^2)}{2R}$$

② Infiltration well.

Infiltration wells are shallow discontinuous constructed along the bank of the river through which the water is utilised by seepage from bottom.

→ Several such infiltration wells are connected through a common well known as Jack well from where water being pumped into the treatment plant.

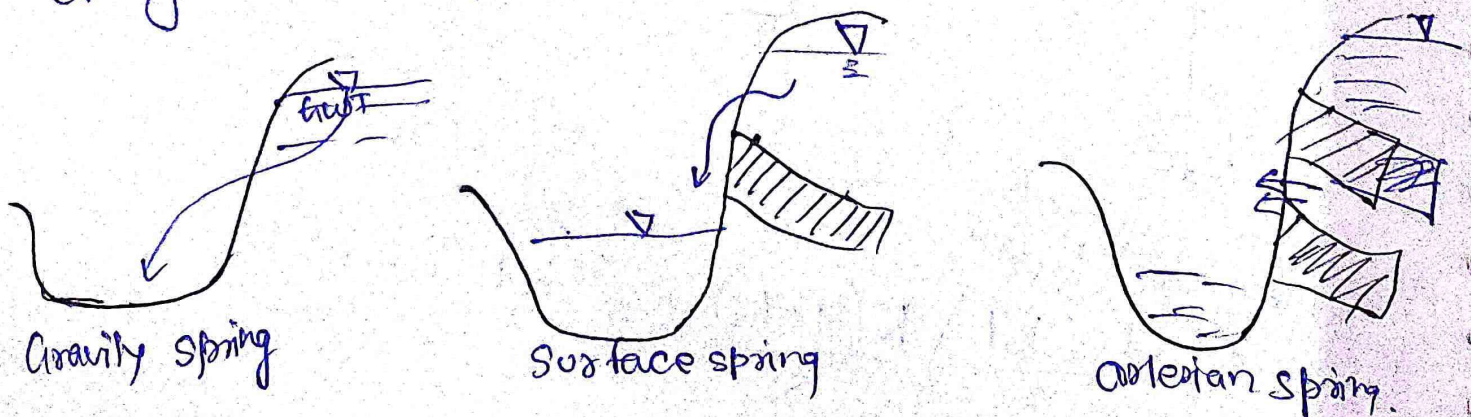


$$L = (d_1 + d_2 + d_3) \dots d_n$$

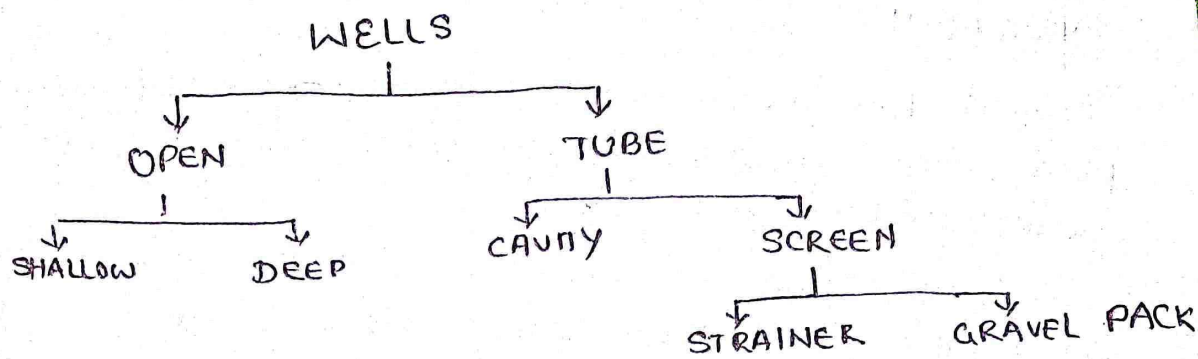
3) Springs.

Natural out of flow of ground water ~~at~~ over the ground surface is termed as spring.

→ only artesian spring are considered as the potential source of ground water { since water flow under pressure }.



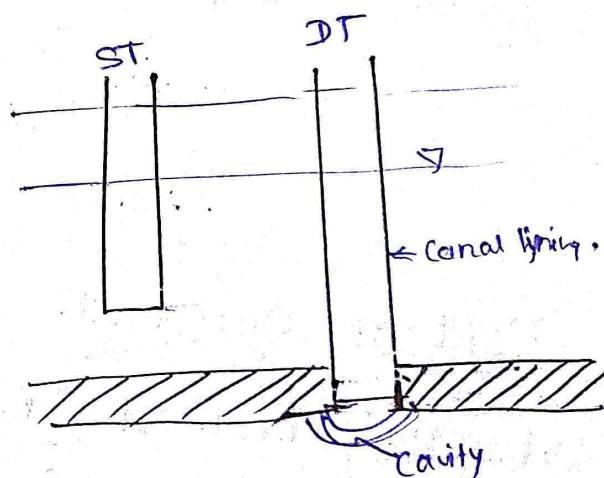
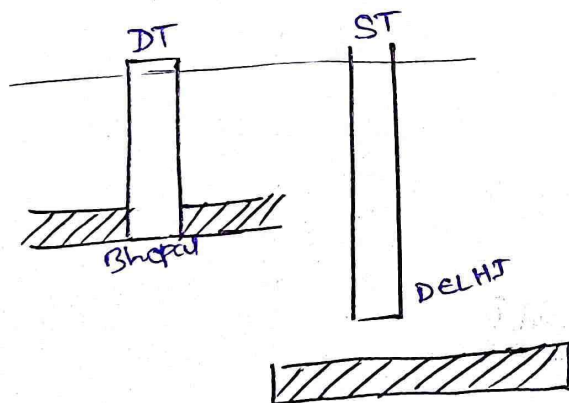
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	OW (2.5m)	TW (20cm)
Dia -	↑	↓
depth -	↓	↑
Discharge -	↓↓	↑↑↑

SHALLOW WELL :- are those which derive water from upper water bearing strata.

DEEP WELL :- derive water from lower water bearing strata.



	ST	DT
Quality	Poor	better.
Quantity	less	More.

Open Well

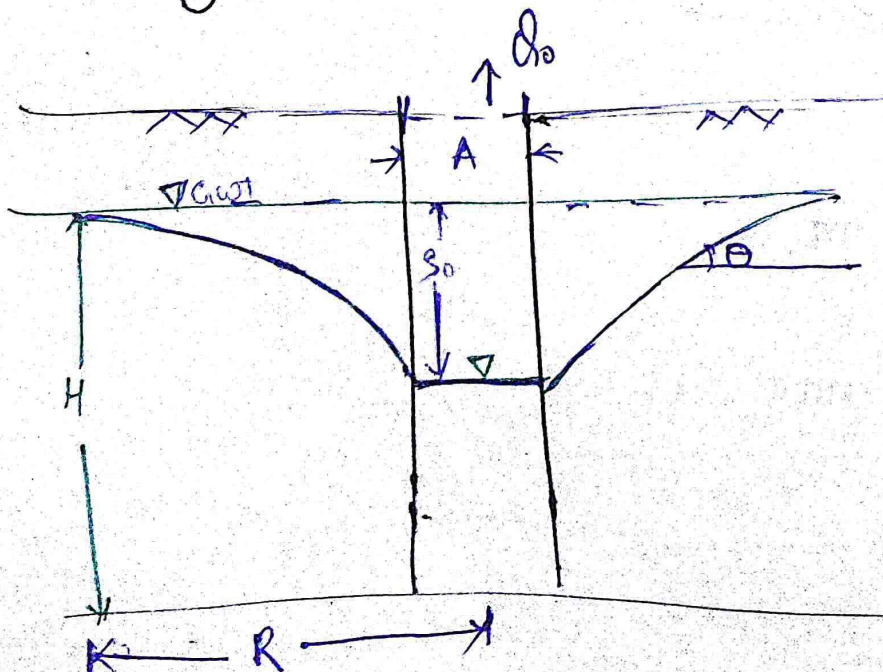
Shallow type open wells are those which utilize water from upper water bearing strata. that is liable to be contaminated to higher extent.

- Discharge from the shallow well is also very limited as if the discharge through those wells is increased critical velocity & settling velocity of medium particle may also lead to the removal of medium particle along with the flowing water and if this process continuous it may also lead to the sinking of the well into the medium.
- However this process is not observed in deep type wells as they derive their support from the lower impermeable layer but cavity is formed at the bottom of these wells due to the removal of the medium particle along with following water.

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Yield of Open well.

↳ Pumping method {field test}



$[V_0, t_0, S_0]$
 Volume time depression.

$v = \text{velocity}$

$$S_0 = H - h_0$$

$$Q_0 = \frac{V_0}{t_0} \quad \text{--- (i)}$$

$$Q_0 = A v$$

$$Q_0 = A K i$$

$$= A K \frac{S_0}{R}$$

$$Q_0 = \frac{K}{R} A S_0$$

$$\boxed{\frac{K}{R} = C \cong \text{constant}}$$

$$Q_0 = C A S_0$$

$$\boxed{C = \frac{Q_0}{A S_0}}$$

$C = \text{Specific capacity of open well (m}^3/\text{m}^2/\text{m/sec or sec}^{-1})$

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

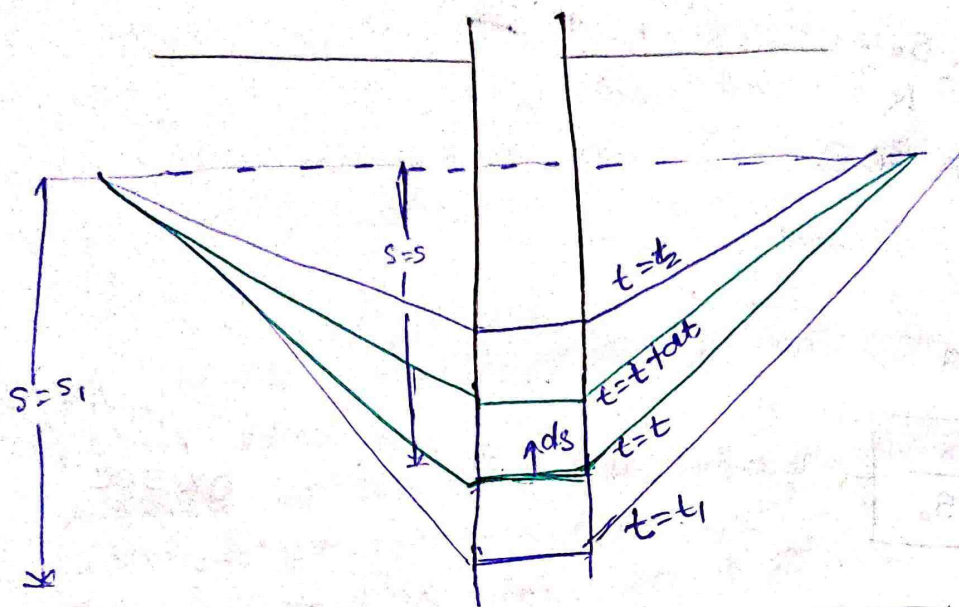
$$Q = \left(\frac{Q_0}{A S_0} \right) A S$$

Drawback: Not use for unsteady state.

$\rightarrow$ Discharge constant

(ii) RECUPERATION TEST

As it is difficult to maintain constant level (steady state) of water in well recuperation test is being performed in order to find the specific capacity of the well in which level of water recuperated in the well well in a given time is noted in order to find its yield.



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Volume of water recuperated in well in time dt .

$$dv = -Q dt = +A ds \quad \begin{matrix} (+ \text{ increasing}) \\ (- \text{ decreasing}) \end{matrix}$$

$$-CAs dt = A ds$$

$$C dt = -\frac{ds}{s}$$

$$C \int_{t=t_1}^{t=t_2} dt = - \int_{s=s_1}^{s=s_2} \frac{ds}{s}$$

$$C [t]_{t_1}^{t_2} = \left| \ln s \right|_{s_2}^{s_1}$$

$$C (t_2 - t_1) = \ln(s_1/s_2)$$

$$C = \frac{\ln(s_1/s_2)}{t} = \frac{2.303 \log_{10}(s_1/s_2)}{t} \quad (m^3/m^2/m/sec)$$

As change in s

I	II
10	10
8	12
$\frac{10}{8}$	$\frac{12}{10}$

let $t_2 - t_1 = t$

$$1 \text{ liter} = 10^{-3} \text{ m}^3$$

$$Q' = C A S'$$

$$= \left(\frac{2.303 \log_{10} (s_1/s_2)}{t} \right) A S'$$

Q During a recuperation test water is pumped upto 3m and it recuperated by 1.1m in 90 minutes. find the size of well to yield 10 liters/second under depression head of 2.5m.

$$Q' = 10 \text{ liters/second}$$

$$= 10 \times 10^{-4} / \text{s} \text{ m}^3/\text{s}$$

$$s_1 = 3 \text{ m}$$

$$s_2 - s_1 = 1.1$$

$$s = 3 - 1.1 = 1.9$$

$$t = 90 \text{ min}$$

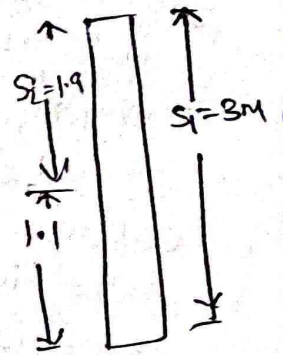
$$Q' = \left(\frac{2.303 \log_{10} (s_1/s_2)}{t} \right) A S'$$

$$10 \times 10^{-3} = \frac{2.303 \log_{10} \left(\frac{3}{3-1.1} \right)}{90 \times 60} \times \frac{\pi}{4} d^2 \times 2.5$$

$$d^2 = 60.20$$

$$d = 7.75$$

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Q Water is pumped from a fine sand aquifer region at a rate of 12 lit/s find size of the well to give the required yield. at the draw down of 2m take $\frac{C'}{A} = 0.06 \text{ m}^3/\text{hr} \cdot \text{m}^2 \cdot \text{m}$.

$$Q = 12 \text{ lit/s} = 12 \times 10^{-3} \text{ m}^3/\text{s}$$

Specific capacity
= C

$$Q' = C A S'$$

$$12 \times 10^{-3} = \frac{0.06}{60 \times 60} \times \frac{\pi d^2}{4} \times 2 \text{ m}$$

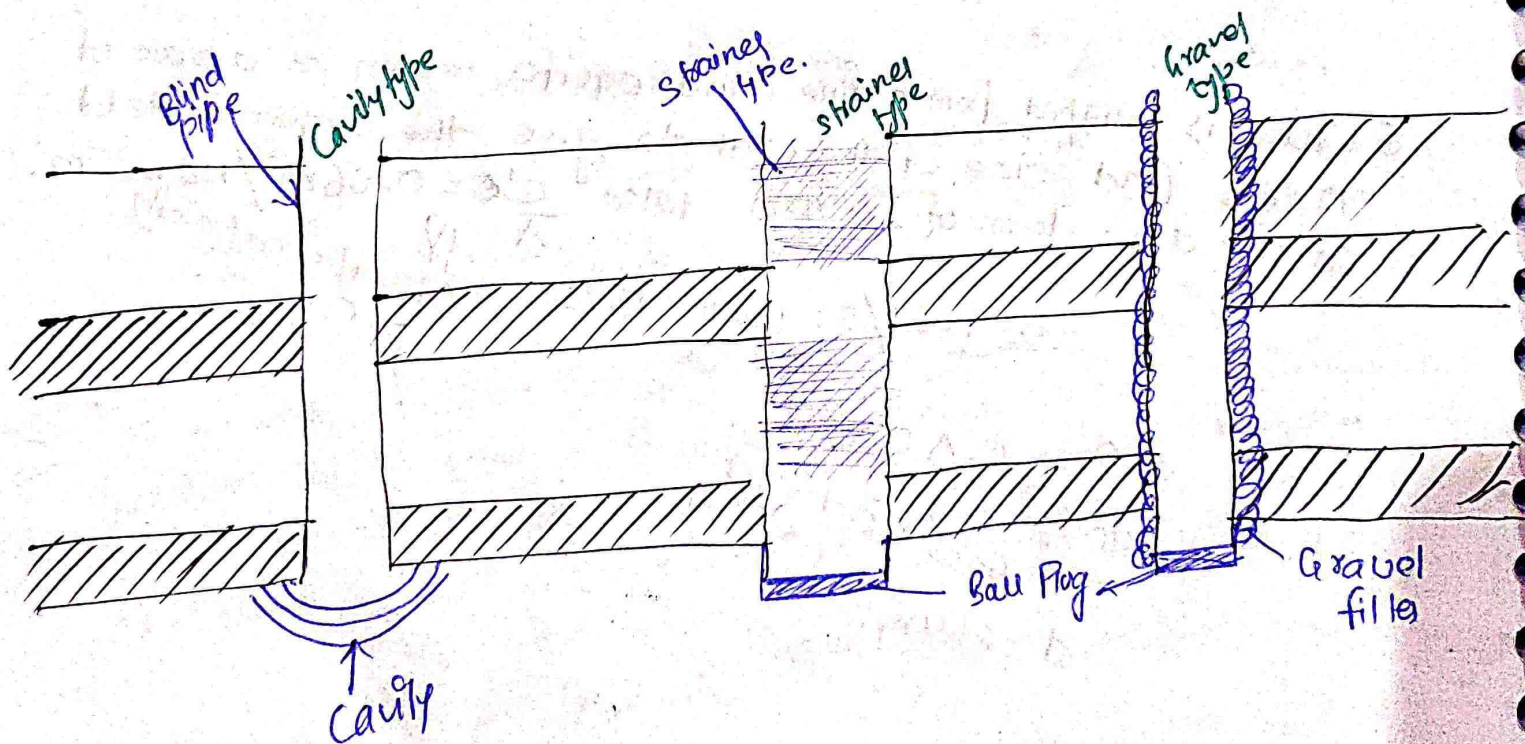
$$d = 21.4 \text{ m}$$

TUBE WELL

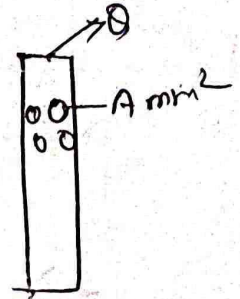
Discharge from the open well is limited in the range of 5-10 lit/sec whenever discharge from tubewell extends upto 200-220 lit/sec as they derive their water from depth or 70-300 m.

- Gravity type tube well are those tube well which utilize the water from the bottom.
- Whereas screen type tubewell are those which utilise water from the surface.
- In order to prevent the entry of the medium particles into the well along with the flowing water screens are used & gravel pack type tube well.

Capacity of well will reduce if medium particles will inters into the screen type well not in cavity type since there is no option of sedimentary of particles in cavity type.



Q what is the strainer length required for a tube well giving a discharge of 8 l/s ? Assume permissible entrance velocity of ~~20~~ 2 m/s . It is desired to have the strainer of slot $20 \text{ mm} \times 0.2 \text{ mm}$ with no. of slots per cm of the strainer as 100.



$$Q = A_s \times V$$

$$A_s = \frac{Q}{V} = \frac{8 \times 10^{-3}}{2 \times 10^{-2}}$$

$$A_s = 0.4 \text{ m}^2$$

$$A_s = n A_{s1}$$

$$0.4 = L \times 10^4 \times (20 \times 0.2) \times 10^{-6}$$

$$L = 10 \text{ m}$$

$$1 \text{ cm} = 100$$

$$1 \text{ m} = 100 \times 100$$

Yield of Tube Well

① DUPUIT'S theory

② Thiem's theory.

! Unconfined Aquifer:-

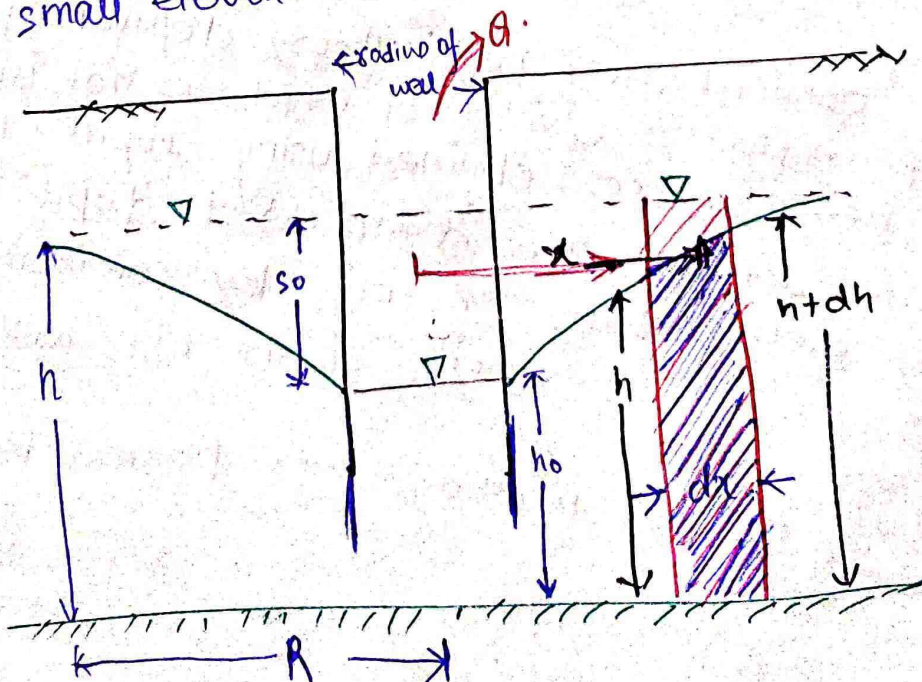
Dupuit's theory:-

Assumptions:-

(i) Medium is homogenous, isotropic, infinite.

(ii) Flow is radial, horizontal and laminar in the vicinity of the well.

(iii) for small elevation [water surface] $\sin \theta = \tan \theta$.



$$q_x = q_x V_x \\ = (2\pi x h) K \frac{dh}{dx}$$

$$Q = \int q_x = \int 2\pi x h \frac{dh}{dx}$$

$$\Rightarrow Q \int_{x=r_w}^{x=R} \frac{dx}{x} = \int_{h=h_0}^{h=H} 2\pi h dh$$

$$\Rightarrow Q \left[\ln x \right]_{r_w}^R = 2\pi K \left[\frac{h^2}{2} \right]_{h_0}^H$$

$$\Rightarrow Q = \frac{2\pi K \left(\frac{H^2 - h_0^2}{2} \right)}{2.303 \log_{10} (R/r_w)}$$

$$\Rightarrow Q = \frac{\pi K (H^2 - h_0^2)}{2.303 \log_{10} (R/r_w)}$$

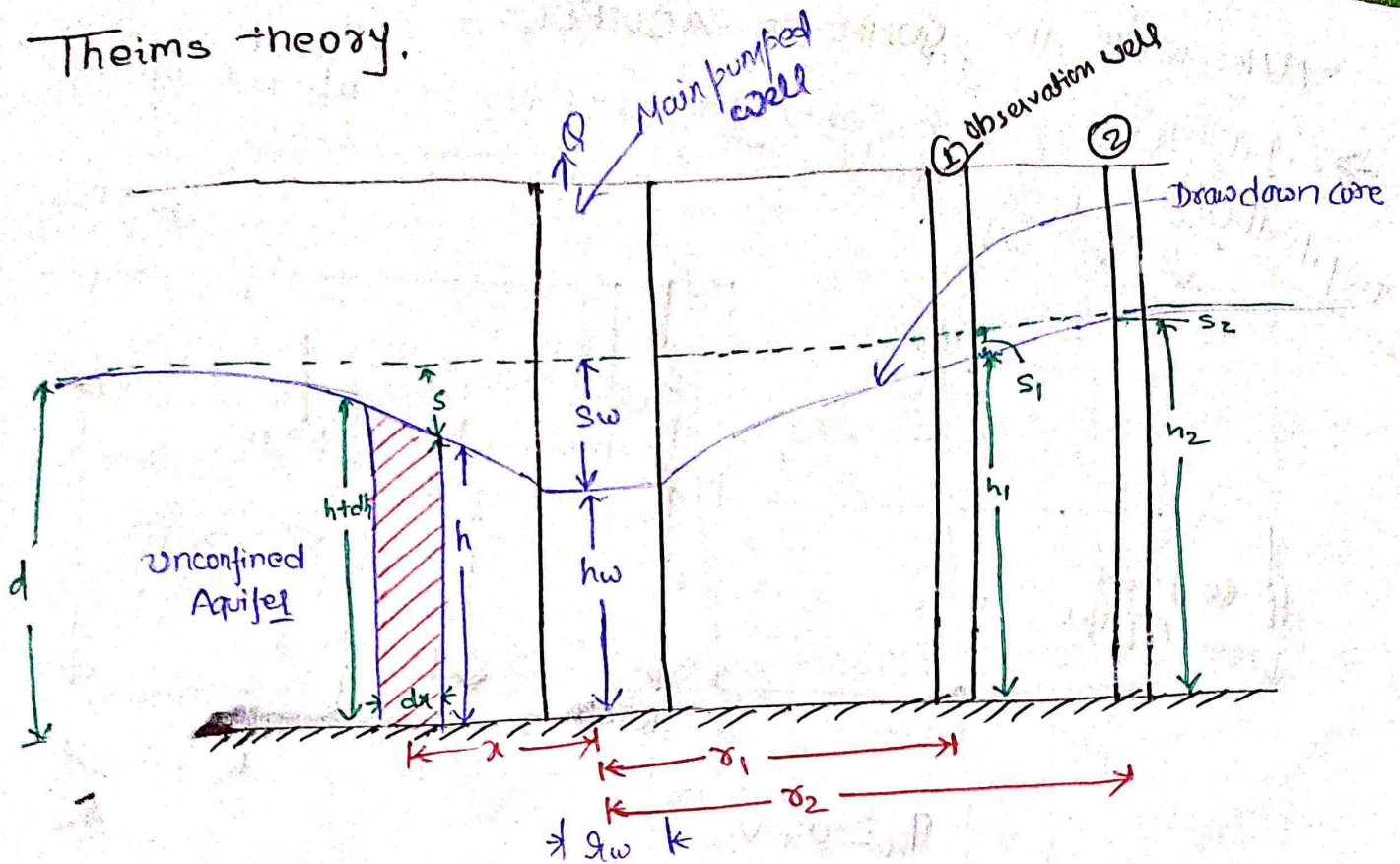
$$R = 150 - 300 \text{ m.}$$

$$\text{Schidout } eq^n = \frac{R}{(m)} = 3000 \frac{S \sqrt{K}}{(m)} \text{ m/sec.}$$

$S = \text{draw down due to correction to constant discharge.}$

Note:- Discharge obtained using Dupit's theory depend upon radius of influence value of which is not known precisely hence stresses obtained using Dupit's theory deviates from actual. This drawback of Dupit's theory was further eliminated by Thiem's by considering two observation well in the vicinity of the test well to the drawback. Corresponding a particle discharge being pumped from the test well.

Theims theory.



$$q_r = a_r \times v_r = (2\pi r h) K i_r$$

$$= (2\pi r h) K \cdot \frac{dh}{ds}$$

($\sin 0 = \tan 0$)
 $\frac{dh}{ds} = \frac{dr}{dr}$

$$\Rightarrow q_r = (2\pi r h) K \cdot \frac{dh}{dr}$$

$$\Rightarrow Q = \int q_r = \int 2\pi r h \frac{dh}{dr} \frac{dr}{h}$$

$$\Rightarrow \int_{r_w}^{r_1} \frac{dr}{r} = 2\pi K \int_{h_w}^{h_1} h dh$$

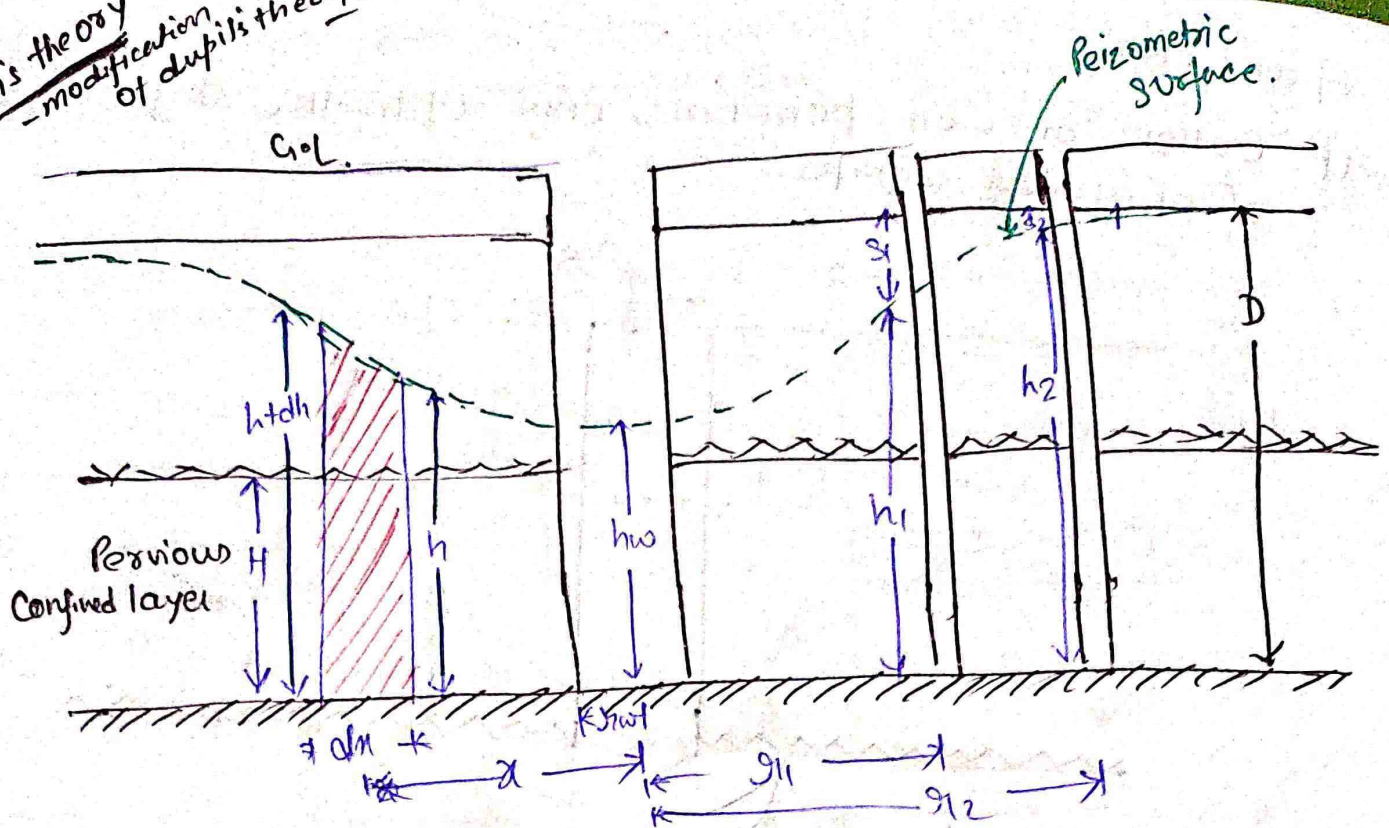
$$\Rightarrow Q \ln r \Big|_{r_w}^{r_1} = 2\pi K \left[\frac{h^2}{2} \right]_{h_w}^{h_1}$$

$$\Rightarrow Q = \frac{\pi K [h_1^2 - h_w^2]}{2.303 \log_{10} \left(\frac{r_1}{r_w} \right)}$$

$$\text{or, } Q = \frac{\pi K [h_2^2 - h_1^2]}{2.303 \log_{10} \left(\frac{r_2}{r_1} \right)}$$

$$\text{or } Q = \frac{\pi K [h_2^2 - h_1^2]}{2.303 \log_{10} \left(\frac{r_2}{r_1} \right)}$$

Thiem's theory
- modification
of Dupuit's theory.



$$q_r = q_r V_r$$

$$= (2\pi r H) K i_r$$

$$= (2\pi r H) K \frac{dh}{dr}$$

$$\Rightarrow Q = \int q_r = \int 2\pi r H K \frac{dh}{dr}$$

$$\Rightarrow Q \int \frac{dr}{r} = \int 2\pi H K dh$$

$r = r_w$ $h = h_w$

$$\Rightarrow Q \left[\ln r \right]_{r_w}^{r_1} = 2\pi H K [h]_{h_w}^{h_1}$$

$$\Rightarrow Q = \frac{2\pi H K [h_1 - h_w]}{2.303 \log_{10} \left(\frac{r_1}{r_w} \right)}$$

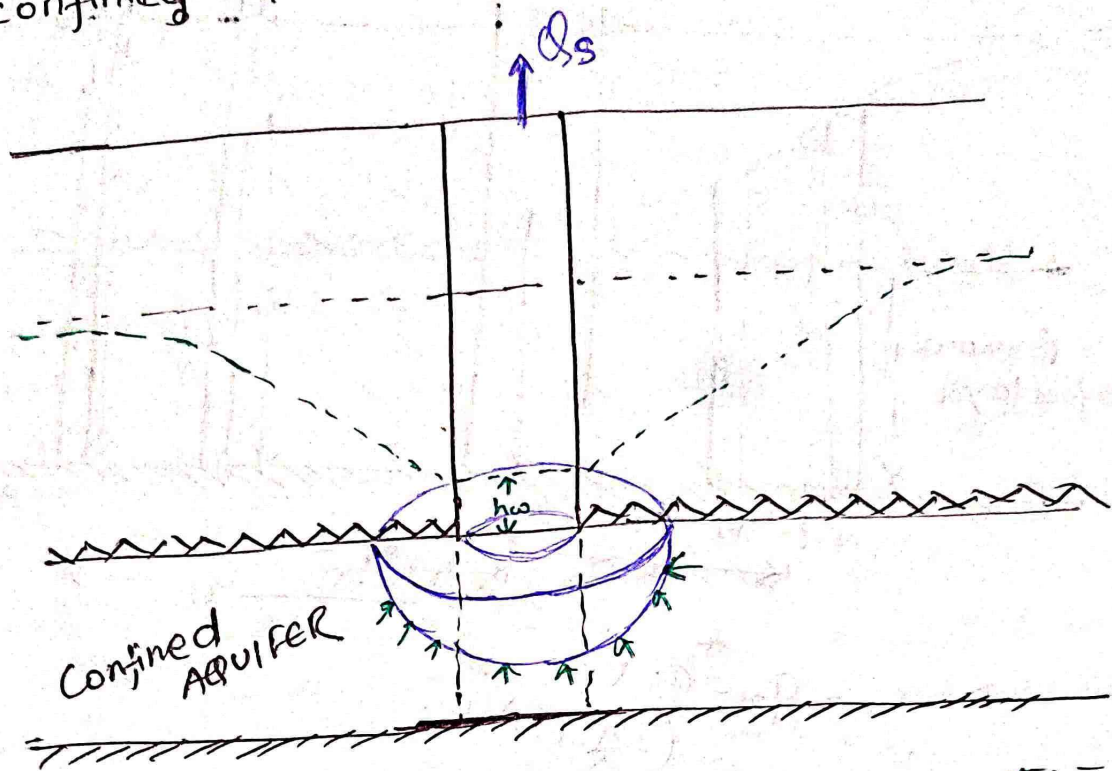
or,

$$Q = \frac{2\pi H K [h_2 - h_w]}{2.303 \log_{10} \left(\frac{r_2}{r_w} \right)}$$

$$Q = \frac{2\pi H K [h_2 - h_1]}{2.303 \log_{10} \left(\frac{r_2}{r_1} \right)}$$

Special case.

21f Bottom of well penetrates only upto the top surface of confined aquifer.



$$Q_s = A \cdot V$$

$$= (2\pi r_w^2) K i$$

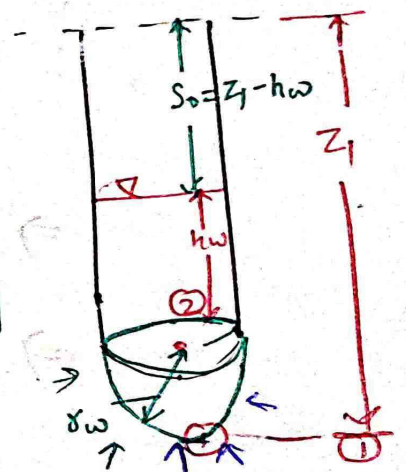
$$= 2\pi r_w^2 \cdot K \cdot \frac{S_0}{r_w}$$

$$Q = 2\pi r_w K S_0$$

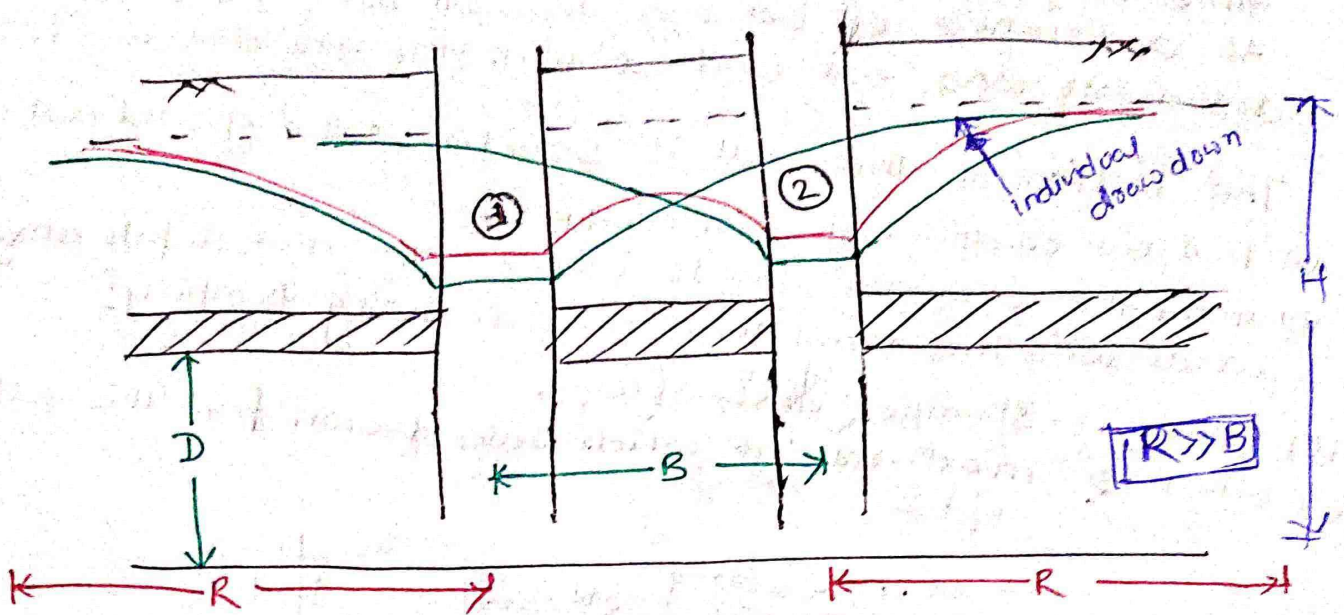
$$Q_R = \frac{2\pi K H (D - h_w)}{2.303 \log_{10} (R/r_w)}$$

$$Q_R = \frac{2\pi K H S_0}{2.303 \log_{10} (R/r_w)} \quad (i)$$

$$Q_s = \left(\frac{1}{20} \text{ to } \frac{1}{30} \right) Q_R$$



If two wells are situated nearby to each other such that their draw down interfere within the radius of influence.



$$Q_1 = Q_2 = Q_T = \frac{2\pi KD(H-h_w)}{2.303 \log_{10}(R/r_w)}$$

$$Q_1' = Q_2' = \frac{2\pi KD(H-h_w)}{2.303 \log_{10}\left(\frac{R^2}{2r_w B}\right)}$$

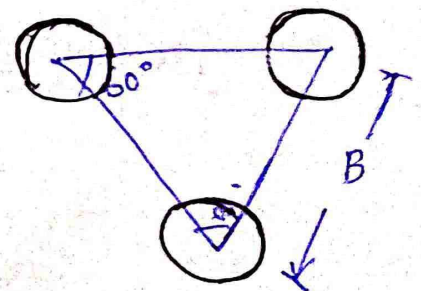
$$< Q_1 = Q_2$$

$$Q_T' = Q_1' + Q_2' > Q_T$$

$$\textcircled{2} \quad Q_1'' = Q_2'' = Q_3'' = \frac{2\pi KD(H-h_w)}{2.303 \log_{10}\left(\frac{R^3}{2r_w B^2}\right)}$$

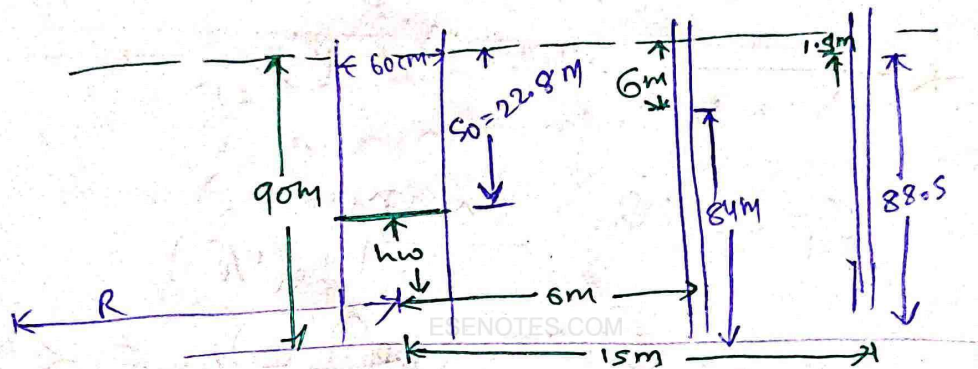
$$Q_1'' = Q_2'' = Q_3'' < Q_1' = Q_2' < Q_1 = Q_2$$

$$Q_T'' = Q_1'' + Q_2'' + Q_3'' > Q_T' > Q_T$$



Q A 60cm dia well is pumped at a rate of 1360 l/m. Measurement in a nearby test well are made at the time as follow:-
At a distance of 6m from the well being pumped, the drawdown was 6m and at 15m the drawdown was 1.5 m.
The bottom of the well is 90m below the ground water table.

- find out coefficient of permeability.
- if all the observed points where one the dupits case, what was the draw down in well, during pumping?
- What is specific ^{capacity} gravity of well.
- What is maxm rate at which water drawn from this well.



$$Q = 1360 \text{ l/min} \\ = \frac{1360 \times 10^{-3}}{60} \text{ m}^3/\text{s} \\ = 0.0226 \text{ m}^3/\text{s}$$

$$Q = \frac{\pi K [h_2^2 - h_1^2]}{2.303 \log_{10} \left(\frac{r_2}{r_1} \right)}$$

$$\frac{1360 \times 10^{-3}}{60} = \frac{\pi K [88.5^2 - 84^2]}{2.303 \log_{10} \left(\frac{15}{6} \right)}$$

$$K = 8.51 \times 10^{-6} \text{ m/sec.}$$

$$r_w = \frac{60}{2} \\ = 30 \text{ cm} \\ = 0.3$$

$$Q = \frac{\pi K [h_1^2 - h_w^2]}{2.303 \log_{10} \left[\frac{r_1}{r_w} \right]}$$

$$\frac{1360 \times 10^{-3}}{60} = \frac{\pi \times 8.51 \times 10^{-6} \times [84^2 - h_w^2]}{2.303 \times \log_{10} \left[\frac{6}{0.3} \right]}$$

$$1540.326 = 84^2 - h_w^2$$

$$h_w = 67.198$$

$$\text{drawdown} = 22.80$$

10

for, $Q = Q_{sc} \Rightarrow S_0 = 1 \text{ m}$

$h_w = H - S_0 = 90 - 1 = 89 \text{ m}$

By Dupuit's theory.

$$Q = \frac{\pi K (H^2 - h_w^2)}{2.303 \log_{10} (R/r_w)}$$

$$\frac{1360 \times 10^{-3}}{60} = \frac{3.14 \times 8.51 \times 10^{-6} [90^2 - 67.22^2]}{2.303 \log_{10} (R/0.3)}$$

$R = 20.5 \text{ m}$

$$Q_{sc} = \frac{\pi K (H^2 - h_w^2)}{2.303 \log_{10} (R/r_w)}$$

$$= \frac{\pi \times 8.51 \times 10^{-6} (90^2 - 89^2)}{2.303 \log_{10} (20.5/0.3)}$$

$Q_{sc} = 1.13 \times 10^{-3} \text{ m}^3/\text{sec}$

or $Q_{sc} = \frac{Q}{S} = \frac{\frac{1360 \times 10^{-3}}{60}}{22.78} = 9.9 \times 10^{-4} \text{ m}^3/\text{sec}$

Non-Equilibrium flow [Unsteady flow]

Theis theory! → Gives the solution of this in terms of well function. $w(u)$

Well function → it is a complex integral which is used to find drawdown in well pumping.

$$w(u) = -0.5772 - \ln u + u + \text{Higher } \xrightarrow{\text{neglected}} \text{ terms of } u$$

$$u = \frac{r^2 S}{4 T t}$$

S = storativity.

T = transmissibility.

t = time after which draw down is calculated in observation well.

$$s(t, r) = \frac{Q}{4 \pi T} \times \text{well function}$$

$$s(t, r) = \frac{Q}{4 \pi T} \times w(u)$$

As per Jacob's Modification,

$$w(u) = -0.5772 - \ln \left(\frac{r^2 S}{4 T t} \right)$$

$$w(u) = \ln \left(\frac{4 T t}{r^2 S} \right) - 0.5772$$

$$s(t, r) = \frac{Q}{4 \pi T} \left[\ln \left(\frac{4 T t}{r^2 S} \right) - 0.5772 \right]$$

Q Give the characteristic of different formation in which ground water exist. A fully penetrating artesian well is discharging at a rate of 25 l/s. The storage coefficient and transmissibility of the aquifer are 4.5×10^{-4} and $0.15 \text{ m}^2/\text{min}$ respectively. Find the drawdown at,

- (i) A radius of 5m distance after 2 hours pumping.
- (ii) A radius 180m distance after 1 day pumping.

Using the following approximation well function,
 $w(u) = -0.5772 - \ln(u) + U$

$$U = \frac{r^2 S}{4Tt}$$

① $r(5\text{m}, 2\text{hrs})$

$$U = \frac{5^2 \times 4.5 \times 10^{-4}}{4 \times 0.15 \times 2 \times 60} = 1.5625 \times 10^{-4}$$

$$\begin{aligned} w(u) &= -0.5772 - \ln(u) + U \\ &= -0.5772 - \ln(1.5625 \times 10^{-4}) + 1.5625 \times 10^{-4} \\ &= 8.187 \end{aligned}$$

$S(5\text{m}, 2\text{h})$

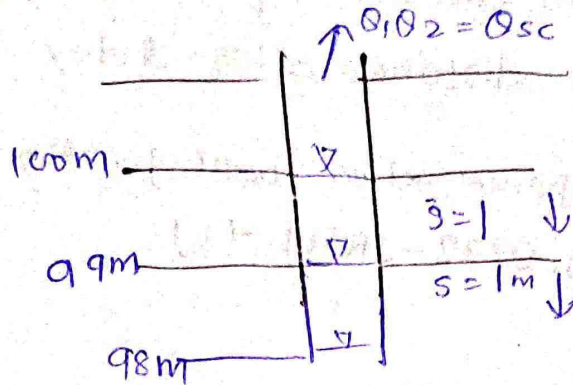
$$= \frac{Q}{4\pi T} w(u)$$

$$\begin{aligned} &= \frac{25 \times 10^{-3}}{4 \times \pi \times 0.15} \times 8.187 \\ &= 6.51 \text{ m} \end{aligned}$$

$$\textcircled{2} \quad r(180\text{m}, 24\text{hrs}) = 3.087$$

Specific Capacity of tube well.

- Rate of flow through the well per unit drawdown.
- It is calculated for initial 1st meter fall of height of water in well.
- And it is not same for all the unit drawdown.



Note:-

$$\begin{aligned}
 Q &= \frac{\pi K (H^2 - h_o^2)}{2.303 \log_{10} (R/r_w)} \\
 &= \frac{\pi K (H - h_o)(H + h_o)}{2.303 \log_{10} (R/r_w)} \\
 &= \frac{\pi K S_o (H + h_o)}{2.303 \log_{10} (R/r_w)}
 \end{aligned}$$

$$\boxed{S_o = C_1 Q} \quad \text{Aquifer loss.}$$

$$h_f = \frac{f l v^2}{2 g d} = \frac{f l Q^2}{2 g d A^2}$$

$$\boxed{S_w = C_2 Q^2} \quad \text{well loss.}$$

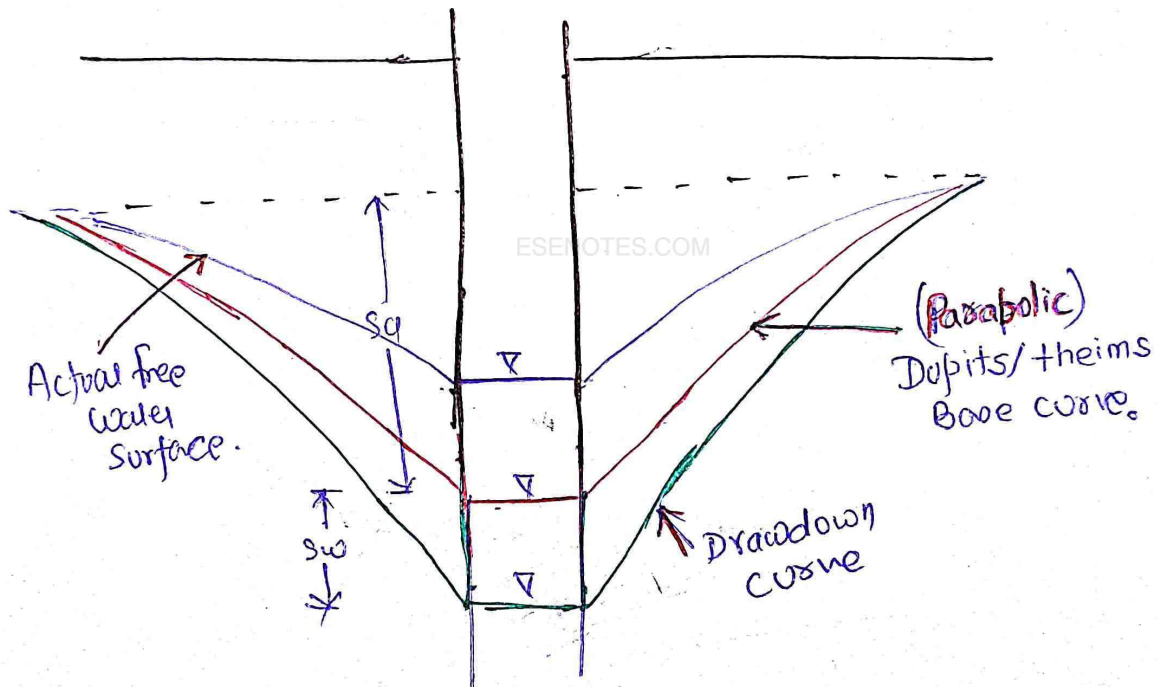
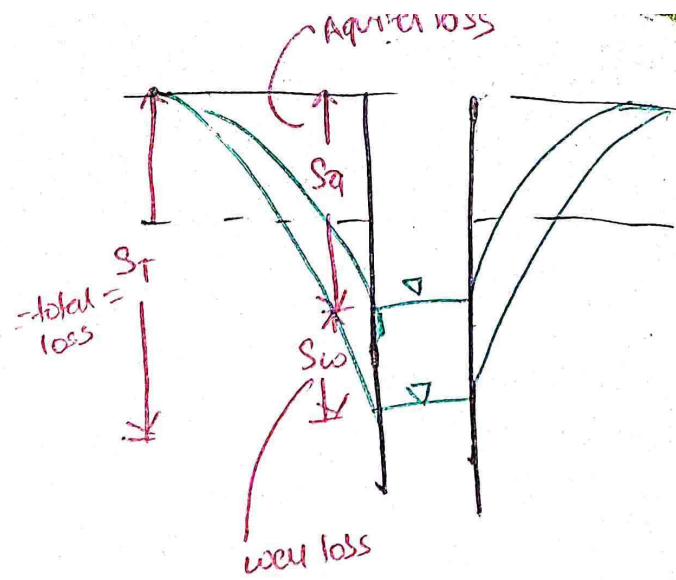
$$S_T = S_a + S_w = C_1 Q + C_2 Q^2$$

Specific capacity = $SC = \frac{Q}{S_T}$

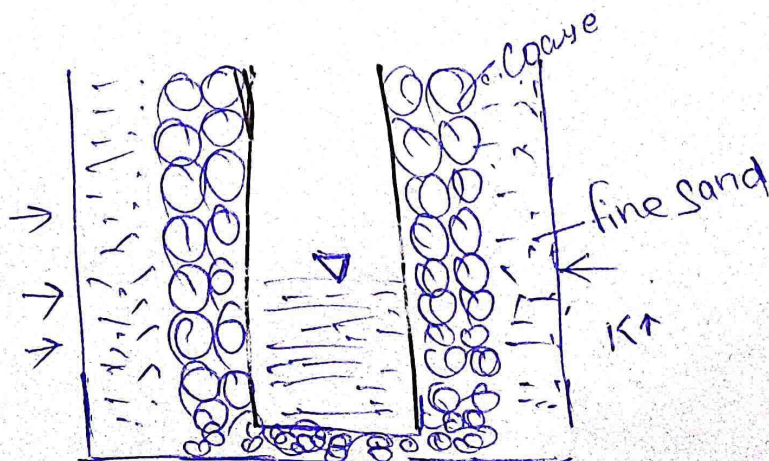
$$= \frac{Q}{C_1 Q + C_2 Q^2}$$

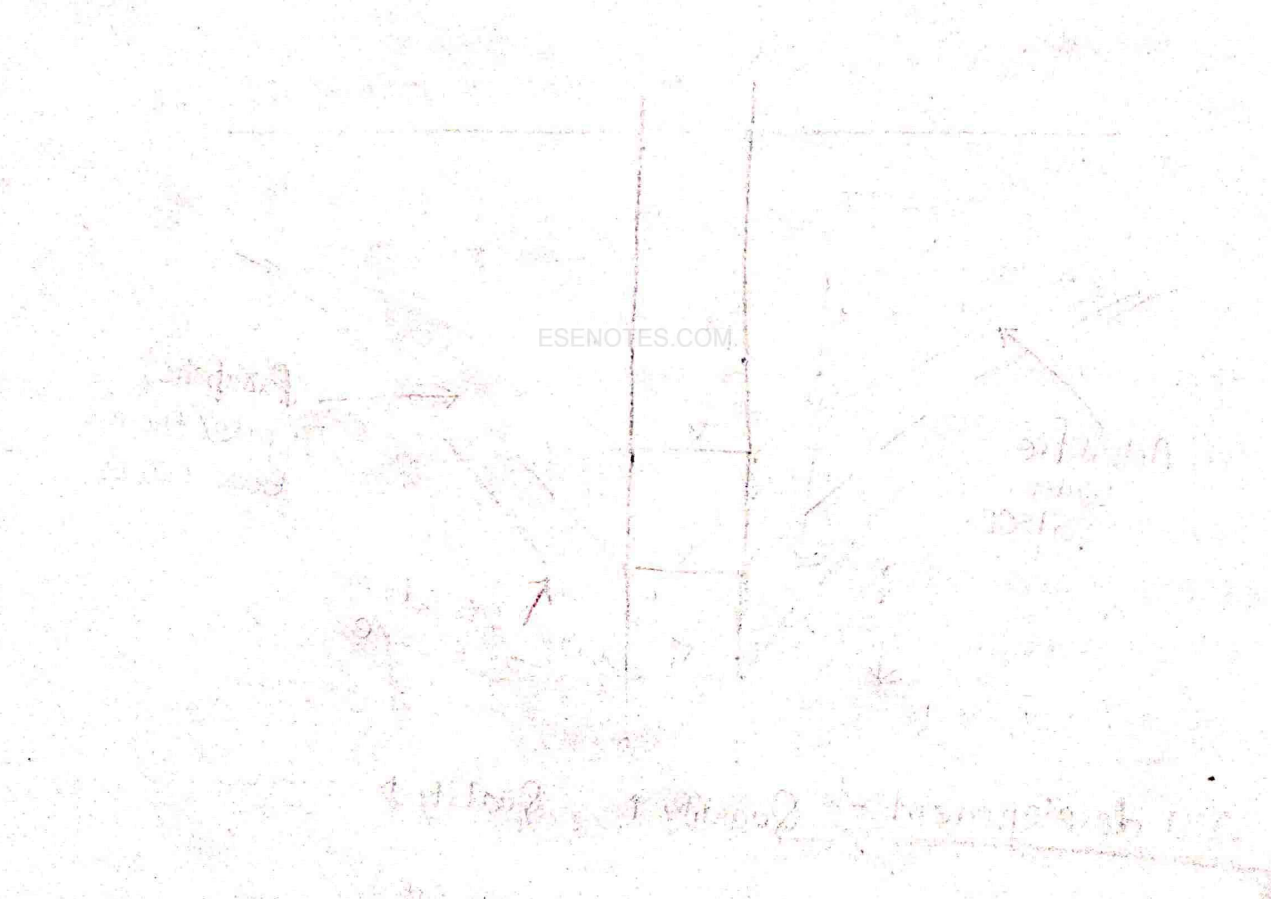
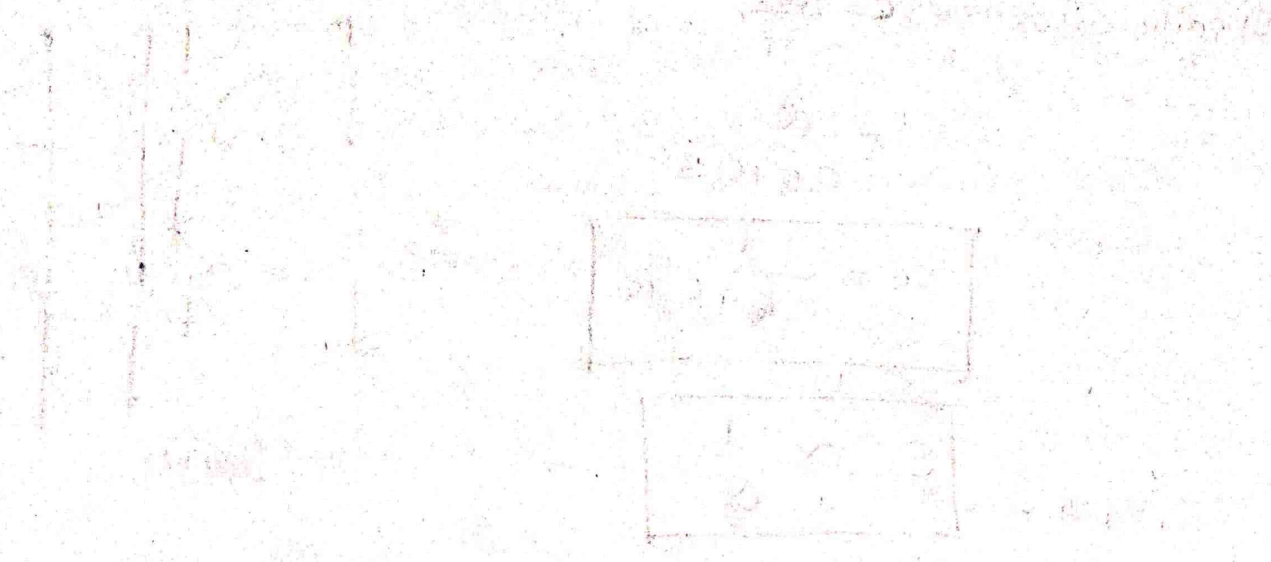
$$SC = \frac{1}{C_1 + C_2 Q}$$

$$SC \propto \frac{1}{Q}$$



well development:- Quantity $\uparrow$, Quality $\uparrow$





Quality of Water

Quality of water is found with the help of sudden indicators or parameter term as water quality parameter which are further classified into:-

- (i) Physical Quality Parameter.
- (ii) chemical Quality. Parameter.
- (iii) Biological Quality "

Physical Quality Parameter

These are parameter which indicate physical quality of water and can be sense by any our senses.

- (i) Source
- (ii) Impact
- (iii) Measurement
- (iv) Limits.

1) Suspended Solids.

Source: Suspended solid may come from organic or inorganic matter.

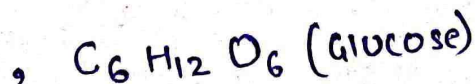
IS : silt, clay, sand, soil etc.

OS : plant, algae, plankton etc.

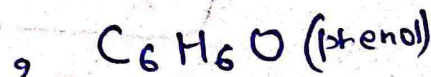
Carbon present
organic



$$a=1, b=4$$



$$a=6, b=12, c=6$$



$$a=6, b=6, c=1$$

Organic Matter.

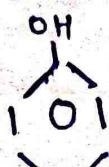
Microorganism
can break

Biodegradable



Aromatic Compound.

Non-biodegradable



Cross linked polymer.

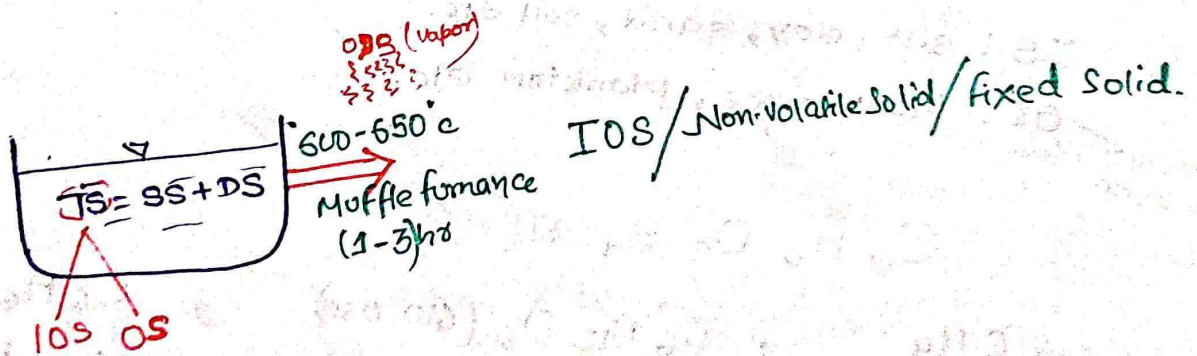
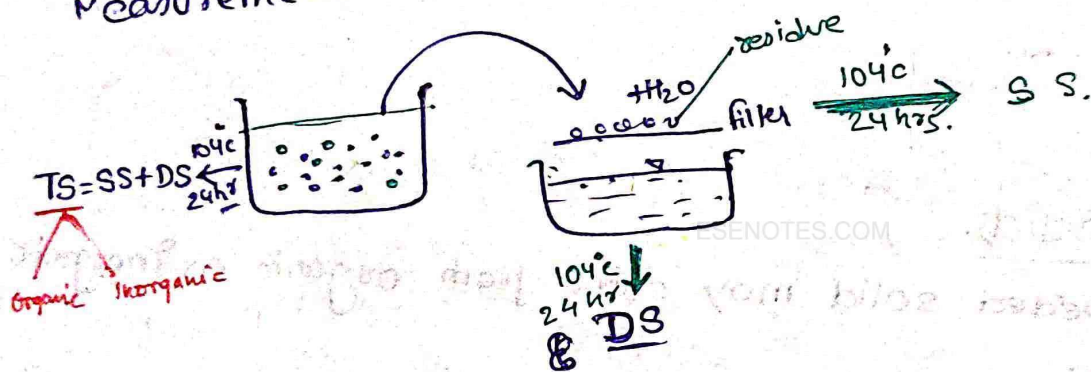
Impact:

- i) Suspended solid is biological in nature may promote growth of disease causing M/O.
- (ii) It makes the water aesthetically displeasing. (psych. effect)
- (iii) It leads to incrustation in pipes hence reduces its discharge carrying capacity. (deposition)

(iv) SS interfere with treatment of water (either increase cost or reduce its efficiency).

Measurement

Measurement of solid is done by Gravimetric Technique.



$$OS = TS - IOS$$

$$SS \xrightarrow[1-3 \text{ hrs}]{600 \text{ to } 650^{\circ}\text{C}} \text{LOSS}$$

$$OSS = SS - \text{LOSS}$$

$$ODS = OS - OSS$$

$$IODS = DS - ODS$$

EPA: Environment protection Agency

Acceptable

Cause for rejection limit

500 mg/lit

2000 mg/lit.

Limits of ~~suspended~~ ^{total dissolve} solid =

Q Following observation were made on raw water sample.

Chiradish

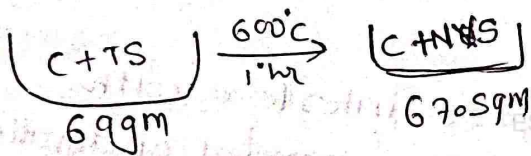
(i) A crucible was dried to a constant mass of 65gm.

(ii) 150 ml of raw water sample was placed in crucible and was dried to a constant mass of 69gm at 104°C.

(iii) Crucible and residue was placed in muffle furnace for 1 hour at 600°C and reduced to constant mass of 67.5gm. Find concentration of TS and volatile solid in mg/liter.

$$TS = \frac{(69 - 65) \times 1000}{\frac{150}{1000}} = 2666.6 \text{ mg/lit}$$

$$VS = \frac{(69 - 67.5) \times 1000}{\frac{150}{1000}} = 1000 \text{ mg/lit}$$



Q Find the concentration of TS, FDS, OSS in given sample of water.

- MgCO_3 - 20 - 10, DS
- Mg(OH)_2 - 30 - 10SS
- $\text{C}_6\text{H}_{12}\text{O}_6$ - 15 - ODS
- SiO_2 - 20 - 10SS
- NaCl - 35 - 10DS
- Algae - 50 - OSS
- $\text{C}_6\text{H}_6\text{O}$ - 5 → ODS

- Fe_2O_3 - 2 - 10SS
- CaCO_3 - 3 - 10SS
- Plankton - 5 - OSS
- MgSO_4 - 15 - 10DS
- CO_2 - 40 - 10DS

ppm = mg/lit
only for water

Total solid = 240ppm = 240mg/lit

FDS = $\text{MgCO}_3 + \text{NaCl} + \text{MgSO}_4 + \text{CO}_2$
 $= 20 + 35 + 15 + 40 = 110$

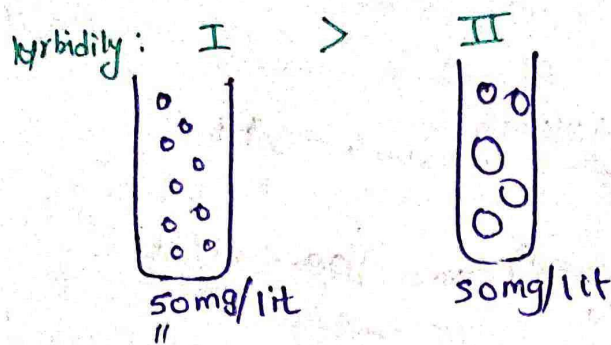
OSS = 10

Limits of

(2) Turbidity

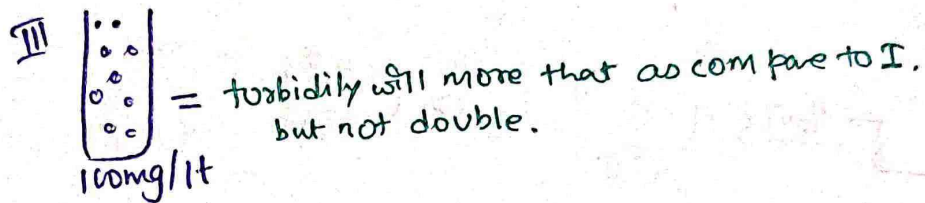
Turbidity is extended to which light is either scattered or obstructed due to presence of suspended particle in water. but rather no relation between two.

So



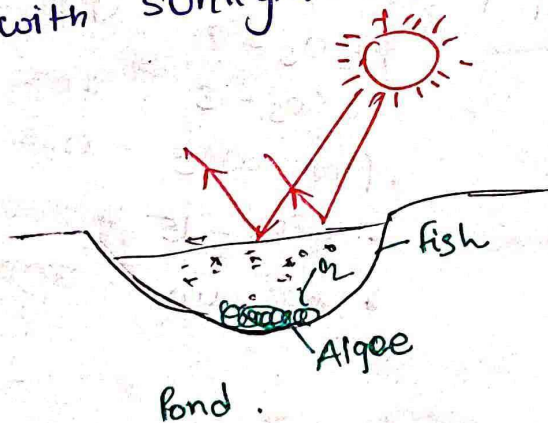
$$\uparrow \text{SSA} \propto \frac{1}{d \downarrow}$$

specific surface area



Impact

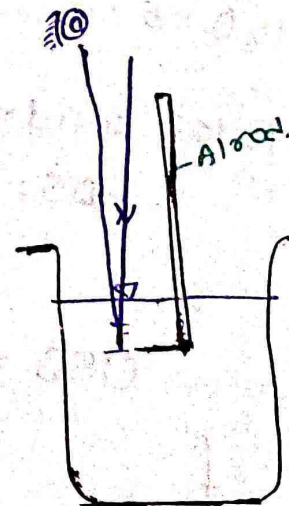
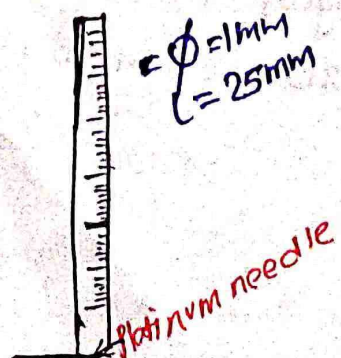
- same as suspended solid.
- Turbidity of water body surface interfere with penetration with sunlight. effects the survival of aquatic life.

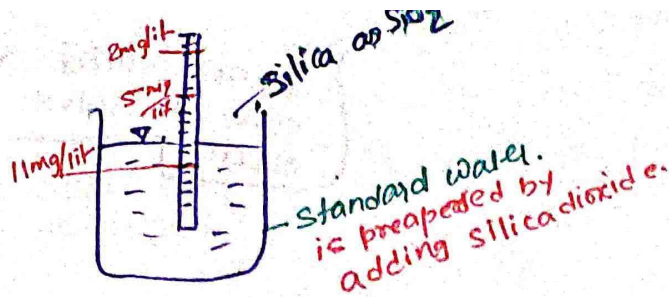


Method Measurement

- (i) Turbidity Rod Method. (F)
- field method.

- depth Calculated.
and calibrated with
standard water.





mg/lit	d
2 mg	10
5 mg/lit	6
11 mg/lit	2

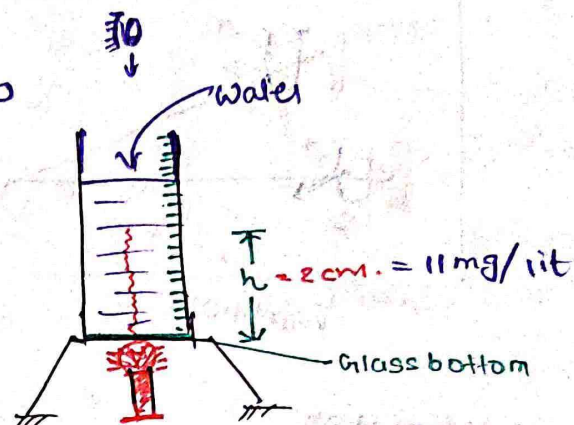
→ mg/lit, parts per million (ppm), STU (silica turbidity unit).

(ii) Jackson Turbidimeter Method. (lab test)

→ how to measure turbidity → 25 to 1000

→ Based on absorption principle.

→ mg/lit, ppm, STU, JTU



→ 1 JTU means 1mg of finely divided silica if mixed in 1 liter distilled water then turbidity produced is taken as 1 JTU.

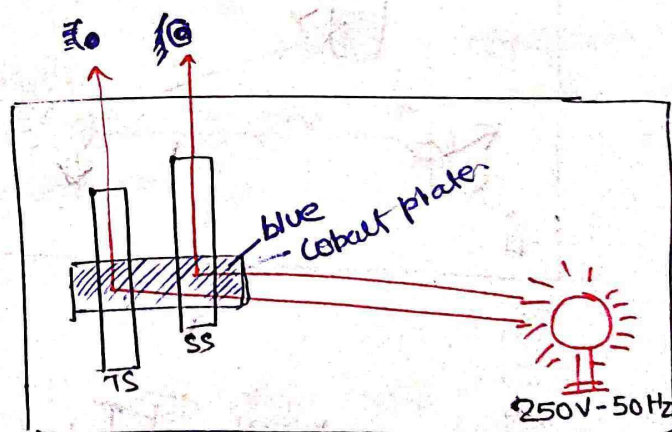
(iii) Baylis turbidimeter & Nephelometer. (0-1) ppm.

→ colour matching technique.

→ Used for domestic water supply.

SS = 1 mg/lit

TS = χ =



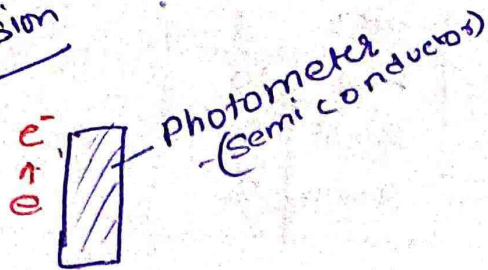
Nephelometer
→ In this method,

Formazine polymer is added if mixed in 1 liter of distilled water then turbidity produced called as 1 NTU or 1 FTU.

→ Baylis turbidity meter, light intensity is measured in the direction of incident ray only whereas in nephelometer light is measured at right angles to the incident ray.

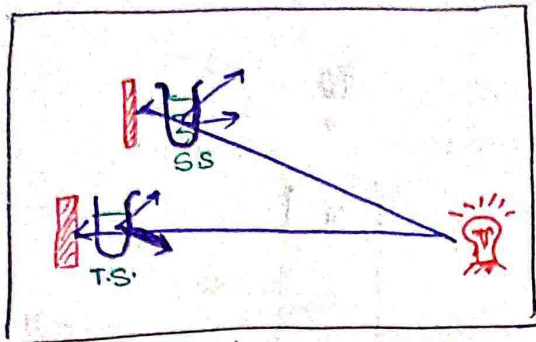
60

New version



Ionic solid
are dissolve
solid.

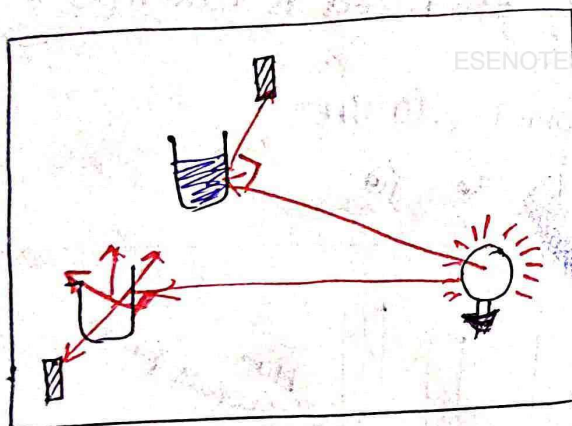
Baylis



→ light pass through sample. and photometer kept just back of sample.

Absorption.

Nephelometer

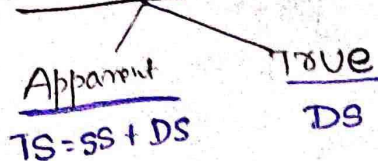


→ It works on scattering principle.
→ In nephelometer we use 1mg formazine in 1 lit of distilled water.
→ light is measured at right angle.

mg/lit, ppm, FTU, NTU,
most appropriate unit:

Limits:	Acceptable	Cause for rejection
	1 NTU	5 NTU

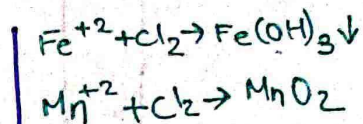
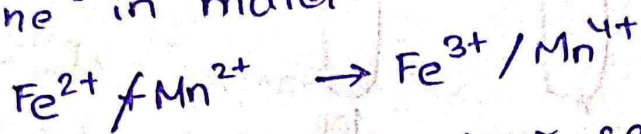
(iii) Colour



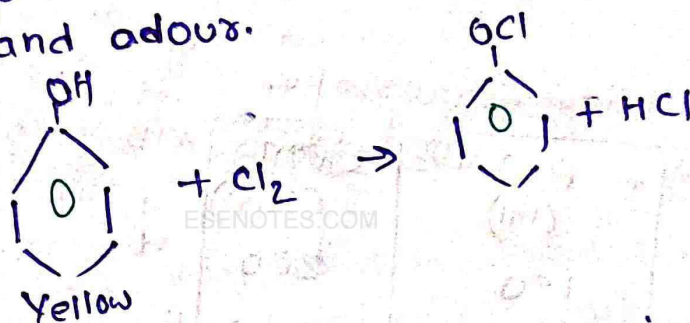
Source: organic & inorganic.

Impact: Colour water can't be used for washing or drying purpose.

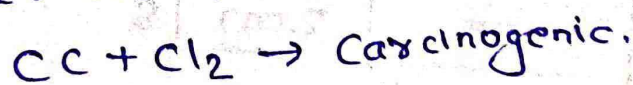
(ii) There are certain colour causing compound, Iron, manganese, which increase the demand of chlorine in water.



(iii) There are certain colour causing compound like phenol which reacts with chlorine to give taste and odour.



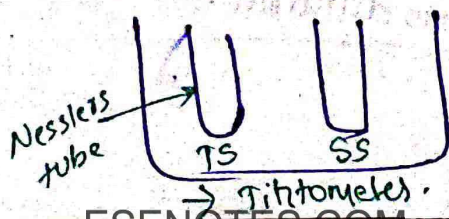
(iv) There are certain colour causing compound which reacts with chlorine to form **Carcinogenic** (cancer causing compound).



Measurement:

→ Colour is measured by using colour matching technique.
→ Colour is measured on platinum cobalt scale in terms of true colour unit (TCU).

1TCU = 1mg of platinum cobalt in form of chloroplatinate ion is mixed in 1 liter distilled water. then colour produced called 1TCU. (Hazen unit)



	AL	CFR
limits	5	15 (TCU)

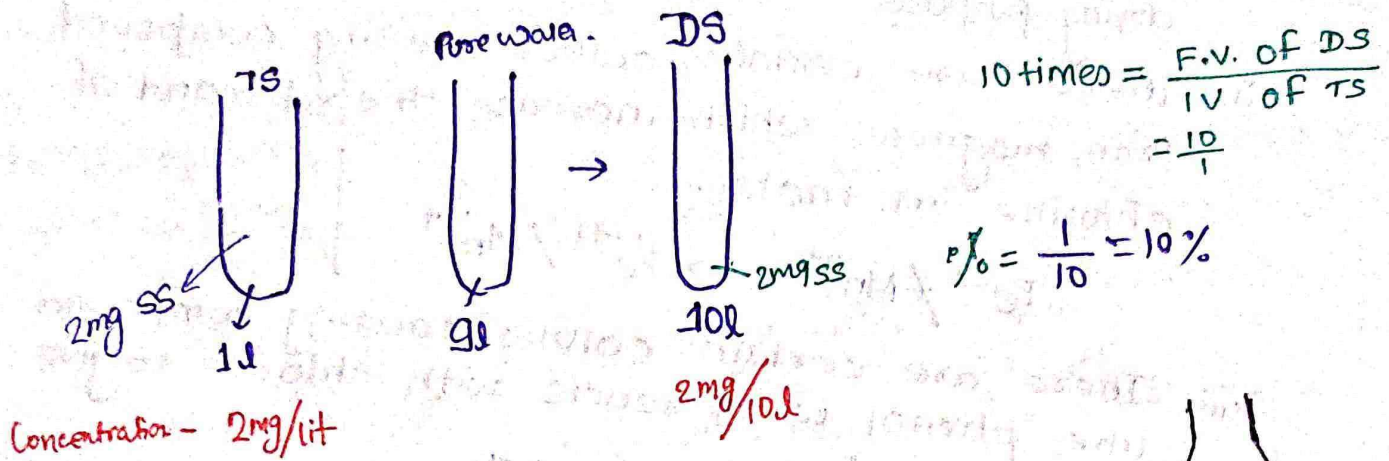
iv Taste and Odour.

Source: Organic & Inorganic, solids & gases.

Impact: Psychological effect,

Measurement: Dilution.

↓
Artificial increase (augmentation) in vol. of water.



→ Measured using Osmoscope.

TS (A)	+ PW (B)	TS (200)	A+B
50	150	200	4
40	160	200	5
...	...	...	...
25	175	200	8

TON (Threshold Odour No.) = Dilution ratio.

$$= \frac{\text{Final volume of DS free from T\&O}}{\text{Initial Vol. of TS}}$$

$$= \frac{A+B}{A}$$

$$A+B = 200 \text{ ml}$$

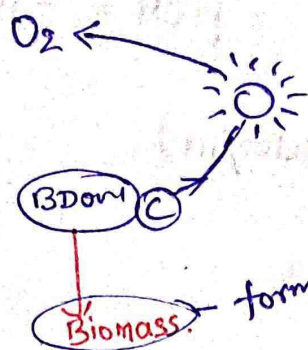
	AL.	CFR
Limit =	1 TON	3 TON

1 TON means same water no dilution.

(v) Temperature.

→ For water supply temp in rang (10-25)°C.

→ Higher temp increase decomposition rate.



formed after decomposition of organic particles

[CO_3^{2-} , NO_3^- , SO_4^{2-} , acids, alcohol, gases: CO_2 , CH_4 , H_2S , N_2]

CHEMICAL WATER QUALITY PARAMETER

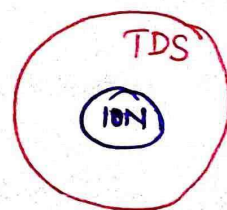
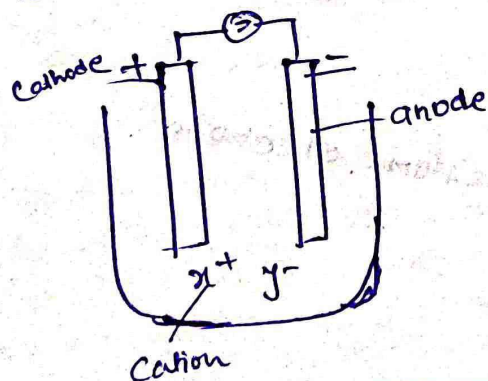
1. Dissolve solid.

Source: Organic & Inorganic

Impact: Same as of taste, colour, odour.

Measurement: Gravimetric technique.

→ Dissolved solid can also be determined approximately by measuring electrical conductivity of water using ~~Di-ionic~~ Di-ionic tester.



$$\left[\text{Electrical Conductivity in } \frac{\mu\text{mho}}{\text{cm}} @ 25^\circ\text{C} \right] 0.65 = \text{TDS (mg/lit)}$$

→ All ions are dissolved solids but all dissolve solids are not ion.
temp ↑ → electrical conductivity ↑

→ E

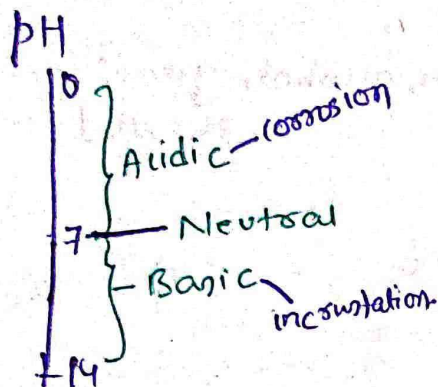
(ii) pH: artificial scale (0-14)

power potential. acidity basicity

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

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

$$[H^+], [OH^-] = (\text{moles/lit})$$



$$[H^+][OH^-] = 10^{-14} (\text{moles/lit})^2$$

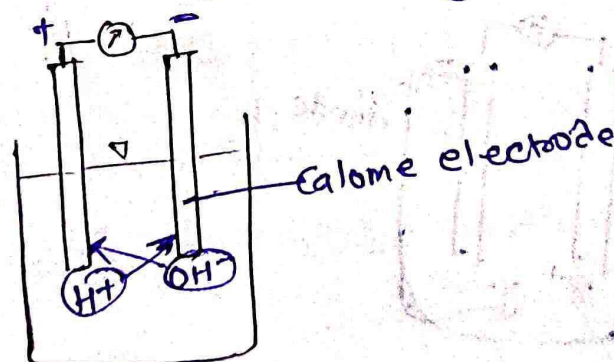
$$\log [H^+][OH^-] = \log 10^{-14}$$

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

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

$$pH + pOH = 14$$

→ pH of water can be found using potentiometer or pH meter.



Limit = (6.5 - 8.5)

	AL	CFR
PH	6.5-8.5	No relaxation

Q

$$pH_A = 4.5$$

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

$$pH_B = ?$$

$$pH_A = 4.5$$

$$pOH_A = 9.5$$

$$- \log_{10} [OH^-] = 9.5$$

$$\log_{10} \left[\frac{1}{[OH^-]} \right] = 9.5$$

$$\frac{1}{[OH^-]} = 10^{9.5}$$

$$[OH^-]_A = 10^{-9.5}$$

$$[OH^-]_B = 2 \times 10^{-9.5}$$

$$pOH_B = \log_{10} \left[\frac{1}{2 \times 10^{-9.5}} \right]$$

$$pOH_B = 9.19$$

$$pH_B = 14 - 9.19$$

$$= 4.8$$

$$pH = \log \frac{1}{[H^+]}$$

$$= \log 1 - \log [H^+]$$

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

$H^+ = \text{mole/lit.}$

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

$$[H^+]_B = \frac{10^{-14}}{2 \times 10^{-9.5}}$$

$$= 1.5 \times 10^{-6.5}$$

$$pH_B = -\log (1.5 \times 10^{-6.5})$$

$$= 4.8$$

Q $t = 0, pH = 6$

$t = 24h, pH = 8.5$

Find the time mean pH value of effluent over 24 hr. period

- Assume hydronium ion concentration varies linearly.
- Assume hydronium ion concentration varies parabolic.
- Assume pH varies linearly.

① $[H^+]_0 = 10^{-6} \text{ moles/lit}$

$$[H^+]_{24} = 10^{-8.5} \text{ moles/lit}$$

$$[H^+]_{avg} = \frac{10^{-6} + 10^{-8.5}}{2} = 6.299$$

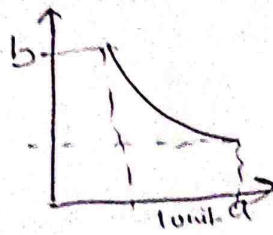
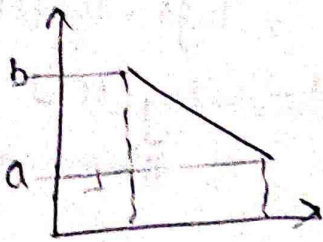
② $[H^+]_{avg} = [H^+]_0 - \frac{2}{3}([H^+]_0 - [H^+]_{24})$

$$= 10^{-6} - \frac{2}{3}[10^{-6} - 10^{-8.5}]$$

$$= 3.35 \times 10^{-7} \text{ moles/lit}$$

$$pH = 6.474$$

$$(ii) p_H \text{ avg} = \frac{16 + 8.5}{2} = 7.25$$



pH

- It is an artificial scale in the range of 0-14 which is used to indicate acidity or basicity of water.
- pH of water can be found with the help of colour indicators like phenolphthalein, methyl orange, bromothymol blue, is used to find the pH apparatus is termed as AQUASCOPE.
- pH can also be determined by help of potentiometer.

Acceptable limit - 6.5 - 8.5

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Basic Stoichiometry

① Molecular weight (MW) = Σ atomic weight of all atoms in the molecule.
Std unit - g

eg. ① $\text{H}_2\text{SO}_4 = (2 + 32 + 64) = 98\text{g}$

② $\text{Al}_2(\text{SO}_4)_3 \cdot 18\text{H}_2\text{O} = (27 \times 2) + (32 \times 3 + 16 \times 12) + 18 \times (2 + 16)$
 $= 666\text{g}$
Hydrated aluminium sulphate

③ $\text{CaCO}_3 \rightarrow 40 + 12 + 16 \times 3 = 100\text{g}$

② No. of moles (n)

$$n = \frac{\text{given weight in (g)}}{\text{molecular weight (g)}}$$

eg. find no. of moles in 350g of CaCO_3

$$n = \frac{350}{100} = 3.5$$

eg. Find no. of moles in 196 mg of H_2SO_4

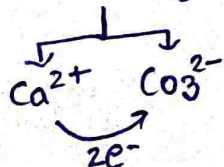
$$n = \frac{196 \times 10^{-3}}{98} = 0.002 \text{ moles.}$$

$$\text{no. of millimoles} = \frac{196}{98} = 2 \text{ millimoles} = 2 \text{ mmoles.}$$

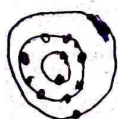
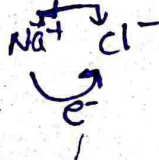
③ Equivalent Weight (g)

$$\text{eq. wt (g)} = \frac{\text{Molecular weight (g)}}{\text{Valency} = \text{the no. of electrons which can be transferred}}$$

(i) $\text{CaCO}_3 = \frac{100}{2} = 50\text{g}$



(ii) $\text{NaCl} = \frac{23 + 35.5}{1} = 58.5$



$$(iii) Mg^{2+} = \frac{24}{2} = 12 g$$

$$(iv) SO_4^{2-} = \frac{96}{2} = 48 g$$

$$(v) H_2SO_4 = \frac{2 + 32 + 64}{2} = \frac{98}{2} = 49$$

$$(vi) MgSO_4 = \frac{120}{2} = 60 g$$

1. H-1	11. Na-23
2. He-4	12. Mg-24
3. Li-7	13. Al-27
4. Be-9	14. Si-28
5. Boron-11	15. Phosphorus-31
6. Carbon-12	16. Sulphur-32
7. N-14	17. Cl-35.5
8. O-16	18. Ar-40
9. F-19	19. Potassium(K)-39
10. Neon-Ne-20	20. Ca-40

K	L	M	N
2	8	8	...

(iv) Gram equivalent.

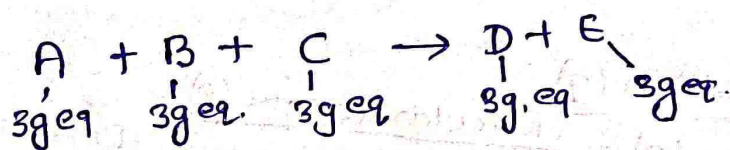
$$g \text{ eq. / no. of eq.} = \frac{\text{Given wt (g)}}{\text{eq. wt (g)}}$$

(i) find the g.eq. in 350g of $CaCO_3$

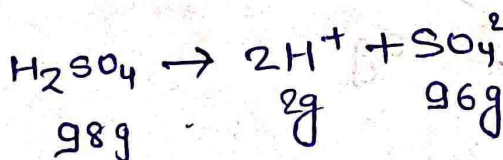
$$g.\text{eq} = \frac{350}{50} = 7 g.\text{eq.}$$

Gram equivalent principle.

"1gm eq. of any compound react with 1g equivalent of any other compound & produce 1g.eq. each of the respective products in a reaction."



Proof



let given weight of H_2SO_4 600g

$$600 = 600 \times \frac{2}{98} \quad 600 \times \frac{96}{98}$$

$$12.24g \quad 587.76g$$

600
98

$$\begin{aligned} \text{g. eq. of 600g of H}_2\text{SO}_4 \\ = \frac{600}{(98/2)g} = \frac{12.24g}{1g} = \frac{587.76g}{48g} \\ 12.24g \cdot g. \quad 12.24g \cdot eq. \quad 12.24g \cdot eq. \end{aligned}$$

Given



A forms E

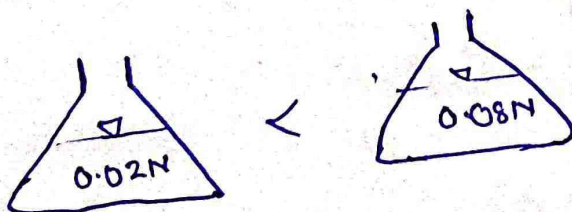
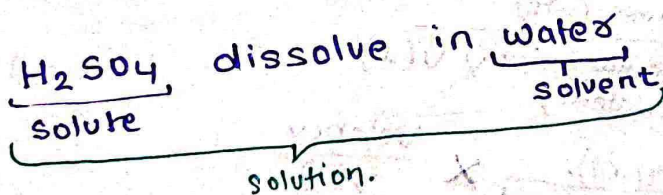
$$\text{g. eq. of A} = \frac{\text{given wt. of A}}{\text{equi. wt of A}} = \text{g. eq. of E}$$

$$\boxed{\text{Wt. of E} = \text{g. eq. of E} \times \text{Equivalent weight of E}}$$

⑤ Normality (N)

→ strength of a solution.

$$\boxed{N = \frac{\text{no. of g. eq. of solute}}{\text{volume of solution (l)}}$$



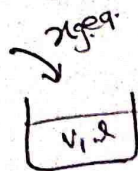
Prepare 1N solution of NaCl.

$$1N = \frac{1 \text{ g. eq.}}{1l}$$

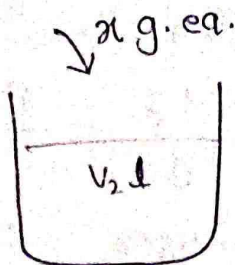
$$\therefore 1 \text{ g. eq.} = \frac{\text{wt(g)}}{58.5}$$

$$\text{wt.} \Rightarrow 58.5g$$

Normality Principle.



$$N_1 \uparrow = \frac{x}{V_1 \downarrow}$$



$$N_2 \downarrow = \frac{x}{V_2 \uparrow}$$

$$x = N_1 V_1 = N_2 V_2$$

when no. of g.eq in the two case are equal.

⑥ Molarity (M)

→ also indicates strength of solution.

$$M = \frac{\text{no. of moles of solute}}{\text{Vol. of solution (l)}}$$

Relation between N & M.

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

$$N = \frac{\text{Given (wt)}}{\text{equivalent wt.} \times V(l)}$$

$$N = \frac{\text{Given (wt)}}{\frac{\text{Molecular weight}}{\text{Valency}} \times V(l)}$$

$$N = \frac{\text{Given (wt)}}{\text{molecular weight}} \times \frac{\text{Valency}}{V(l)}$$

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

Alkalinity

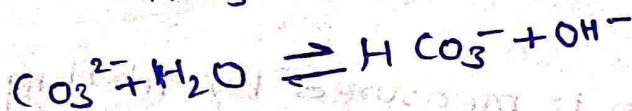
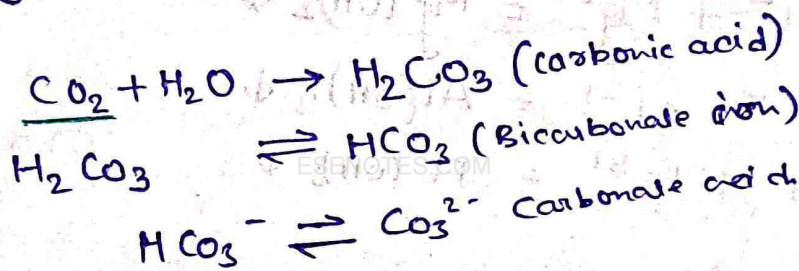
→ it is capacity to neutralise acid.

→ Ability of water to neutralize the hydronium ion or concentration of all the ion in water which are capable of neutralising hydronium ion is known as alkalinity of water.

Carbonate CO_3^{2-} } Major Alkalinity. (95-97%)
Bicarbonate HCO_3^- }
Hydroxide OH^- }
(caustic alkalinity)

B(OH)_4 } Minor Alkalinity. (0.01%)
 $\text{H}_3(\text{SiO})_4$ }
 HS^- }
 NH_3^0 }
 H_2PO_4^- }
 HPO_4^{2-} }

Source:- CO_2 organic or inorganic particles and dissolve gases. ($\text{CO}_2, \text{H}_2\text{S}$)



* reversible reaction depend on pH and temp.
rate depend on concentration of ION.

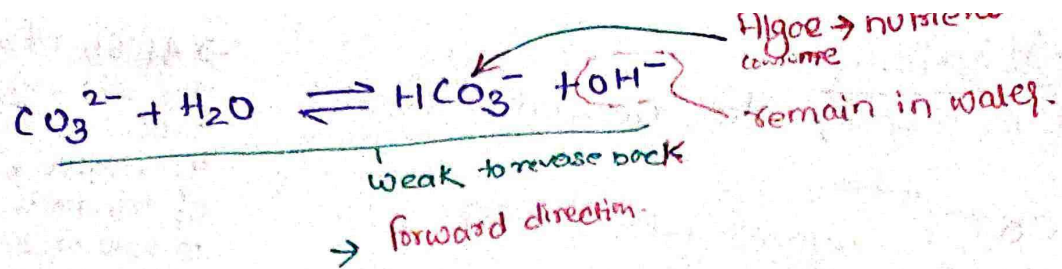


a, b, c, d - moles.

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

$$K_b = \frac{1}{K_f}$$

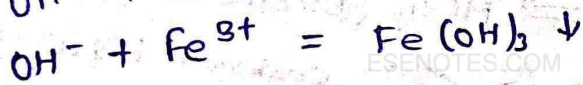
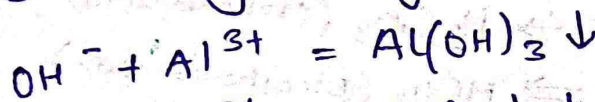
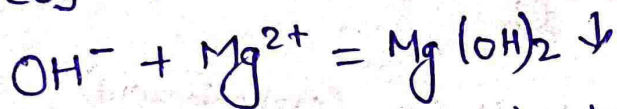
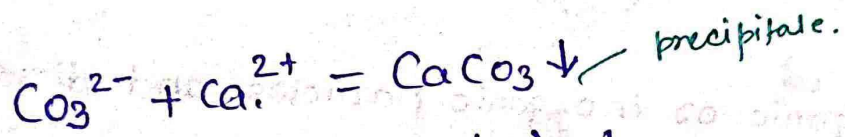
$$\boxed{K_f \cdot K_b = 1}$$



→ If algae is found in water then water will be basic pH = 8-10

Impact:-

- (i) Excess of alkalinity imparts bitter taste in water.
- (ii) Prime objection of alkalinity is that it leads to incrustation in pipe which leads to reduce in discharge carrying capacity.



Measurement
→ Alkalinity of water is measured by titrating.

Molecular weight.

→ Weight of 1 mole = Σ atomic weight.

$$\text{eg. } \text{CaCO}_3 = 40 + 12 + 16 \times 3 = 100 \text{ g}$$

$$\text{H}_2\text{SO}_4 = 1 \times 2 + 32 + 16 \times 4 = 98 \text{ g}$$

$$\text{HCO}_3^- = 1 + 12 + 16 \times 3 = 61 \text{ g}$$

$$\text{OH}^- = 16 + 1 = 17 \text{ g}$$

$$\text{SO}_4^{2-} = 32 + 16 \times 4 = 96$$

$$\text{No. of moles} = \frac{\text{Given weight}}{\text{Molecular weight.}}$$

Q No. of moles in 300 kg of calcium carbonate.

$$\text{moles} = \frac{300 \times 10^3}{100} = 3000 \checkmark$$

Equivalent Weight

→ Weight of 1 gm equivalent.

$$\Rightarrow \text{Equivalent weight} = \frac{\text{Molecular weight}}{\text{Valency}}$$

$$\text{eq. wt. of 1 gm equivalent weight } \text{CaCO}_3 = \frac{100}{2} = 50 \text{ g}$$

$$\text{H}_2\text{SO}_4 = \frac{98}{2} = 49 \text{ g}$$

$$\text{HCO}_3^- = \frac{61}{1} = 61 \text{ gm}$$

$$\text{OH}^- = \frac{17}{1} = 17 \text{ g}$$

$$\text{Al(OH)}_3 = \frac{27 + (16+1) \times 3}{3} = \frac{78}{3} = 26$$

$$\underline{\underline{\text{Q}} \text{ wt. of 5 gm equivalent of } \text{CO}_3^{2-} = 5 \times \frac{(12 + 16 \times 3)}{2} = \frac{60 \times 5}{2} = 150 \text{ gm}}$$

no. of gm. equivalent in 300 gm of CaCO_3 .

$$= \frac{300 \times 10^{-3} \times 1000}{50} = 6 \times 60 = 60$$

$$\boxed{\text{No. of gm. equivalent} = \frac{\text{Given weight}}{\text{equivalent weight.}}}$$

$$\text{Normality} = \frac{\text{No. of gm. eq.}}{\text{lt}} = N$$

$$\text{Molarity} = \frac{\text{No of moles}}{\text{lt}} = M.$$

① water is 20N H_2SO_4 .

② water is 10M H_2SO_4 .

Q 2 lit of water has 196 gm of H_2SO_4 .

$$\text{concentration of } \text{H}_2\text{SO}_4 = \frac{196}{2} = 98 \text{ gm/lit}$$

in terms of moles (i) $\text{moles} = \frac{\text{given weight}}{\text{molecular weight}}$

$$= \frac{196}{98} = 2$$

$$\text{moles/lit} = \frac{2}{2} = 1 \text{ moles / lit}$$

Gram equivalent (ii) $\frac{196}{49 \times 2} \frac{\text{given weight}}{\text{Equivalent weight} \times L} = 2 \text{ gm-eq/liter.}$

Molarity - (iii) $\text{molarity} = \frac{\text{no. of moles}}{\text{liter}} = \frac{196/98}{2} = \frac{2}{2} = 1 \text{M } \text{H}_2\text{SO}_4$

Normality (iv) $\text{Normality} = \frac{\text{No. of gm. equivalent}}{\text{liter.}}$

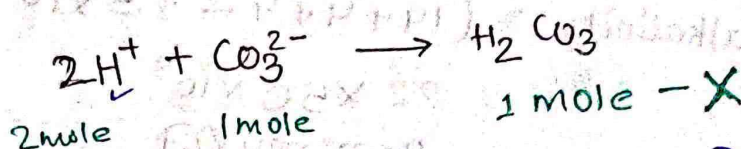
$$= \frac{196}{49} \div 2 = \frac{4}{2} = 2 \text{N } \text{H}_2\text{SO}_4.$$

Note!
 ① 1 gm. eq. of anything = 1 gm. eq. of any other thing.

$$N_1 V_1 = N_2 V_2$$

$$(\text{total eq})_{\text{anything}} = (\text{total eq})_{\text{any other thing}}$$

② 1 gm. eq. of anything reacts with 1 gm. eq. of any other thing and leads to formation of one gm. eq. of any other thing.



2 mole

1 mole

1 mole - X

2 gm

60 gm

62 gm. X

conservation of mass.

given wt.
Equivalent weight

gram equivalent =

$$\frac{2}{1} = 2 \text{ gm. eq.}$$

$$\frac{60}{30} = 2 \text{ gm. eq.}$$

$$\frac{62}{31} = 2 \text{ gm. eq.}$$

$$1 \text{ gm. eq.} \neq 1 \text{ gm. eq.} \quad 1 \text{ gm. eq.}$$

Q If water contains 420 gm of CO_3^{2-} , 244 gm of HCO_3^- and 68 gm of OH^- report the alkalinity of this water as CaCO_3 (mg/lit)

$$X \text{ in terms of } Y = X \text{ mg/lit} \times \frac{\text{Equivalent wt. of } Y}{\text{Equivalent of } X}$$

$$\text{eq. OH}^- \text{ in terms of } \text{CaCO}_3 = \text{OH mg/lit} \times \frac{50}{17}$$

$$\frac{\text{Given wt}}{\text{Equivalent weight}} = \text{gm. eq. of } \text{CO}_3^{2-} = \frac{420}{30} = 14 = \text{gm. eq. of } \text{CaCO}_3$$

$$\text{gm. eq. of } \text{HCO}_3^- = \frac{244}{61} = 4 = \text{gm. eq. of } \text{CaCO}_3$$

$$\text{gm. eq. of } \text{OH}^- = \frac{68}{17} = 4 = \text{gm. eq. of } \text{CaCO}_3$$

ANSWER

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

$$[H^+] = \frac{10^{-14}}{6.8/17} = 0.25 \times 10^{-14}$$

mole/lit

$$[H] = \frac{2.5 \times 10^{-15}}{1} \text{ moles/lit}$$

$$= 2.5 \times 10^{-5} \times 1 \text{ gm/lit}$$

$$= 2.5 \times 10^{-5} \times \frac{1}{1} \text{ gm-equiv/lit} = \text{gm-equiv of CaCO}_3$$

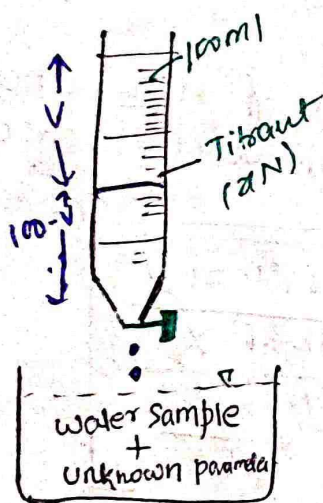
$$\text{Total alkalinity} = (14 + 4 + 4 - 2.5 \times 10^{-5}) \frac{\text{gm-equiv}}{\text{lit}}$$

$$= 22 \times 50 \times 10^3$$

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

$$= 11 \times 10^5 \text{ mg/lit}$$

Measurement of Alkalinity (titration)



$$1 \text{ l} \rightarrow x \text{ gm-equiv.}$$

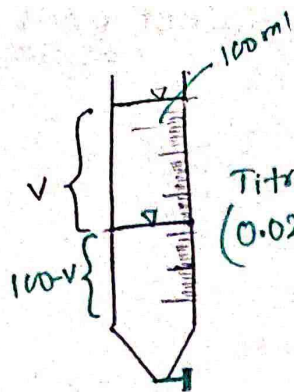
$$1000 \text{ ml} \rightarrow x \text{ gm-equiv.}$$

$$1 \text{ ml} \rightarrow \frac{x}{1000} \text{ gm-equiv.}$$

$$100 \text{ ml} \rightarrow \frac{x}{10} \text{ gm-equiv.}$$

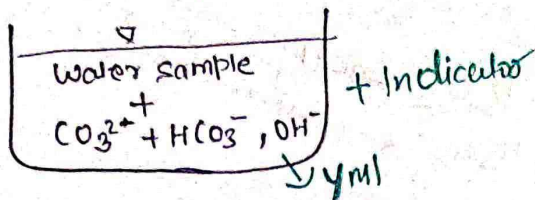
$$100 \text{ ml} = 0.1x$$

$$V \text{ ml} = \frac{x}{1000} \times V \text{ gm-equiv}$$



Titrant
(0.02N H_2SO_4)
= 0.02 g/g/lit.

$$= \frac{0.02}{1000} \times 100 = 0.002$$



$$V_{ml} = \frac{x}{1000} \times V \text{ gm eq.}$$

$$= \frac{0.02}{1000} \times V$$

Equivalent alkali = Acid = $CaCO_3$

$$= \frac{0.02}{1000} \times 100 \times 50 \times 10^3 \left(\frac{mg}{y_{ml}} \right)$$

$$= V \left(\frac{mg}{y_{ml}} \right)$$

$$\text{Normality} = \frac{\text{no. of gm eq.}}{\text{lit.}}$$

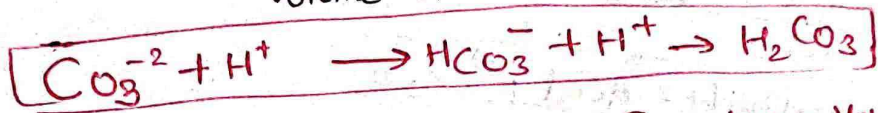
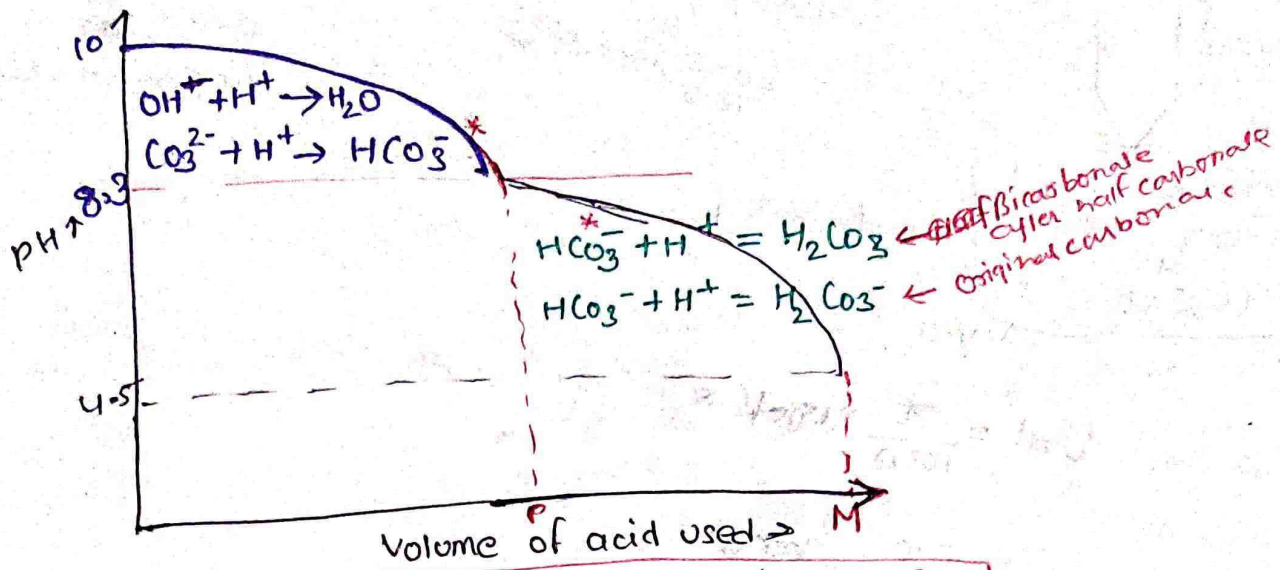
$$\text{Gram equivalent} = \frac{\text{gram wt.}}{\text{Equivalent Wt.}}$$

$$\text{Eq. wt} = \frac{\text{molecular wt.}}{\text{Valency}}$$

$$0.02 = \frac{\text{gram wt.}}{49}$$

$$\text{gram wt} = 0.98 \text{ gm}$$

Fact :- $P_{H_2O} \geq 10$, (HCO_3^- , OH^-) alkalinity do not exist together at same time.

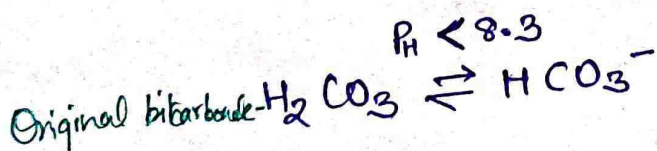


(i) phenolphthalein (8.6-10.3) [colourless, yellow]

→ when we add it before titration, ~~our~~ water change its colour ~~from~~ to pink.

→ Then we start titration and when our sample become (yellow). colourless, it means.

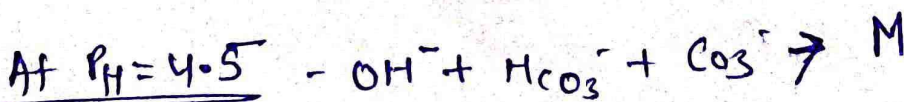
- water pH is 8.3, -
- Half carbonate is neutralise, (Half carbonate means whole CO_3^{2-} converted into HCO_3^-)
- OH^- full neutralise. (caustic Alkalinity)



At $P_H = 8.3 - 4.5$

→ Half carbonate neutralise
→ HCO_3^-

acid.
(M-P)



When second stage start,

we add (Methyle orange) ^{2.8-4.4} ~~(2.8-4.4)~~ (orange red)

→ we add it at 8.3 pH.

→ when we add it in water, ~~water~~ colour change from colourless to orange red.

→ ~~when~~ then again we start titration and when ~~the~~ water colour become colourless ~~means~~ it means water pH is at 4.04. ~~and~~

P = Acid use in first stage.
M = ~~strong~~ Acid used in whole stage.

- ① IF $P=0$, $M \neq 0 \Rightarrow$ ONLY HCO_3^- is present.
- ② IF $P=M \Rightarrow$ ONLY CAUSTIC alkalinity present in water
- ③ IF $P=\frac{M}{2} \Rightarrow$
 - ① ONLY carbonate (CO_3^{2-})
 - ② $[\text{OH}^-]$, $[\text{HCO}_3^-] \Rightarrow$ concentration of both same.
 - ③ $[\text{CO}_3^{2-}]$, $[\text{OH}^-] = [\text{HCO}_3^-]$

④ IF $P > \frac{M}{2} =$ OH^- , CO_3^{2-} alkalinity is predominant.
or $[\text{OH}^-] > [\text{HCO}_3^{2-}]$ concentration

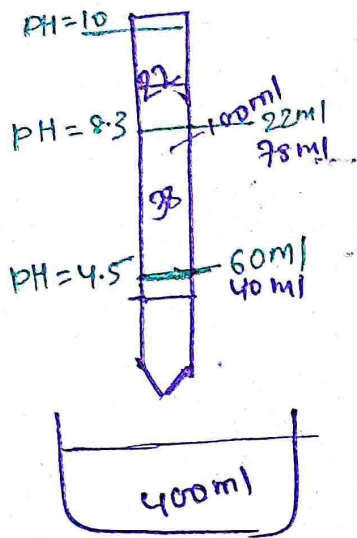
⑤ IF $P < \frac{M}{2} =$ CO_3^{2-} , HCO_3^{2-} alkalinity is predominant
 $[\text{HCO}_3^{2-}] > [\text{OH}^-]$.

→ In raw water we get 5th observation.

	Acceptable.	Repease for rejection.
Limits	200 mg/lit as CaCO_3	500 mg/lit as CaCO_3

Q 8 400ml sample of water has initial pH=10, 60ml of 0.02N H_2SO_4 is used to bring the water at 4.5 and 22ml of 0.02N H_2SO_4 is used to bring to 8.3.

- ① Total Alkalinity, $CaCO_3$ mg/lit. = V used.
- ② Concentration of all alkalinity as $CaCO_3$ (mg/lit)
- ③ Methyl orange and Phenolphthaleine alkaline.



① → If 400ml water sample 60ml acid is used to bring the water to neutralise total alkalinity.

$$\Rightarrow (OH^- + CO_3^{2-} + HCO_3^-) \text{ as } CaCO_3 = \frac{60}{400} \text{ mg}$$

$$= \frac{60 \times 1000}{400} = 150 \text{ mg/lit.}$$

② 22ml used to bring pH 8.3. means to neutralize $(OH^- \text{ and } \frac{1}{2} CO_3^{2-})$ alkalinity.

$$\begin{aligned} (OH^- + \frac{1}{2} CO_3^{2-}) \text{ alk as } CaCO_3 &= 22 \text{ mg} / 400 \text{ ml.} \\ &= \frac{22}{400} \times 1000 \\ &= 55 \text{ mg/lit} \end{aligned}$$

Phenolphthaleine alkalinity = 55 mg/lit
Methyl orange alkalinity = 150 - 55 = 95 mg/lit.

$$pH=10 \Rightarrow H^+ = 10^{-10} \text{ moles/lit} = 10^{-10} \times 1 \text{ gm/lit}$$

$$[H^+] = \frac{10^{-10} \times 1}{1} \text{ gm-eq/lit as } CaCO_3$$

$$H^+ \text{ alkalinity as } CaCO_3 = 10^{-10} \times 50 \times 10^{-3} = 5 \times 10^{-6} \text{ mg/lit}$$

$$pOH = 14 - 4 = 4$$

$$\Rightarrow [OH^-] = 10^{-4} \text{ moles/lit}$$

$$= \frac{10^{-4} \times 17}{17} \text{ gm-eq. } ~~(10^{-4})~~$$

$$OH^- \text{ alkalinity as } CaCO_3 = \frac{10^{-4} \times 50 \times 10^3}{17} = 5 \text{ mg/lit} \quad \text{--- (i)}$$

$$CO_3^{2-} \text{ alkalinity} = 55 - 5 = 50 \text{ mg/lit.}$$

$$HCO_3^{2-} = 150 - 5 - 100 = 45 \text{ mg/lit.}$$

$$\begin{aligned} \text{Total Alkalinity} &= Ca_3^{2-} + HCO_3^- + OH^- \\ &= 100 + 45 + 5 - 5 \times 10^{-6} \\ &= 149.9999995 \text{ mg/lit} \end{aligned}$$

Hardness

→ It is ability of water to destroy surfactant property of the soap or may also be referred as concentration of multivalent cation in water.

eg. $\rightarrow \begin{matrix} Ca^{2+} \\ Mg^{2+} \end{matrix}$ } Major multivalent cation.

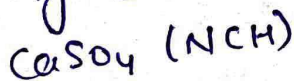
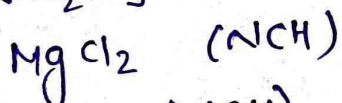
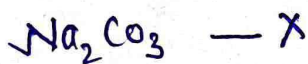
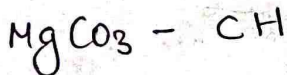
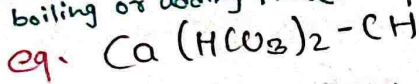
$\rightarrow Al^{3+}, Fe^{2+,3+}, Mn^{2+}, Pb^{2+}, Sn^{2+}$ } Minor multivalent cation.

Hardness

↓
Carbonate

Hardness due to carbonate (CO_3^{2-}) and (HCO_3^-) of multivalent cation is termed as carbonate hardness.

→ temporary hardness - because it remove easily by boiling or adding lime.



Non-Carbonate Hardness.

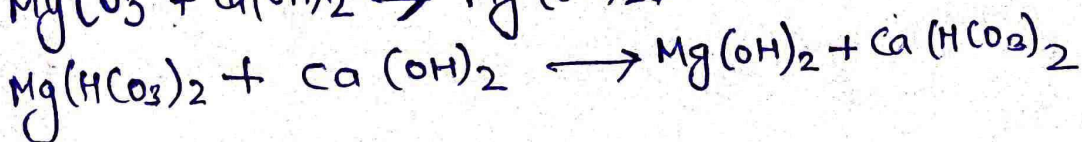
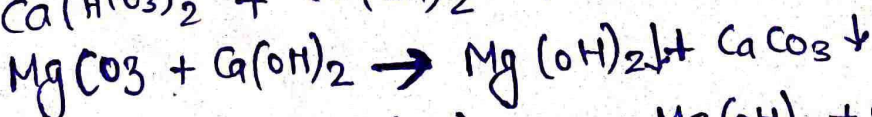
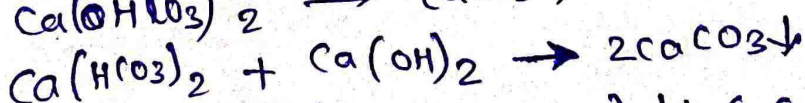
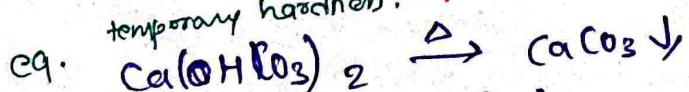
Hardness due to (Cl^-), (SO_4^{2-}), (NO_3^-) of multivalent cation is termed as Non carbonate.

→ Permanent hardness.

→ Removed by ion exchange method, zeolite process etc.

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temporary hardness. - It can be removed by boiling or adding lime.



Clark degree ($^{\circ}$ Clark) or English degree of hardness.
 $= 14.254 \text{ mg/lit of hardness as CaCO}_3$

1 French degree ($^{\circ}$ fn) or ($^{\circ}$ F) of hardness
 $= 10 \text{ mg/lit of hardness as CaCO}_3$.

Q A water sample has 120 mg/lit of Ca^{2+} , 96 mg/lit of Mg^{2+} , 27 mg/lit of Al^{3+} in it. find total hardness.

$$\text{TH} = 120 \times \frac{50}{20} + 96 \times \frac{50}{12} + 27 \times \frac{50}{27}$$

$$= 850 \text{ mg/lit}$$

$$\frac{27}{3} = 9$$

$$\text{TH} = \text{Ca}^{2+} \left(\frac{\text{equivalent weight of CaCO}_3}{\text{equivalent of Ca}} \right) + \text{Mg}^{2+} \left(\frac{\text{equivalent weight of CaCO}_3}{\text{equivalent wt of Mg}} \right)$$

Source :- Organic or Inorganic particle.

Impact:

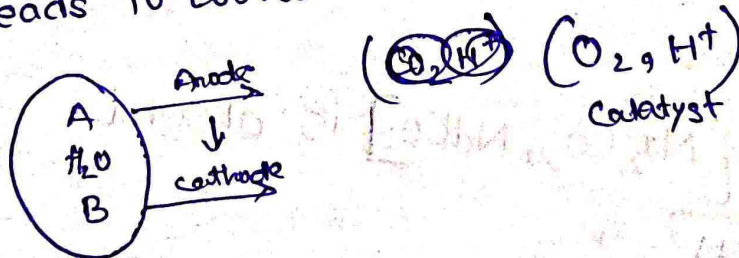
(i) It increases the consumption of soap.

(ii) It leads to incrustation.

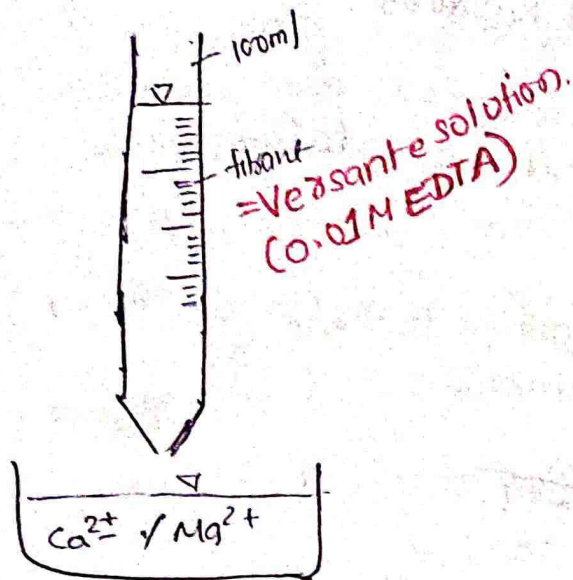
(iii) It imparts bad taste in water.

(iv) If $\text{MgSO}_4 > 50 \text{ ppm}$, it causes

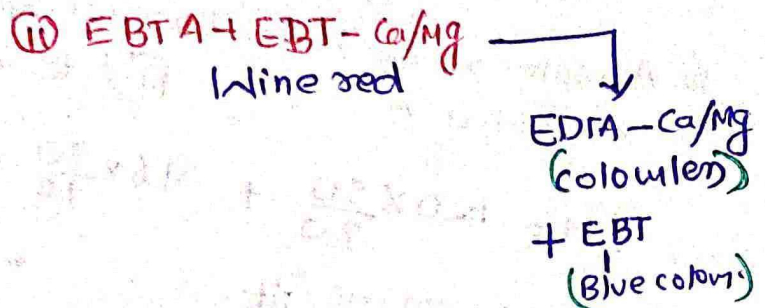
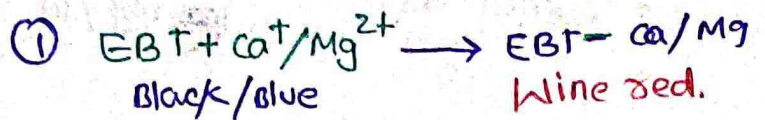
(v) It leads to corrosion in pipe. (laxative effect (inse motion, digestion problem)
 (reducer Discharge carrying capacity).



Measurement: titration.



Indicators - EBT (Eriochrome black-T)
Black/Blue colour



EDTA = Ethylene diamine tetraacetic acid.
($C_{10}H_{16}N_2O_8$)

<u>Limits</u>	<u>Acceptable</u>	<u>Rejection</u>
	200 mg/lit as $CaCO_3$	600 mg/lit as $CaCO_3$

* For domestic water supply hardness must be in range of 75-115 mg/l.

* Hardness and Alkalinity:-

$Ca^{2+}, Mg^{2+}, Na^+, CO_3^{2-}, HCO_3^-$	Alkalinity.
Ca^{2+}, HCO_3^-	Carbonate hardness

① If Na Alkalinity [$Na_2CO_3, NaHCO_3$] is absent

$\boxed{\text{Alkalinity} = CH}$ - (i)

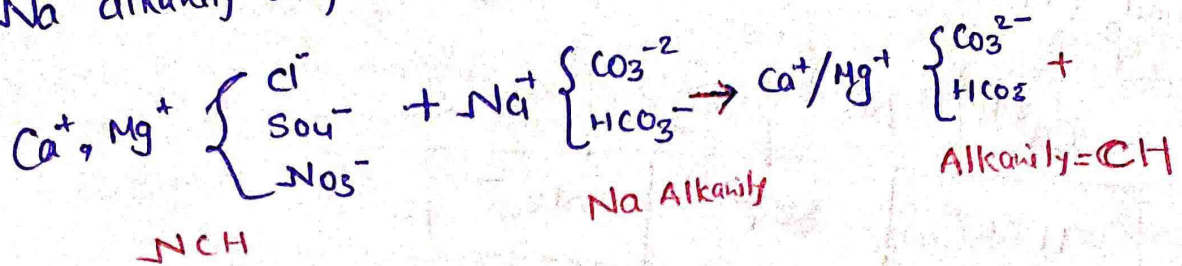
$TH = CH + NCH$

$TH = \text{Alkalinity} + NCH$

$\boxed{TH > \text{Alkalinity}}$ - (ii)

$\boxed{NCH = TH - CH}$

* Na alkali only observe only when NCH is present.



② If Na alkali is present.

$$\Rightarrow \text{NCH} = 0$$

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

$$\text{TH} = \text{CH} \quad \text{--- (i)}$$

$$\text{Alkalinity} > \text{CH}$$

$$\text{Alkalinity} > \text{TH} \quad \text{--- (ii)}$$

from (i), (ii)

$$\boxed{(\text{TH}, \text{Alkalinity})_{\min} = \text{CH}}$$

Q Find TH, CH, NCH and Alkalinity of given water sample.

$$\checkmark \text{Ca}^{2+} = 40 \text{ mg/lit} - \text{hard}$$

$$\checkmark \text{Mg}^{2+} = 48 \text{ mg/lit} - \text{hard}$$

$$\text{Na}^+ = 46 \text{ mg/lit} -$$

$$\checkmark \text{Al}^{3+} = 18 \text{ mg/lit} - \text{hard}$$

$$\text{HCO}_3^- = 244 \text{ mg/lit}$$

$$\text{SO}_4^{2-} = 96 \text{ mg/lit. } \text{Ca}$$

$$\text{TH} = 40 \times \frac{50}{20} + 48 \times \frac{50}{24.2} + 18 \times \frac{50}{27.3}$$

$$= 400 \text{ mg/lit}$$

Mg = 12

$$\text{T.A} = \text{HCO}_3^- \times \frac{50}{61} = 244 \times \frac{50}{61} = 200 \text{ mg/lit}$$

$$\text{TH} > \text{T.A.}$$

$$\text{T.A.} = \text{CH} = 200 \text{ mg/lit}$$

$$\text{NCH} = 400 - 200 = 200 \text{ mg/lit.}$$

$$\text{pH} = 7 \Rightarrow [\text{OH}^-] = 10^{-7} \text{ mole/lit.}$$

$$= 17 \times 10^{-7} \text{ mg/lit}$$

$$= 17 \times 10^{-7} \times 10^3 \text{ g/lit}$$

$$\text{T.A} = 244 \times \frac{50}{61} + 10^{-7} \times 17 \times 50$$

$$= 200 \text{ mg/lit.}$$

Q Find Non-Carbonate hardness for given sample of water.

meq/lit	0	2	4.5	6	8.5
	Ca^{2+}	Na^{2+}	Mg^{2+}	Fe^{3+}	
meq/lit					
	HCO_3^-	SO_4^{2-}	NO_3^-		
		3	5.5	8.5	

$$\text{TH} = (2 + 1.5 + 2.5) \times 50 = 300 \text{ mg/lit}$$

$$\text{Alkalinity} = 3 \times 50 = 150$$

$$\text{TH} = (\text{TH}, \text{Alkalinity}) \text{ min}$$

$$\text{CH} = 150 \text{ mg/lit as } \text{CaCO}_3$$

$$\text{NCH} = 150 \text{ mg/lit } \text{CaCO}_3$$

5 Chloride content

Limits $\rightarrow$ Acceptable $\rightarrow 250 \text{ mg/lit}$
C.F.R $\rightarrow 1000 \text{ mg/lit}$

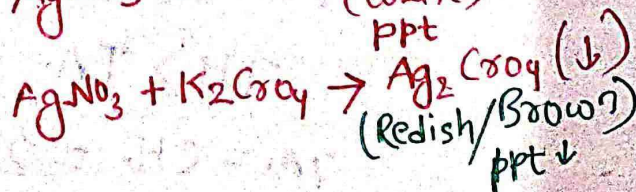
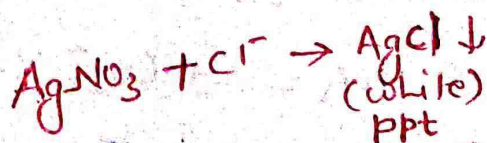
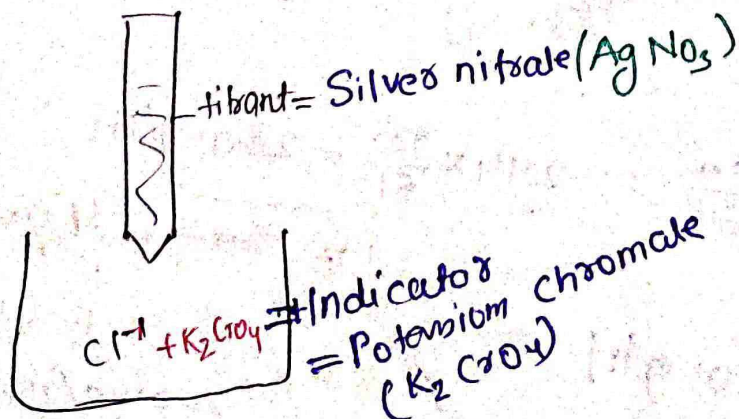
$\rightarrow$ Presence of chloride in water in excess indicates its pollution either due to sewage or due to industrial waste.

Impacts:

- (i) It gives bad taste in water.
- (ii) It cause hypertension.
- (iii) It affect heart & kidney.

Measurement: Titration - work on principle of 1 meq of any thing react with one gram of any other thing.

$\rightarrow$ Mohr's method



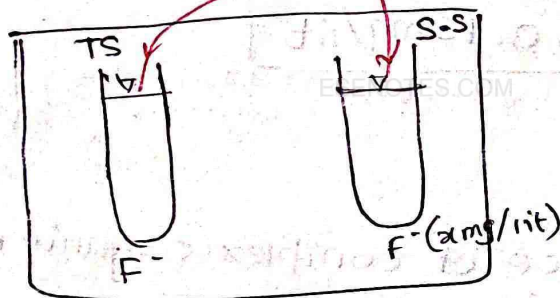
Fluoride Content

- Fluoride upto 1mg/lit is required to prevent dental cavity and for growth of permanent teeth.
(It combines with tooth enamel & makes it hard & strong)
- Beyond 1.5 ppm it cause decolourisation of molting of teeth & result in disease called as fluorosis.
- Beyond 5ppm it causes chemical deformation of bones and result in BONE FLUOROSIS and other skelton abnormalities.

Limit →	Acceptable	C.F.R
	1mg/lit	1.5 Mg/lit

✓ Measurements:-

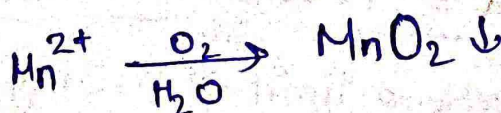
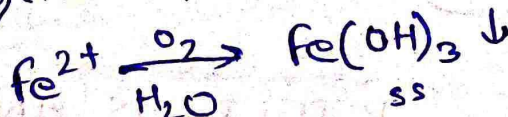
Fluoride is measured by colour mating technique.
ZIRCONIUM ION + ALAZARINE



* Fluoride is dissolve solid.

	Ground water	Surface water
SS.	↓ (Filtration)	↑ (Turbulence)
DS.	↑ (O ₂ ↓)	↓ (O ₂ ↑)

→ In presence of oxygen dissolve solid oxidise,



(ii) Nitrogen Content

- Minor constituent of water
- Presence of nitrogen in water indicates its pollution due to organic matter.
- It is found in following different forms:-

① Free Nitrogen (Ammonia)

→ It indicates recent pollution due to organic water.

→ Measurement: ^{By} Boiling ammonia gas liberated.



→ By boiling of water decomposition of organic matter takes place.

→ To separate ammonia and water by distillation.

Limit:- Acceptable - 0.15 mg/lit

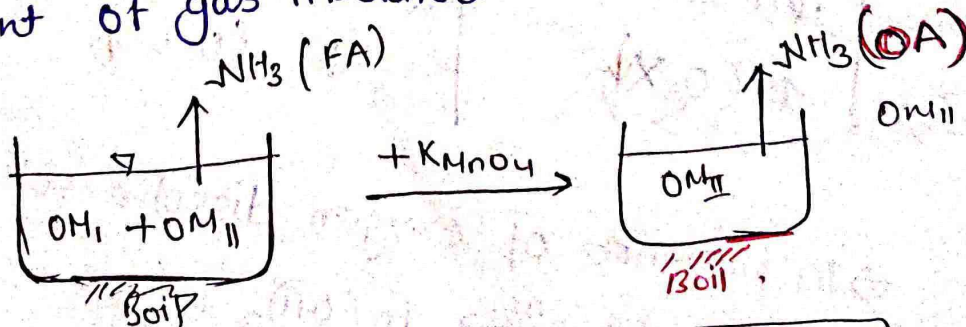
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② Organic Ammonia.

→ It signifies presence of complex organic matter in water.

Measurement: Boiling.

→ It is determined by boiling the already boiled sample of water with addition of strong alkaline reagent like Potassium permagnate and noting the amount of gas liberated.



FA + OA = KJELDHA'S AMMONIA

Limit → Acceptable limit → 0.3 mg/lit.

③ Nitrite (NO_2^-)

→ Presence of nitrite in water is highly dangerous as it indicates the partial decomposition of organic matter.

→ Limit → 0 mg/lit.

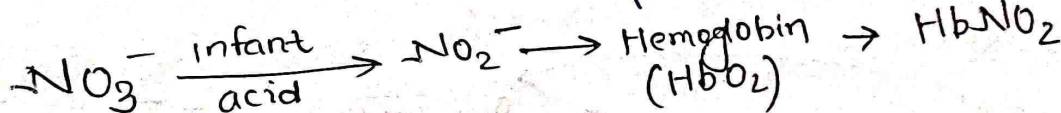
→ Measurement:- Colour matching technique. and colour is induced by adding of sulphonic acid and Naphthamine.

④ Nitrate (NO_3^-)

→ Presence of nitrate is not harmful. as it signifies the complete decomposition of organic matter.

→ Limit - 45 mg/lit

→ But if it is present more than 45 mg/lit it affects infant & causes MARHEMO GLOBINEMIA and Blue Baby disease.



displacement reaction
oxygen displaced

Measurement:-

Colour matching technique.

→ Colour induced by - Phenol Di-sulphonic Acid + Potassium hydroxide

✶ Different types of gases in water.

Hydrogen Sulphide H_2S :- Rotten egg / Pungent smell.

CO_2 :- It adds taste and odour in water and causes corrosion. catalyst.

NH_3 :- Ammonia. (←)

Methane CH_4 :- It is an explosive gas. High calorific value.

due to presence of O_2 in water

Q10 Oxygen

→ Max^m oxygen that may be dissolve in water at any given temperature is termed as saturation dissolve oxygen

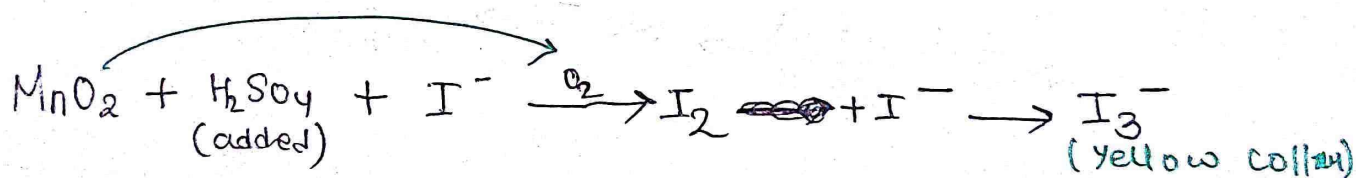
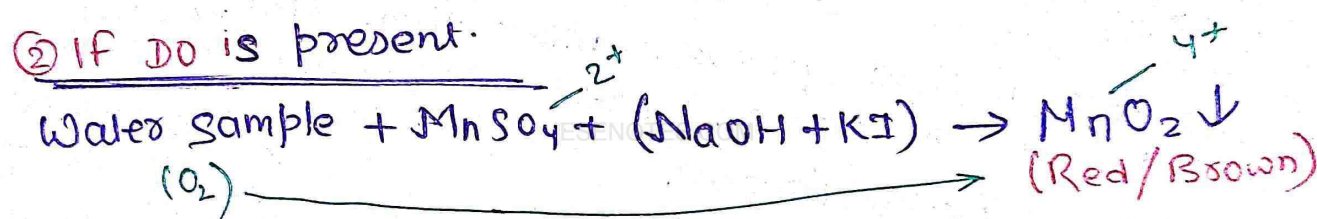
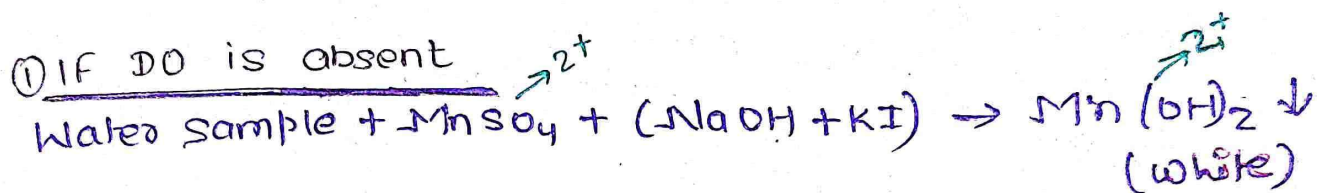
$$D = DO_{sat} - DO.$$

→ $DO_{sat} @ 20^{\circ}C = 9.2 \text{ mg/lit}$

$DO_{sat} @ 100^{\circ}C = 0$

→ Any deficiency of O_2 in water indicates the presence of organic matter.

→ DO in water is measured by Winkler's test.



↓ Starch (Indicator)

BLUE COLOURED COMPLEX SOLUTION

V(ml) of 0.01N $Na_2S_2O_3$ required to decolourised the sample.
 ← (Titrate with 0.01N $Na_2S_2O_3$)

$$\frac{0.01}{10^3} \times V = I_3^- = I_2 = O_2$$

Modified Whitley test

on when other oxidising agent present eg. NO_3^- - then add NaN_3 (Sodium azide)

→ NaN_3 is a strong reducing agent reduces the oxidation of other source.

BIOLOGICAL WATER QUALITY PARAMETER

→ Organic ~~matter~~ matter in water may be either be biodegradable or non-biodegradable

→ Biodegradable organic matter is that which can be decomposed by action of micro-organism either in presence of ~~oxygen~~ or absence of oxygen.

→ If the decomposition takes place in the presence of oxygen then it is termed as aerobic decomposition.

→ Aerobic decomposition is carried by aerobic micro-organism.

→ End product of aerobic decomposition is comparatively stable which includes carbonate, nitrate & sulphate.

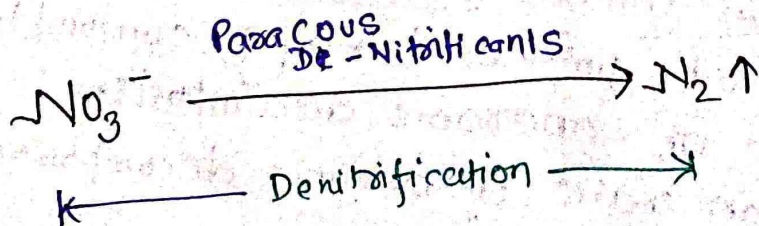
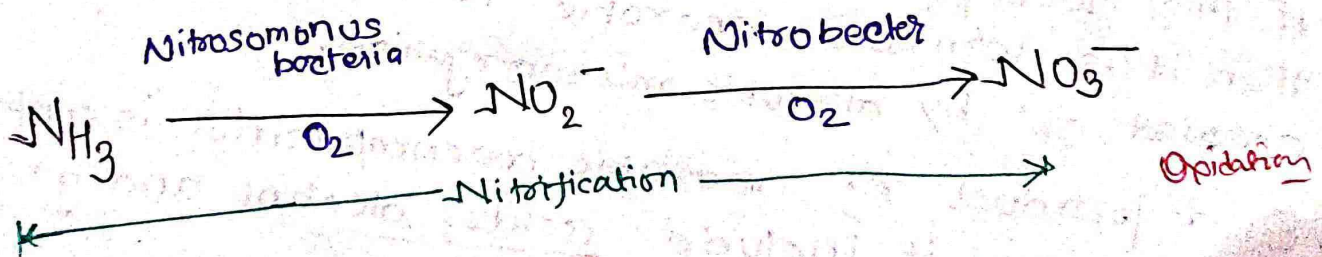
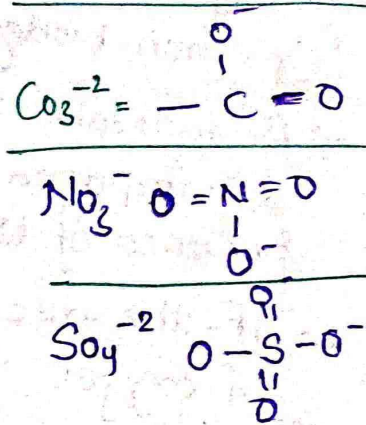
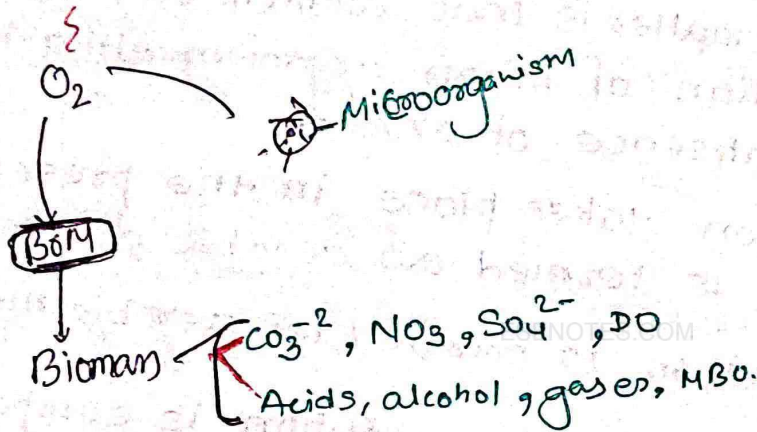
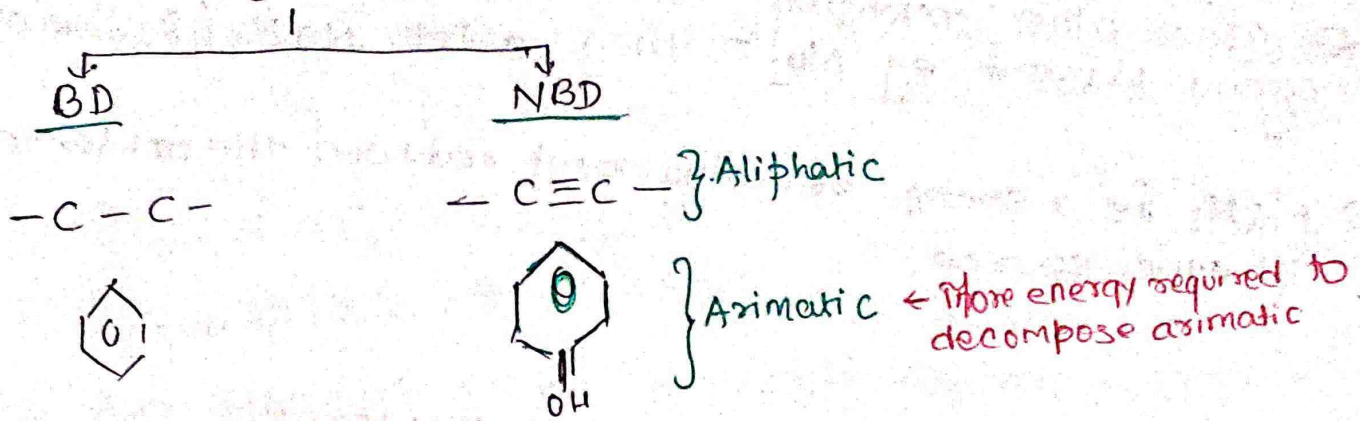
→ If this decomposition takes place in the absence of O_2 then it is termed as anaerobic decomposition which is carried out by anaerobic micro-organism.

→ End product of anaerobic decomposition is comparatively unstable and it includes acids, alcohol and gases.

→ Anaerobic microorganism utilises molecular bond oxygen which is both time and energy consuming process. Hence rate of anaerobic decomposition is 3 times slower than rate of aerobic decomposition.

Note:- There are certain type of micro-organism which can act both in presence or absence of oxygen these are termed as Facultative microorganisms.

Organic matter



→ The most important type of micro-organism present in water which are capable of carrying disease is termed as pathogens.

→ Testing & counting of pathogen is difficult and test involved are comparatively costly & time taken. Hence instead of testing the water for pathogen it is being tested for Coliform which themselves are harmless. but their presence or absence indicates the presence or absence of pathogens in water. Moreover they are easier and economical to be tested.

→ Coliforms like E-coli & B-coli are present in the intestine of all warm blooded animals.

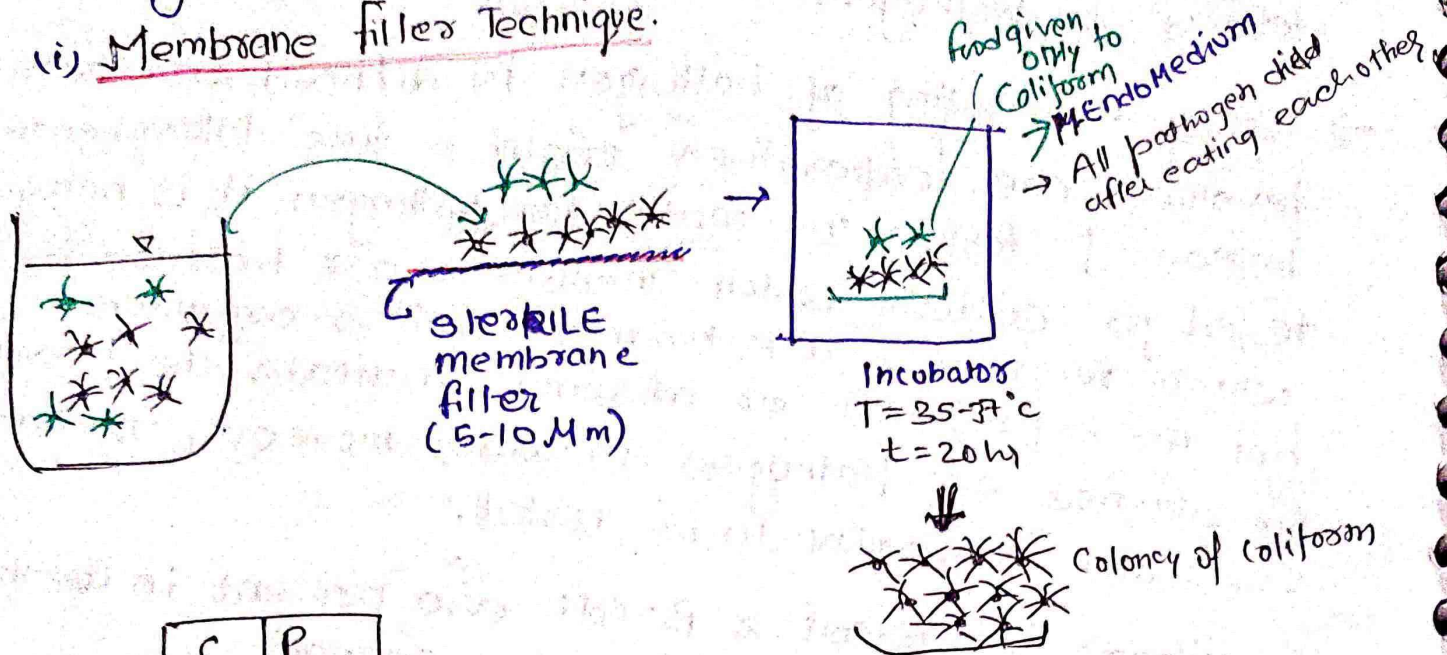
Pathogens — ^{disease}
Virus — Hepatitis / Poliomyelitis / Jaundice.
Bacteria — typhoid / ~~cholera~~ / cholera / Dysentery
Fungi — Valley fever.
Protozoa — Amebic Dysentery
Helminths — Helminthiasis

Coliforms — E-coli
B-coli
Harmless

Testing of Coliforms.

Testing of coliform can be done by any one of following method:

(i) Membrane filter Technique.



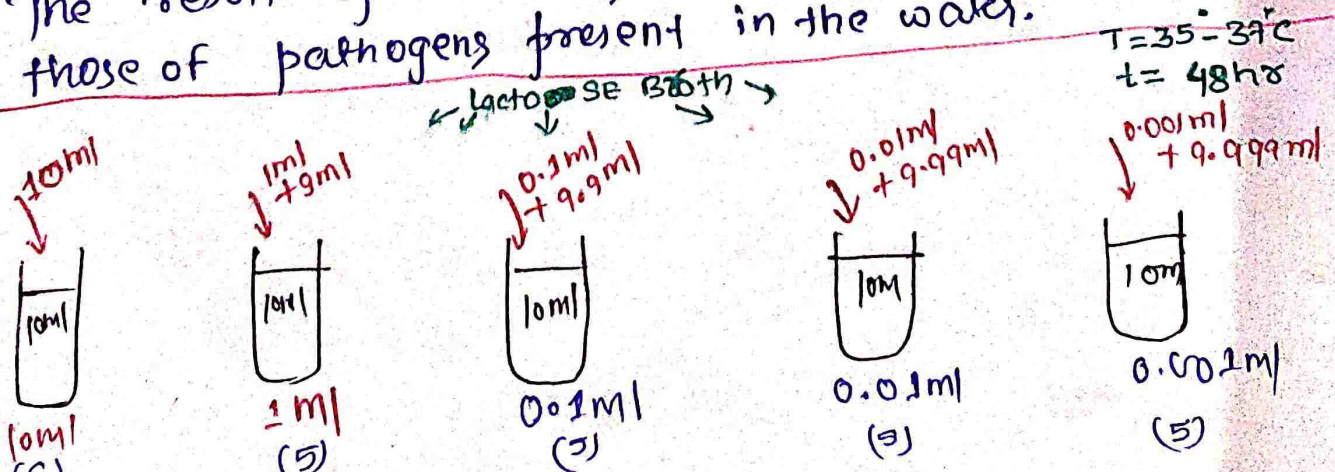
C	P
100	15
80	10

(ii) Most Probable Number (MPN Test)

- It is multiple tube fermentation test having water sample of different dilution ratio.
- In all the sample nutrients of coliforms (LACTO-SE BROTH) is added and samples are placed in incubator at temp of 35-37°C for 48h.
- The sample of water is then removed and tested for presence of coliform indirectly by noting the presence of acids alcohol and gases in the sample.
- The result of test is further calibrated with those of pathogens present in the water.

- (i) Presumptive test
- (ii) Confirmatory test
- (iii) Completed test

KEEP IN INCUBATOR



(5)	(5)	(5)	MPN 100
10ml	1ml	0.1ml	
5	5	5	20
5	4	3	30
3	4	2	5
4	3	2	16
0	2	2	28

density of pathogen which is mostly likely to present.

↓ Age aur हिसाब

Sample Size	Ex ₁	Ex ₂	Ex ₃	Ex ₄	Ex ₅
10ml	5 ✓	5 ✓	5 ✓	4 ✓	0 ✓
0.1ml	5 ✓	5 ✓	3 ✓	3 X	2 ✓
0.1ml	5 ✓	5 ✓	4 X	2 X	2 ✓
0.01ml	5 ✓	4 ✓	2 ✓	0 ✓	0 ✓
0.001ml	5 ✓	3 ✓	0 ✓	0 ✓	0 ✓
	20 X 100	30 X 100	5 X 10	16 X 1	28 X 1
	2000	3000	50	16	28
	MPN 100ml				

Dilution times

MPN value can also be found using Thomas Equation.

$$\frac{\text{MPN}}{100\text{ML}} = \frac{\text{No. of Positive tube} \times 100}{(\sqrt{\text{ml of sample in negative tube}}) \times (\text{ml of sample in all tube})}$$

Used when table is not available.

Q.

Sample Size (ml)	No. of positive tube of 5	No. of negative tube
10	4	1
1 1	2	3
0.1	1	4
0.01	0	5

MPN TABLE is as follows.

Combination of (+ve) 10ml, 1ml, 0.1ml	MPN/100ml
5-4-3	280
4-3-1	33
4- 2 -1	26
2-1-0	7

① $MPN/100ml = 26$

② No. of +ve tube = $4 + 2 + 1 = 7$

Vol of sample in negative tube = $1 \times 10 + 3 \times 1 + 4 \times 0.1 + 5 \times 0.01$
 $= 13.45$

wt of sample in all tube = $5 \times (10 + 1 + 0.1 + 0.01)$
 $= 55.55$

$MPN/100 = \frac{7 \times 100}{\sqrt{13.45 \times 55.55}}$
 $= 25.60$

Coliform Index Test

In real terms it is not method of testing of Coliforms only used to represent the coliform in water in terms of parameter coliform index which can further be calibrated with those of pathogen present in water. Coliform index is defined as reciprocal of smallest quantity of sample which show the presence of coliform in it.

Note - MPN test & coliform index test are absolute.

<u>V (ml)</u>	<u>CI-path</u>	<u>c</u>
0.1	— 10	A
10.0	— 0.01	B
1	— 1	C
10	— 0.1	D

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98

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Treatment of Water

It is based upon the quality of raw water and treated water.

$$\text{TDS} = 2000 \text{ mg/lit} \rightarrow \text{WTP} \rightarrow 500 \text{ mg/lit}$$

$$\eta = \frac{2000 - 500}{2000} = 75\%$$

Treatment of water can be done by any of the following methods:

(i) Screening.

(ii) Aeration.

(iii) Coagulation.

(iv) Flocculation.

(v) Sedimentation.

(vi) Filtration.

(vii) Disinfection.

(viii) Softening of water.

(ix) De-salination.

(x) De-ferrisation.

(xi) De-fluoridation.

(xii) Fluoridation.

Screening

→ It is carried out to remove heavy solids [plants, animals, stones] from the water.

→ It is generally adopted for treatment of surface water.

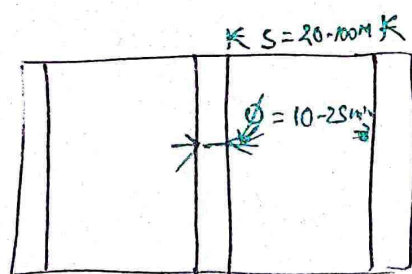
→ Screening is done with the help of screens.

Screens $\left\{ \begin{array}{l} \rightarrow \text{Coarse / Bar / Trash rack} \\ \rightarrow \text{Fine} \end{array} \right.$

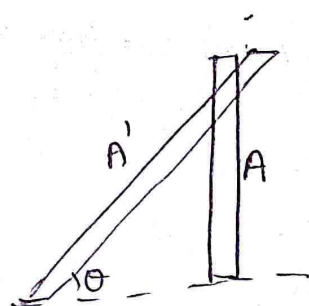
Coarse Screen

→ These are in the form of bars [$\phi = 10-25 \text{ mm}$, $S = 20-40 \text{ mm}$].

→ These screens are always placed at an inclination of $3-6^\circ$ as it effective cleaning of the screens by sinking mechanism moreover it also help in increasing the efficiency of screening process.



Trash $\rightarrow$ Impurities removed by rack $\rightarrow$ Trash rack.



$$A = PL \quad A' = HL$$

$$A' = A \quad \text{--- (i)}$$

$$\text{or } \sin \theta = \frac{A}{A'}$$

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

$$\boxed{\sin \theta < 1} \quad A' > A \quad \text{--- (ii)}$$

Area of flow increases by keeping screens inclined.

$\rightarrow$ When water passes through screen loss in heads takes place.

$$\boxed{h_L = \frac{K}{2g} [v^2 - u^2]}$$

K = constant that depend upon material of screen.

g = acceleration due to gravity.

u = velocity of approach.

v = velocity through screen.

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

Q. A trash screen is provided for treatment of 50000 m³/day of raw water, velocity through the screen is to be maintained at 0.8 m/s size of bars is 20mm and spacing is 60mm. Compute.

(1) Headloss loss through the screen.

(2) Net and gross area of the screen.

(3) If half of the screen is clogged due to leaves and debris again compute the head loss.

Assume any data not given.

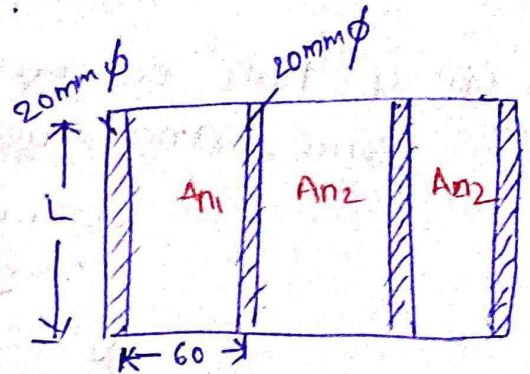
$$d = 20\text{mm}$$

$$S = 60\text{mm}$$

$$S_{cc} = S + \frac{d}{2} + \frac{d}{2}$$

$$= 60 + 20$$

$$= 80\text{mm.}$$



$$Q = 50000 \text{ m}^3/\text{day.}$$

$$V = 0.8 \text{ m/sec.}$$

$$A_{net} = \frac{Q}{V} = \frac{50000 \text{ m}^3/\text{day}}{0.8 \text{ m/sec}} = \frac{50000}{24 \times 60 \times 60 \times 0.8} = 0.723 \text{ m}^2.$$

$$A_{net} = S L n = 0.723 \text{ m}^2$$

$$A_T = S_{cc} L n$$

$$L n = \frac{0.723}{S} = \frac{0.723}{60}$$

$$A_T = S_{cc} L n$$

$$= 80 \times \frac{0.723}{60}$$

$$= 80 \times \frac{0.723}{60} = 0.964 \text{ m}^2$$

$$U = \frac{Q}{A_T} = \frac{50000}{0.964} = 0.6 \text{ m/s.}$$

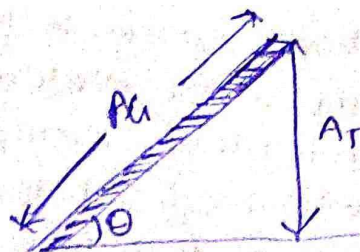
$$(i) h_L = \frac{K}{2g} (v^2 - U^2) = 0.0729 \times (0.8^2 - 0.6^2)$$

$$h_L = 0.0284 \text{ m}$$

(If) θ is 45°

$$\sin \theta = \frac{A_T}{A_h}$$

$$A_h = \frac{0.964}{\sin 45^\circ} = 1.363 \text{ m}^2$$



(iii) If half screen is clogged total area will remain same, And A_{net} will be half.

$U = \text{Constant}$,

$$U' = 2U$$

$$U' = 2 \times 0.8 = 1.6 \text{ m/s}$$

$$h_L' = \frac{K}{2g} [U'^2 - U^2]$$

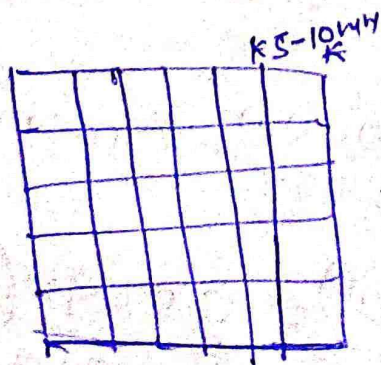
$$= 0.0729 [1.6^2 - 0.8^2]$$

$$= 0.16 \text{ m}$$

Vii) Fine Screen

→ These screen are in the form of wire mesh.

→ Under normal treatment condition, fine screen are generally avoided as they get frequently clogged, required their frequent cleaning which in turn increase the operational cost. Hence it is generally recommended to use coarse screen instead of fine screen and to remove the fine suspended particle in following Sedimentation and filtration process.

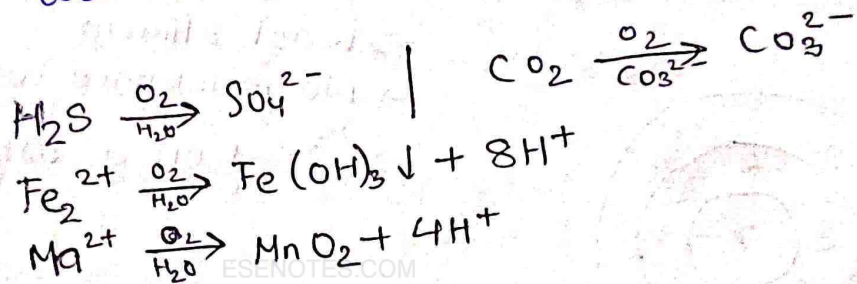


2. Aeration

In this process water is brought in intimate contact with air so it allows the absorption of O_2 in carry out the removal of following dissolved impurities.

- It remove dissolved gases like CO_2 , H_2S from the water.
- It remove dissolved organic matter from water.
- It remove volatile liquid like phenol & humic acid from water.
- It remove dissolved minerals, Fe, Mn
- It is generally adopted for the treatment of water which has deficiency of O_2 .

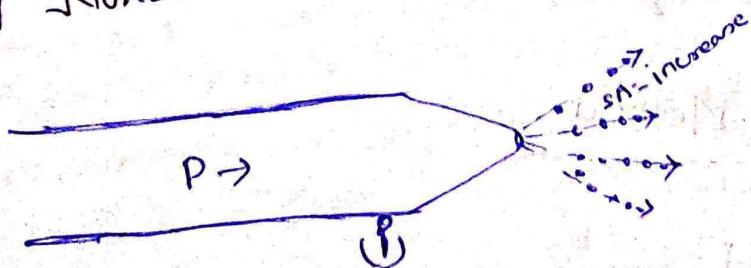
eg. ground water, water from bottom of lakes.



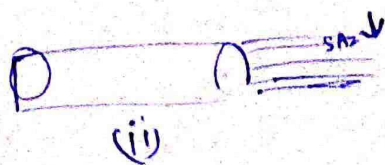
→ By aeration taste, colour, odour, pH reduces but turbidity of water increases.

⚡ Aeration can be done by any of the following Method:-

(i) Spray Nozzle method.



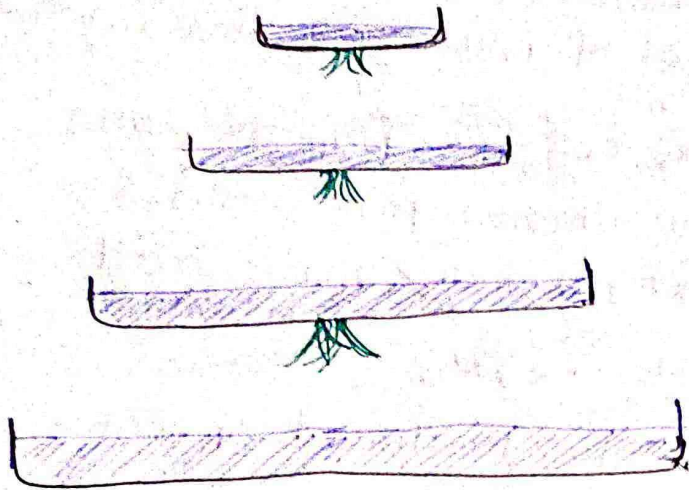
(Requirement of power is more.)
→ uneconomical.



(a) $V = \text{constant}$ (i) $SA \uparrow$ (ii) $SA \downarrow$
(b) $SA = \text{constant}$ $V \downarrow$ $V \uparrow$
eg top

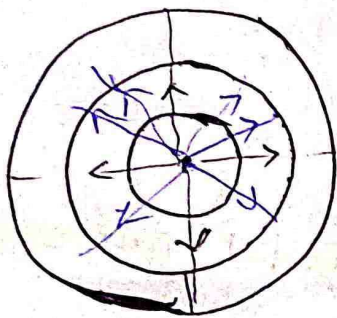
(ii) Tray Tower Method.

→ more area required
→ so it is not suitable
→ **Its efficient is more.**

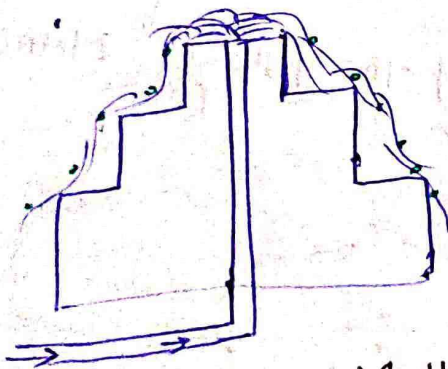


(iii) Cascade Aerator (Used for ~~less treatment of water ex. industry~~)

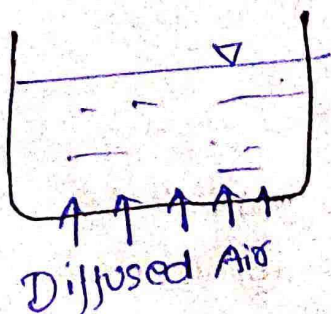
→ least efficient
→ No maintenance cost
→ Used on a large scale.



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(iv) Diffused Air Method.



(3) SEDIMENTATION

→ It is the process of removal of suspended particle from water.

→ The entire theory of sedimentation based on Specific Gravity. (G_s).

$$G_s = \frac{\text{wt. of solid of given volume.}}{\text{wt. of standard fluid of same volume.}}$$

water

$V_s = V_w$

$$G_s = \frac{W_s}{W_w} = \frac{W_s}{V_s} \times \frac{V_s}{W_w} = \left(\frac{W_s}{V_s} \right) \left(\frac{V_w}{W_w} \right)$$

$$G_s = \frac{\gamma_s}{\gamma_w} = \frac{\rho_s}{\rho_w}$$

$$G_s > 1$$

$$\gamma_s > \gamma_w$$

For inorganic solid, $G_s = 2.6 - 2.9$ (2.65)

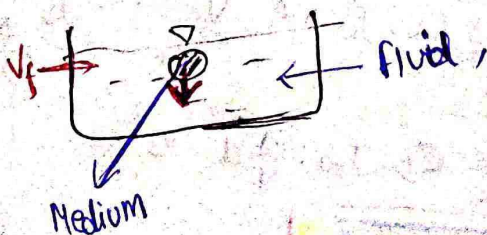
For organic solid, $G_s = 1 - 2$ (1.2)

$\gamma_s > \gamma_w$: Solid are heavier than water.

→ Water is in flowing condition, so due to turbulence no setting takes place.

→ During sedimentation which opposes the tendency of settlement are taken proper care of.

- (i) Turbulence/velocity of flow. $\downarrow$ - $\eta \uparrow$
- (ii) Viscosity $\downarrow$ $\eta \uparrow$



$$\gamma_{\text{liquid}} = f(\text{temp})$$

$$\gamma_{\text{liquid}} \propto \frac{1}{T}$$

$$\gamma_{\text{gases}} \propto T$$

(iii) Size of particles. (d)

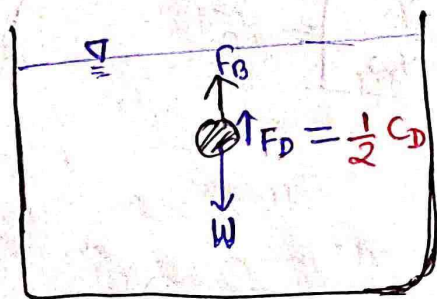
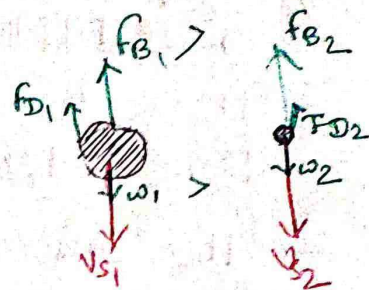
Stoke's law - valid ($d < 1\text{mm}$)

Assumption. of Stokes law:-

- i) Particle undergoing settlement is spherical.
- ii) Medium in which settlement is taking place is infinite.

Discrete settling takes place during settlement

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settling..



$$F_D = \frac{1}{2} C_D \rho_w A V_{sl}^2$$

$$V_{sl} = V_p - V_w$$

$$V_{sl} = V_p$$

C_D = coefficient of drag

$F_{0x}, V_p = V_s$ - terminal velocity / settling velocity.

gravity accⁿ, accelerating $a_p = 0$

$$\sum F_y = 0$$

$$W - F_B - F_D = 0$$

$$W - F_B = F_D$$

Boyant weight, $B_w = F_D$

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

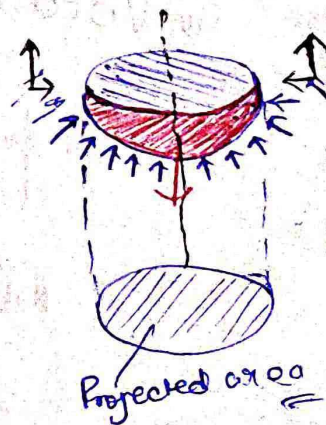
$$V_s \gamma_s - V_s \gamma_w = F_D$$

$$[V_w = V_s]$$

$$V_s \gamma_w \left(\frac{\gamma_s}{\gamma_w} - 1 \right) = \frac{1}{2} C_D \cdot \rho_w \cdot A \cdot V_s^2$$

$$\frac{\pi d^3}{6} \gamma_w \left(\frac{\gamma_s}{\gamma_w} - 1 \right) = \frac{1}{2} C_D \cdot \rho_w \cdot \frac{\pi}{4} d^2 \cdot V_s^2$$

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



(i) For laminar flow condition. ($d < 0.1 \text{ mm}$), ($Re < 1$)

$$C_D = \frac{24}{Re}, \quad Re = \frac{\text{Inertial force}}{\text{Viscous force.}}$$

$$Re = \frac{\rho V d}{\mu} = \frac{V d}{\nu}$$

$$\nu = \frac{\mu}{\rho}$$

$$V_s = \frac{(G-1) \gamma_w d^2}{18 \mu} = \frac{(G-1) g d^2}{18 \nu} \quad \nu = \text{kinematic viscosity } m^2/sec.$$

Settling velocity can also be written as.

$$V_s = 418 (s-1) \frac{d^2 (3T+70)}{100} \quad \text{mm/sec}$$

(ii) For Transition flow condition. ($0.1 < d < 1 \text{ mm}$)
 $1 < Re < 1000$

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

$$Re = \frac{\rho V_s d}{\mu}$$

Modified
Hazen formula.

$$V_s = 418 (s-1) \frac{d (3T+70)}{100}$$

(iii) For Turbulent flow Condition ($d > 1 \text{ mm}$)
 $Re > 10^4$

$$C_D = 0.4$$

$$V^2 = \frac{4}{3} \frac{g d (s-1)}{C_D}$$

$$V_s^2 = \frac{4}{3 \times 0.4} \times g d (s-1)$$

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

* Bigger the size of particle more the efficiency of settlement.

- * Viscosity & size of particle can be altered during operation.
- * Size of particle normally altered, alternation in viscosity needs extra cost.

→ Sedimentation is carried out in the units termed as Sedimentation tank.

Two types:-

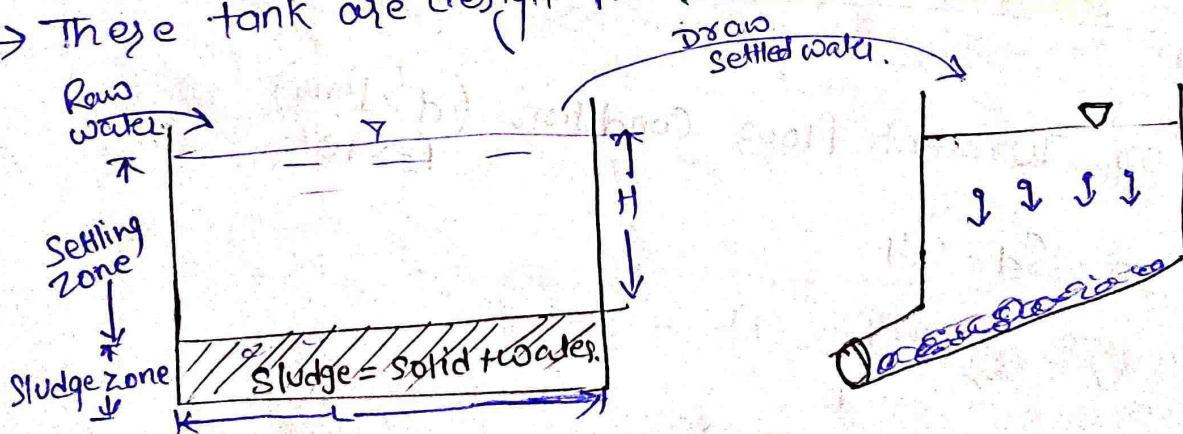
- Quiescent type / Fixed type / Fill & draw type.
- Continuous flow type:-
 - Horizontal flow type.
 - Vertical flow type.

* Quiescent type Sedimentation tank.

→ These are the type of tank in which the flow of water is completely stopped for 24 hours. during which the solid present in it get settled in the tank resulting in the formation of sludge which is further followed by the removal of settled water from the tank for the next treatment.

→ Detention time for these tank are 24 hours, cleaning period is 6-12 hours.

→ These tank are design for max^m daily demand.



$$Q_0 = \text{Design discharge} \\ = Q_{\max}$$

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

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

- * Minimum unit is depend upon detention time.
So minimum unit should be 3. Two for operation and one for standby.
- * May^m unit depend upon daily demand.

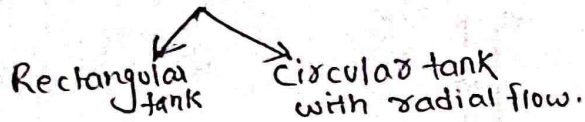
Continuous flow type.

→ In this type of tank in which flow of water is ensured to be continuous, for carrying out sedimentation.
(either in horizontal or vertical)

Type:-

a) Horizontal flow type.

b) Vertical flow/or upper flow type.



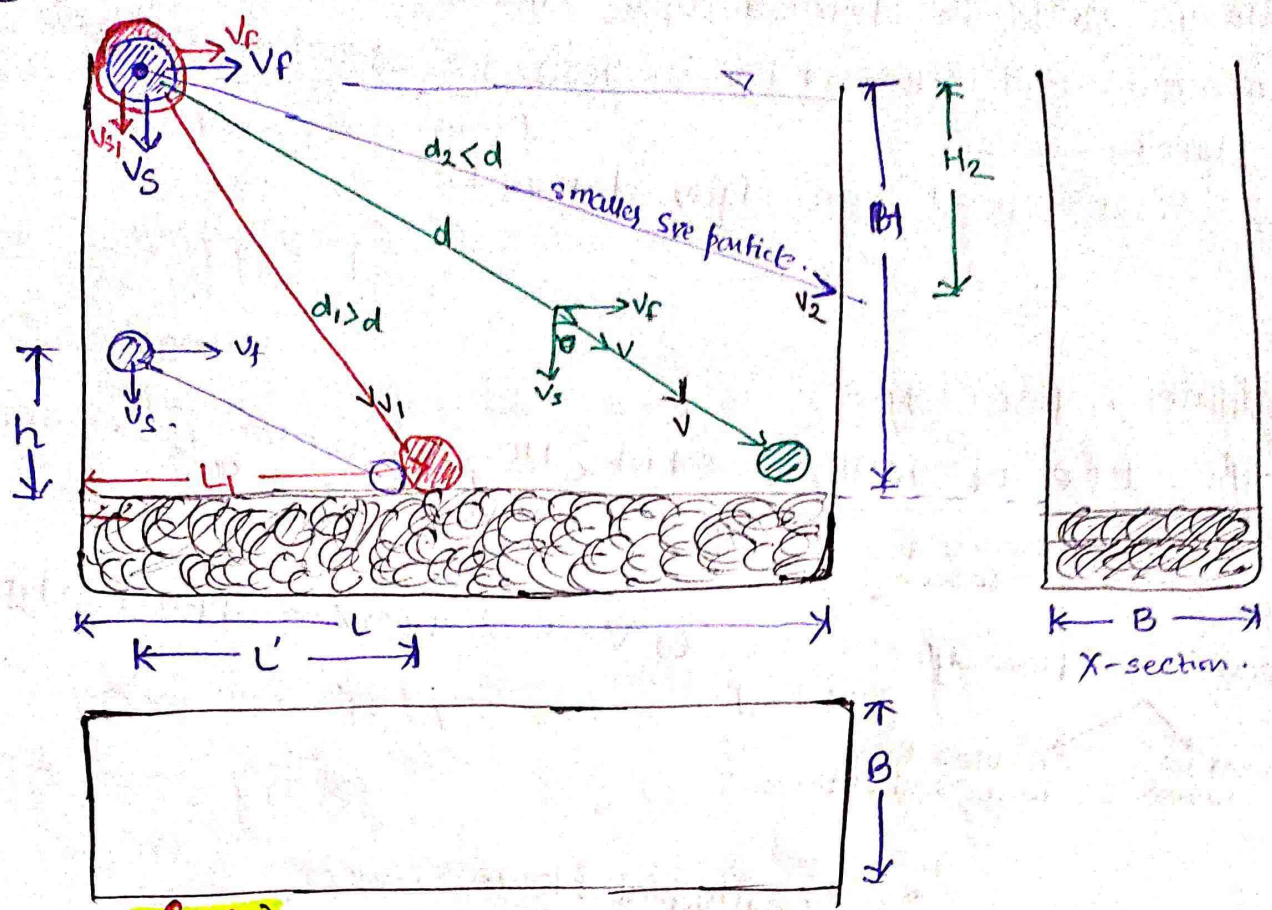
* Rectangular tank with Longitudinal Flow.

Assumption

- A particle is assumed to be settled in the tank if it reaches the bottom of the settling zone.
- Concentration of particle of all the sizes is same at all the points of the vertical section at the inlet end.

Size(mm)	Conc mg/lit	% R
6	200	100%
5	50	100%
5	250	100%
4	150	<100%
2	350	<100%
1	1000 mg/lit	

Conc'd size of particle



Width (H)

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

$t_d = \frac{H}{v_s}$

$v_f = \frac{Q_D}{\text{X-area}}$

$v_f = \frac{Q_D}{BH}$

$\frac{L}{v_f} = \frac{H}{v_s}$

$v_s = \frac{H v_f}{L}$

$v_s = \frac{H \times Q_D}{L BH}$

$v_s = \frac{Q_D}{LB}$

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

(ii) $d(h)$, $h < H$

$$h < H$$

$$t_d' = \frac{h}{V_s} , t_d = \frac{H}{V_s}$$

$$h < H$$

$$t_d' < t_d$$

⇒ 100% removal of concerned size of particles takes place in the tank.

$$L' = V_f t_d' , L = V_f t_d$$

$$L' < L$$

(iii) $d_1(H)$, $d_1 > H$, $V_{s1} > V_s$

$$t_{d1} = \frac{H}{V_{s1}} , t_d = \frac{H}{V_s}$$

$$V_{s1} > V_s$$

$$t_{d1} < t_d$$

→ If particle of greater size than concerned size of particle it will settle quickly than concerned size of particle

(iv) $d_1(h)$, $h < H$

$$t_{d1}' < t_{d1} < t_d$$

100% removal of bigger sized particle also takes place in the tank.

$$L_1 = V_f t_{d1}' < L = V_f t_d$$

(v) $d_2(h)$, $d_2 < d$, $v_{s2} < v_s$

Small particles

$$t_{d2} = \frac{H}{v_{s2}} \quad , \quad t_d = \frac{H}{v_s}$$

$$\boxed{t_{d2} > t_d}$$

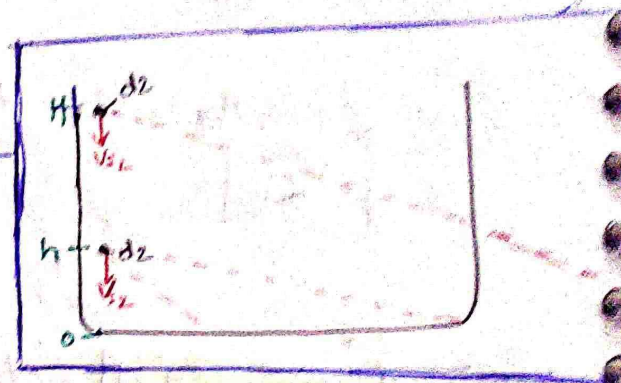
Or, $H_2 = v_{s2} \cdot t_d$, $H = v_{s2} \cdot t_{d2}$

($t_d < t_{d2}$)

$$\boxed{H_2 < H}$$

Or, $L = v_f \cdot t_d$, $L_2 = v_f \cdot t_{d2}$

$$\boxed{L_2 > L}$$



(vi) $d_2(h)$, $d_2 < d$, $v_{s2} < v_s$, $h < H$

$$t_{d2} = \frac{h}{v_{s2}} \quad , \quad t_d = \frac{H}{v_s}$$

→ For some $h \neq v_{s2}$ for which $t_{d2}' = t_d \Rightarrow \frac{h}{v_{s2}} = \frac{H}{v_s}$ - settle

→ For some $h \neq v_{s2} \Rightarrow t_{d2}' > t_d$ ($h > H$) - not settle

→ For some $h \neq v_{s2} \Rightarrow t_{d2}' < t_d$ ($h < H$) - settle

⇒ Smaller size particles are partially removed in sedimentation tank.

$$\eta(\%) = f(I, O)$$

$$\eta\% = \frac{I - O}{I} \times 100$$



% Removal of small size particle.

$$\% R = \frac{\left(\frac{W}{H}\right)h}{W} \times 100$$

$$\% R = \frac{h}{H} \times 100$$

$$\% R = \frac{V_{s2}}{V_s} \times 100 = \frac{(G-1) \gamma_w d_2^2}{18(G-1) \gamma_w d_1^2} \times 100 = \frac{d_2^2}{d_1^2}$$

$$\% R = \frac{d_2^2}{d_1^2} \times 100 \quad \left\{ \text{only if flow is laminar } (d < 9 \mu\text{m}) \right\}$$

Q

Size (mm)	Concentration	% R
6	200	100%
5	50	100%
(4)	250	100%
3	150	20%
1	350	15%

$$\eta(\%) = \frac{[(200+50+250) \times \frac{100}{100} + 150 \times \frac{20}{100} + \frac{15 \times 350}{100}] \times 100}{1000}$$

$$= 58.25\%$$

$$\eta\% = \sum_{i=1}^n \% C \times \% R$$

→ Settling velocity of concerned sized particle is termed,
by Surface Overflow Rate and SOR governs
governs the efficiency of sedimentation tank.

$$\eta \propto \frac{1}{\text{SOR}}$$

4) A tank having overflow rate of 2m/hr. is designed for treatment of flowing water sample

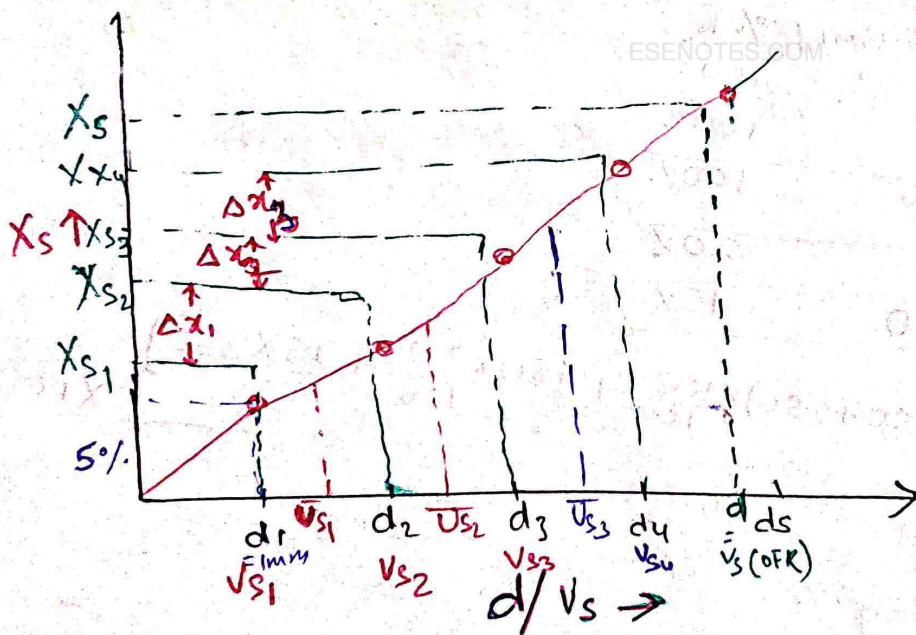
type of particle	velocity (m/hr) (v_s)	concentration (mg/lit)	%R
I	3	300	= 100%
II	2	250	= 100%
III	1	450	= 50%

Find the concentration of particle is settled water.
 (water that comes out of tank.)

$$\%R = \frac{v_{s2}}{v_s} \times 100 = \frac{1}{2} \times 100 = 50\%$$

$$450 \times \frac{50}{100} = 225 \text{ mg/lit}$$

* If all size of particles present in water.
 $X_s = \% \text{ finer} = \% \text{ of particles having size less than stated size / velocity less than stated velocity.}$



mg/lit	(mm)
200	6
50	5
250	4
350	2
100	1

$$\bar{v}_{s1} = \frac{v_{s1} + v_{s2}}{2}$$

$$\bar{v}_{s2} = \frac{v_{s3} + v_{s2}}{2}$$

$$\bar{v}_{s3} = \frac{v_{s4} + v_{s3}}{2}$$

$$\eta(\%) = \sum \% CX \% R$$

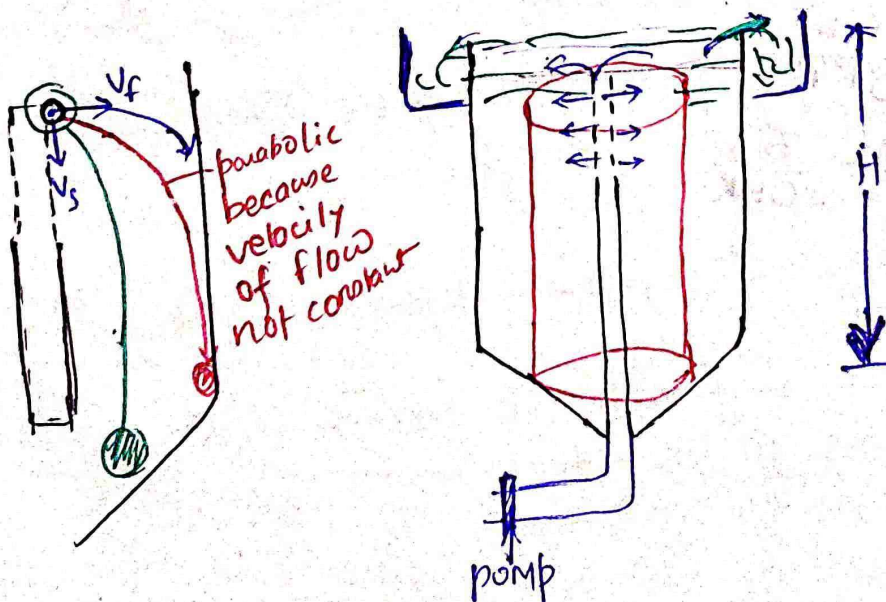
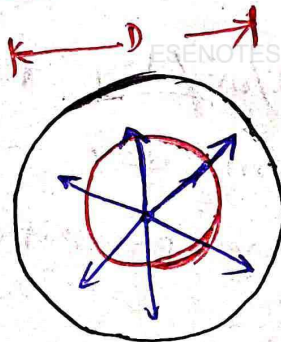
$$= (100 - X_s) \times 100 + \Delta x_1 \left(\frac{\overline{U_{s1}}}{V_s} \right) + \Delta x_2 \left(\frac{\overline{U_{s2}}}{V_s} \right) + \Delta x_3 \left(\frac{\overline{U_{s3}}}{V_s} \right) - \dots - \Delta x_4 \left(\frac{\overline{U_{s4}}}{V_s} \right)$$

$$\eta_i = (100 - X_s) + \sum_{i=1}^n \left(\frac{U_{si}}{V_s} \right) \Delta x_i$$

$$\overline{U_{si}} = \frac{V_{si} + V_{si+1}}{2}$$

$$\text{eg. } U_{st} = \frac{V_{s1} + V_{s2}}{2}$$

⑥ Circular tank with Radial flow.



* Efficiency of Circular tank is compared to ~~circ~~ rectangular tank.

→ In vertical flow type tank, settling velocity of particles remain constant during settlement but horizontal velocity of flow reduces as the particle move from center towards circumference due to increase in area ($2\pi rH$) hence the path followed by the particle during settlement in the tank is a parabolic path unlike the straight line path in the horizontal flow type tank.

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

Design data for horizontal flow type tank.

1) Over flow rate $\frac{12-18 \text{ m}^3/\text{m}^2/\text{day}}$

→ range = 12000 - 18000 $\text{l/m}^2/\text{day}$ for plain sedimentation

→ 24000 - 30000 $\text{l/m}^2/\text{day}$ for coagulation added sedimentation
(24-30 $\text{m}^3/\text{m}^2/\text{day}$)
 $\text{l/m}^2/\text{day}$ - Discharge per unit area.

$$\text{Plan area} = \frac{Q_D}{OFR}$$

$$\text{B.L OR } \frac{\pi D^2}{4} = \frac{Q_D}{OFR}$$

(ii) Detention time (t_d) - avg. theoretical time taken by water to travel from inlet to outlet.

range = 4 to 8 hours plain sedimentation.

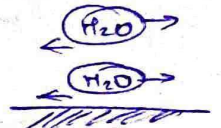
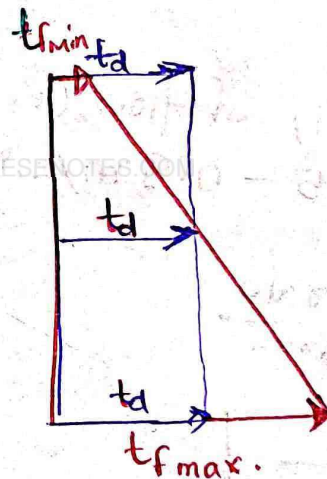
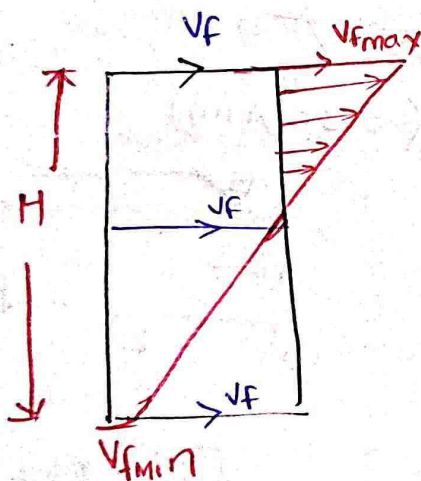
2 to 4 hours for coagulation aided sedimentation.

$$\boxed{\text{Volume} = Q_d \cdot t_d}$$

Volume = LBH - Rectangular tank

$$= \frac{\pi D^2}{4} (0.011D + 0.785H) - \text{Circular tank.}$$

$$H = \frac{\text{Volume}}{B \times L}$$



→ This process is known as short-cutting.

→ Flow-through period: Actual time available with water along the height of tank to travel along the length of tank

→ Impact of short-cutting is, it reduce efficiency of tank.

→ The theoretical detention time is calculated on the basis of avg velocity of flow. But in actual it is more in upper layer. ie results in lesser availability of actual time for particles to settle in upper layer hence these particles do not settle & result in reduced efficiency of tank this is known as "Short cutting".

→ Extent of which is measured in terms of displacement efficiency.

~~$\eta_d = \frac{\text{Theoretical detention}}{\text{Actual detention}}$~~

$$\eta_d = \frac{\text{Avg. flow through period } (t_{f \text{ avg}}) \times 100}{\text{The detention time } (t_d)}$$

Displacement Efficiency, $\eta_d = \frac{(t_{f \text{ max}} + t_{f \text{ min}})}{2} \times 100 > (30-35)\%$

$\eta_d \uparrow \rightarrow t_{f \text{ avg}} \uparrow \rightarrow t_{f \text{ min}} \uparrow \rightarrow \text{Short circuiting} \downarrow$

$$\eta_d \propto \frac{1}{sc}$$

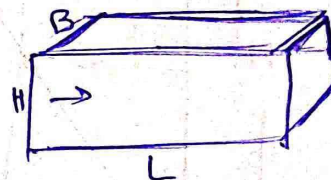
(iii) Horizontal velocity of flow, (Rectangular)
 $V_f = 0.15 - 0.9 \text{ m/min } (0.3 \text{ m/min})$

$$V_f = \frac{\text{flow rate}}{c/s \text{ area}}$$

$$V_H = \frac{Q_D}{B \times H}$$

$$V_H = \frac{L B H}{B \times H \times D_t}$$

$$L = V_H \cdot D_t$$



$$D_t = \frac{V}{Q_D}$$

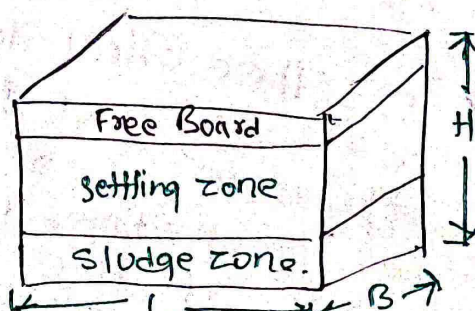
$$Q_D = \frac{V}{D_t} = \frac{L B H}{D_t}$$

(iv) $\frac{L}{B} = 4$

(v) $Q_D = Q_{MD}$

(vi) Free Board = $0.3 - 0.5 \text{ m}$

(vii) Sludge zone = $0.8 - 1.2 \text{ m}$.



Q Consider a plain sedimentation tank in a water treatment plant with $SOFR = 40 \text{ m}^3/\text{m}^2/\text{day}$. The dia of the spherical particle which will have 80% theoretical removal efficiency in the tank _____ μm . Assume Stokes law to be valid.
 $\rho_w = 1000 \text{ kg/m}^3$, $\rho_s = 2650 \text{ kg/m}^3$, $\nu = 1.01 \times 10^{-6} \text{ m}^2/\text{s}$.

~~SOFR = ^{settling velocity} v_s for concerned particle.~~
 ~~$= 40 \text{ m}^3/\text{m}^2/\text{day}$~~
 ~~$= \frac{40}{24 \times 60} \text{ m}^3/\text{m}^2/\text{sec}$~~

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

Soln

$$\% R = \frac{v_{s2}}{v_s} \times 100$$

$$80 \times v_s = v_{s2} \times 100$$

$$80 \times 40 = v_{s2}$$

~~$v_{s2} = 32 \text{ m}^3/\text{m}^2/\text{day}$~~

$$v_{s2} = \frac{80 \times 40}{100} \text{ m}^3/\text{m}^2/\text{day}$$

$$v_{s2} = \frac{80 \times 40}{100 \times 86400} \text{ m/sec}$$

$$= 3.7 \times 10^{-4} \text{ m/sec.}$$

$$v_{s2} = \frac{(G-1) \gamma_w d^2}{18 \mu} = \frac{(G-1) g d^2}{18 \nu}$$

$$\left(\nu = \frac{\mu}{\rho} \right)$$

$$3.7 \times 10^{-4} = \frac{(2.65-1) 9.81 \times d^2}{18 \times 1.01 \times 10^{-6}}$$

$$d = 2.187 \times 10^{-5} \text{ m}$$

$$d = 21.27 \times 10^{-6} \text{ m}$$

$$= d = 21.27 \mu\text{m.}$$

A rectangular sedimentation basin is to handle 10MLD of raw water. A detention basin of ~~length~~ width to length ratio of 1:3 is proposed to trap all particles larger than 0.04mm in size assuming a relative density of 2.65 for the particles at 20°C temp as the avg. temperature compute the basin dimension. If the depth of tank is 3.5m. calculate detention time.

$$Q_D = 10 \text{ MLD}$$

$$= 10 \times 10^6 \text{ L/day}$$

$$= \frac{10 \times 10^6 \times 10^{-3}}{24 \times 60 \times 60}$$

$$Q_D = 0.1157 \text{ m}^3/\text{s}$$

$$\frac{B}{L} = \frac{1}{3}$$

$$3B = L$$

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

$$= 418 (2.65-1) \left(\frac{0.04}{1000} \right)^2 \times \left(\frac{3 \times 20 + 70}{100} \right)$$

$$= \cancel{2.304 \times 10^{-6} \text{ m/sec}}$$

$$= \cancel{0.304 \text{ mm/sec}}$$

$$= 1.43 \text{ mm/sec}$$

$$= 1.43 \times 10^{-3} \text{ m/sec}$$

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

$$\text{plan area} = \frac{Q_D}{V_s} = \frac{0.1157}{1.43 \times 10^{-3}}$$

$$= 80.909 \text{ m}^2 \text{ (i)}$$

$$(B \times L) = 80.909$$

$$(B \times 3B) = 80.909$$

$$3B^2 = 80.909$$

$$B = 5.19 \text{ m}$$

$$L = 15.57 \text{ m}$$

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

$$\frac{H}{V_s} = D_t = \frac{V}{Q_D} = \frac{5.19 \times 15.57 \times 3.5}{10 \times 10^6 \times 10^{-3}} = 0.028 \text{ day}$$

$$= 40.72 \text{ min}$$

Q A settling basin is design to have surface overflow rate of 32.6 m/day. Determine the overall removal obtain for a suspension with size distribution as follow.
 $G = 1.2$ and water temp. is 20°C at which dynamic viscosity = 1.027 centipoise, $\rho_w = 0.997 \text{ gm/cm}^3$.
~~particle size~~ $\frac{NS}{m^2}$

Particle size (mm)	0.1	0.08	0.07	0.06	0.04	0.02	0.01
Weight fraction greater in size (%)	10	15	40	70	93	99	100

If all size of particles present in water

$$\eta = (100 - X_s) + \sum \left(\frac{V_{si}}{V_s} \right) X \Delta x_i$$

Size (mm)	wt. fraction smaller in size (%)	V_s (mm/sec)
0.1	90	1.0581
0.08	85	0.677
0.07	60	0.51
0.06	30	0.38
0.04	7	0.17
0.02	1	0.04
0.01	0	0.01

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

$$V_s = \frac{(1.2-1) \times 0.997 \times 10^3 \times 10^{-6}}{18 \times 1.027 \times 10^{-2} \times 0.01}$$

$$V_s = 105.81 \text{ mm/sec}$$

$$\text{Over flow rate} = 32.6 \text{ m/day} = \frac{32.6 \times 10^3}{86400} = 0.377 \approx 0.38 \text{ mm/s}$$

$$X_s = 30\%$$

$$\Delta x_1 = 1 - 0 = 1\% ; \bar{V}_{s1} = \frac{V_{s1} + V_{s2}}{2} = \frac{0.01 + 0.04}{2} = 0.025 \text{ mm/sec}$$

$$\Delta x_2 = 7 - 1 = 6\% ; \bar{V}_{s2} = \frac{0.04 + 0.17}{2} = 0.105 \text{ mm/sec}$$

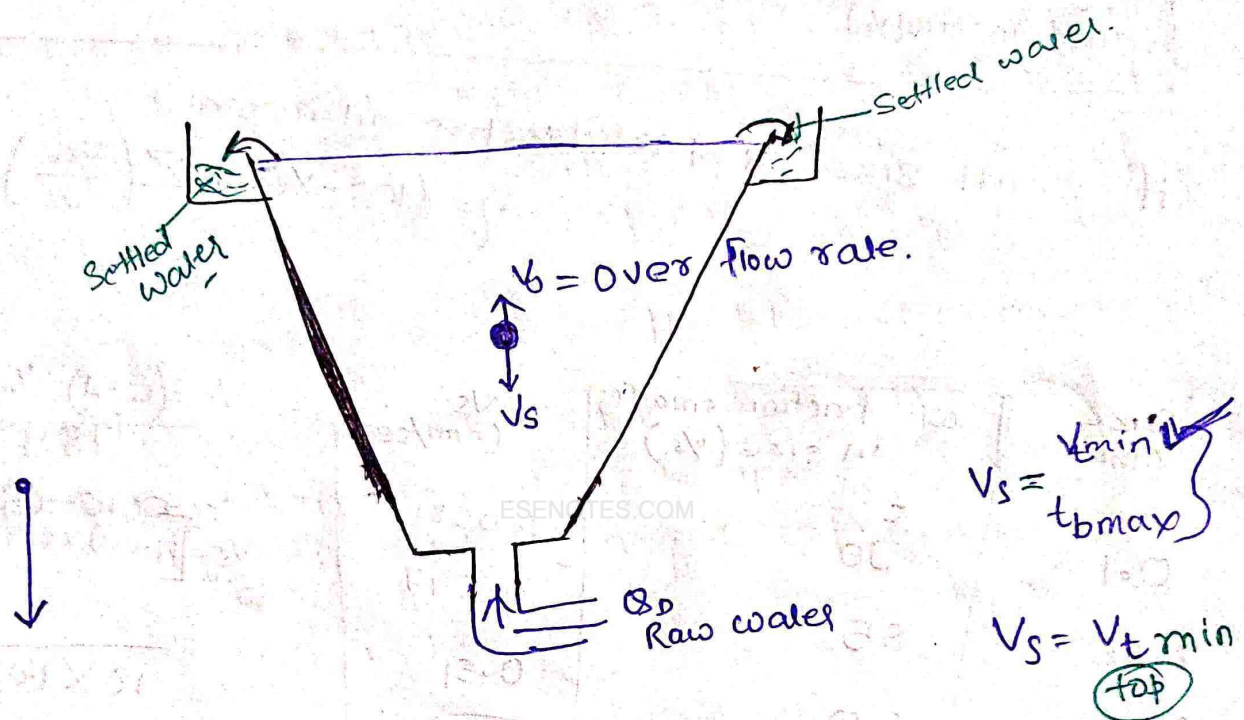
$$\Delta x_3 = 30 - 7 = 23\% ; \bar{V}_{s3} = \frac{0.38 + 0.17}{2} = 0.275 \text{ mm/sec}$$

$$\eta(\%) = (100 - x_s) + 1 \times \left(\frac{0.025}{0.38} \right) + 6 \times \left(\frac{0.105}{0.38} \right) + 23 \times \left(\frac{0.275}{0.38} \right)$$

$$= (100 - 30) + 18.467$$

$$= 88.46\%$$

☀ VERTICAL Flow Type Tank



Coagulation Added Sedimentation.

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

- (i) Coagulation. (fast mixing)
- (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 μ m 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 stability is due to their very small size.

→ Surface phenomena predominate over mass phenomena.

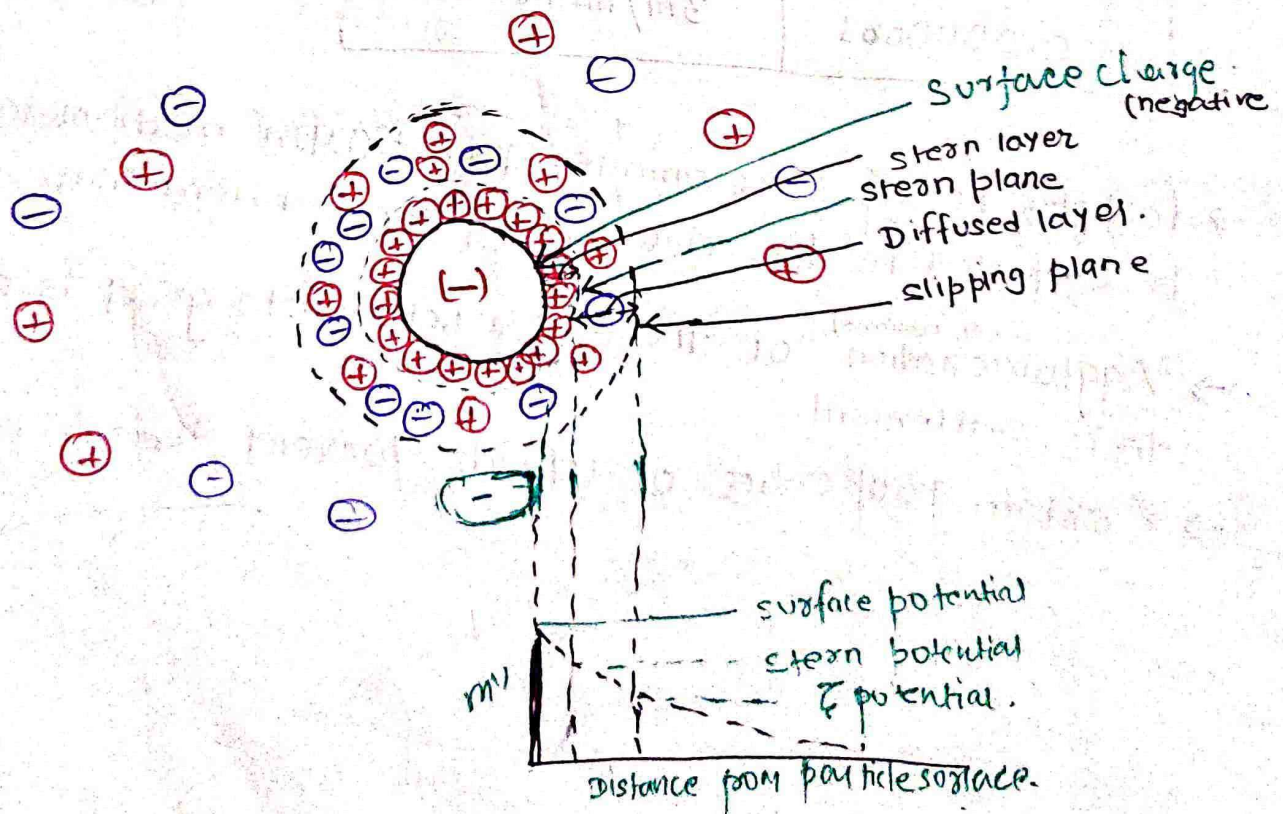
$$\begin{aligned} d \downarrow &\rightarrow SSA \uparrow \uparrow \uparrow \rightarrow G \downarrow \\ d \uparrow &\rightarrow SSA \downarrow \rightarrow G \uparrow \end{aligned}$$

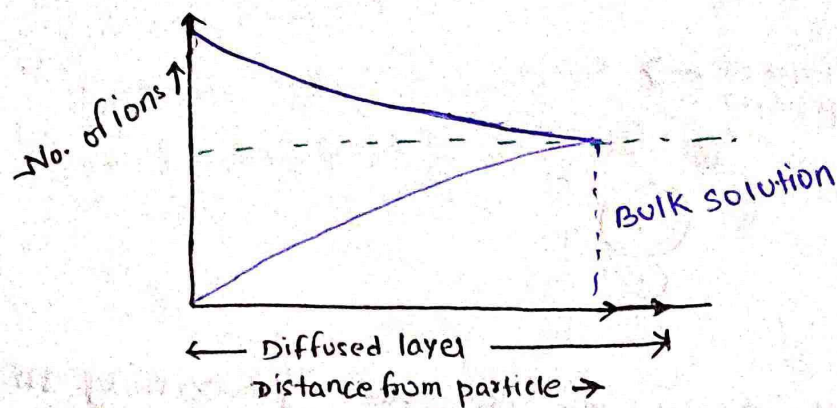
→ The most important surface phenomenon is the accumulation of electrical charges at particle surface.

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

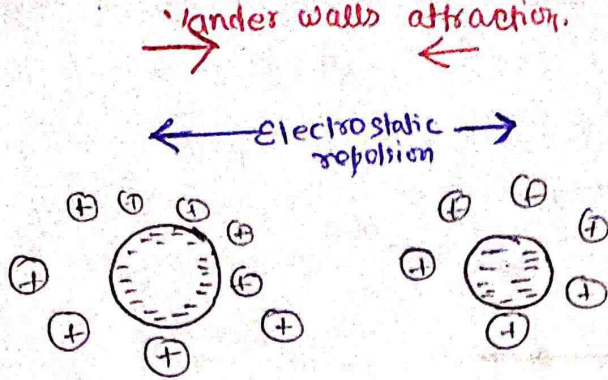
→ In most surface water, colloidal surface are negatively charged.

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





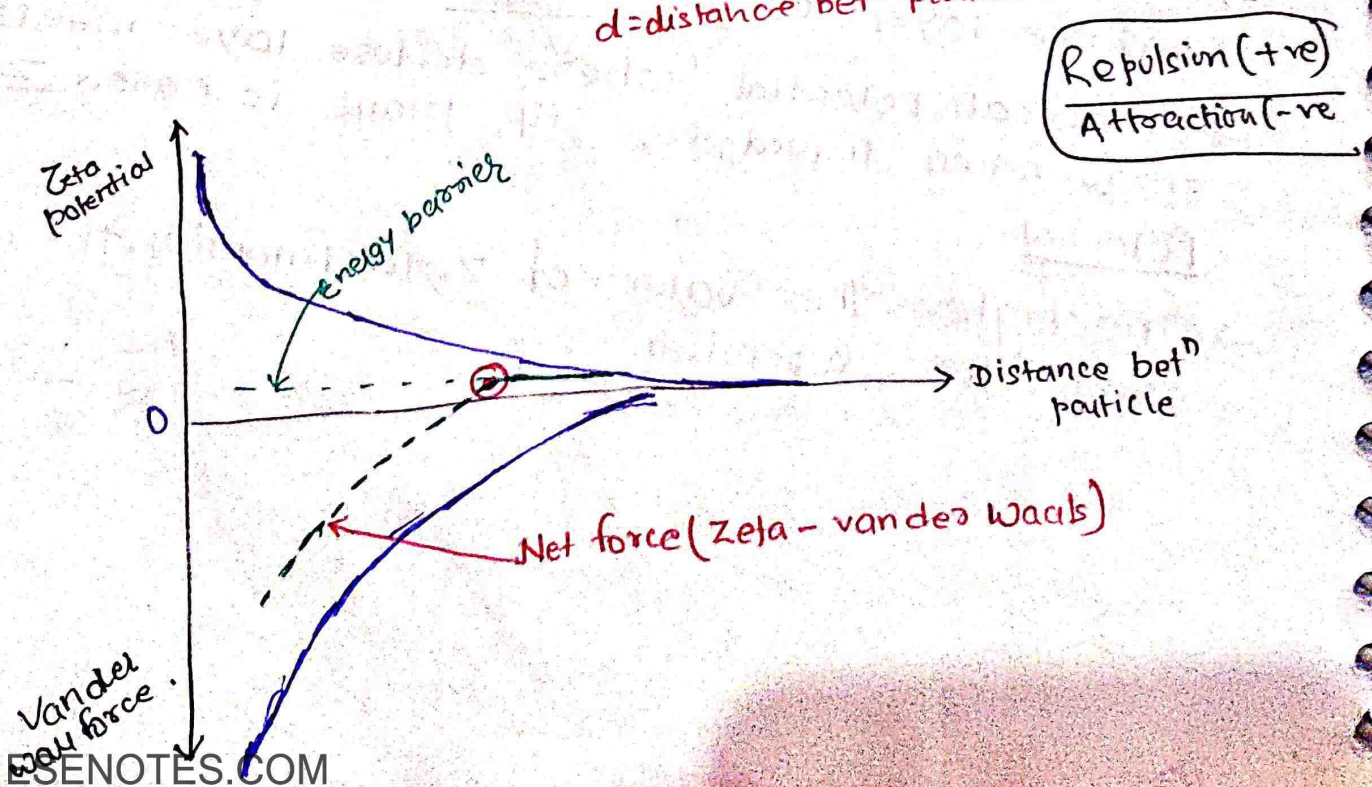
- Positive counter ions first attached to negatively charged particle, forming a rigid layer called STERN LAYER/BOUND.
- The particle continues to attract more counter ions but now these (counter⁺ ions) are being repelled by other counter-ions in the vicinity and STERN LAYER itself.
- The second region 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 i.e., 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 WAALS FORCE, which support contraction.
- Both zeta Potential and VANDER WAALS 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 THRESHOLD 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 impart enough momentum to larger particles to move them across energy barrier.

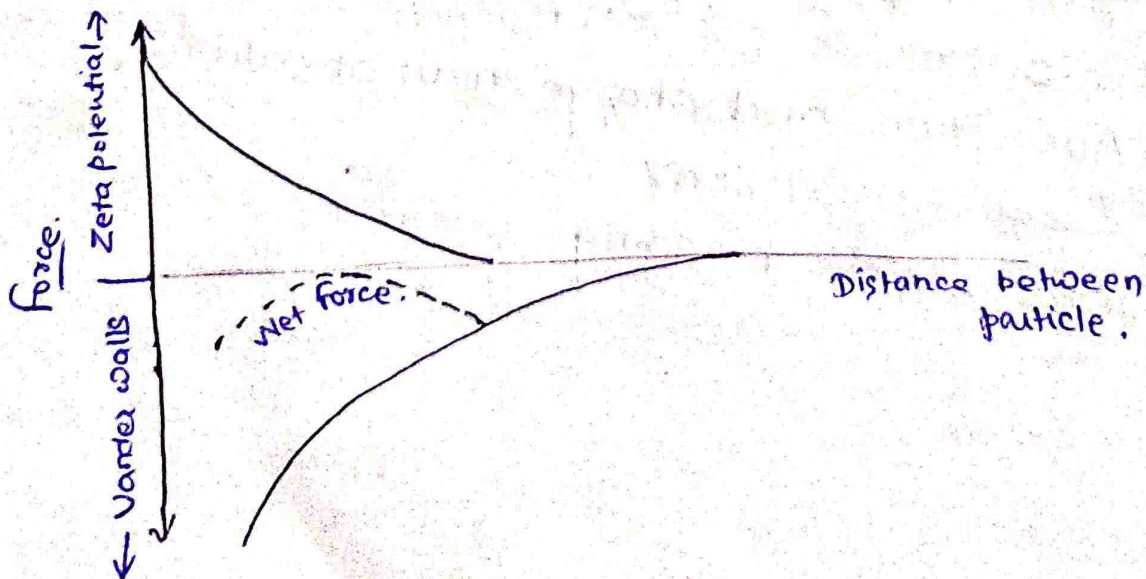
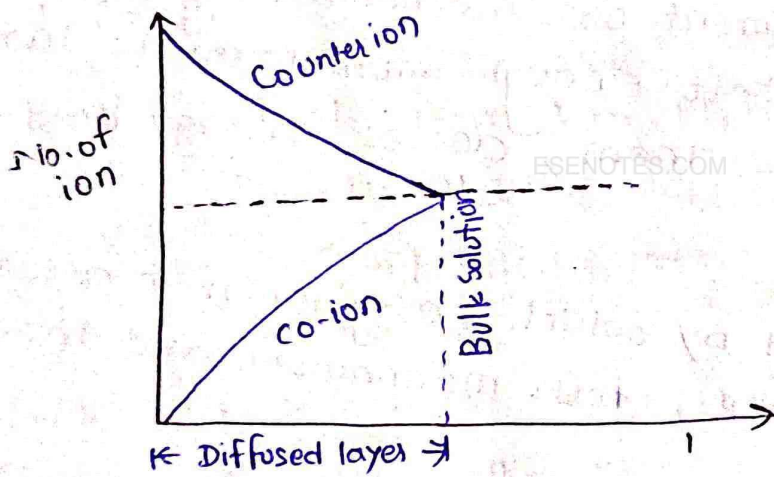
Coagulation theory:-

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

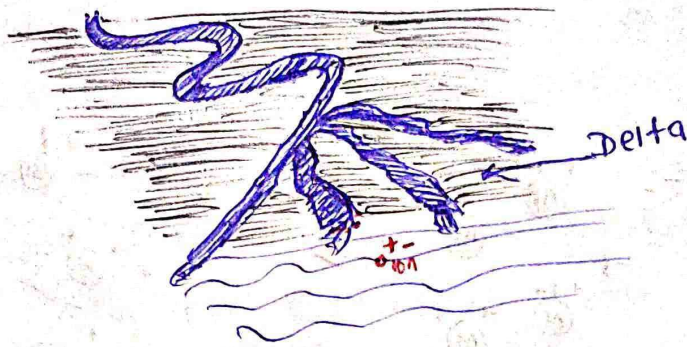
- (i) Ionic layer compression.
- (ii) Absorption and charge neutralisation.
- (iii) Sweep coagulation.
- (iv) Interparticle bridging.

1. Ionic layer Compression. - 10m quantity

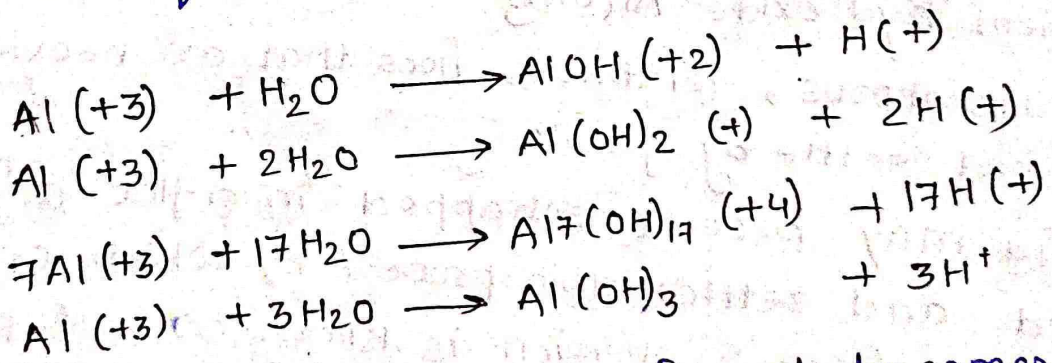
- The quantity of ions in water surrounding the colloid has effect on the decay of electrostatic potential.
- A high ionic concentration compresses the double layer (layers composed predominantly of counterions) towards the surface of colloid.
- If this layer is sufficiently compressed, then the VAN DER WAALS 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.



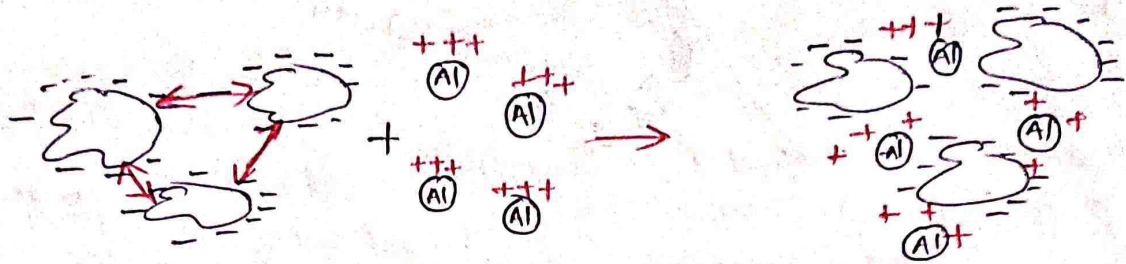
- Adsorption and charge Neutralisation.
 → Here nature, rather than the quantity of the ions is of prime importance
 → If alum (aluminium sulphate) is used as a coagulant $Al(+3)$ cations immediately react with water to form a variety of aquometallic ions and hydrogen ion.



- These aquometallic ions ~~formed~~ formed, become 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 disappears so that contact occurs freely.

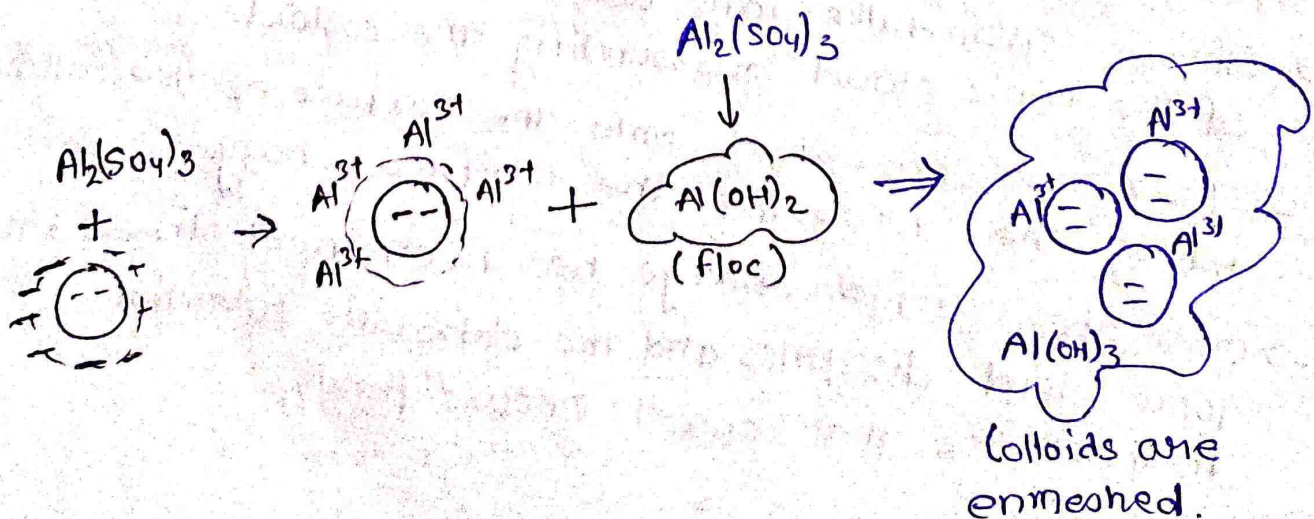
Note: In this overdosing with coagulant can result in restabilizing of the suspension.

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



Sweep Coagulation

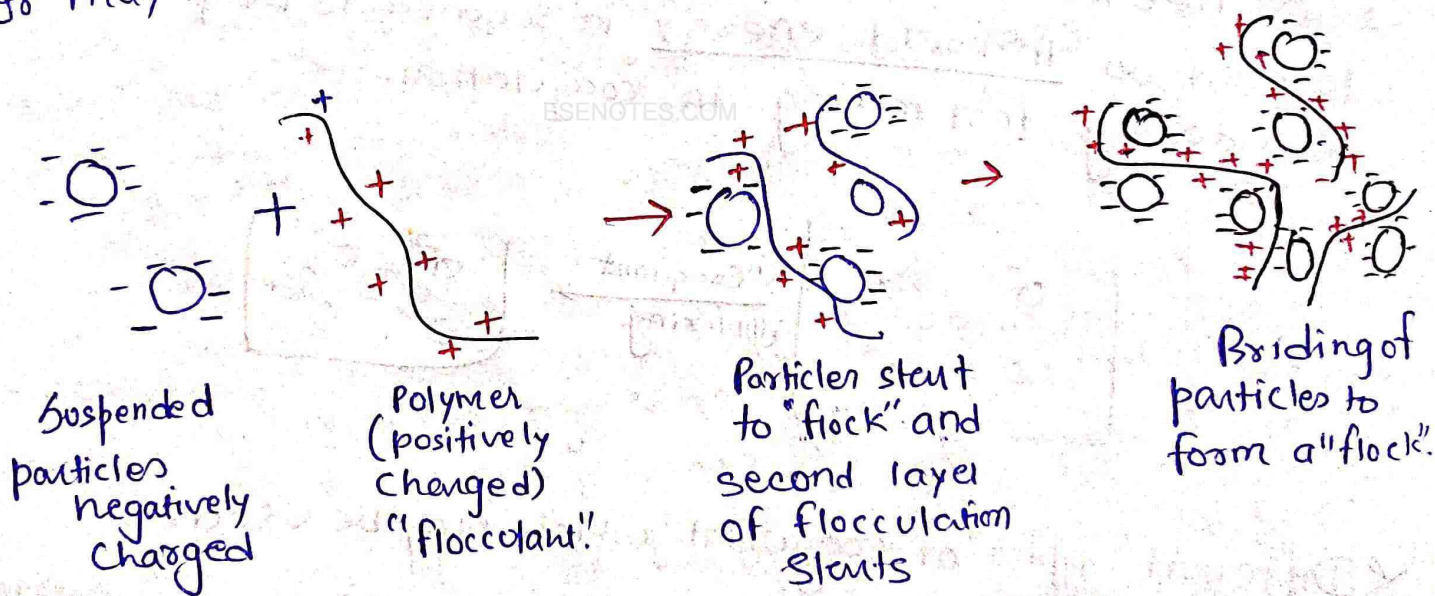
- The last product formed in hydrolysis of alum is aluminium hydroxide $\text{Al}(\text{OH})_3$
- It is amorphous, gelatinous flocs that are heavier than water and settle by gravity.
- Colloid may become entrapped in a floc as it is formed and settles, this process by which colloids are swept from suspension is known as sweep coagulation.



Interparticle Bridging.

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

- Synthetic polymers may also be used ~~there~~ 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.



Coagulation Added Sedimentation.

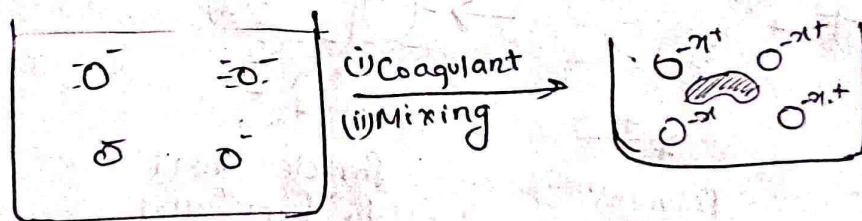
$$\text{I} \quad \frac{\overset{\text{Coagulation}}{\eta(I)}}{SA = I} > \frac{\eta(II)}{SA = II}$$

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

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.

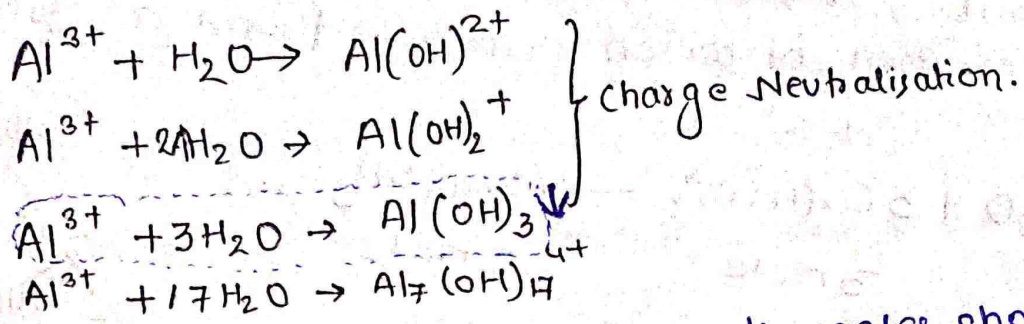
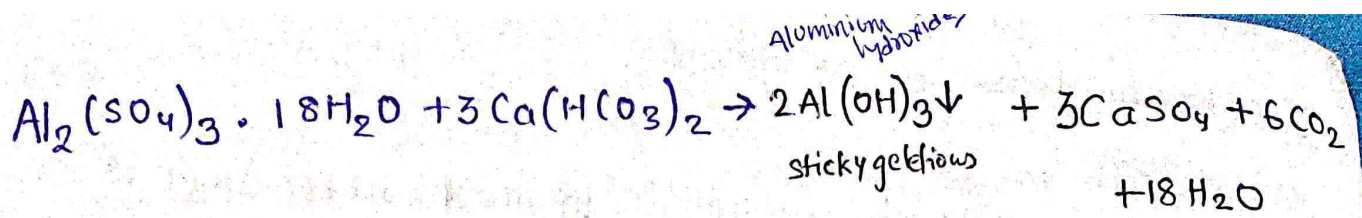
→ To neutralise the charge certain minimum energy termed as threshold energy is required to be provided by inducing fast mixing in coagulation.



* Different types of coagulant which may be used:-

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

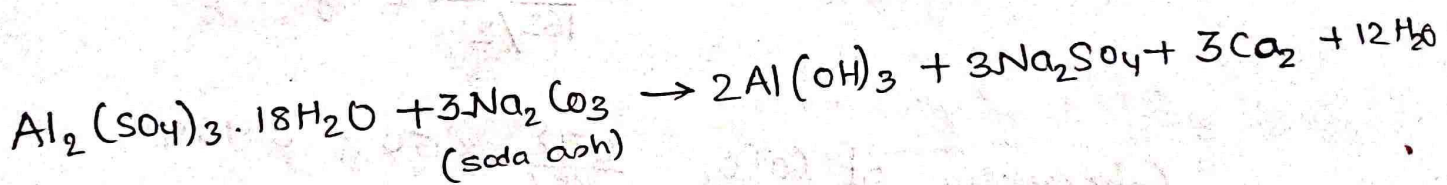
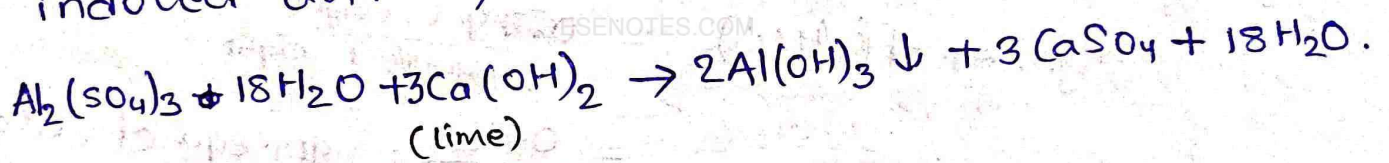
Alum reacts with alkalinity present in water and results in formation of sticky gelatinous precipitate of aluminium hydroxide which attract fine suspended impurities over its surface, thus grow in size and gets easily settled in following sedimentation process.



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.



→ Alum works in pH range of 6.5 - 8.5

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

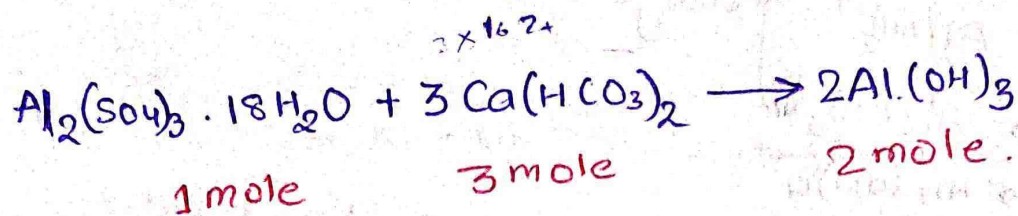
→ Does of alum depend upon turbidity and colour of the water. and it is determine using Jar test.

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

Optimum dose of coagulant

~~Optimum~~

→ 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:-



1 mole

3 mole

2 mole

666 gm

486 gm

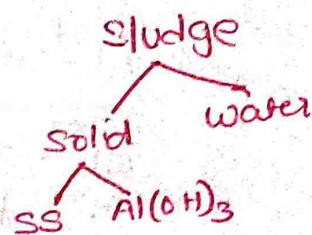
156 gm

1 gm

$$\frac{486}{666} = 0.72$$

0.234 gm

0.45 gm as CaCO_3



$$1 \text{ gm eq. of } \text{Ca}(\text{HCO}_3)_2 = 1 \text{ gmeq. of } \text{CaCO}_3$$

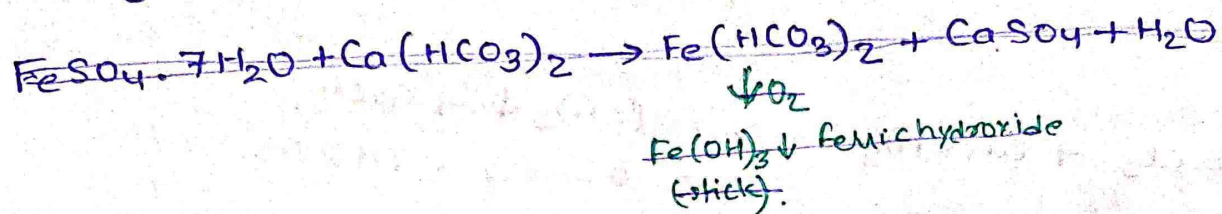
$$\text{gm eq. of } \text{Ca}(\text{HCO}_3)_2 = \frac{\text{given weight}}{\text{equivalent weight}}$$

$$= \frac{0.72}{162/2} = \text{gm. eq. of } \text{CaCO}_3$$

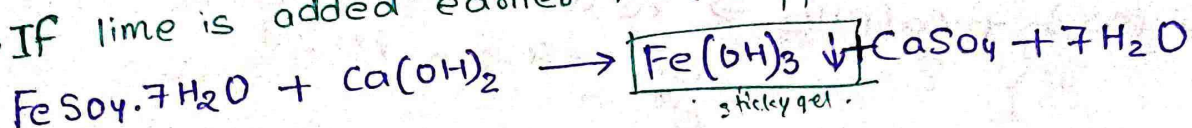
$$\text{Concentration of } \text{CaCO}_3 = \frac{0.72}{62/2} \times 50 = 0.45 \text{ gm}$$

2) Copperas ($\text{FeSO}_4 \cdot 7\text{H}_2\text{O}$) [Hydrated Ferrous Sulphate]

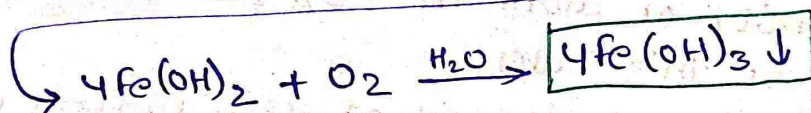
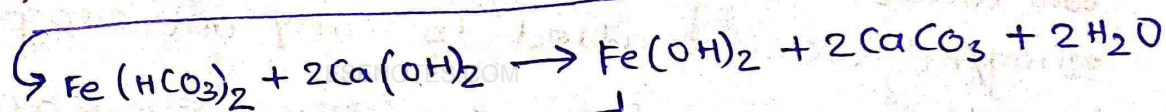
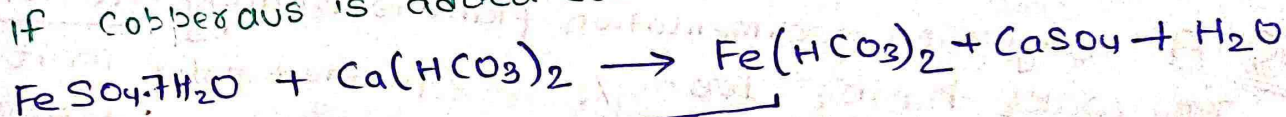
→ It also reacts with alkalinity present in water & result in the formation of sticky gelatinous precipitate of $\text{Fe}(\text{HCO}_3)_2$ which attracts fine suspended impurities over its surface and gets easily settled in following sedimentation tank.



→ If lime is added earlier than Copperas.



→ If Copperas is added earlier than lime.



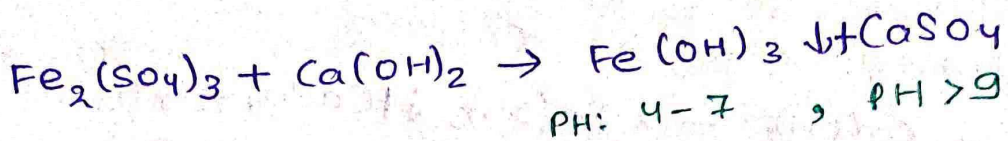
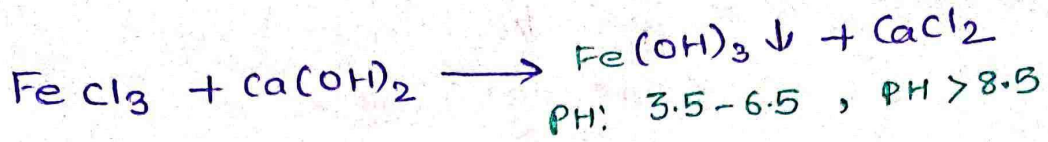
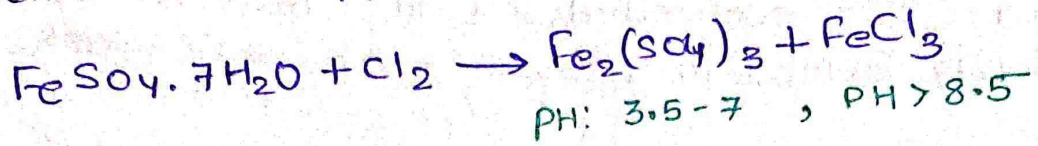
→ It is not used for treatment of coloured water because it itself impart colour in water.

→ Its normal dose 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 conjunction with lime.

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

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



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: Quantity of alum in 85% pure CaO needed

Compute:

① Annual Quantity of alum in 85% pure CaO needed for treatment of this water.

for treatment of this water.

11. Volume of sludge produced at the bottom of Sedimentation tank in a day. Assume efficiency of sedimentation tank 80%, $G_{\text{sludge}} = 1.2$, solid content = 2%.

III change in Ionic character of water in a day.
Assume raw water has alkalinity of 4mg/lit as CaCO_3 .

$1 \text{ ton} = 1000 \text{ kg}$
 $1 \text{ kg} = 1000 \text{ g}$
 $1 \text{ g} = 1000 \text{ mg}$

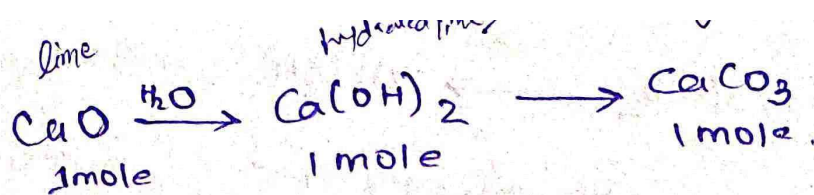
Quantity of alum required = $30 \times 40 \times 10^6 \times 365 \times 10^{-9}$
= 438 tons/year

$\frac{1 \text{ gm}}{30 \text{ mg/lit}}$ of CaCl_2 reacts with 0.45 gm of alkalinity as CaCO_3

" " = 0.45×30

" " = 13.5 mg/lit as CaCO_3

Additional alkalinity required = $13.5 - 4 \text{ mg/l as CaCO}_3$
 = $9.5 \text{ mg/l as CaCO}_3$



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

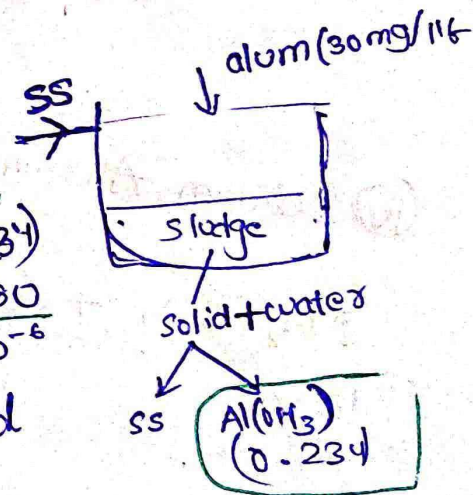
$$\begin{aligned} 100 \text{ gm} & \xrightarrow{\quad\quad\quad} 56 \text{ gm} \\ 9.5 \text{ mg/lit of } \text{CaCO}_3 \text{ alkalinity is introduced by} &= \frac{56}{100} \times 9.5 \\ &= 5.32 \text{ mg/lit} \\ &\text{of 100\% CaO.} \end{aligned}$$

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

$$\begin{aligned} \text{Quantity of 85\% pure CaO} &= 6.26 \times 40 \times 10^6 \times 365 \times 10^{-9} \\ &= 91.396 \text{ tons/year} \end{aligned}$$

III Volume of sludge

$$\begin{aligned} \text{Wt. of solid settle to form sludge} &= (100 + 30 \times 0.234) \\ &\quad \times 0.80 \\ &\quad \times 40 \times 10^6 \times 10^{-6} \\ &= 3424.64 \text{ kg/d} \end{aligned}$$



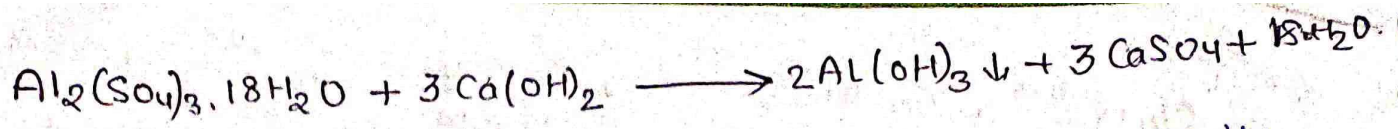
$$\text{Sludge} = \text{solid} + \text{water}$$

$$\begin{aligned} 2 \text{ kg solid} + 98 \text{ kg water} &= 100 \text{ kg of sludge} \\ S = 2\% \end{aligned}$$

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

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

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



1 mole of alum reacts with 0.45 gm of CaCO_3 alkalinity.
 $30 \text{ mg/L} \xrightarrow{\quad\quad\quad} 0.45 \times 30 = 13.5 \text{ mg/Lit}$

Alkalinity of water will be reduced by $= 13.5 \times 40 \times 10^6 \times 10^{-6}$
 $= 540 \text{ kg/day as } \text{CaCO}_3$

1 mole of alum gives 3 mole of CaSO_4 hardness as CaCO_3 ^{3 mole}

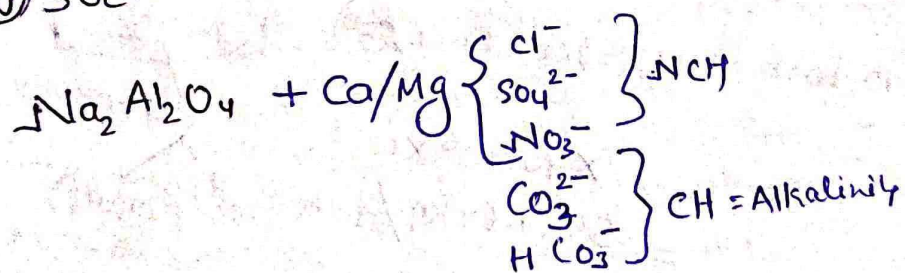
$666 \text{ gm} \xrightarrow{\quad\quad\quad} 300 \text{ gm as } \text{CaCO}_3$

$1 \text{ gm} \xrightarrow{\quad\quad\quad} \frac{300}{666} = 0.45 \text{ gm as } \text{CaCO}_3$

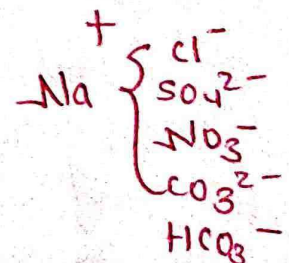
Permanent hardness of water will be increased by

$$= 0.45 \times 40 \times 40 \times 10^6 \times 10^{-6} = 540 \text{ kg/day}$$

(IV) Sodium Aluminate ($\text{Na}_2\text{Al}_2\text{O}_4$)



$\left\{ \begin{array}{l} \text{CO}_3^{2-} \\ \text{HCO}_3^- \end{array} \right\} \text{CH} = \text{Alkalinity}$



→ 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 dose is $(10-30) \text{ mg/lit}$ 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 aluminium salt.
- (iii) Iron flocs are heavier than aluminium floc.
- (iv) Iron flocks are formed earlier than aluminium floc.

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

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Drawback

Use of iron salts causes pitting & tuberculation in pipes.

Pitting → corrosion

tuberculation → Incrustation.
(deposition of solid)



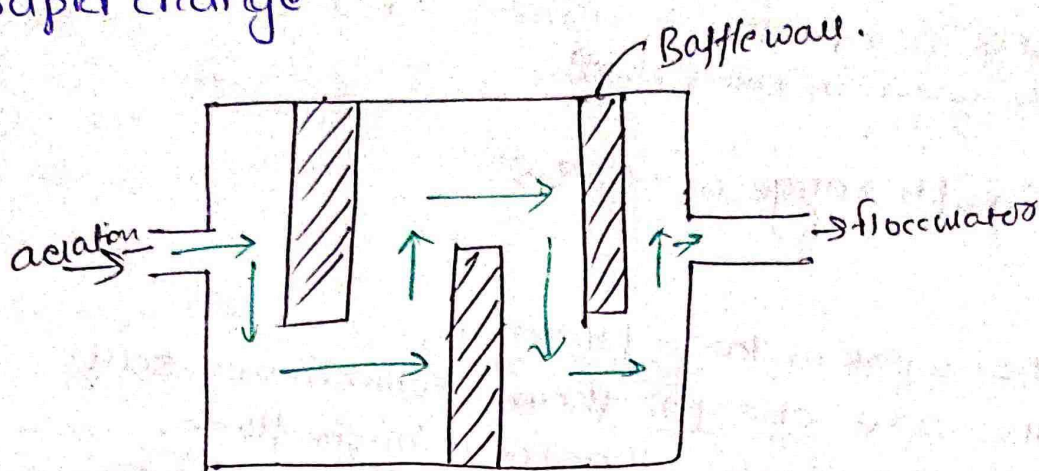
→ To neutralise the charge or for particles to combine minimum energy is required to be given termed as threshold energy. which is provided by ~~inducing~~ inducing fast mixing in the process in any of the following forms.

(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

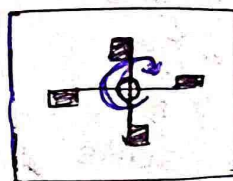
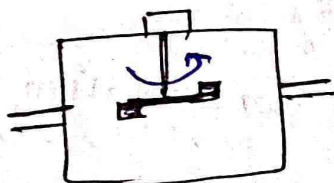
In mixing basin, mixing is induced by generating the turbulence which is achieved by bringing the rapid change in the direction of flow of water.



Plan: A & E
X-X: 0.8 V

(ii) Mechanical Mixer

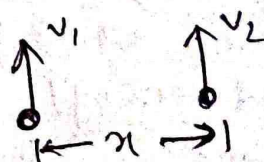
- Horizontal shaft type
- Vertical shaft type.



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

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

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



$$\text{Unit} = \text{m/m/sec} \\ = \text{sec}^{-1}$$

$$G = f(P, M, v)$$

$$G \propto \frac{P}{M v}$$

$$G = P^a M^b v^c$$

By dimension analysis.

$$G = \sqrt{\frac{P}{M v}}$$

$$a = \frac{1}{2}$$

$$b = c = -\frac{1}{2}$$

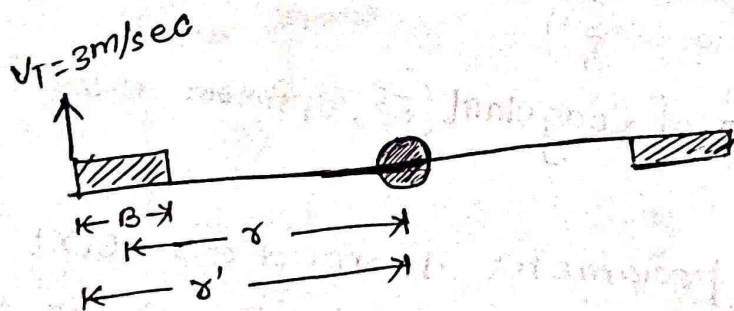
→ Depth to width ratio = 1:1 - 3:1 = $\frac{4}{B \text{ or } D}$

→ Detention time = 30-60 sec.

→ $G > 300 \text{ sec}^{-1}$

→ Power supply to shaft = 1-3 watts/(m³/hr) = 1-3 watt/discharge

→ Speed of shaft should be velocity at tip of paddle is more than 3m/sec. = V_T



$$V_T = \omega r'$$

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

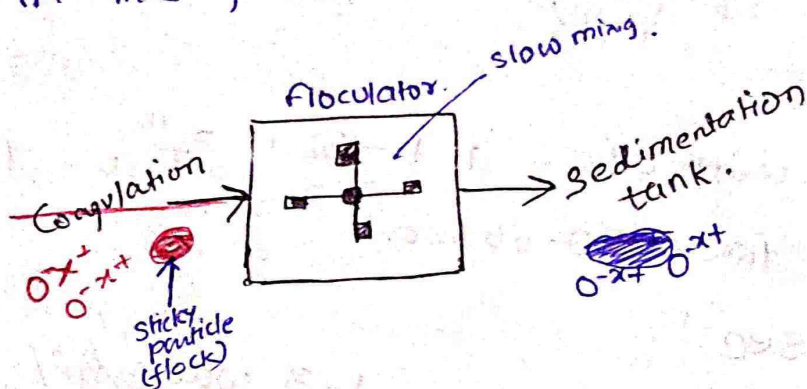
$$\omega = \frac{2\pi N}{60}$$

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

II FLOCCULATION (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.

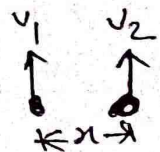


Rate of flocculation depend upon following parameters:-

- (i) Turbidity. $\uparrow$ $\frac{ROF}{ROF}$ $\uparrow$
- (ii) Mixing $\uparrow$ (Optimum range) $ROF \uparrow$
($G = 10-75 \text{ sec}^{-1}$)
- (iii) Type and does of Coagulant (Fe, optimum does) $ROF \uparrow$

In flocculation a parameter termed as $G \times t$ represent the conjugation (to combine) opportunity as it signifies no. of collision between the particles, thereby helps in identifying the types of flocs formed.

G - TMVS $\propto$ Intensity of mixing $= \frac{v_1 - v_2}{x}$
 t - time of contact.



$$Gt = \text{Constant}$$

$$G = \frac{(V_1 - V_2) \cdot \text{Constant}}{x}$$

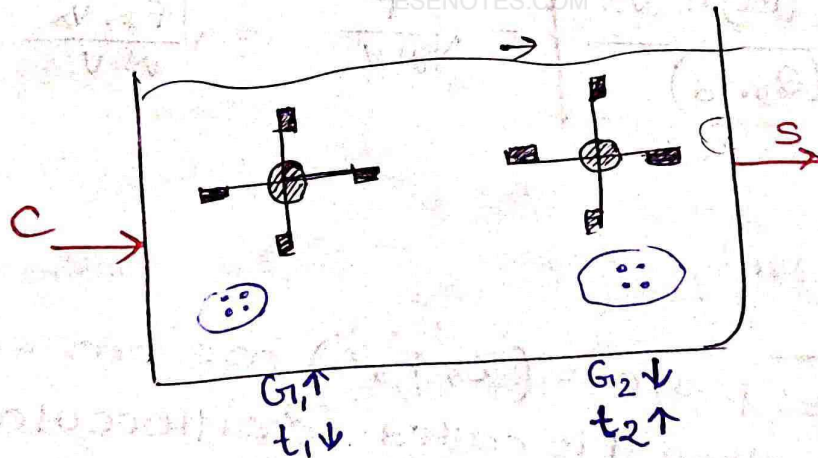
(i) $G \uparrow, t \downarrow = \text{Dense, small} = \text{DS} = \text{small dense floc}$

(ii) $G \downarrow, t \uparrow = \text{Light, large} = \text{LL} = \text{large light floc}$

→ 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 ($G \uparrow, t \downarrow$) combine with large & light flocs formed in later section of the tank ($G \downarrow, t \uparrow$) & result in the formation of large & dense-floc. Hence,

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



Design data

Depth = 3-3.5m

Detention time = 10-30 min

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

Total Area of Redals = (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)

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

$$\text{Volume, } V = \theta_D \cdot t_D$$

$$\text{Power, } P = F_D \cdot V_x$$

$$V_x = V_p - V_w$$

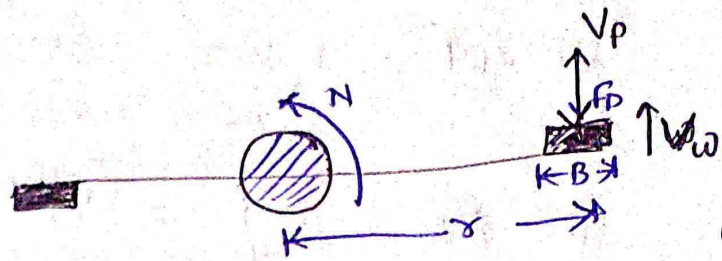
$$V_p = \omega r$$

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

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

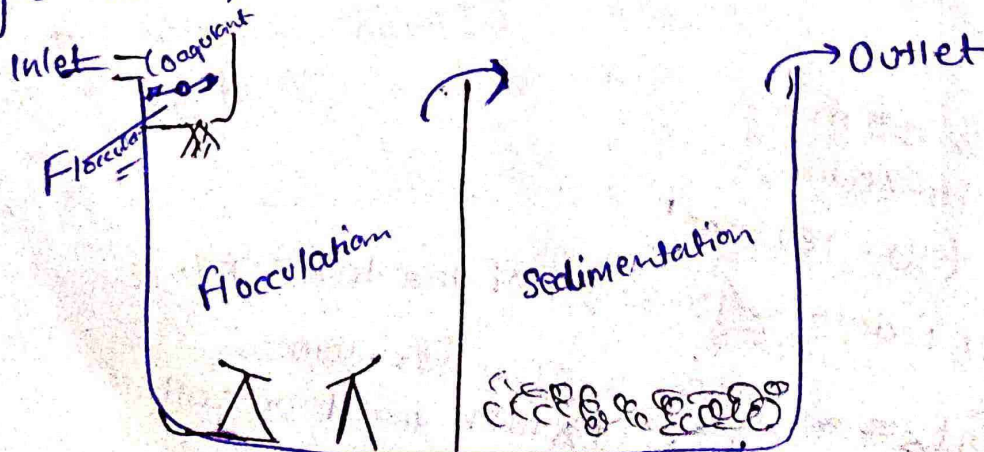
$$A_T = (BL)n, \quad C_D \approx 1.8$$

$$G = \sqrt{\frac{\frac{1}{2} C_D \cdot \rho_w \cdot (BL)n \cdot V_x^2}{\mu \cdot (\theta_D \cdot t_D)}} = \sqrt{\frac{P}{\mu V}} = \sqrt{\frac{F_D \cdot V_x}{\mu V}}$$



Clariflocculator

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



- Q. A flash mixer is treating 2m^3 of water @ mixing rate of 300 sec^{-1} at the temp of 20°C . where viscosity $= 0.01\text{ poise}$. compute the power requirement of the mixer for treating the water per unit volume of water.

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

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

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

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

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

- Q. A flocculation 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 centerline of the pedal is 1.8m from the center of shaft. Size of pedal is $0.3\text{m} \times 12\text{m}$. 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)

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

$$= \frac{30 \times 12 \times 4}{10 \times 10^6 \times 10^{-3}}$$

$$= 0.144\text{ day} = 3.456\text{ Hour.}$$

$$(ii) \text{ Power requirement} = 0.5 \cdot f_D \cdot V_s.$$

$$= \frac{1}{2} C_D \cdot f_w \cdot A_T \cdot V_{s1}^3$$

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

$$= 1961.806\text{ KW}$$

$$= 1961.806\text{ KW}$$

$$C_D = 1.8 \\ f_w = 1000\text{ kg/m}^3$$

$$V_{s1} = (V_p - V_w) \\ = \frac{3}{4} V_p$$

$$V_p = \omega r$$

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

$$V_{s1} = \frac{3}{4} V_p = \frac{3}{4} \times 5.654 = 4.241\text{ m/sec.}$$

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

$$= \sqrt{\frac{1961.806}{0.01 \times 10^{-1} \times (30 \times 12 \times 4)}}$$

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

IV Filtration

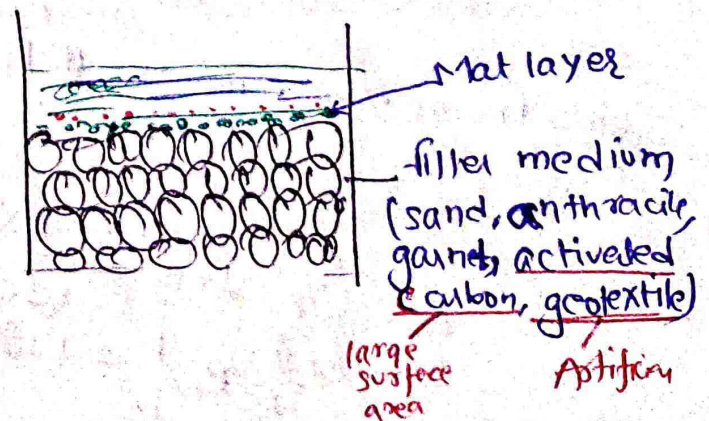
- Filtration is provided for removal of fine suspended particle and floc from water.
- It also removes organic matter, micro-organism, dissolve solid from water.
- Filtration is carried out in the units termed as filter. broadly classified into:
 - (i) Gravity filter. -
 - Slow sand filter
 - Rapid sand 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 following mechanism:-

(i) Mechanical straining.

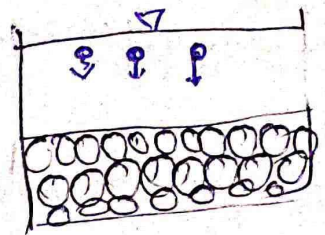
- When the water passes through the filter medium, suspended solid of size greater than the size of void of filter medium are strained (removed) out from it resulting in the formation of layer of entrapped impurities over the filter medium (mat layer).



→ 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 in this layer is less than the size of the voids of medium.

(i) Sedimentation

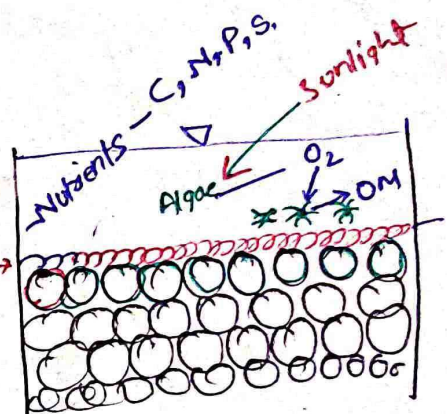
Due to the removal of turbulence from the water over the filter medium, solid get settled over the water free from suspended solids.



(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.

Schmutzdeke
(0.2-1mm)



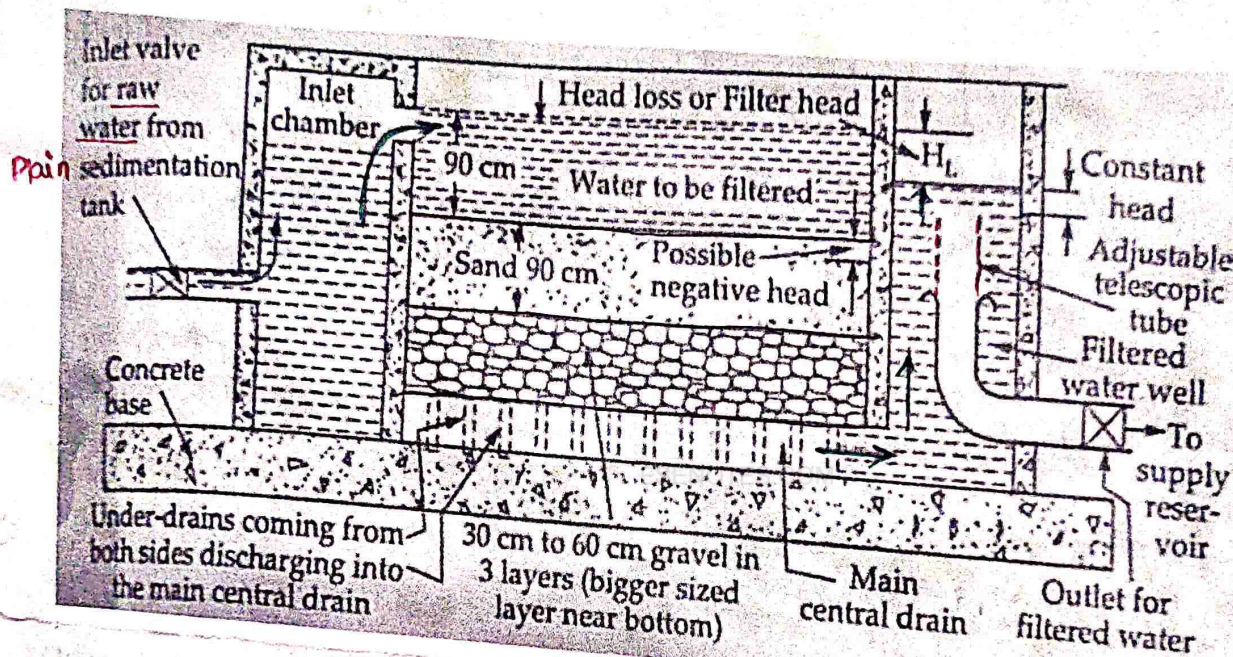
(A thin layer in which entire activity takes place is referred as Schmutzdeke). Over a period of time due to removal of organic matter by the micro-organism endogenous respiration takes place which also reduce the microbial load over the filter.

* 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 oppositely 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 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 (settled water)
- Gravel layer is provided only to support the sand medium.
- Depth of tank is 2.5-3.5m.
- Plan area required is 100-200m².
- Filter medium used sand, anthracite, garnet, activated carbon, geotextile.

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

Sand $\frac{75\% \rightarrow 10\% D_{10} \rightarrow 90\% \rightarrow 4.75 \text{ mm}}{[0.2 - 0.3 \text{ mm}]}$

→ As D_{10} is comparatively less, efficiency is completely more and rate 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.

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

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

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

$$C_u = 1 \quad (\text{Poorly graded})$$

$$C_u = 5$$

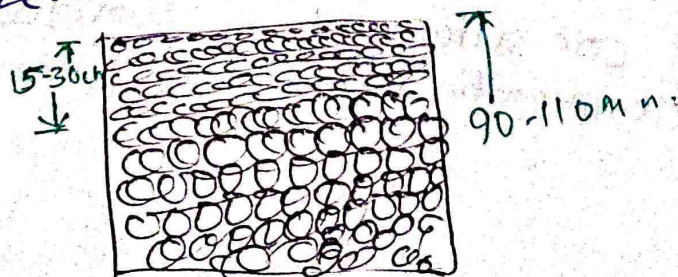
$D_{10} = 0.2 \text{ mm.}$
10% particle are
finer than 0.2 mm.

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

→ Depth of sand = 90 - 110 cm.

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

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



→ Freshly cleaned filter has a head loss of 10-15 cm 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 3 cm 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 24-36 hr as sufficient time provided for regrowth of Schmutzdecke that is removed during cleaning & for maturity of filter.

→ Frequency of cleaning is 1 to 3 month.

→ This cleaning is termed as surface cleaning.

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

$$\rightarrow R_{OF} = 2400 - 4800 \frac{\text{l/m}^2/\text{d}}{\text{Discharge/unit area}}$$

$$\boxed{\text{Plan area} = \frac{Q_D}{R_{OF}} \text{ (LXB)}}$$

$$\text{eg } Q_D = 1.8 \text{ m}^3/\text{s} = 1.8 \times 3600 \times 24 \times 10^3 = 1.6 \times 10^9 \text{ l/d}$$

$$PA = \frac{Q}{R_{OF}} = \frac{1.6 \times 10^9}{4800} = 333333 \text{ m}^2 = 0.333 \text{ km}^2 = (1 \text{ km} \times 333) \text{ m}$$

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

→ Minimum no. of Slow sand filter required ~~for~~ operation 1
for standy by 1.

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

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

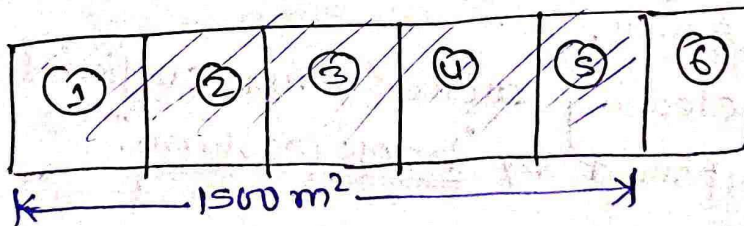
eg

$$SA = 1500 \text{ m}^2$$

$$N = 6 (s+1)$$

$$SA_f = \frac{1500}{5} = 300 \text{ m}^2 \quad \cancel{250 \text{ m}^2}$$

$$SA_T = 1500 \text{ m}^2 (1500 + 300) \text{ m}^2$$

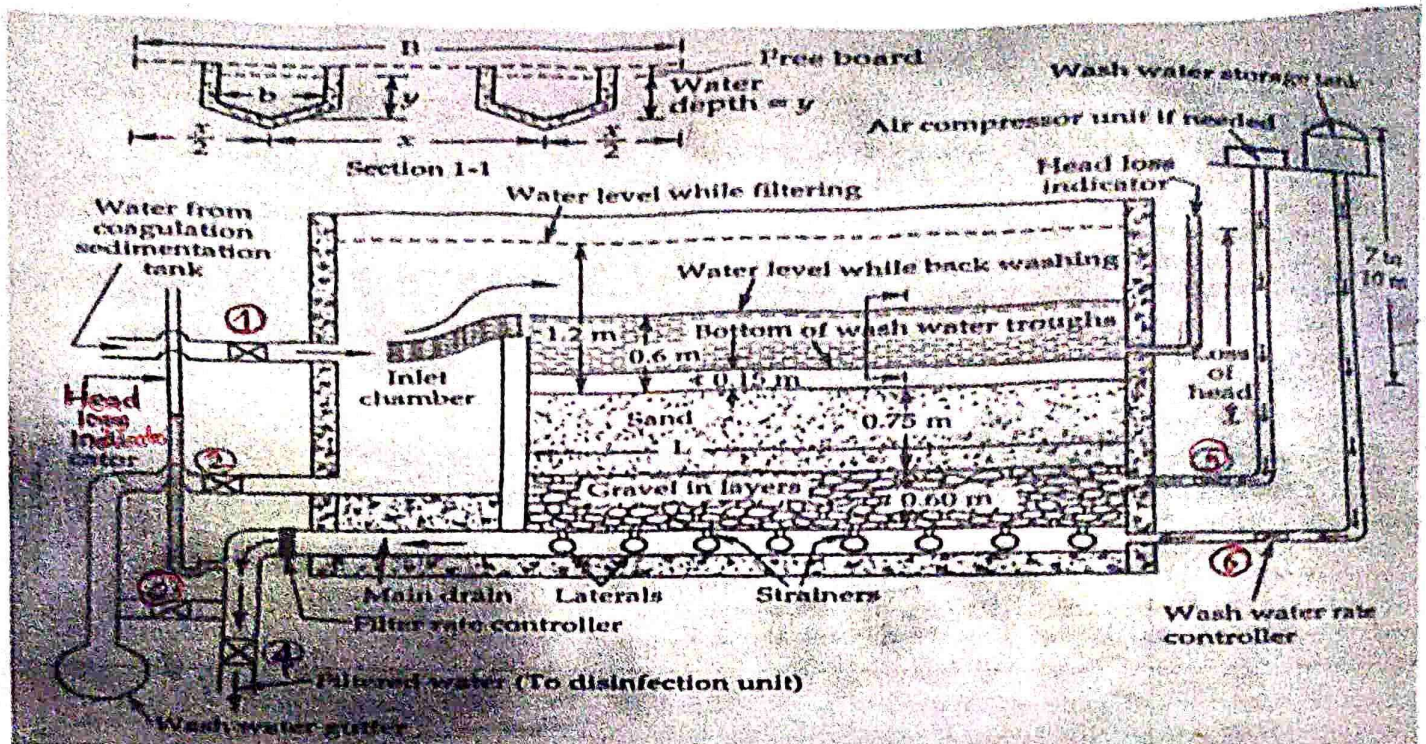


→ Efficiency of slow sand filter for removal of organic matter and MO is 98-99%.

→ Effluent from slow sand filter 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 50 NTU as it result in frequent chucking 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 penetrate 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 where in settled water is fed into the filter through valve no. 1. and filtered water is collected through valve no. 4.

→ During backwashing valve no. 1 & 4 are closed. Valve 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 increased porosity that in turn increased the opportunity of entrapped impurities to get washed away along with backwater into ~~wash~~ wash trough from where it is further disposed into washwater gutter through valve no. 2.

→ Once backwashing is completed, valve 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 *Schmutzdecke* and the removal of left over entrapped impurities.

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

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

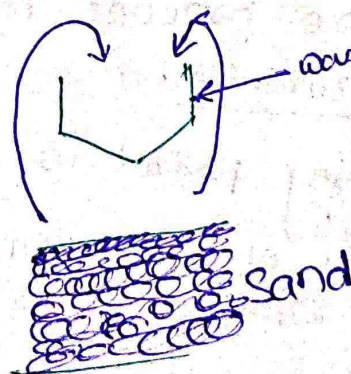
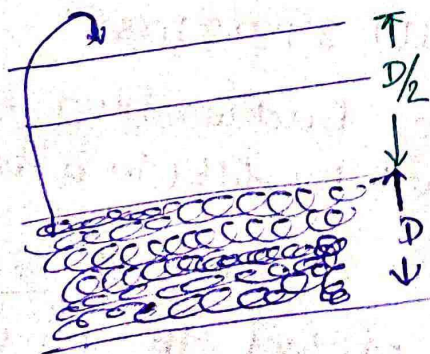
→ The washing period is normally 24-42 hrs.

→ 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 15-90 cm/min

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

- ROF → $3000 - 6000 \text{ l/m}^2/\text{hr}$, ($\approx 30 \text{ SSF}$)
- 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 - $10-80 \text{ m}^2$.
- minimum no. of unit is required $2(1+1)$ & additional no. of unit can be found. $N = 1.22 \sqrt{\frac{Q}{Q(\text{MLD})}}$
- Sand layer depth is $60-90 \text{ cm}$.
- D_{10} of sand = $0.35-0.55 \text{ mm}$
- 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.

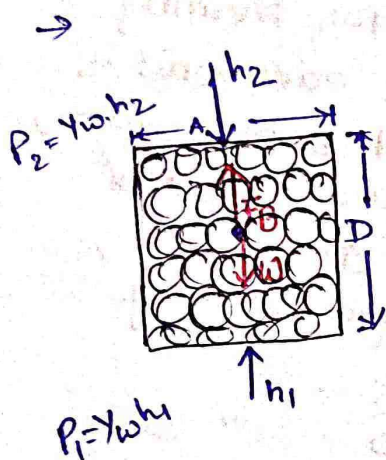


→ To hydraulically expand the porous bed, loss of head through it is equal to buoyant weight of particle forming the bed.

→ 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 buoyant weight of particle during expansion.



$\sum F_y = 0$ [for medium to be in equilibrium]

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

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

$$\text{loss of Energy} = \text{Buoyant weight} = B_w$$

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

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

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

$$\gamma_w A \cdot H_L = \underline{V(1-\eta)} \gamma_w (G-1)$$

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

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

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

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

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

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 treat water for 275000 people with water demand of 200 l/c/d. Rate of filtration = $15 \text{ m}^3/\text{m}^2/\text{hr}$. Determine the no. of filters required for treating this water. The available surface area of each unit is $10 \times 4 \text{ m}^2$. 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, $G = 2.5$, $d = 0.6 \text{ mm}$, $C_D = 5.02$. Flow is in transition during back washing.

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(i) Design Discharge through filter $Q_D = Q_{MD}$
 $= 1.8 \times Q_{AD}$
 $= 1.8 \times 275000 \times 200$
 $= 99 \times 10^6 \text{ l/d}$

Assuming 5% of back wash water is used in 30 minutes
 Net design discharge through filter = $Q_D + 5\% Q_D$
 $= 1.05 \times 99 \times 10^6$
 $= 103.95 \times 10^6 \text{ l/d}$

Net effective design discharge through filter,
 $Q_{ND}' = \frac{103.95 \times 10^6 \text{ l}}{23.5 \text{ hr.}}$

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

Plan area = $294.8 \sim 295 \text{ m}^2$

$$PA_{\pm} = 10 \times 4 = 40 \text{ m}^2.$$

$$N = \frac{PA}{PA_{\pm}} = \frac{295}{40} = 7.375 = 8 + 1 = 9$$

Standby unit

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

$$h_L = 0.6(1-0.5)(2.5-1) = 0.45 \text{ m}$$

$$(ii) h_L' = h_L$$

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

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

$$n' = 0.545$$

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

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

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

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

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

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

$$\log 0.545 = 0.22 [\log V_B - \log V_s]$$

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

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

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

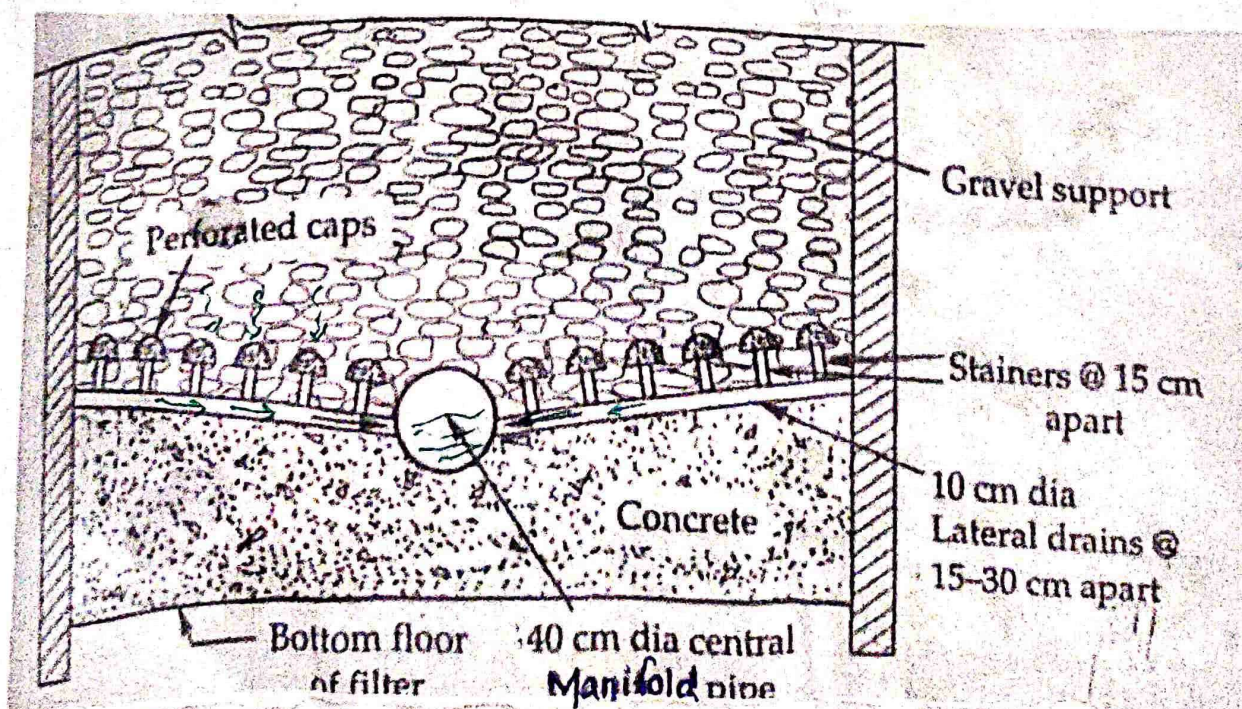
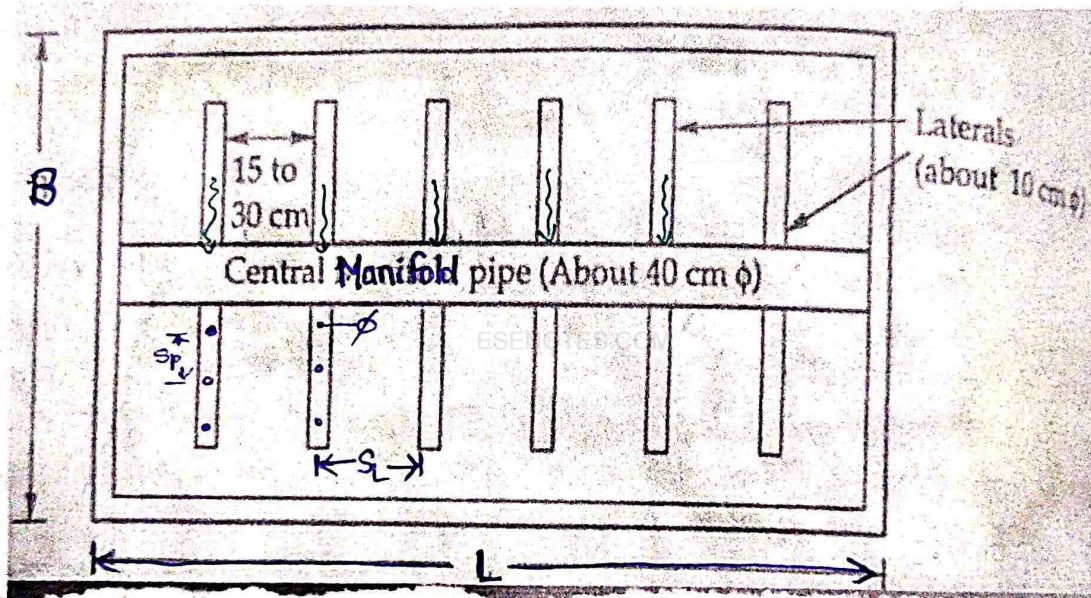
$$= 3.04 \times 10^{-3} \text{ m/sec.}$$

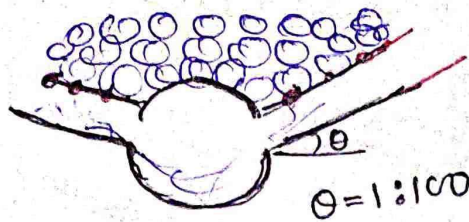
→ When water passes through filter medium loss in its head takes place to overcome the resistance offered by medium particles. Initial head loss through the filter is given by "Roes Equation".

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

ϕ = shape factor.

Design of Under-Drainage System.





→ Lateral and manifold system of under drain is 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.

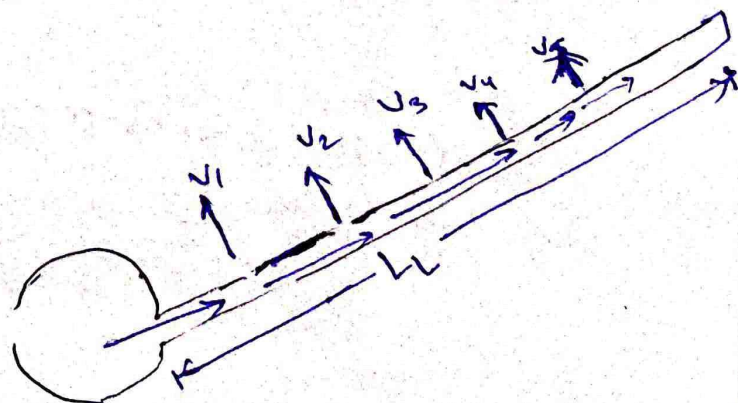
Factor 2 is used when 13mm perforation adopted.
Factor 4 is used when 6mm perforation adopted.

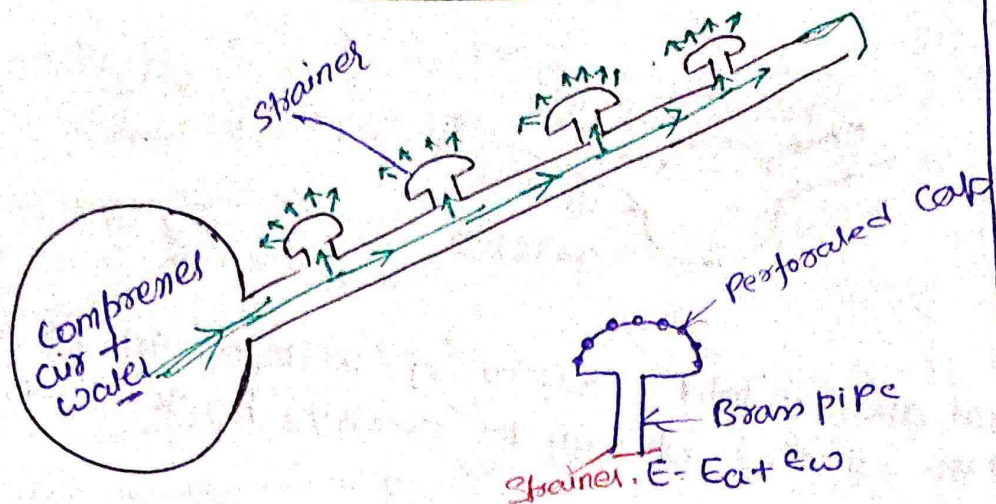
$$\left(\frac{\pi}{4} \times D_L^2 \right) = \frac{2}{4} \text{ times of x-x area of perforation in it.}$$

→ X-X area of Manifold = (cross-section of all laterals).

$$\frac{\pi}{4} D_M^2 = (\pi \times r_L) \cdot N.$$

Check
 $\frac{\text{length of lateral}}{\text{dia of lateral}} > 60$





$E = E_a + E_w$
Slow velocity wash

$E = E_w$
high velocity wash

→ High velocity wash is done when we ~~use~~ do not use strainer

→ slow velocity wash is done when we use strainer.

Q Same Question Design lateral and Manifold.

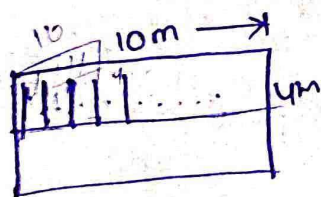
Area = 10×4

let, dia of perforation = 13mm

let, spacing of perforation = 20cm c/c.

let, spacing between lateral = 20cm c/c.

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



Cross-section area of perforation area = 0.2% of A

$$\begin{aligned} &= \frac{0.2 \times 10 \times 4}{100} \\ &= 0.08 \text{ m}^2 \\ &= 0.08 \times 10^4 \\ &= 800 \text{ cm}^2 \end{aligned}$$

Cross-sectional area of one lateral = $2 \times$ total cross-section area in one lateral

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

$$D_L = 4.5 \text{ cm}$$

Cross section area of Manifold. =

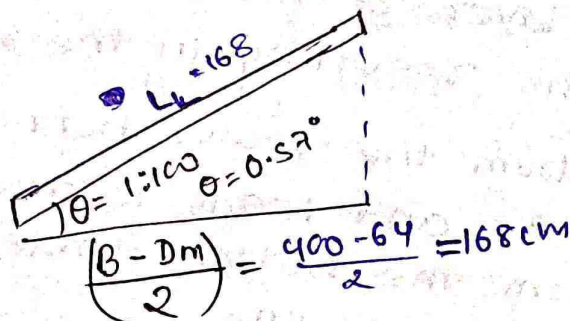
$$\frac{\pi D_m^2}{4} = 2 \times 16 \times 100$$

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

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

Check

$$\frac{\text{length of lateral}}{\text{dia of lateral}} = \frac{168.00}{4.5} = 37.33 > 60 \quad \text{OK}$$

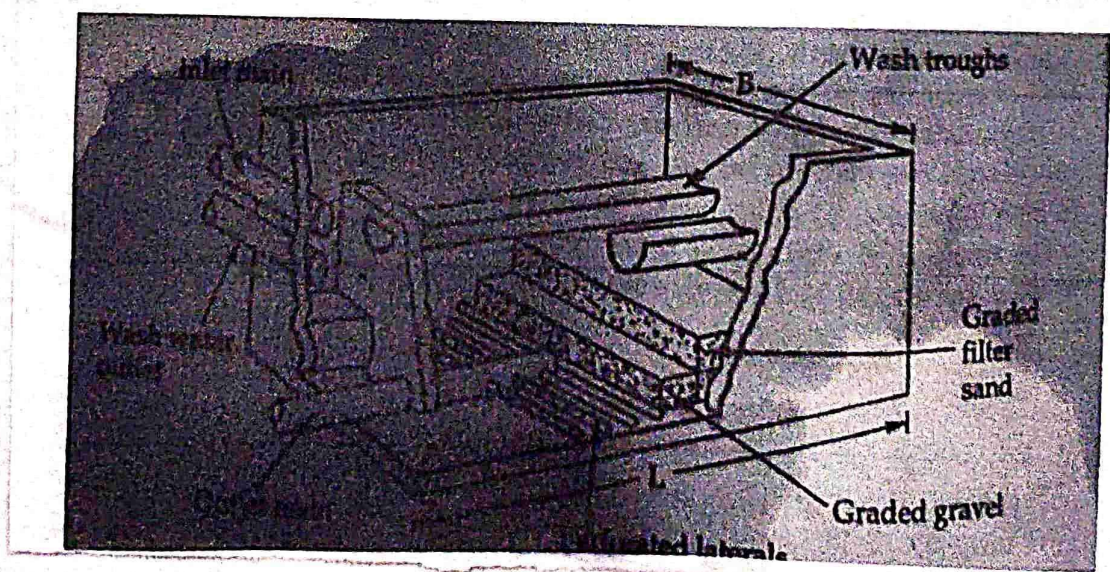


$$L = \frac{B - D_m}{2} \times \frac{1}{\cos \theta}$$

$$= \frac{168}{\cos 0.57} = 168.0083$$

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

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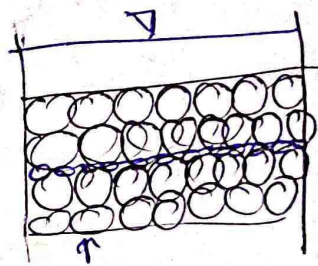


☀ Operational Problem of RSF

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 vacuum 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 affecting the operation of the filter.



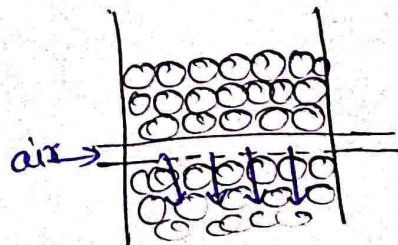
→ To avoid Air Binding proper cleaning of filter should be done when head loss becomes 2.5-3.5m or when negative head becomes 1.2m.

→ Avoid increase in temperature during filtration.

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

→ Pumping in air.

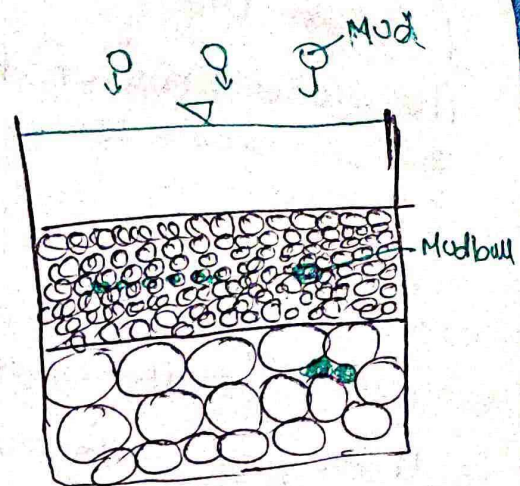
→ It only takes place in RSF because, in SSF, impurities are entrapped in upper layer only.



(ii) Mudball Formation.

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

→ It is not found in SSF because no backwashing is done in that case, no effect 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 turbulence around them during backwashing leading to removal of medium particles along with the backwash water out from the filter.

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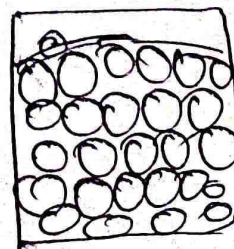
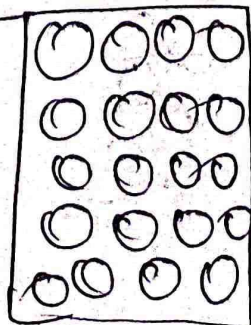
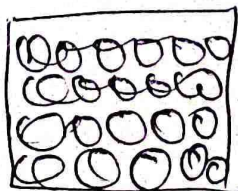
→ To avoid mud ball formation compress air scouring for 3-5 min is done during backwashing along with surface racking.

(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

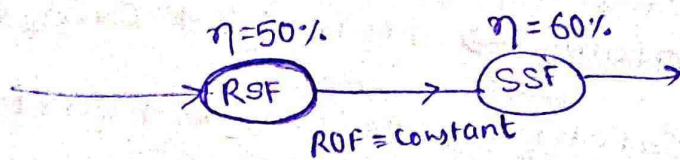
These are types of filter in which energy required by water to overcome the resistance of medium provided by application of pressure.

- These filter are same as RSF with a difference, here treated water 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 $6000 \text{ l/m}^2/\text{hr}$
- Dia of filter = $1.5 - 3 \text{ m}$
- Depth = $3.5 - 8.5 \text{ m}$
- They are suitable for treatment of limited quantity of water, eg. water in swimming pool or industrial water.
- Their size is comparatively much smaller than gravity filter hence are more flexible.
- Their cleaning is also done by backwashing.

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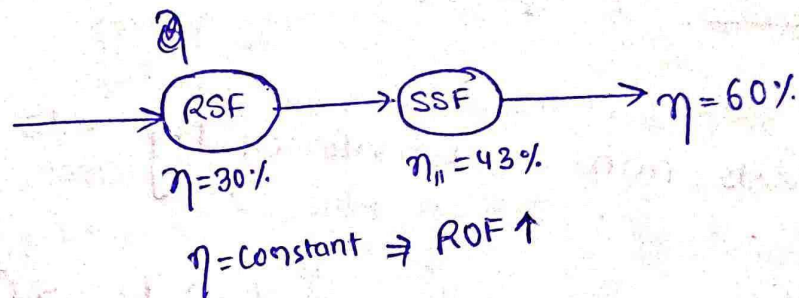
Roughing filter and Double filtration.

→ 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 filtration. & rapid sand filter used is termed as roughing filter.



$$\eta = 80\%$$

$$\eta_o = \eta_I + (1 - \eta_I) \eta_{II}$$



IV Diatomaceous Earth filter / Diatomite filter.

→ These filters are used to remove chlorine resistant dysentery cysts, or larvae of Schistosomiasis.

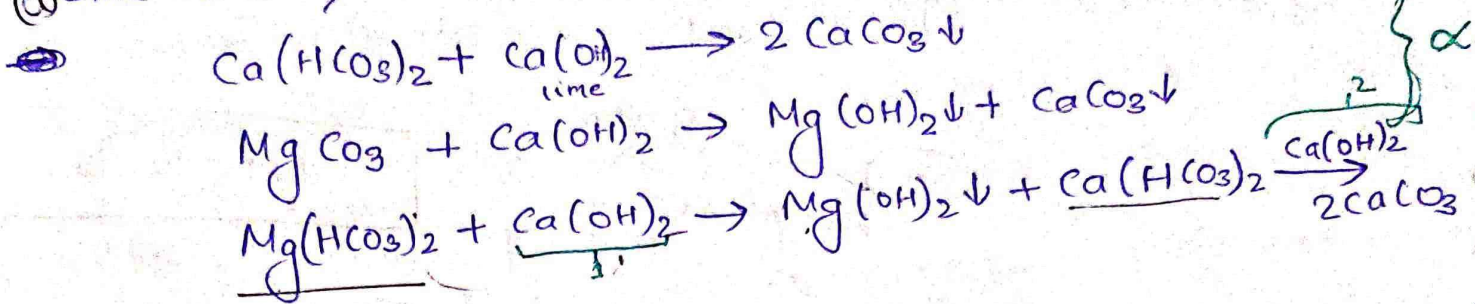
→ Diatomaceous earth or diatomite is a naturally occurring soft silicious sedimentary rock that is easily crumbled into a fine white to off-white powder.

Softening

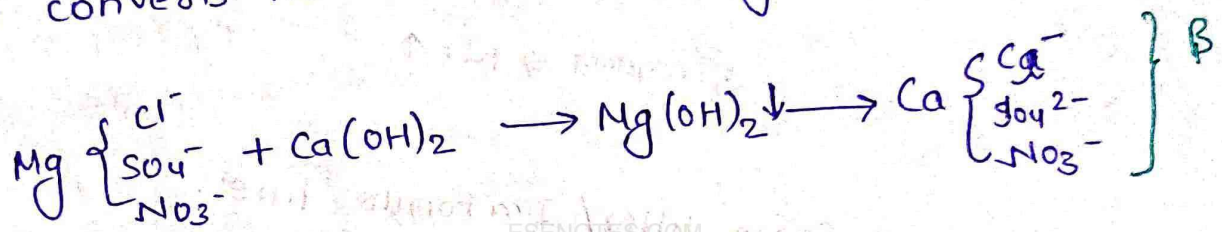
- It is the process of removal of hardness from water.
- It can be achieved by any of the following methods:-

Lime Soda Method

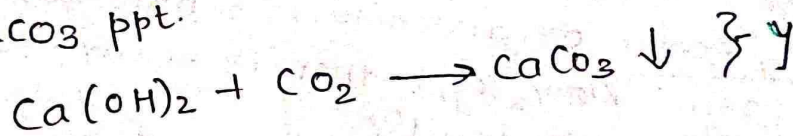
(a) Lime removes all the carbonate hardness from the water.



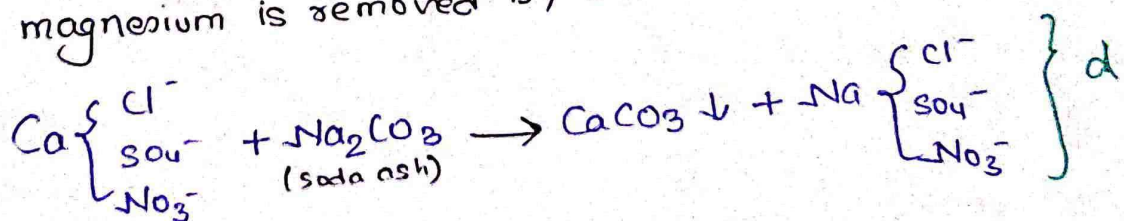
(b) Lime converts non-carbonate of Mg into NCH of Ca (B)



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



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

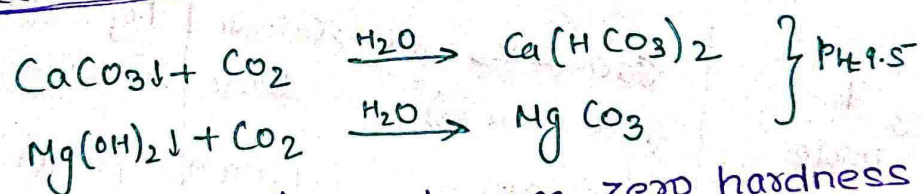


→ Precipitate of calcium carbonate is formed at pH greater than 9 to 9.5. Precipitate of $Mg(OH)_2$ is formed at $pH > 11$, hence pH of water is not above range it is first increased by addition of base. likewise.

→ 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.

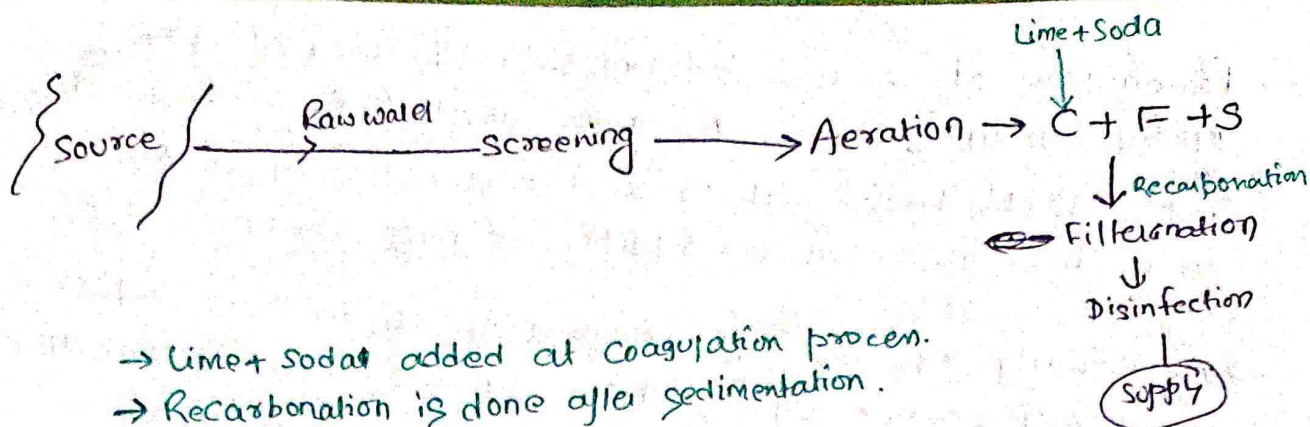
→ This method also helps in removing micro-organism and avoiding corrosion in pipe.
↳ catalyst - oxygen, acidity.

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



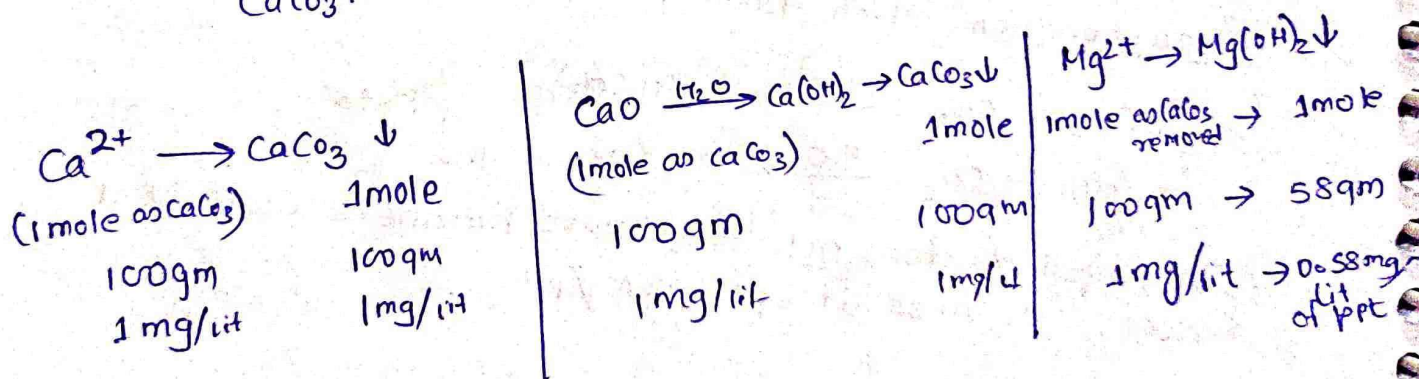
→ This method does not gives zero hardness water, but reduces hardness it upto 50 mg/lit

SO



→ 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:-

- 1mg of Ca hardness removed as CaCO_3 forms 1mg of sludge of CaCO_3 .
- 1mg of Mg hardness removed as CaCO_3 form 0.58gm of sludge of $\text{Mg}(\text{OH})_2$.
- 1mg of lime added as CaCO_3 forms 1mg of sludge as CaCO_3 .



$$\text{Dry sludge (mg/lit)} = \left[\text{Ca}^{2+} \text{ hardness removed as } \text{CaCO}_3 (\text{mg/lit}) + \text{lime added as } \text{CaCO}_3 (\text{mg/lit}) + 0.58 \text{ mg}^{2+} \text{ hardness removed as } \text{CaCO}_3 \right]$$

make correct

8. 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 50mg/lit & metallic flocculating alum used is 20mg/lit. Solid content of sludge, $S = 1.5\%$
Specific gravity = 2.5.

	ppm
precipitate $\rightarrow$ CaCO₃ $\rightarrow$ CaCO ₃ (ppt)	100
Lime $\rightarrow$ Mg(HCO ₃) ₂	150
(salt) $\rightarrow$ NaCl	-120
Lime $\rightarrow$ MgSO ₄	-110

	ppm
soda-CaCl ₂	50
ppt-SiO ₂	60
Lime CO ₂	10
Soda Mg CaSO ₄	100.

(1 mole of Mg(HCO₃)₂ react with 2 mole of lime)

militiequivalents $\rightarrow$ 40 + 16
(CaO)

I) Quantity of lime = $\left(\frac{150}{146} \times 2 + \frac{110}{120} \times 1 + \frac{10}{44} \times 1 \right) \times 56 \times 10 \times 10^6$
 $= 653.82 \text{ tones/year.}$

Quantity of soda = $\left(\frac{110}{120} \times 1 + \frac{50}{100} \times 1 + \frac{100}{100} \times 1 \right) \times 106 \times 10 \times 10^6$
 $= 813.42 \text{ tones/year.}$

Soda ash = Na₂CO₃
 $= 23 \times 2 + 12 \times 16 \times 3$
 $= 106$

II wt of solid in sludge = $\left[\frac{50}{11/2} + \frac{100}{136/2} + \frac{179.13}{56/2} + 0.58 \left(\frac{150}{146/2} + \frac{110}{120/2} \right) \right] \times 50$

$+ 50 + 20 \times 0.234 + 100 + 60$
 (solid) Due to alum (CaCO₃) (SiO₂)
 $\times 10 \times 10^6 \times 10^6$

$= 4978.6 \text{ kg/day.}$

$$\text{Volume of solids in sludge} = \frac{4978.6}{2.05 \times 10^3} = 1.99 \text{ m}^3/\text{day}$$

$$S = 1.5\% \Rightarrow 1.5 \text{ kg of solid} + 98.5 \text{ kg of water} = 100 \text{ kg of sludge}$$

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

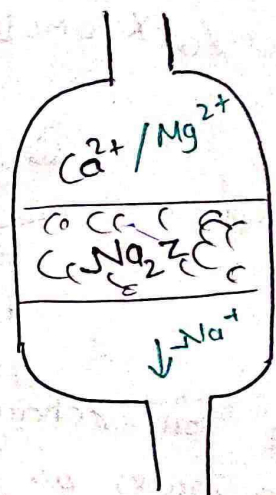
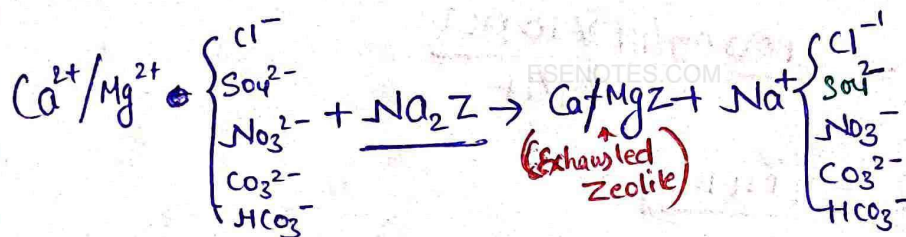
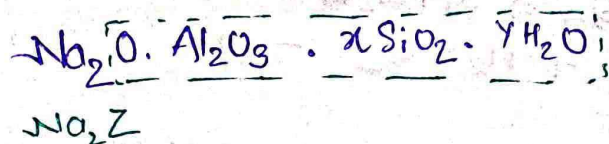
$$\text{Volume of water} = \frac{326928 \text{ kg/day}}{1000} = 326.928 \text{ m}^3/\text{day}$$

$$\begin{aligned} \text{Volume of sludge} &= \text{Volume of (water + solid)} \\ &= 326.928 + 1.99 \\ &= 328.918 \text{ m}^3/\text{day} \end{aligned}$$

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(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 solution (NaCl) through it before carrying ^{out the} further treatment with it.



Regeneration of exhausted Zeolite,



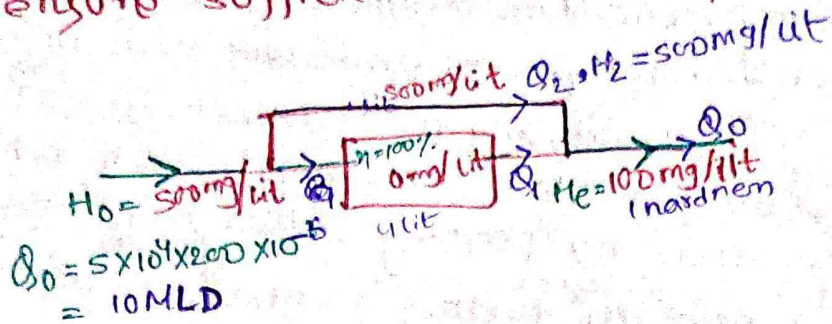
→ Hardness comes in water used for regeneration, will be thrown out.

→ 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.



Q A ground water having hardness of ~~500~~ 500 mg/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 100 mg/lit of hardness is required in treated water to ensure sufficient taste compute the by pass flow rate.



$$Q_1 + Q_2 = Q_0 \Rightarrow Q_1 + Q_2 = 10 \text{ MLD}$$

$$H_0 \cdot Q_0 = H_1 \cdot Q_1 + H_2 \cdot Q_2 \quad \text{--- (i)}$$

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

$$Q_2 = \frac{100 \text{ mg/lit} \times 10 \text{ MLD}}{500 \text{ mg/lit}}$$

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

Q An ion exchange softener is to be ~~remov~~ used to treat the water as given below. The medium selected has an absorptive capacity of 90 kg/m^3 @ a flow rate of $0.4 \text{ m}^3/\text{min}/\text{m}^2$.

Regeneration is accomplished 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 filter bed also determine the chemical requirement and the regeneration cycle time.

Ca^{2+}	Mg^{2+}	Na^+
CO_3^{2-}	HCO_3^-	SO_4^{2-}
0.8	1.7	3.83
0.3	1.33	3.83

Assume regeneration loading rate $0.64 \text{ m}^3/\text{m}^2/\text{min}$. rate of flow $25000 \text{ m}^3/\text{d}$. and required hardness = 75 mg/lit .

Sol $TH = 1.7 \times 50 = 85$

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

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

$$Q_2 = 22059 \text{ m}^3/\text{d}$$

Water to be treated, Q_1

$$Q_e - Q_2 = 25000 - 22059 \\ = 2941 \text{ m}^3/\text{day}$$

Total hardness to be removed, $= 2941 \times 85 \times 10^{-3} \times 10^{-6}$
 $= 250 \text{ kg/day}$

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

$$\text{Area of medium require} = \frac{2941 \text{ m}^3/\text{day}}{0.4 \text{ m}^3/\text{min}/\text{m}^2} \\ = \frac{2941}{0.4 \times 24 \times 60} \\ = 5.106 \text{ m}^2$$

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

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

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

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

$$S_{A_f} = 1.02 \text{ m}^2$$

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

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

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

$$\text{III Quantity of Sodium chloride waste} = 150 \times 2.78 = 417 \text{ kg.}$$

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

$$= 4170 \text{ kg/day.}$$

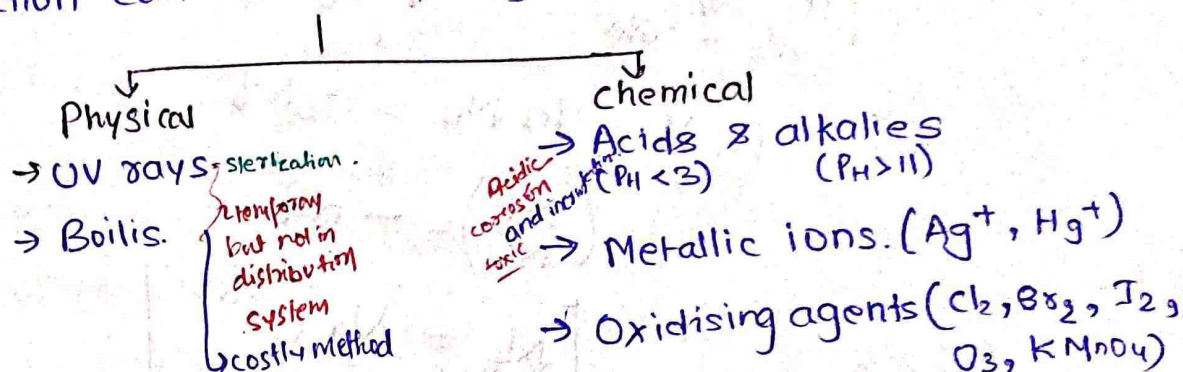
$$\text{Volume of brine sol}^n = \frac{4170}{10^3} = 4.17 \text{ m}^3/\text{day.}$$

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

$$= 20.44 \text{ minute.}$$

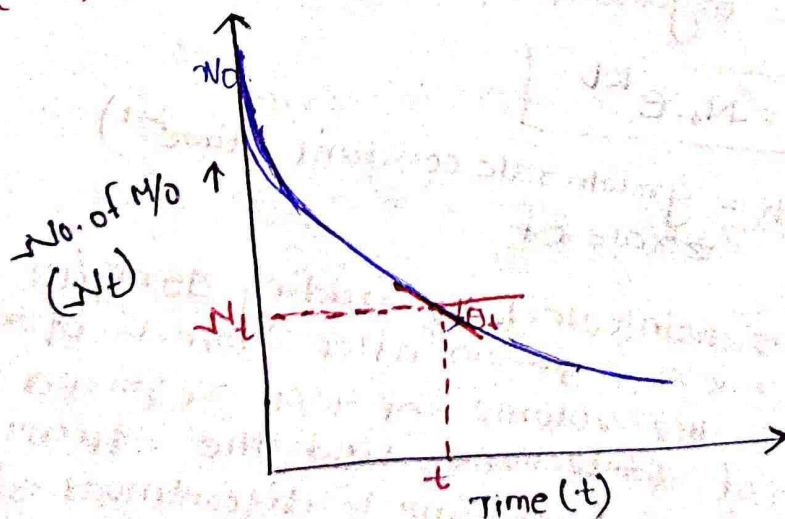
Disinfection of Water.

- 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 Sterilization.
- Disinfection is a type of Sterilisation
- 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.



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



$$\frac{dN_t}{dt} = -N_t [Ox \propto y]$$

→ nature of change of variable.

$$\frac{dN_t}{dt} = -KN_t$$

K = disinfection const.
(time⁻¹)

= It signify rate of disinfection.

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

$$\ln N_t \Big|_{N_0}^{N_t} = -K \Big|_0^t$$

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

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

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

$$y = f(x)$$

Note → Growth of micro-organism is also observed to be exponential.

$$N_t = N_0 \cdot e^{Kt}$$

Here, K = growth rate constant (time⁻¹)
= rate of

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 $k_d = 0.234 \text{ min}^{-1}$, $K_g = 0.465 \text{ min}^{-1}$.

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

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

$$= 4.65 \text{ mg/lit}$$

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

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

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

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

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

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

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

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

$$-0.234 \times t = -4.605$$

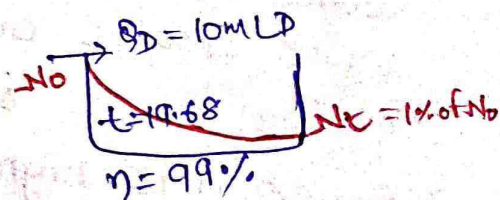
$$t = 19.68 \text{ min.}$$

$$(iii) \quad dt = 19.68$$

$$V = Q_D \times dt$$

$$= 10 \times 10^6 \times 10^{-3} \times 19.68$$

$$= 196.8 \text{ m}^3$$



III

$t = 19.68$

% removal of micro-organism after half operation; 4% left in water.

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

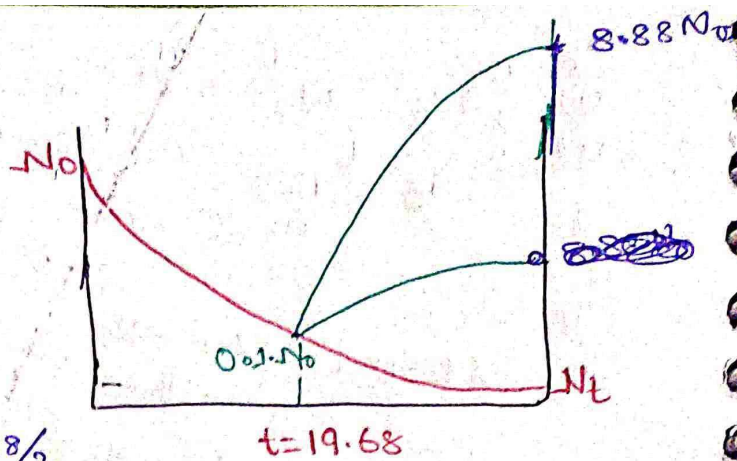
$$= 15 \times e^{-0.234 \times \frac{19.68}{2}}$$

$$= N_0 \times 0.1$$

After half operation,
growth.

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

$$\begin{aligned} N_t &= 0.1 N_0 \cdot e^{kt} \\ &= 0.1 \times N_0 \times e^{0.456 \times 19.68/2} \\ &= 8.88 \times N_0 \end{aligned}$$



$$\begin{aligned} \% \text{ efficiency} &= \frac{N_0 - 8.88 N_0}{N_0} \times 100 \\ &= -788\% \end{aligned}$$

Minor Method of Disinfection.

1) Treatment with ozone. (future)

- It is a strong ~~re~~ oxidising reagent which when added in water dissociates into molecular and nascent oxygen which carries out disinfection and oxidation of organic matter.

$$O_3 \xrightarrow{H_2O} O_2 + \overset{\text{nascent oxygen}}{O}$$
- Since it is highly reactive it quickly disappears in water, and does not safeguard it against future recontamination.
- It gives pleasant taste in water unlike other disinfectants but its comparatively costlier.
- It destroys the cell wall of micro-organism to carry out disinfection.
- Its normal dose varies in range of 2-3 ppm.
- It is less effective than chlorine in killing bacteria and virus but more effective in killing protozoa.

→ future recontamination chances.

(ii) Treatment with KMnO_4 - C

- 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 disappears 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-organisms to carry out disinfection.
- It removes 98% of bacteria in general, and 100% of cholera causing bacteria.
- Its normal dose is 1-2 mg/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 to well.

Note Watson's law.

Solubility of any chemical compound = $f[C, \text{Time}]$

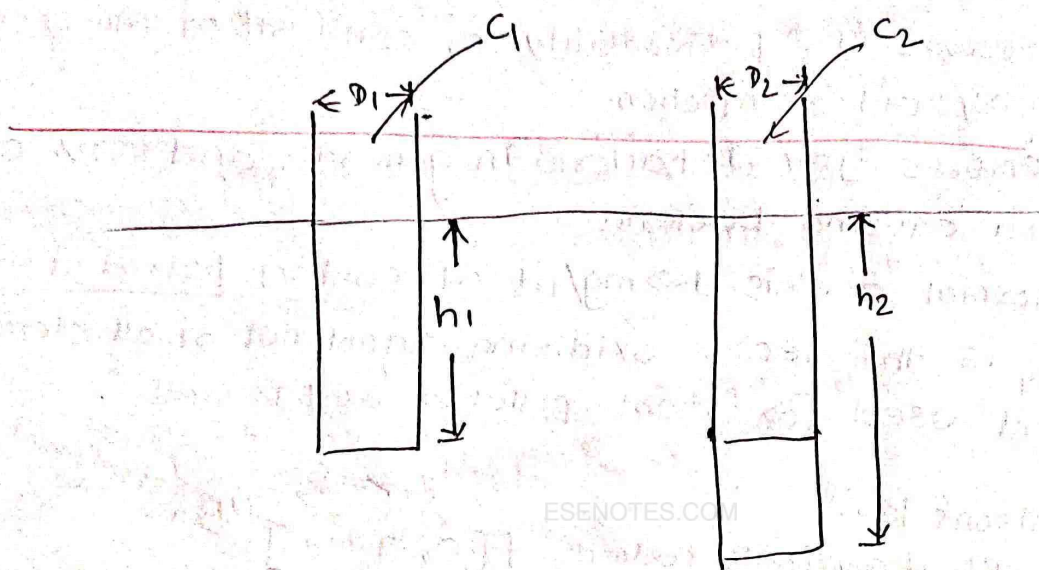
for given solubility = $t C^\eta = \text{constant}$

η = dilution co-efficient.

Q. QOK

Q80

40 kg of KMnO_4 is added in a well of dia-5m, & having water standing water depth of 10m in it. Compute the ~~the~~ ^{water} ~~of~~ KMnO_4 is required level at which same disinfection would be achieved by 30kg of KMnO_4 added in another well of dia-3m in same medium in half time. Assume $n=1$



for same degree of disinfection. Solubility of KMnO_4 must be same in both the well

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

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

$$t_1 \left(\frac{w_1}{v_1} \right) = t_2 \left(\frac{w_2}{v_2} \right)$$

$$\cancel{t_1} \times \frac{40}{\cancel{\pi} \times 5^2 \times 10} = \frac{\cancel{t_2}}{2} \times \frac{30}{\cancel{\pi} \times 3^2 \times h_2}$$

$$h_2 = 10.416 \text{ m}$$

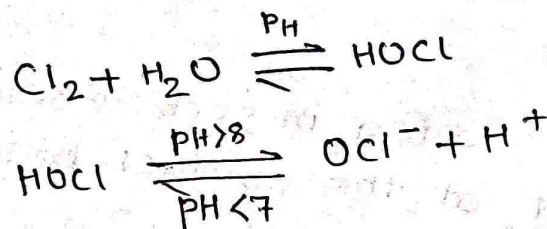
Major method of Disinfection.

(i) Treatment with chlorine (Cl)

→ When chlorine added with water reacts it at $\text{pH} > 5$ & heads to formation of highly unstable Hypochlorous acid (HOCl) which further dissociates into Hypochlorite ion (OCl^-) at $\text{pH} > 7$.

→ Here all three above forms of chlorine are collectively termed as freely available chlorine and is capable of carrying out disinfection. But HOCl is most destructive disinfectant amongst them. (80 times more effective than hypochlorite ion) Hence pH during disinfection is maintained slightly less than < 7 .

→ Chlorine enters into cell wall of m/o and destroy the enzyme responsible for metabolism in them in order to carry out disinfection. **"ENZYMATIC HYPOTHESIS"**

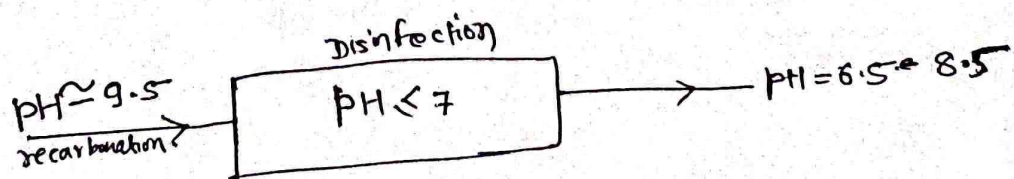


metabolism = Catabolism + Anabolism

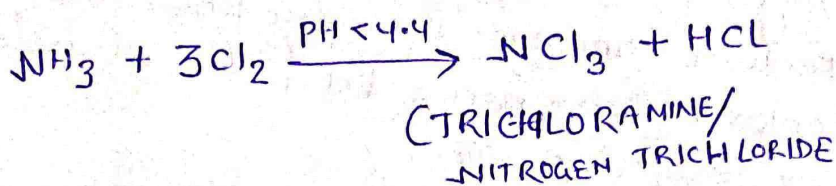
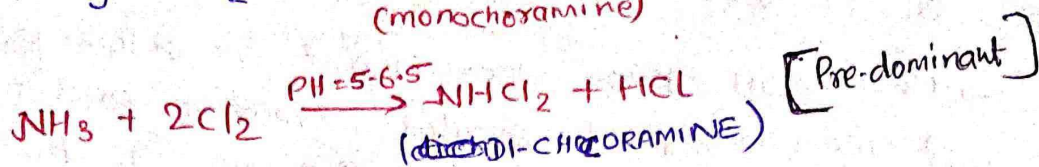
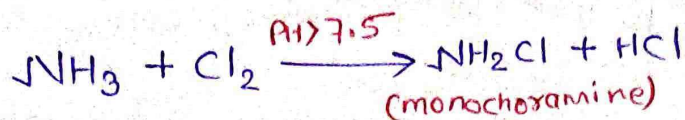
Metabolism is the process in which bigger nutrients consumed by living organism are broken into lighter nutrients and energy is released (CATABOLISM) and this energy is further used by body for the growth of new cell (ANABOLISM)

$\text{Cl}_2, \text{HOCl}, \text{OCl}^-$
Free available chlorine.

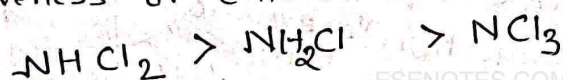
pH	form of chlorine.
< 5	Cl_2
<u>5-7</u>	<u>HOCl</u>
7-8	$\text{HOCl}, \text{OCl}^-$
> 8	OCl^-



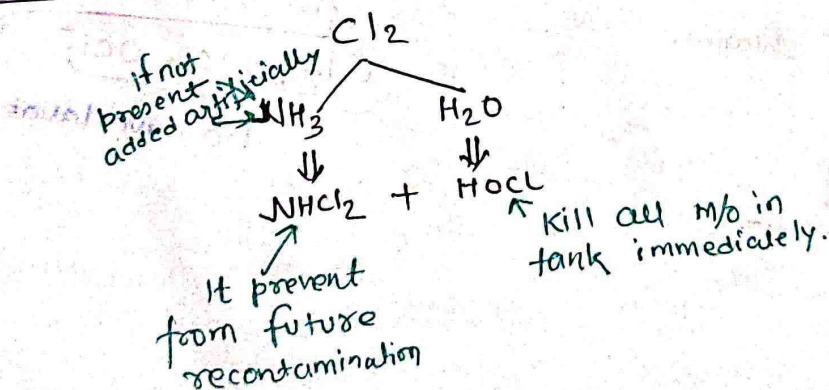
→ Chlorine when added in water immediately reacts with ammonia and form chloramine which are also termed as combined form of chlorine and are capable of carrying out disinfection but are 25 times less effective than free chlorine.



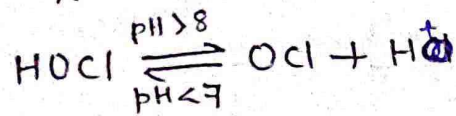
→ Effectiveness of chloramine in killing M/O.



→ Dose of chlorine is added in such a residual of 0.2 mg/l is insured at the contact period of 10 minutes.



Q 20mg/lit of chlorine is added in water to carry out disinfection equilibrium of which is governed by following reaction.



$$K = 2.5 \times 10^{-7} \text{ moles/lit.}$$

(i) compute

(ii) fraction of HOCl in water at pH=7.

(iii) Concentration of OCl⁻ at this pH, considering chlorine added dissociates in HOCl & OCl⁻.

(iv) At what pH of water 0.7 fraction of OCl⁻ would be available in water.

(i) pH=7 $\Rightarrow [\text{H}^+] = 10^{-7} \text{ moles/lit}$

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

$$2.5 \times 10^{-7} = \frac{[\text{OCl}^-][10^{-7}]}{[\text{HOCl}]}$$

$$[\text{HOCl}] = 0.4 [\text{OCl}^-]$$

$$f[\text{HOCl}] = \frac{[\text{HOCl}]}{[\text{HOCl}] + [\text{OCl}^-] + [\text{H}^+]} \xrightarrow{\text{neglect.}}$$

$$= \frac{0.4}{1.4} = 0.285$$

$$f[\text{HOCl}] = 0.285$$

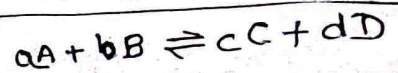
$$= 0.285 \times 100$$

$$= 28.5\%$$

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

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

$$K_f \cdot K_b = 1$$



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

where a, b, c, d are no. of moles.

At pH=7
OCl⁻ & HOCl
Both present

(i) At pH = 7

$$[\text{HOCl}] = 0.4 [\text{OCl}^-] \quad \text{--- (i)}$$

moles of chlorine added. = $[\text{Cl}_2] = [\text{HOCl}] + [\text{OCl}^-]$

$$\frac{20 \times 10^{-3}}{71} = [\text{HOCl}] + [\text{OCl}^-] \quad \text{--- (ii)}$$

$$\frac{20 \times 10^{-3}}{71} = 0.4 [\text{OCl}^-] + [\text{OCl}^-]$$

$$[\text{OCl}^-] = \frac{20 \times 10^{-3}}{71 \times 1.4} = 2.01 \times 10^{-4} \text{ moles/lit}$$

(iii) $f[\text{OCl}^-] = 0.7$

$$\frac{[\text{OCl}^-]}{[\text{OCl}^-] + [\text{HOCl}] + [\text{H}^+]} = 0.7 \quad \text{--- neglect } [\text{H}^+]$$

$$[\text{OCl}^-] = 0.7 [\text{OCl}^-] + 0.7 [\text{HOCl}]$$

$$0.3 [\text{OCl}^-] = 0.7 [\text{HOCl}]$$

$$\frac{[\text{OCl}^-]}{[\text{HOCl}]} = \frac{7}{3}$$

$$\Rightarrow K_f = \frac{[\text{OCl}^-][\text{H}^+]}{[\text{HOCl}]}$$

$$2.5 \times 10^{-7} = \frac{7}{3} [\text{H}^+]$$

$$[\text{H}^+] = 1.07 \times 10^{-7} \text{ moles/lit.}$$

$$\text{pH} = -\log_{10}(1.07 \times 10^{-7})$$
$$= 6.97$$

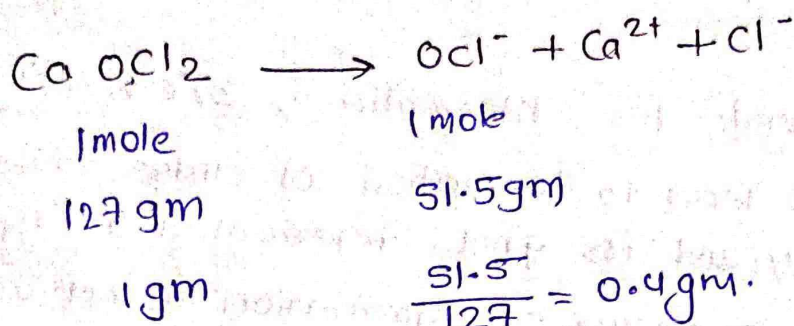
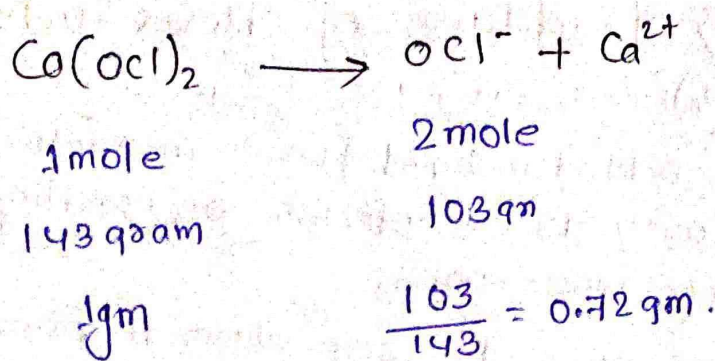
Different forms of addition of Chlorine in Water.

(i) Free chlorine (Cl_2) (solid, liquid, gas)

- Free chlorine is added in liquid phase in treatment plant in order to carry out disinfection as handling of liquid chlorine is much easier.
- It is highly sensitive to temperature. It freezes below 10°C and burns the container above 92°C in which it is kept for storage.
- Optimum temp. for chlorination is 32°C to 48°C .
- It does not lead to formation of sludge while disinfection and its quality ~~and~~ is not deteriorated during storage.
- Temperature problem, so chlorination is not used in European countries (Distribution system temp.).

(ii) Hypochlorites and Bleaching Powder.

- In this case either hypochlorite ion and ~~hypochlorite~~ HOCl carries out the disinfection depending upon the pH of water.
- For 100% pure calcium hypochlorite freely available chlorination availability is approximately 72% which also goes on decreasing with time.
- For 100% pure bleaching powder, freely available chlorination availability is approximately 40% which also goes on decreasing with time.
- Use of these forms also leads to formation of sludge while disinfection. Hence it is not suitable for treatment in plant but can be used for swimming pool.

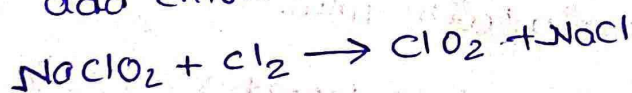


(iii) Chloramine

→ It is added in treatment plant for disinfection in case when phenol is present in water.

(ii) Chlorine dioxide (ClO₂)

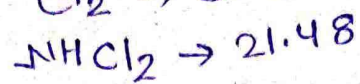
When we add chlorine is sodium chlorate



→ It is strong oxidising reagent (2.5 times more effective than free chlorine.) which can remove ~~not~~ both microorganisms and OM from water but it is not capable of safeguarding water against future recontamination.

→ It can also be used when phenol is present in water.

Q freely available Unit oxidising power of free chlorine is 35.5 and that of dichloramine is 21.48. Find the % of free Cl_2 in dichloramine.



$$\chi(35.5) = 21.48$$

$$\chi = \frac{21.48}{35.5} = 0.6$$
$$= 60\%$$

Q A city with the population of 50000 is to be provided disinfected water @ of 250 l/c/d. Find the amount of 25% pure bleaching powder required for treatment of this water @ dose of 20 mg/lit.

$$\text{Qty of CaOCl}_2 = \frac{5 \times 10^4 \times 250 \times 20 \times 10^{-6}}{0.25}$$
$$= 1000 \text{ kg/day}$$

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Factor effecting efficiency of chlorination

- (i) Turbidity ↑ ↓
(ii) Presence of metallic ion. (Fe, Mn²⁺) ↑ ↓
(iii) Presence of ammonia (NH₃) ↑ ↓
(iv) pH ↑ ↓ (optimum pH = 5-7)
(v) Temperature ↑ ↑
(vi) Time of contact ↑ ↑

Types of chlorination:-

(i) Plain chlorination.

- If the only treatment given to water is chlorination it is termed as plain chlorination.
- It removes taste, colour, odour and micro-organism from water.
- It is not done if turbidity of water is beyond permissible limit.
- Normal dose of chlorine in this case is 0.5 mg/lit.

(ii) Pre-chlorination:-

- If chlorination is done during coagulation and or during filtration. it is termed as pre-chlorination.
- It is adopted when quality of water is poor with respect to organic matter and micro-organism.
- Pre-chlorination is always succeeded by post chlorination but vice-versa not.
- Dose of chlorine in this case depend upon point of addition of chlorine.
- In ~~coagulation~~ ^{filtration} it is 0.1-0.5 mg/lit and in ~~filtration~~ ^{coagulation} it is 5-10 mg/lit

(iii) Post-chlorination

- If chlorination is done after filtration or it is the last treatment given to water it is termed as post chlorination.
- Does of chlorine in ~~each~~ this case such a residual of 0.2 mg/lit of free Cl_2 is insured at contact period of 10 min to safeguard the water against future contamination.

(iv) Double-chlorination

- If both pre and post chlorination is done it is termed as double-chlorination.
- It is adopted when higher efficiency of chlorination is required.

(v) Super-chlorination

- ~~Whenever a para~~
- If excess of chlorine is added in the range of $5-15 \text{ mg/lit}$ so as to ensure residual of $1-2 \text{ mg/lit}$ at contact period of 10 minute during epidemic, it is termed as super-chlorination.

(vi) De-chlorination

It is process of removal of excess chlorine from the water. It can be achieved by any of following chemicals.

Effective → NaAs ✓

cheap → $\text{Na}_2\text{S}_2\text{O}_3$

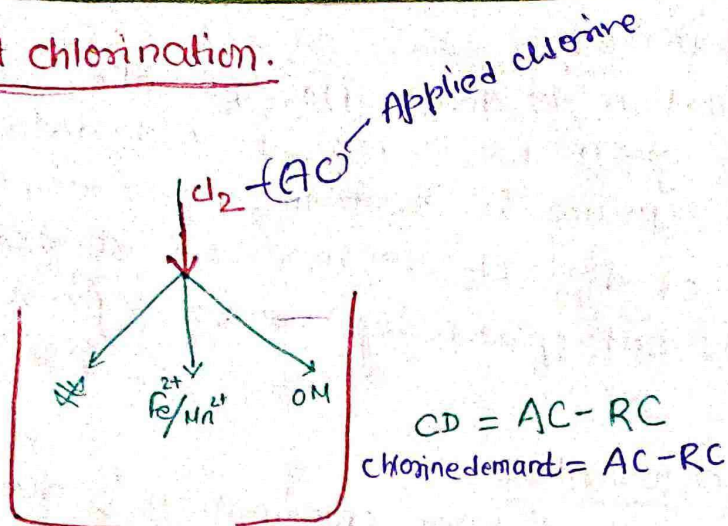
→ NH_4OH

→ Activated carbon

→ SO_2

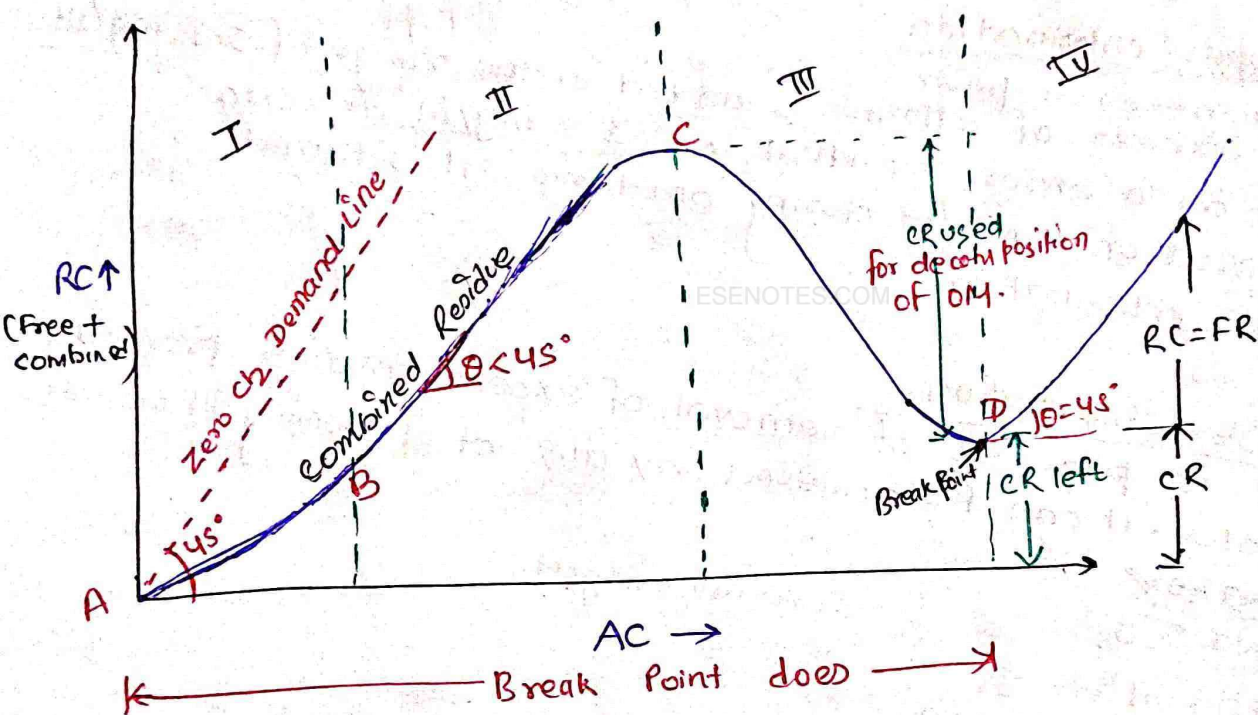
} De-chlorinating Reagent

(iii) Break Point chlorination.



$$\text{RC} = \text{FR} + \text{CR}$$

Residual chlorine = Free Residual + Combined residual



- In phase-I, chlorine first performs the function of disinfection and oxidation of mineral in the water resulting in high chlorine demand and leading to lesser residual chlorine in water.
- As most of the demand for disinfection and oxidation of mineral is satisfied in phase-I, residual chlorine increase in phase-II.

- At no point in phase II, residual chlorine equals applied chlorine as some amount of disinfection still takes place in this phase
- At the end of this phase i.e., at point C, bad smell oxidation of organic matter started by chlorine.
- As the result of this high chlorine demand, destruction of chloramines and chloroorganic compounds takes place in this phase.
- At the end of this phase i.e., point D, bad smell stop coming out of the water indicate complete decomposition of organic matter.
- After this point D, whatever chlorine is added breaks free from water i.e., appear as a residual chlorine, hence this point is termed as break point & the amount of chlorine required to reach upto this point is termed as break point dose.
- Theoretically no chlorine is required to be added in water after this point but practically chlorine is added to ensure the residual of 0.2 mg/lit after the contact period of 10 minutes.

	I	II	III	IV
AC	10	10	10	10
CD	9.8	0.3	8.5	0
RC	<u>0.2</u> CR	<u>9.7</u> CR	<u>6.9</u> CR	<u>16.9</u> FR + CR

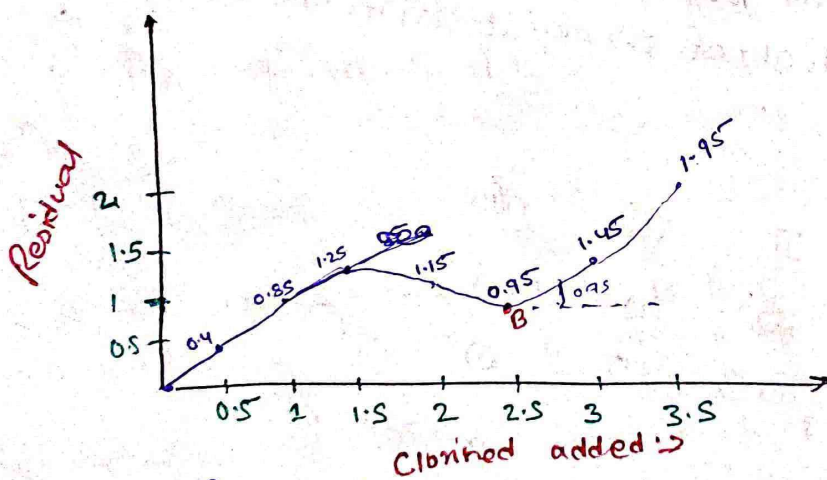
$$3CR \rightarrow 3FR \rightarrow 75CR$$

Q For the given data, compute (i) CD (ii) Break point Does, (iii) CD at does of 3mg/lit. (iv) Does of chlorine required for free residual of 0.75

Ac (mg/lit)	RC (mg/lit)	CD'	CD
0.5	0.4	$0.5 - 0.4 = 0.1$	$0.5 - 0.4 = 0.1$
1.0	0.85	$0.5 - 0.45 = 0.05$	$1 - 0.85 = 0.15$
1.5	1.25	$0.5 - 0.4 = 0.1$	$1.5 - 1.25 = 0.25$
2.0	1.15	$0.5 - (-0.15) = 0.65$	$2 - 1.15 = 0.85$
(ii) <u>2.5</u> Break point	0.95	$0.5 - (-0.2) = 0.7$	$2.5 - 0.95 = 1.55$ (iii)
3.0	1.45	$0.5 - 0.5 = 0$	$3 - 1.45 = 1.55$ (iii)
3.5	1.95	$0.5 - 0.5 = 0$	$3.5 - 1.95 = 1.55$

(iv) Break point =
after = free residual of 0.75, chlorine demand = 1.55

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B = Break point

(i) Chlorine demand = $2.5 - 0.95 = 1.55$

(ii) Break point does = 2.5

(iii) Chlorine demand @ 3 mg/lit = it is after Break point
= 1.55 mg/lit.

(iv) Does of chlorine required for free residual of 0.75
= $1.55 + 0.75$

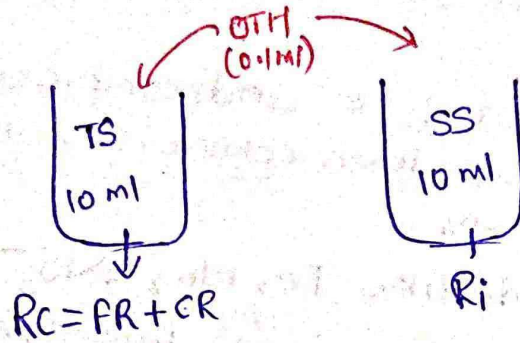
Testing of chlorine Residual.

Testing of chlorine residual can be done by any one of following methods:-

① ORTHOTOLIDINE TEST

→ This test is based upon colour matching technique. & the colour is induced by addition of orthotolidin.

(i)

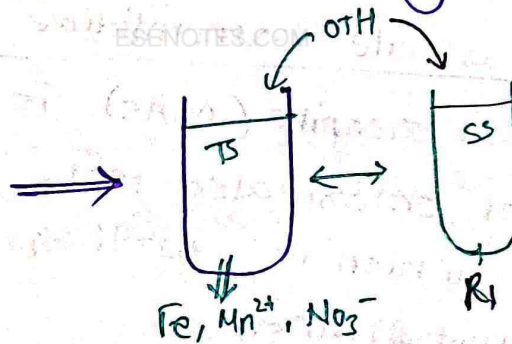
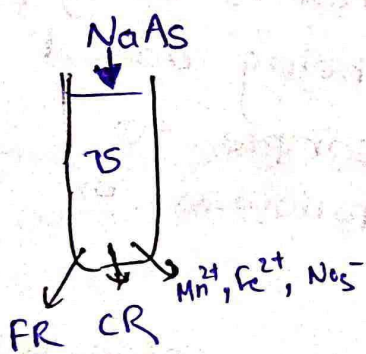


$$\frac{5 \text{ sec}}{FR = Ri}$$

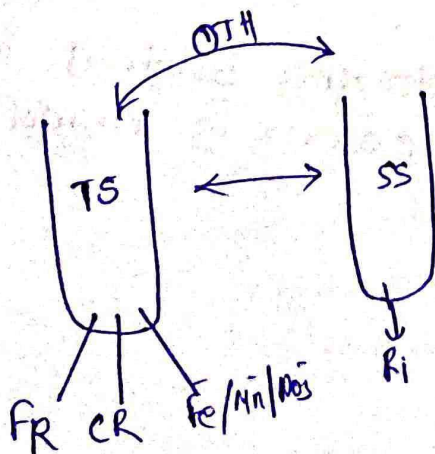
$$FR + CR = R_2$$

$$CR = R_2 - R_1$$

ii) If colour causing compound (Fe, Mg) are present in water orthotolidine arsenite test is being performed. (OTA).



$$Fe^{2+}, Mn^{2+}, NO_3^- = R_1$$



$$FR + Fe^{2+}/Mn^{2+}/NO_3^- = R_2$$

$$FR = R_2 - R_1$$

$$\frac{5 \text{ min}}{FR + CR + Fe^{2+}/Mn^{2+}/NO_3^- = R_3}$$

$$CR = R_3 - R_2$$

→ Practically this test obsolete now.

Procedure

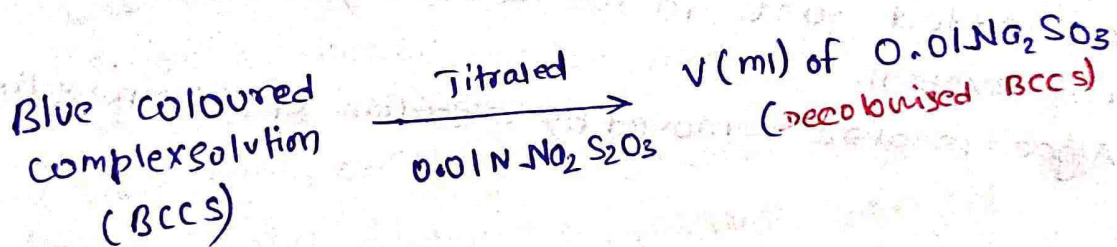
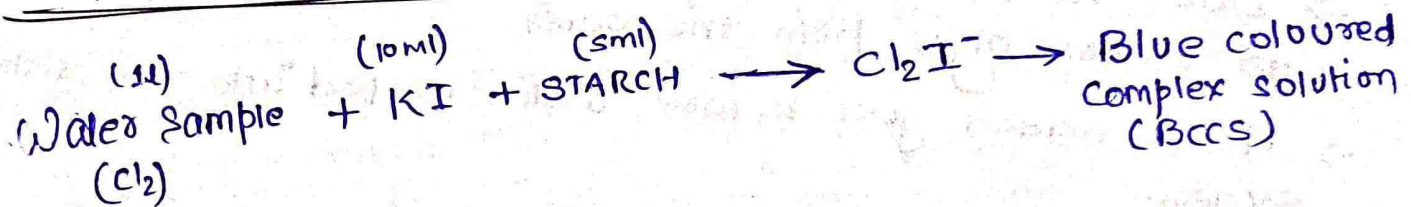
- In this test 10ml of chlorinated water sample is mixed with 0.1 ml of orthotolidine which gives yellow colour in it. Intensity of which is further matched with that
- Orthotolidine or 2-Tolidine is $C_{14}H_{16}N_2$ of standard sample. If the intensity of colour of test sample is same with that of the standard sample, then chlorine in standard sample is also same as that of test sample.
- With separate concentration of free and combined chlorine in residual form is desired, then colour is noted after 35 and 5 min respectively.
- If colour causing compounds like Fe, Mn, NO_3^- , are also present in the water, they give false yellow colour during the test resulting in higher value of residual chlorine than actual.
- In such case, arsenite orthotolidine test is being performed in which sodium arsenite (NaAs) is being added to the original chlorinated test sample to dechlorinate it which is further followed by addition of orthotolidine.
- Intensity of the colour induced by these colour causing compound is matched with that of the standard sample
- Orthotolidine is also added to the original chlorinated test sample and colour is noted after 5 minute respectively.

In today's time Orthotolidine test DPD (Diethyle phenylene diamine) and chloroex test is used which are also based upon colour matching technique. and are more accurate than orthotolidine test but are also comparatively costlier.

→ These two reagent DPD and chloroex are also termed as BDH (British Drug house) reagent.

Note:- Orthotolidine test may also be referred as ~~orthotolidine~~ method of STORT (stabilized neutral-o-tolidine) method

(ii) Starch Iodide Test. (Iodimetric Test)



$$\text{gm. equivalent of Na}_2\text{S}_2\text{O}_3 \text{ used} = \frac{0.01}{10^3} \times V = \text{gm. eq. of Cl}_2$$

$$\text{Cl}_2 (\text{mg/lit}) = \frac{0.01}{10^3} \times V \times 35.5 \times 10^3 = 0.355 V$$

* Starch reacts with complex of halogen gases. [Chlorine, fluorine, bromine, iodine, astatine].

Minor Methods of Treatment:

(i) Treatment with Activated Carbon.

→ Activated carbon in powdered form is added in water to remove taste, colour, odour, phenol, flowid, Cl^- , H_2S , Fe^{2+} , Mn^{2+} etc from water by absorbing it over the surface.

→ It is added either during coagulation or during filtration.

→ Its normal dose varies between 5 to 20 mg/lit.

(ii) Treatment with CuSO_4

→ Due to its strong ~~algae~~ algacidal property, it is used to remove algae from the water.

→ It is added just before water is fed into the distribution system.

→ Its normal dose is 0.5 - 0.6 mg/lit.

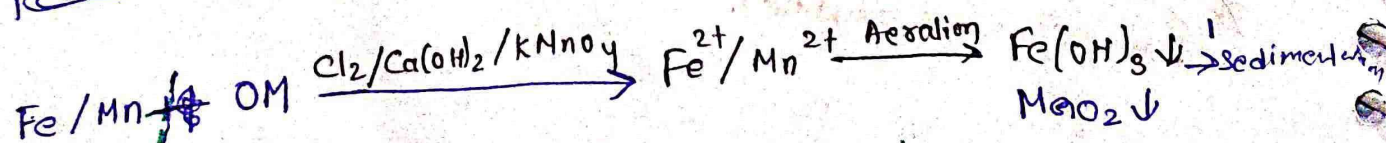
→ It is ~~just~~ added just in the water just before water is fed into distribution system.

Note:- Algae is also removed by addition of chlorine & lime.



(iii) Removal of Fe and Mn

Iron and manganese are generally present in conjunction with organic matter in water. Hence in order to remove iron manganese by aeration, bond between Fe/Mn and OM is first broken by addition of lime, ~~potassium~~ KMnO_4 or Cl_2 .



(Fe and Mn are removed during aeration process.)

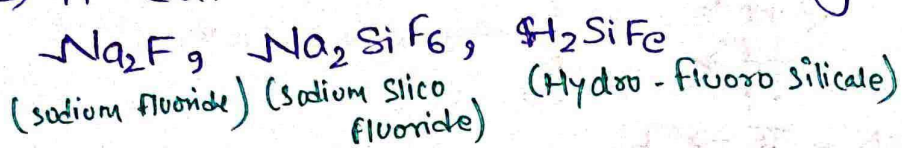
→ It is also known as De-ferrization (removal of Fe)

- Fe/Mn can also be removed with the help of magnese zeolite. which is a natural green sand, coated with the magnese dioxide.
- After magnese zeolite becomes exhausted it is regenerated by addition of potassium permagnet (KMnO_4)

4) Fluoridation fluoride (1ppm - 1.5ppm)

→ It is the process of addition of fluoride in water. if less than 1ppm.

→ It can be achieved by following chemicals,



5) De-fluoridation

→ The process of removal of excess fluoxine from water if its concentration is greater than 1.5ppm.

→ It can be achieved by any of following chemicals; -

(i) Calcium Phosphate ($\text{Ca}_3(\text{PO}_4)_2$),

(ii) Bone charcoal

(iii) Activated carbon.

(iv) (Alum + $\text{Ca}(\text{OH})_2$) = Nallaconda technique.

(v) Activated Alumina = Prashanti method.

6) Desalination

→ It is process of removal of salts from water.

→ It can be achieved by any of the following method.

(a) Reverse Osmosis.

(b) Evaporation

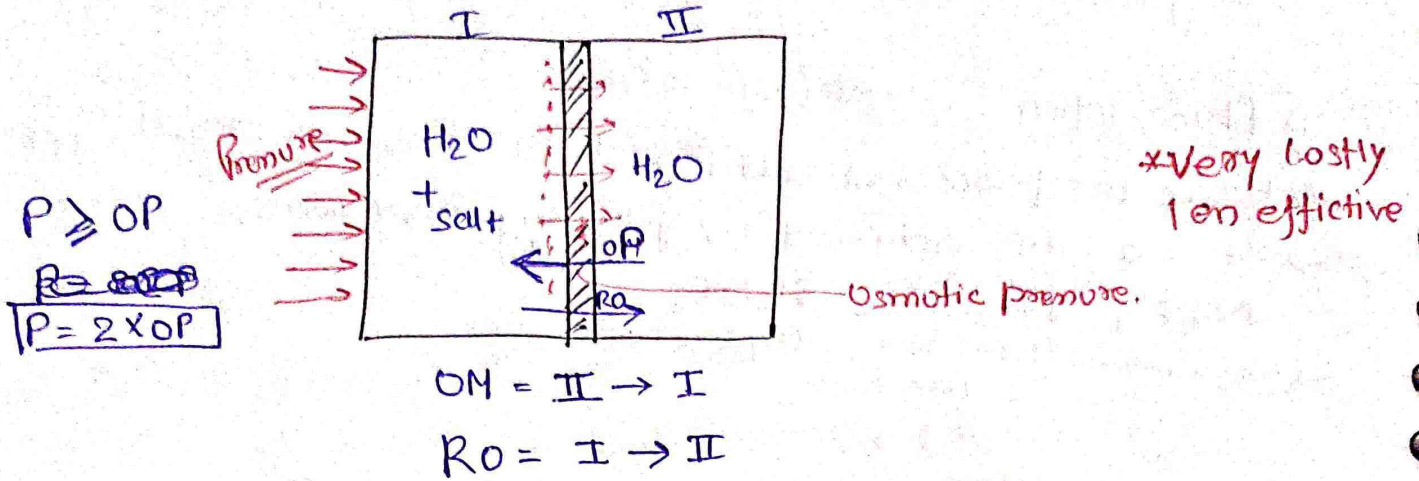
(c) Freezing

(d) ~~Electrolysis~~
Electrolysis.

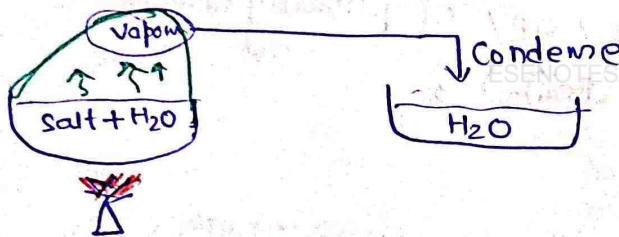
Reverse osmosis

→ It is just opposite of osmosis process.

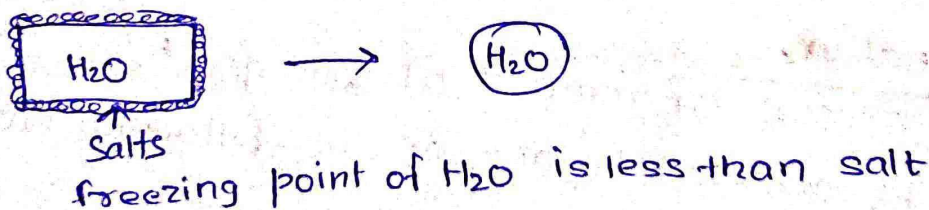
→ Osmosis is the movement of water through a semi permeable membrane from a region of high concentration to a region of low concentration.



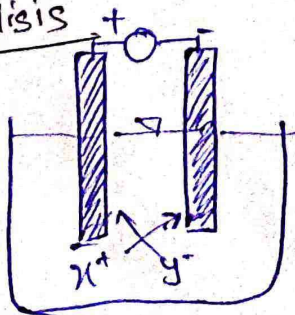
Evaporation (distillation)

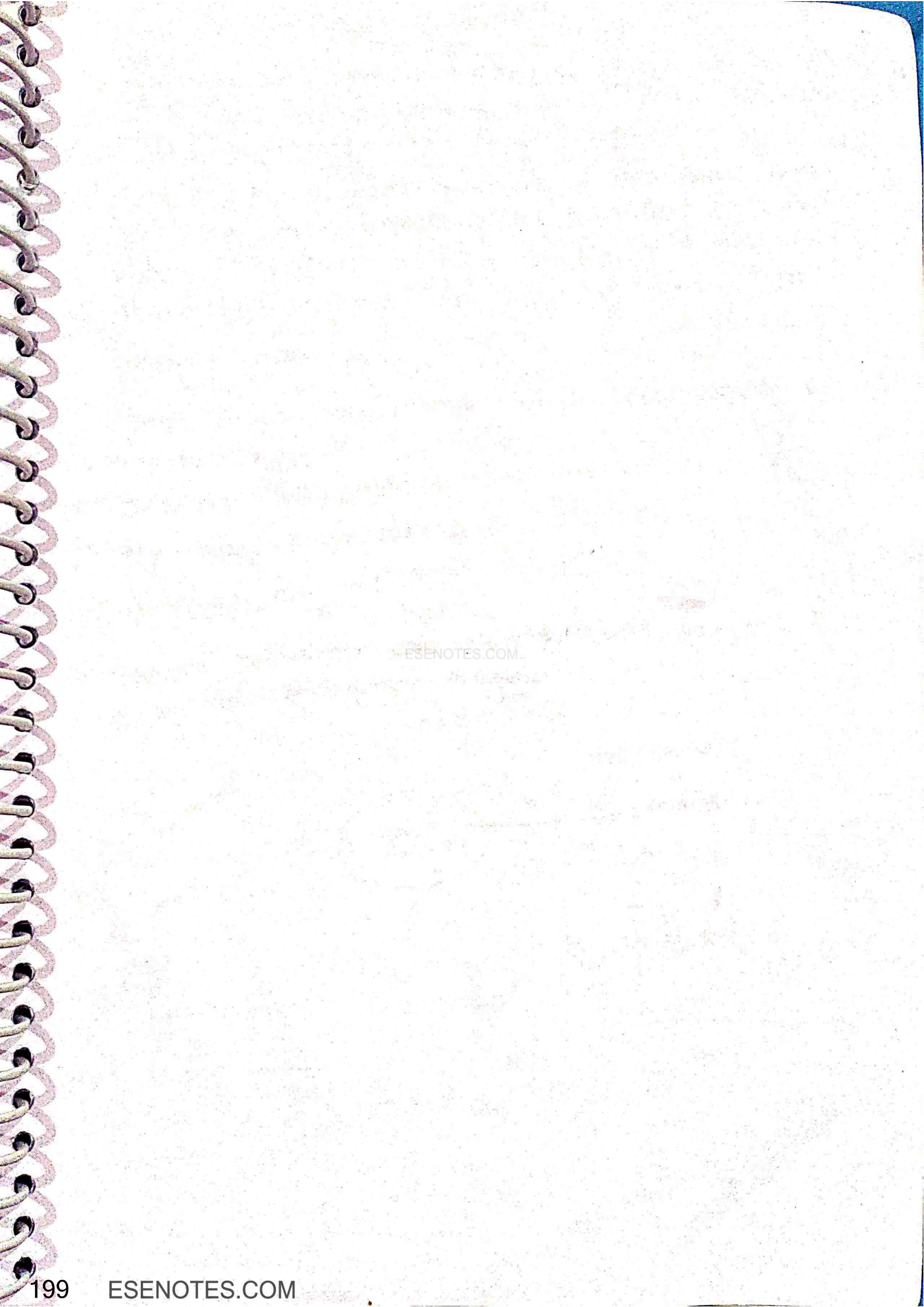


Freezing



Electrolysis





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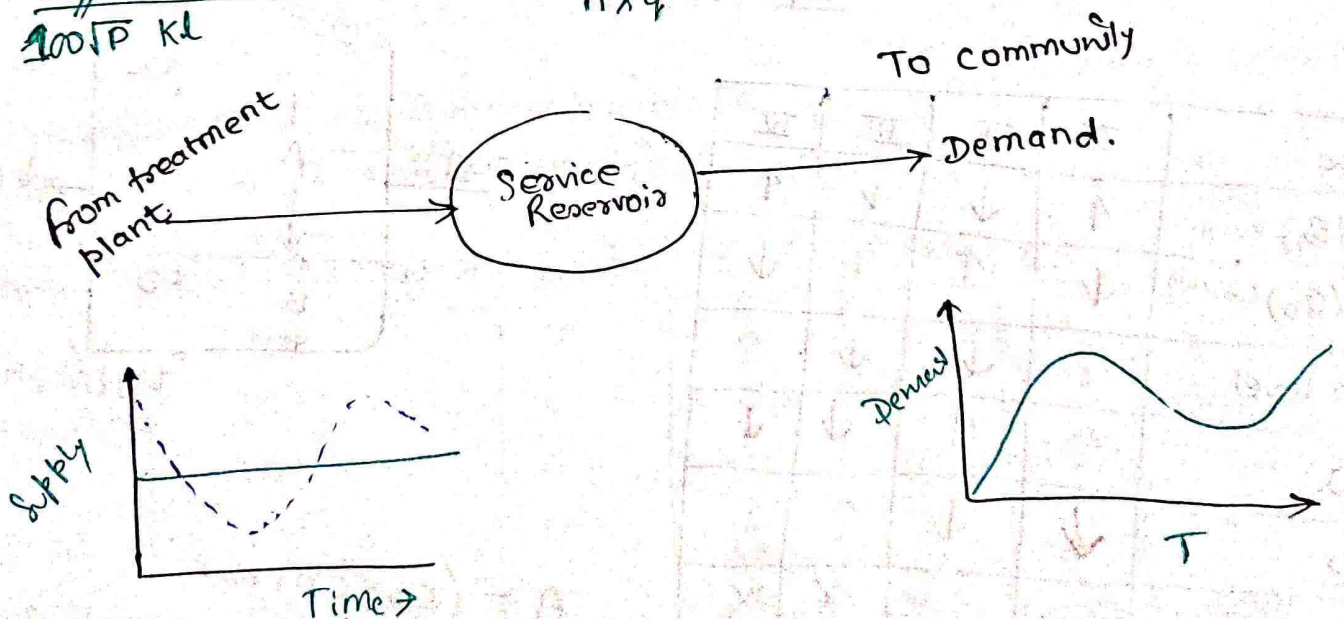
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Service Reservoir

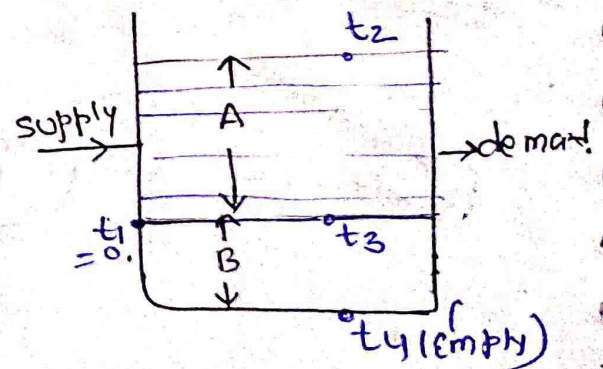
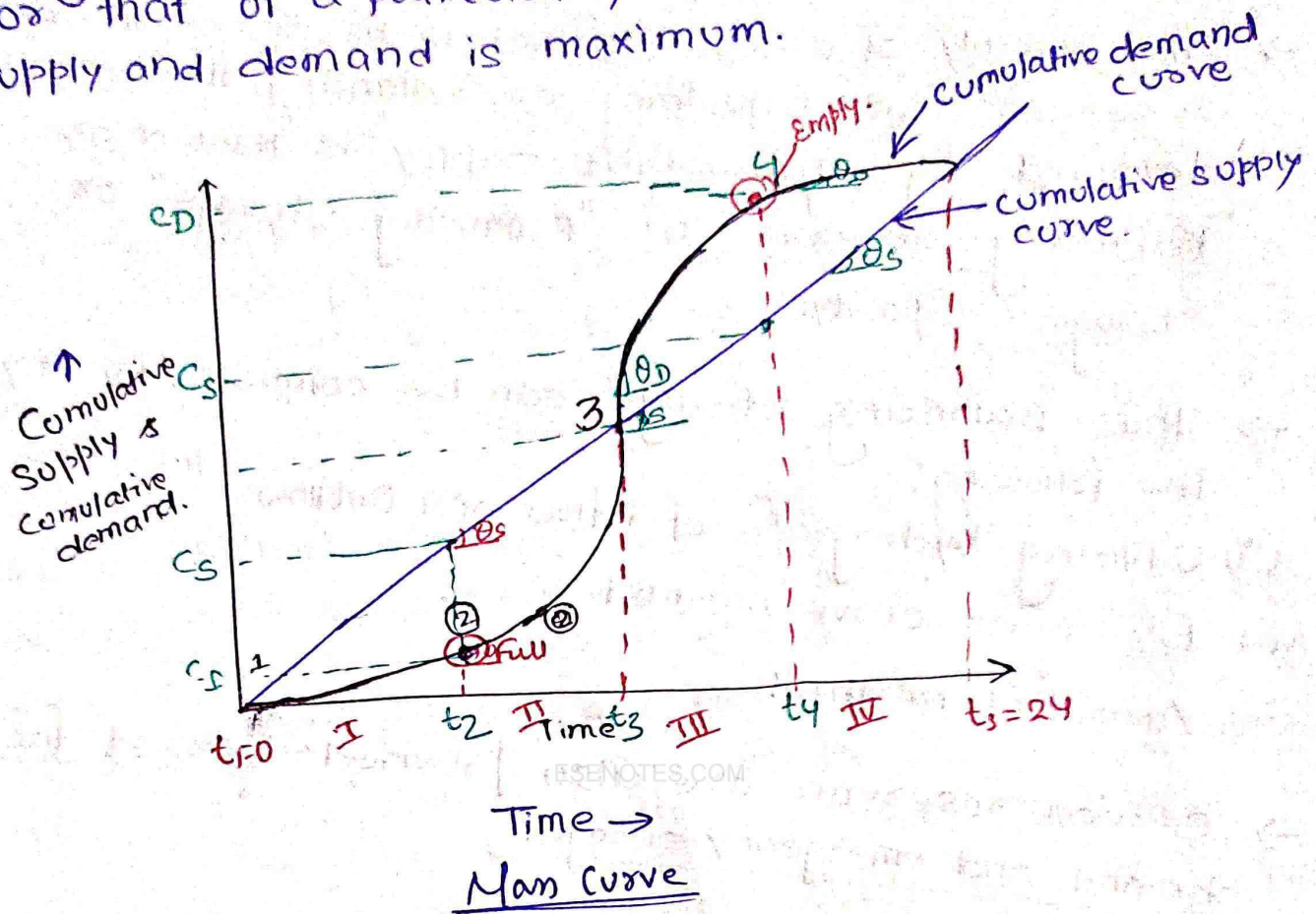
- The main and primary function of distribution reservoir is to meet the fluctuating demand with a constant rate of supply from the treatment plant.
- The quantity of water required to be stored in the reservoir for equalising or balancing this variable demand against constant supply is termed as "Balancing reserve" or "Balancing storage" or storage capacity.
- This Balancing storage can be computed by any of the following:-
 - (i) Utilizing hydrograph of inflow and outflow.
 - (ii) By Mass curve method.
 - (iii) Analytical method.

→ Service reservoir are also provided ^{storage} to meet fire demand and emergency storage.



Mass Curve Method

- It is the plot of accumulated inflow or supply or outflow (demand) ^{with time}
- Using this mass curve balancing storage is found for that of a particular year when variation of supply and demand is maximum.



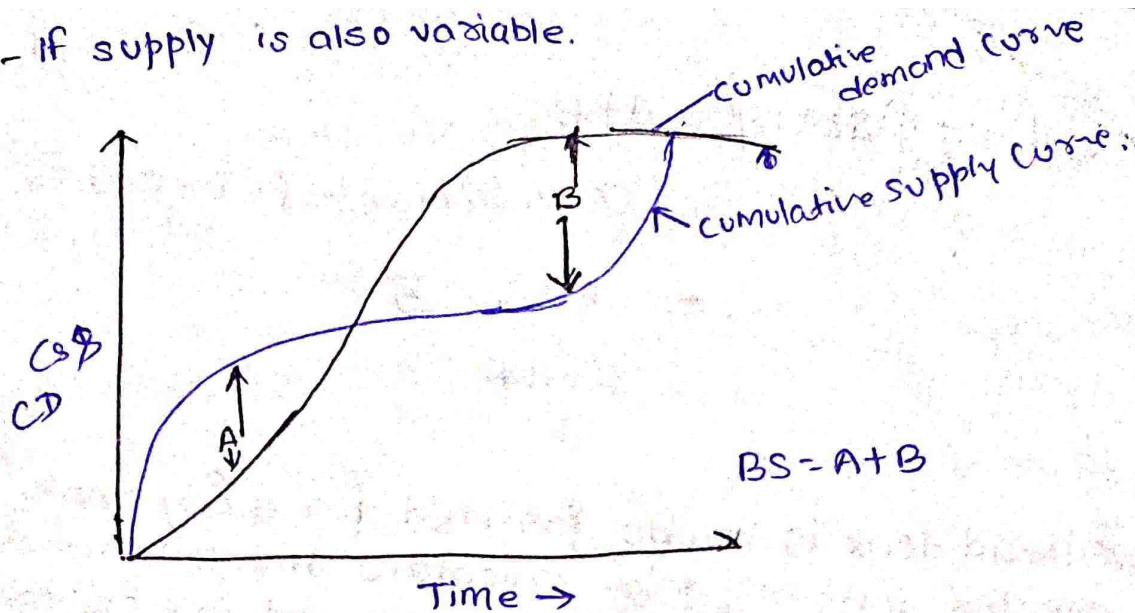
$$A = (CS - CD)_{\text{Max}} = \text{Maximum accumulation.}$$

$$B = (CD - CS)_{\text{Max}} = \text{Maximum depletion}$$

$$\text{Balancing storage} = A + B$$

Phase	I	II	III	IV
ROS (θ_s) (coml.)	↑	↓	↓	↑
ROD (θ_D) (downlike)	↓	↑	↑	↓
Water level	↑	↓	↓	↑
CS	↑	↑	↓	↓
CD	↓	↓	↑	↑
Accumulation	✓	✓	X	X
Depletion	X	X	✓	✓

Note:- If supply is also variable.



Q A town having a population of 1 lakh is to be supplied with water @ rate of 200 l/c/d. The variation in the demand is as follows:-

Time	% of total discharge	
6-9am	40%	80
9-12am	10%	20
12-3pm	10%	20
3-6pm	15%	30
6-9pm	25%	50

Determine the minimum capacity of reservoir when pumping is done at uniform rate from 6 am to 6 pm.

Demand = $200 \times 10^5 \times 10^{-6}$
= 20MLD

Time	% of total discharge	Demand	CD	Supply	CS	CD - CS
6-9am	40	8	8	5	5	3
9-12am	10	2	10	5	10	0
12-3	10	2	12	5	15	-3
3-6	15	3	15	5	20	-5
6-9	25	5	20	5	20	0

= Supply = demand = 20MLD

= Rate of supply = $\frac{20ML}{12hr} = 5M/ghr$

$$\text{Balancing storage} = A + B$$

$$= (CS - CD)_{\text{max}} + (CD - CS)_{\text{net}}$$

$$= 5 + 3$$

$$= 8 \text{ ML}$$

Q An overhead tank is to be provided for a city water supply given the following data. Calculate the minimum capacity of the tank without any fire demand, and fire storage. Assuming the tank to be empty between 12 to 15 hours. Compute the water level in reservoir at different time interval.

Time	Water pumped	Water Consumed.	CD	CS	CD-CS	Water level
			13333	0	13333	8833
0-4	0	13333	53333	50000	3333	18333
4-6	50,000 >	40000	173333	150,000	23333	21333
6-10	1,00,000 <	1,20,000	206666	200000	6666	25000
10-12	50,000 >	33333	231666	200000	31666	0
12-15	0	25000	239999	225000	14999	16667
15-16	25000	8333	346665	325000	21665	10001
16-20	100000	106,666	396665	400000	-3335	35001
20-23	75000	50,000	89999	400000	0	3666
23-24	0	3333	400000			

$$\text{Balancing storage} = A + B$$

$$= 31666 + 3335$$

$$= 35001 \text{ liter.}$$

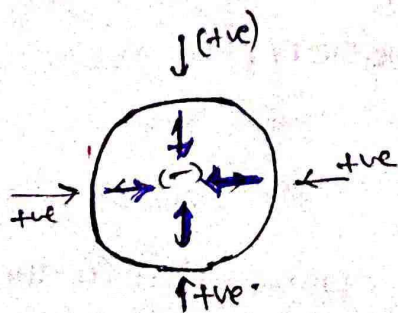
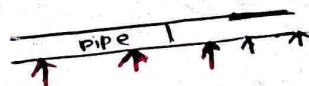
$$\text{Water level} = (\text{supply} - \text{demand}) + \text{water level of previous interval}$$

Different types of pipes require to carry water.

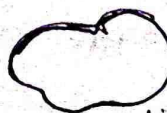
→ Pipes carrying water from one point to another in raw water scheme are being subjected to following types of forces:-

- (i) Internal Pressure of water including water hammer pressure (to be resisted by using material which is strong in tension)
- (ii) Pressure due to external loads in the form of Backfill, traffic load (to be resisted by using material strong in compression).
- (iii) Longitudinal temperature stress which are created when pipes are laid over the ground to be resisted by providing expansion joints.
- (iv) Longitudinal stress created due to unbalanced pressure at bends or at point of change in cross-section (to be resisted by holding the pipes firmly by anchoring it in massive block of concrete or stone masonry).

(v) Flexural Stresses. produced when pipes are supported over restles.



$$\frac{(+ve)}{\text{from top of pipe}} = \frac{(-ve)}{\text{from inside the pipe.}}$$



crushing failure.

Different types of pipes available to be used to carry the water:-

(i) Cast-iron^(CI) pipe / Galvanized pipe (GI pipe).

- These pipes are moderate in cost.
- Easy to join.
- Strong and durable. (compression)
- Sufficiently resistant against corrosion.
- Long life upto 100 years.
- Service connection can be made easily.
- ^{disadvantage} Water carrying capacity decreases with time as the value of friction increases due to deposition ferric hydroxide (Fe(OH)_3) (tuber cules). termed as tuberculation.
→ Use GI pipe to prevent from it (1)
- It can't resist water pressure greater than 700 kN/m^2 .
- When these pipes are large they are heavy and uneconomical.
- They are likely to break during transportation or while making connection.
- They are suitable to be used as Distribution main.

(ii) Steel Pipe / Hume steel Pipe.

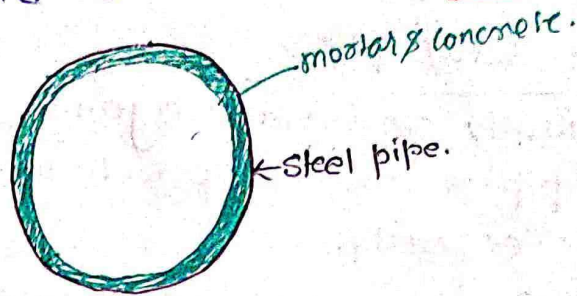
- These pipes can ~~resist~~ withstand high water pressure as they are strong in tension but ~~it~~ can't take heavy external load or vacuum.
- These are susceptible to corrosion.
- Have high maintenance cost.
- Shorted life (25-50) years.
- It is difficult to provide service connection in them.

→ They are light in weight hence offer low transportation cost.

→ They are suitable to be used as pumping main.

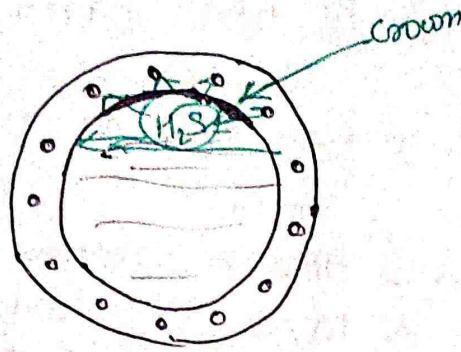
Note:-

In order to increase the resistance of these pipe against corrosion a layer of mortar/concrete is provided over their surface and is termed as "TUME Steel pipe".



(3) Reinforced Cement Concrete Pipe (RCC pipe)

- They can resist external compressive load and do not collapse under nominal vacuum and traffic load.
- They are not corroded from inside by normal potable water. & from outside by ordinary soil but may undergo hydrogen sulphide corrosion (H_2S) at its crown.
- They are quite strong, and their useful life upto 75 years.
- They are easy to construct at site or in factory.
- Their coefficient of thermal expansion is low hence expansion joints are not required.
- If laid under water, the empty pipes do not float because of their own weight.
- They are difficult to be repaired.
- They can't withstand high water pressure.
- They are heavy and bulky, ~~and~~ hence it's difficult to transport.
- Making connection in them is difficult and they tend to leak due to shrinkage crack.



4) Vitrified Clay pipes.

- It is virtually resistant against corrosion.
- These pipes are smooth hence offer high discharge carrying capacity.
- These pipes are not used as pressure pipe for carrying water. but are used for sewage and drainage at partial depth. (because clay is weak in tension and formation of water tight joint is difficult in them).

5) Asbestos Cement Pipe (AC pipe)

- These pipes are light hence easy to transport
- They can be easily assembled without skilled labour.
- They are highly resistant to corrosion.
- They are highly flexible (may permit deflection of 12°) when laying around the curves.
- They are smooth hence offer high discharge carrying capacity. ~~and~~ do not have much strength and are brittle and soft
- Very costly.
- The rubber joint seal may deteriorate if exposed to petroleum product. hence not used in this case.

Pipe Appurtenance.

Different units or accessories provided in the pipe for its proper functioning is termed as pipe appurtenance.

like:- (i) Valves.

(ii) Joints.

(iii) Arch or ages

Valves

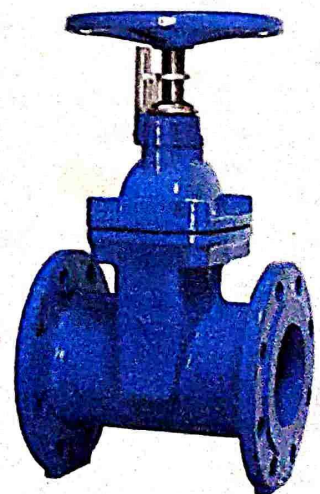
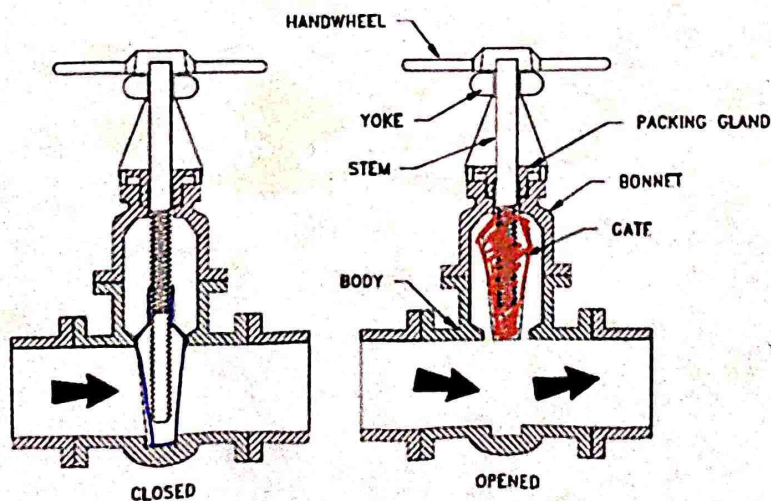
(i) Gate Valves or Sluice Valve or cut off valve.

→ These valve are used to regulate the flow of water through pipes. by dividing it into number of section

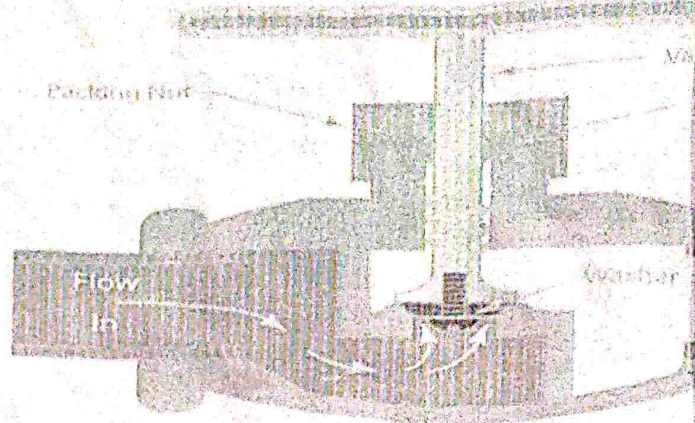
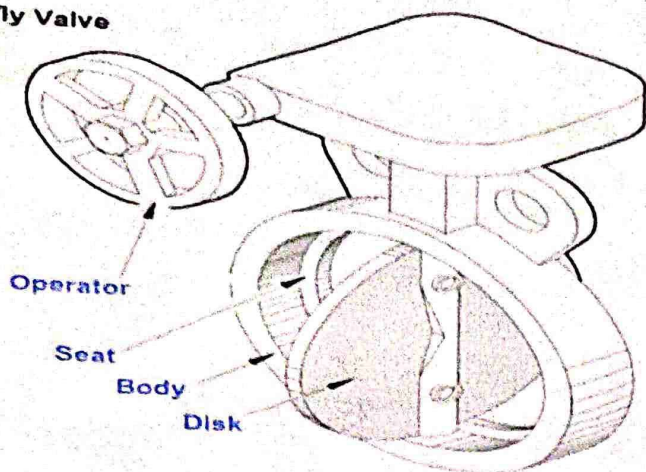
→ These valve are placed at the summit point in the pipes as pressure to be resisted by these valves is minimum at this points, (Because of conservation of energy) requiring lower strength of the material that inturn reduces the cost of the valves.

→ The valve is made of cast iron and with Brass, Bronze, or stainless steel mountings.

→ These are manually operated valves.



Butterfly Valve



Air Valves

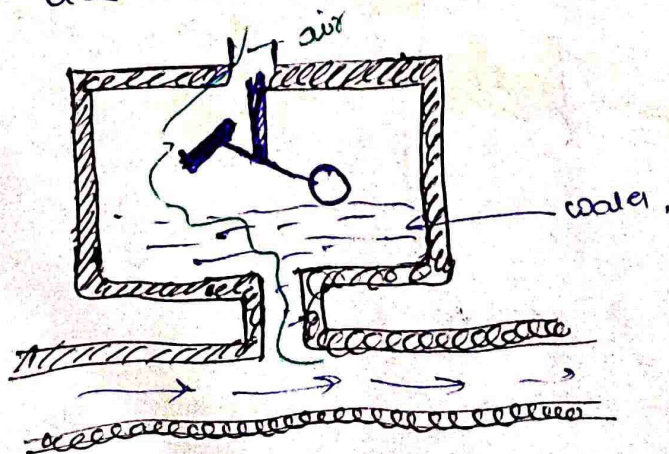
- These valves are used to ensure the safety of the pipes against failure.
- Air valves are the special kind of valves which are generally placed along the pipe lines at "summit" on both sides of sluice valves and also on the downstream side of all the other sluice valves.
- These valves are automatic valves.

It is further divided into two parts:-

(i) Air inlet valve - The pressure in the pipe falls below a certain fixed predetermined value and thus allowing air to enter the pipe. Such a valve is known as Air inlet valve.

(ii) Air Relief Valve:-

These valve are required to be provided to all summits to remove accumulated air.

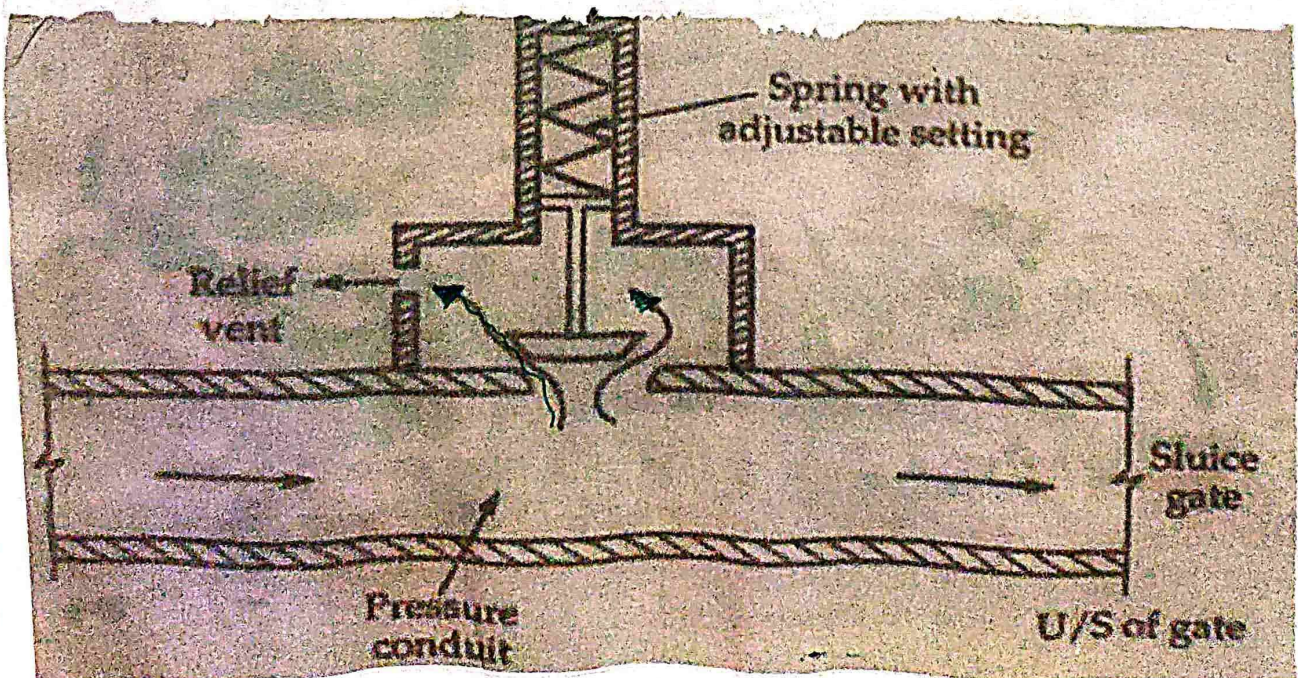


Drain Blow off/Scour Valve

- These valves are used to remove the entire water from within a pipe (after closing the supply), small gate off-takes are provided at low points.
- These valves are provided to drain the water & Scour off the settle suspended impurities out from the pipe network.
- These valves are placed at the lower most point in the pipe network in order to ensure the gravity drainage from either side of the gate valve.
- For safety purpose, two drains valves are generally placed in series, so as to reduce the chances of pollution reaching the water in the conduit.
- These valves are manually operated.

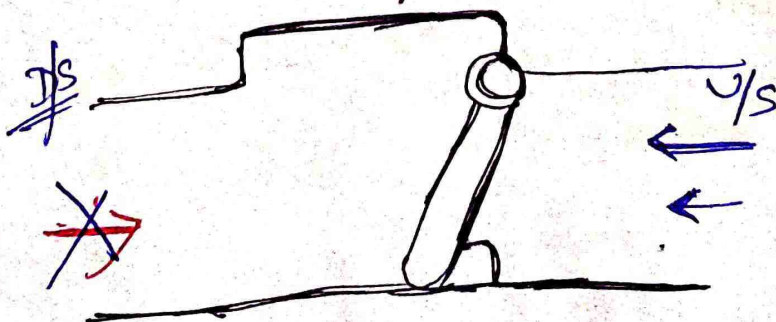
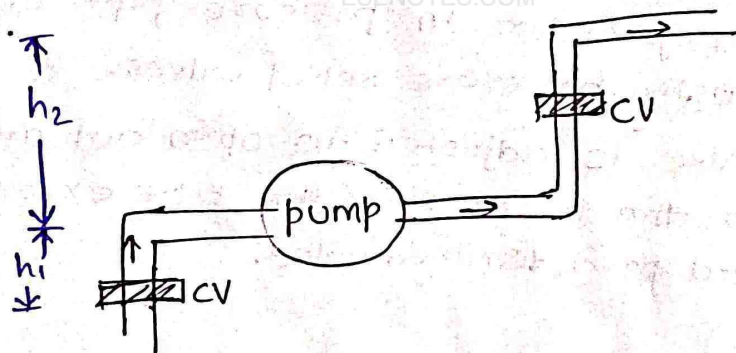
Pressure Relief Valve.

- Water hammer pressure in pressure pipe can be reduced by using pressure relief valves.
- Such a valve is adjusted to open out automatically as soon as the pressure in the pipe exceeds a certain fixed predetermined value.



Check Valves or Reflux Valves.

- They are also called as Non-return valves because they prevent to flow back in the opposite direction.
- These valves are also installed on pump discharge to reduce water hammer forces on the pump.
- These valves are placed for either side of the pump to check the flow of the water in single direction.
- These valves are required at inter-connections between a polluted water system and a potable water system so as to prevent the entry of pollution into the pure water.
- ~~These valves are required at inter-connection betw.~~
- A check valve installed at the end of a suction line is called foot valve. and prevents drainage of the suction when the pump stops.



Joints

There are following types of Joint provided in pipe network:-

- (i) Socket and Spigot Joint.
- (ii) Flanged Joint
- (iii) Mechanical Joint called dresser coupling.
- (iv) Flexible Joint.
- (v) Expansion Joint.

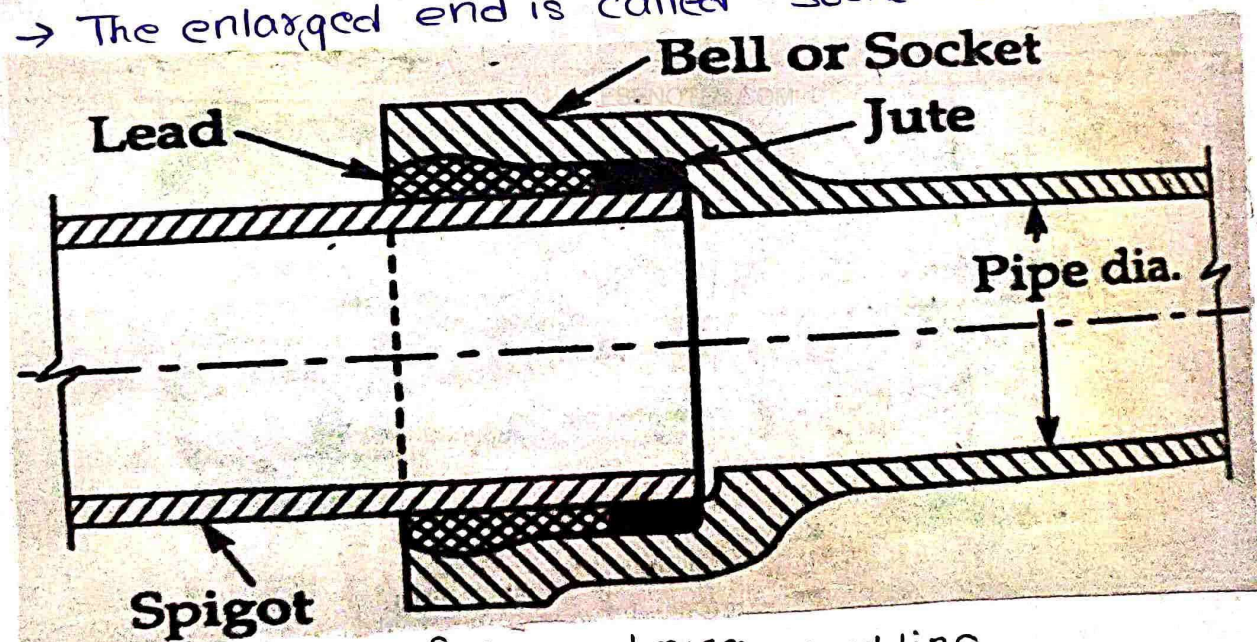
i) Socket and Spigot Joint

→ Cast iron pipes are being joined by a "socket and spigot joint".

→ Also used today on large scale.

→ These joints are made in such a way that one of their end is enlarged, whereas the other end is called is normal.

→ The enlarged end is called "Socket" and "bell" while

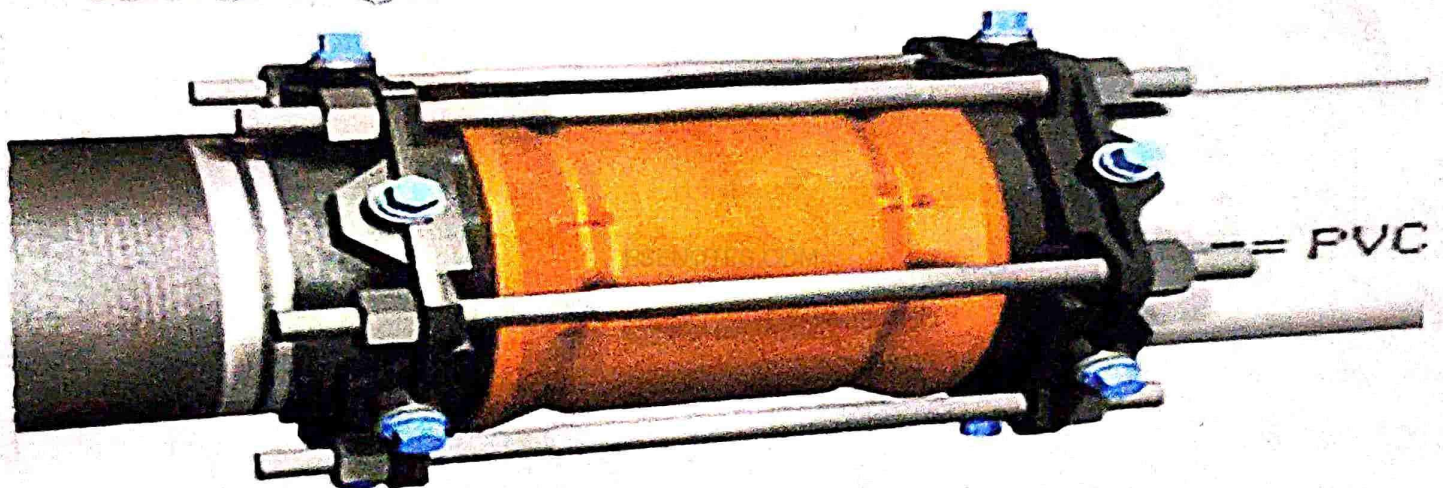
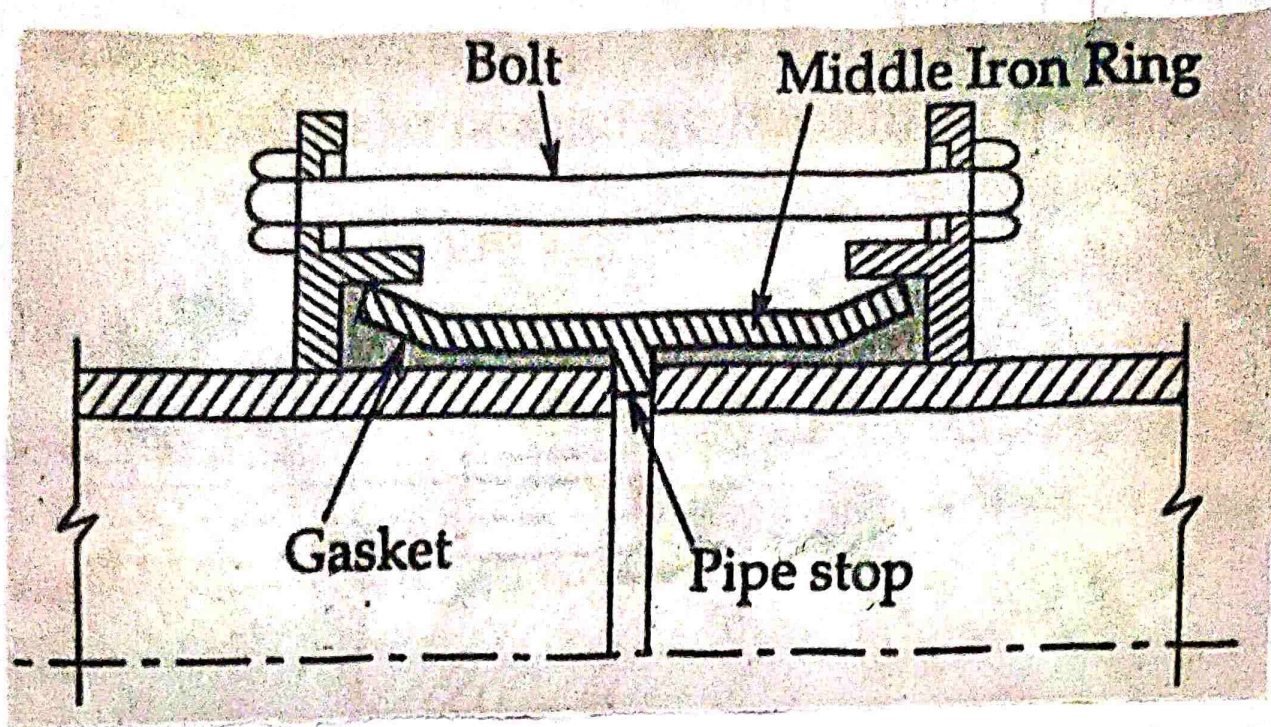


(ii) Mechanical Joint or dresser coupling.

→ These type of joint is used when it is required to joint the plain ends of cast iron pipes.

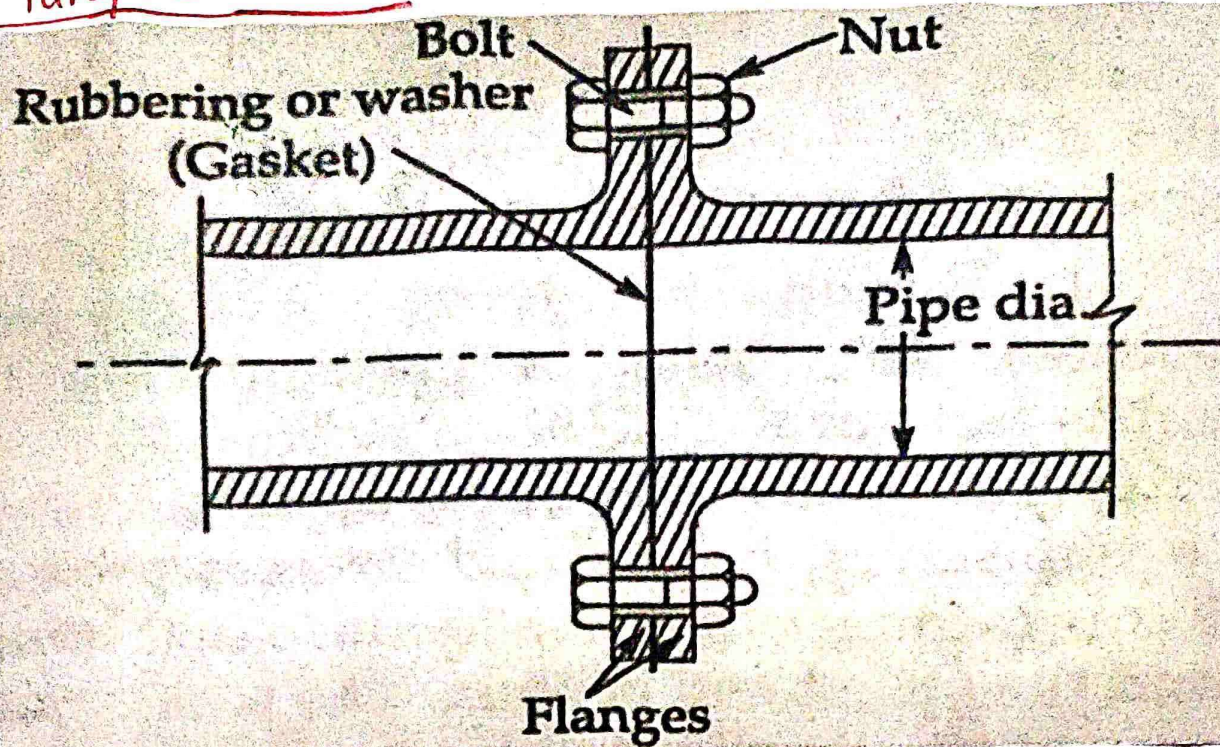
→ A special type of metallic collar is then fitted and tightened ~~and~~ over the abutting ends. thus forming a mechanical joint.

→ one of the most commonly used type of mechanical joint is a "dresser coupling".



→ These joint are strong and rigid. They can withstand vibrations and are therefore useful for pipes to be carried over bridges or below bridges in hangers.

Flanged Joint

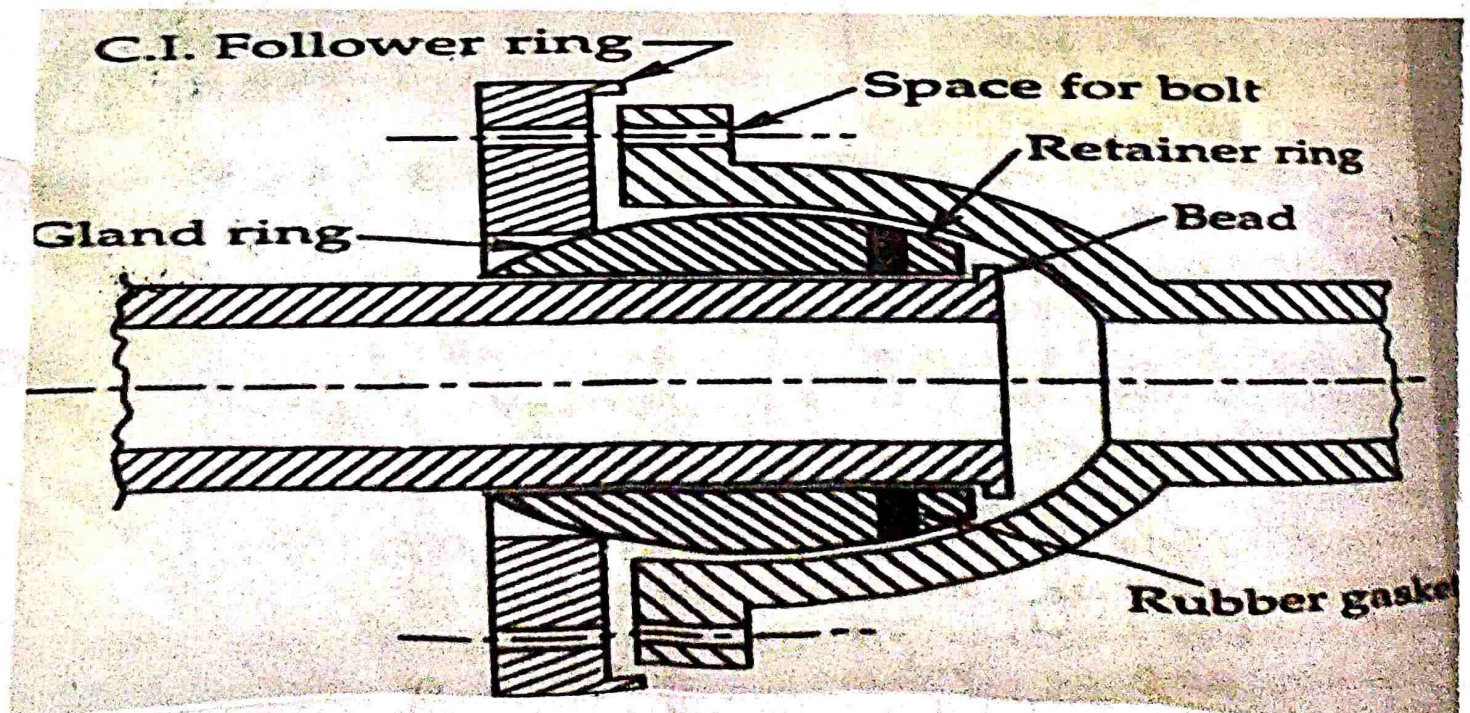


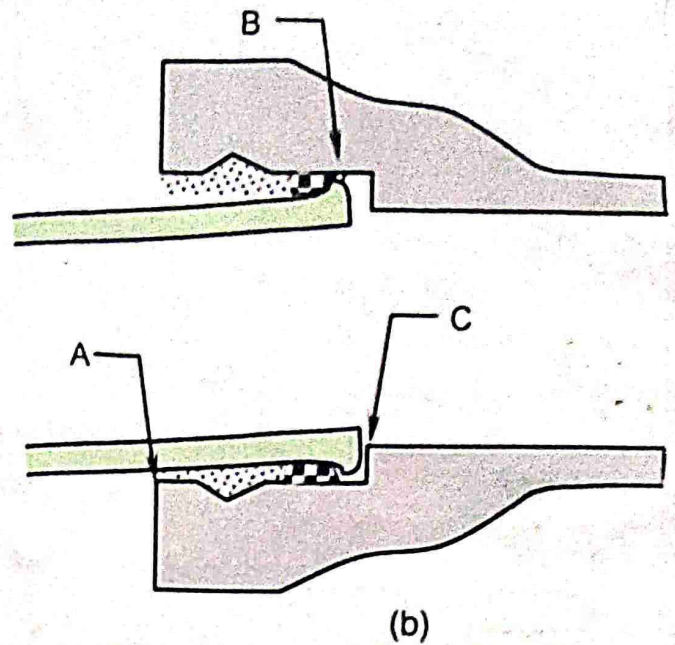
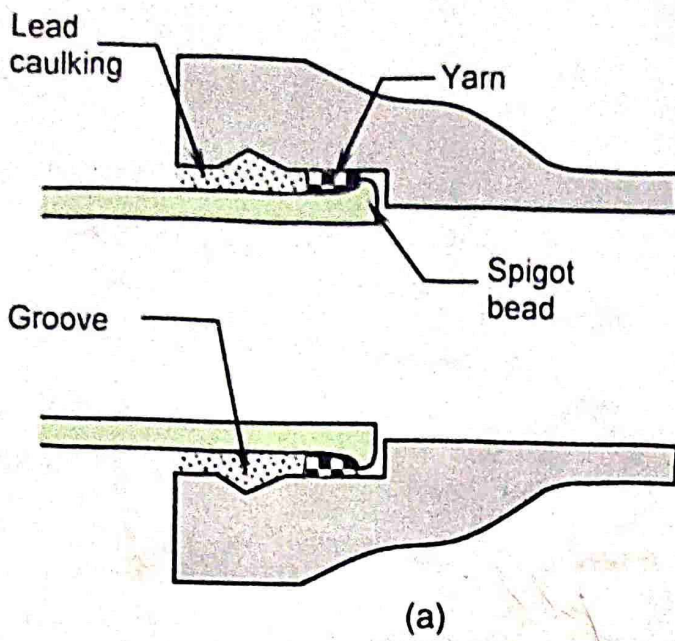
- These joint are used for pumping stations, fillers, points and at other location where it may be necessary to occasionally disjoint the pipe.
- Cast iron pipe length to be joined by this joint are cast in such a way as to have flanges at both ends.
- In case of steel pipes, flanges are separately cast and then screwed down or welded at both the ends of the respective pipe length.
- These joint are strong but rigid, and hence cannot be used where deflection or vibration are expected.
- They are expensive and mostly used for indoor works (such as pumping station, filter plants etc.)

Flexible Joint.

- These are used where large scale flexibilities are required.
- The pipes to be provided with such a joint are cast with special types of ends.
- The socket is spherical, and the spigot, through the plain is having a bead at the end.
- A retainer ring is placed over the bead which keeps the special rubber gasket (rubber gasket linked with a special kind of fabric called "duck".) in position.
- The spigot-end can be moved to give away the desired deflection and nuts are tightened over the gland ring.

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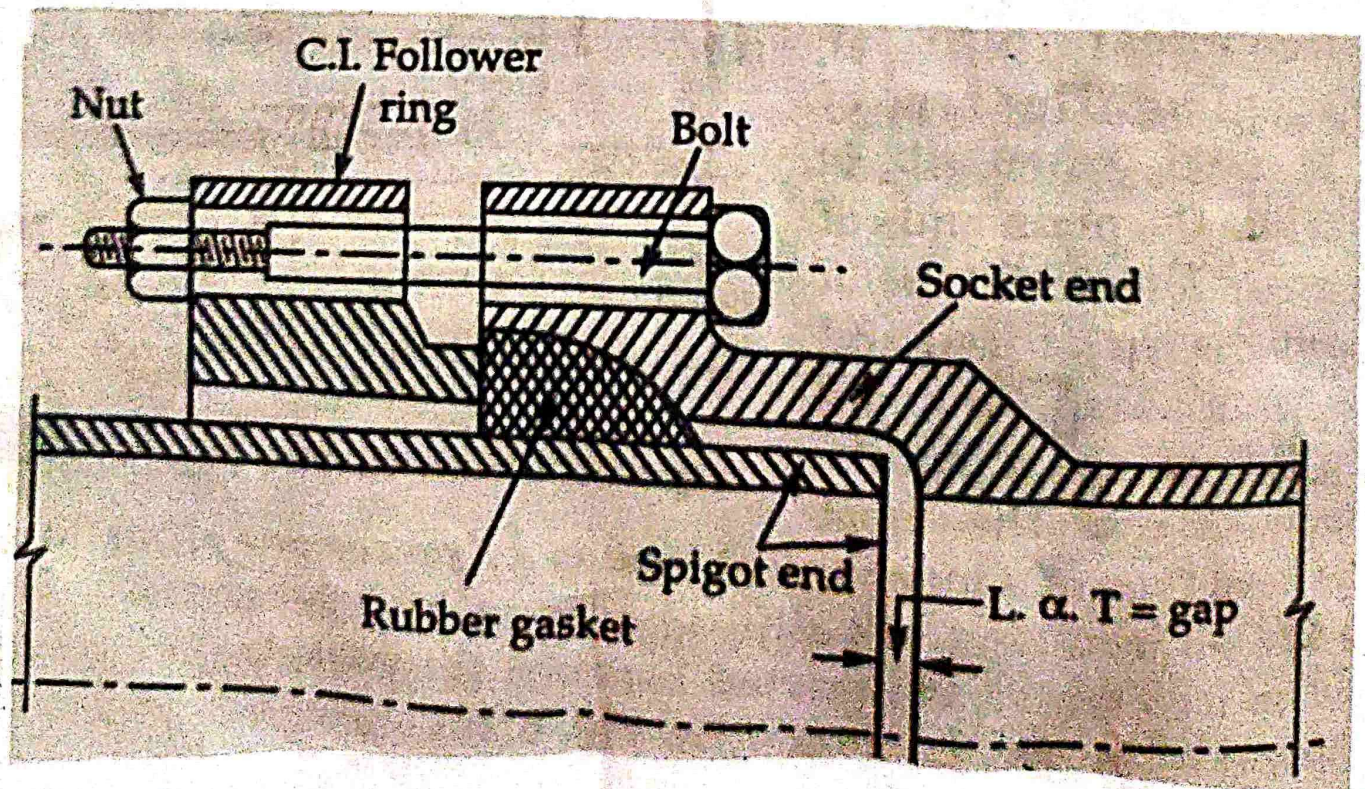


flexible joint

Expansion Joint.

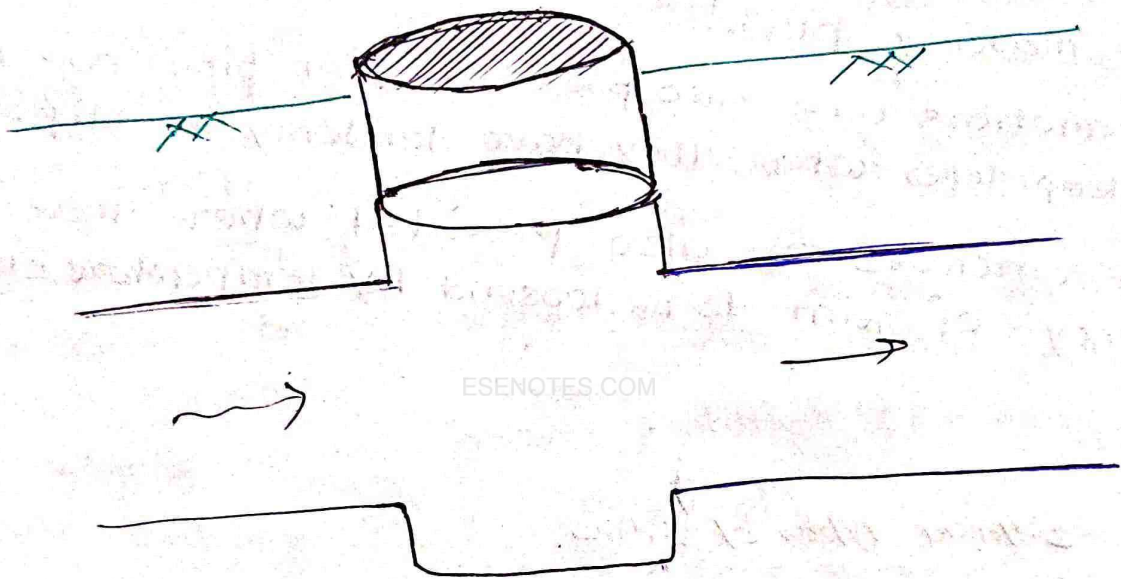
- These joint are provided at suitable intervals in the pipe lines, so as to counteract the thermal stresses produced due to temp. variation.
- For providing expansion joint in cast iron pipes, the socket end is cast flanged and the spigot end is plain.
- The socket end is connected rigidly to an annular ring which can slide freely over the spigot end.
- While making this joint, a small piece space is kept between the face of the spigot and the inner face of the socket, and the spigot is filled up by means of a rubber gasket. The flanges are then tightened by means of nuts and bolts.

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Manholes

- They are provided at suitable interval along the pipe so as to help its laying & to serve for inspection and repair.
- They are generally provided on large pipe, bringing water from source to the treatment plant.
- They are provided at the spacing of 300-600 meter.



Insulation Joint

- These joint are provided along the pipe lines at suitable interval so as to insulate the pipe against the flow of ^{stray} electric current. & thus to check electrolysis.
- Rubber gaskets or rings can be provided as insulator in between the pipe.
- In some cases rubber covering is also provided to resist the flow of current.

5 Anchorage

→ The pipe try to pull apart and get out of the alignment at bends and other point of unbalanced pressure at such ~~low~~ places the forces exerted on the joint due to longitudinal shearing stress caused by these unbalanced pressure are more & the joints may get loosened & leading to excessive leakage.

→ In order to prevent pipes & joint from pulling apart pipes are anchored by firmly embedding these portion of pipe in massive concrete or stone masonry block.

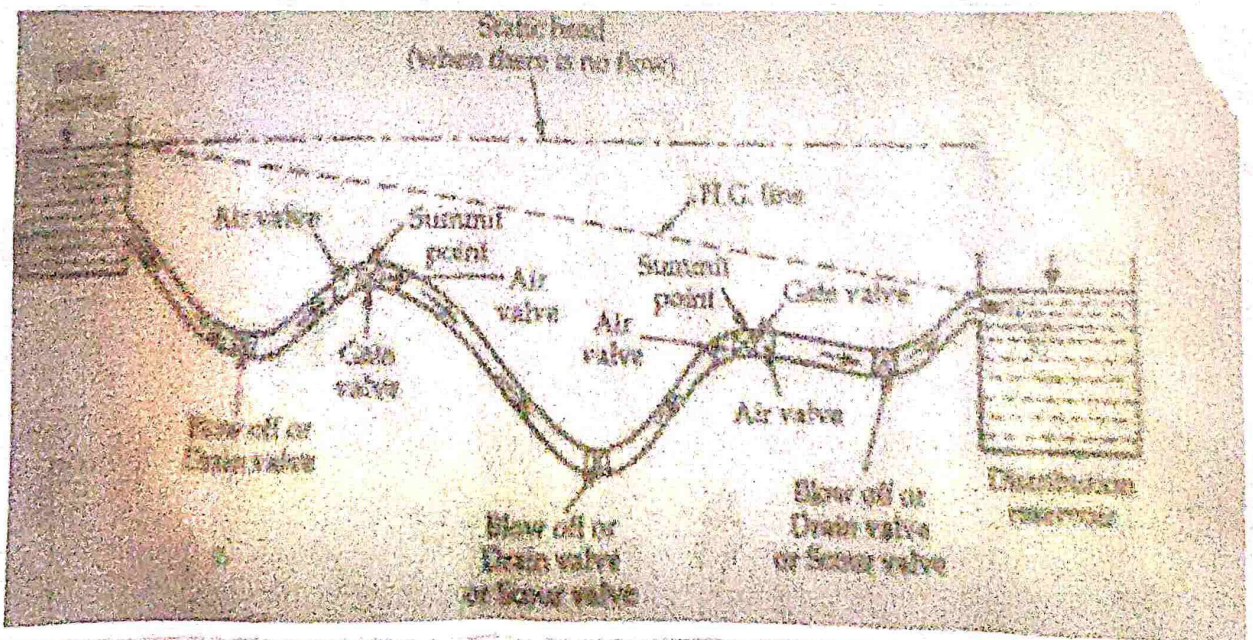
→ These anchors are also provided when pipes are laid on steep slopes where they have tendency to slip.

→ These anchors are also provided where there is tendency of joint to be loosened by temperature stresses

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Different types of Valves



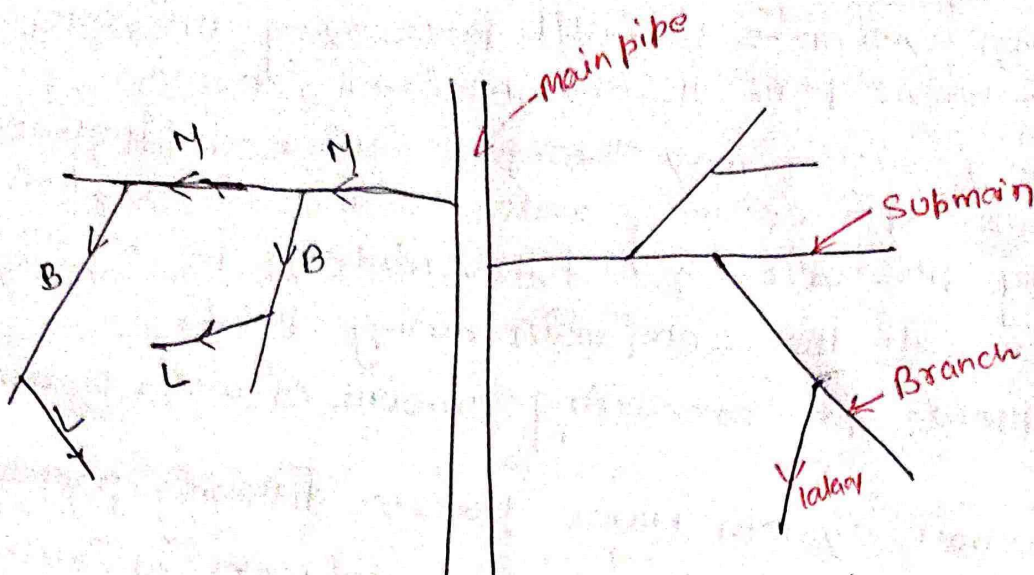
Distribution System.

- Distribution system is provided for carrying the water from treatment plant to the individual homes.
- Distribution system may therefore consist of pipelines of various sizes for carrying water to the streets, valves for controlling flow of water in pipe, hydrants for providing connection with the water main during fires, water meter for measuring amount of water consumed.
- Distribution system must possess following properties:-
 - (i) It should be capable of supplying water at all the intended places.
 - (ii) It should be capable of supplying requisite amount of water for fire fighting.
 - (iii) It should be cheap and simple and easy to design operate & repair.
 - (iv) It should be safe against pollution of water.
 - (v) It should be fairly water tight.

types:-

- (i) Dead end / Tree system.
- (ii) Grid iron / Reticulation / Interlaced system.
- (iii) Ring / circular system.
- (iv) Radial / Stem system.

(i) Dead-End Tree System.

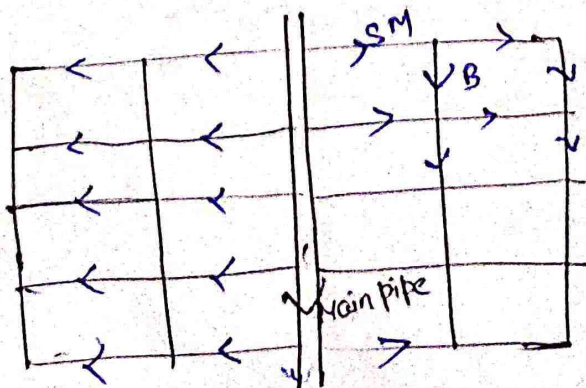


- This system is provided for those community where development is not planned.
- This system can be designed very easily it is comparatively economical as length of the pipe required is small & requirement of valve is less.
- It can be expanded easily.

Drawback

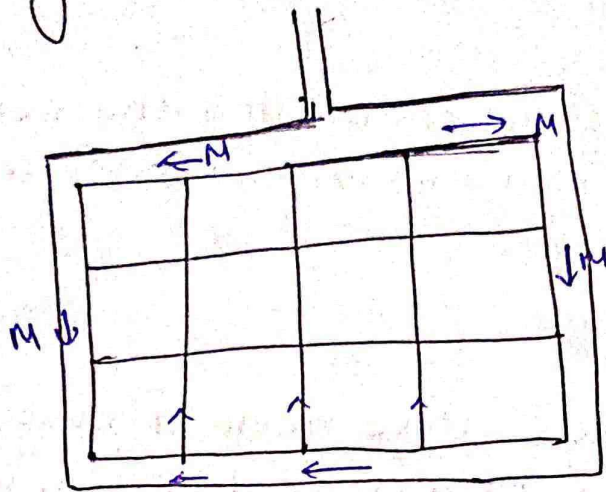
- As there is only one path available with the water to reach at a particular point in case of any obstruction downstream ~~con~~ section is affected.
- Due to number of dead end chances of contamination of water is very high.
- In case of fire breakout demand, can't be met.

(ii) Grid Iron system



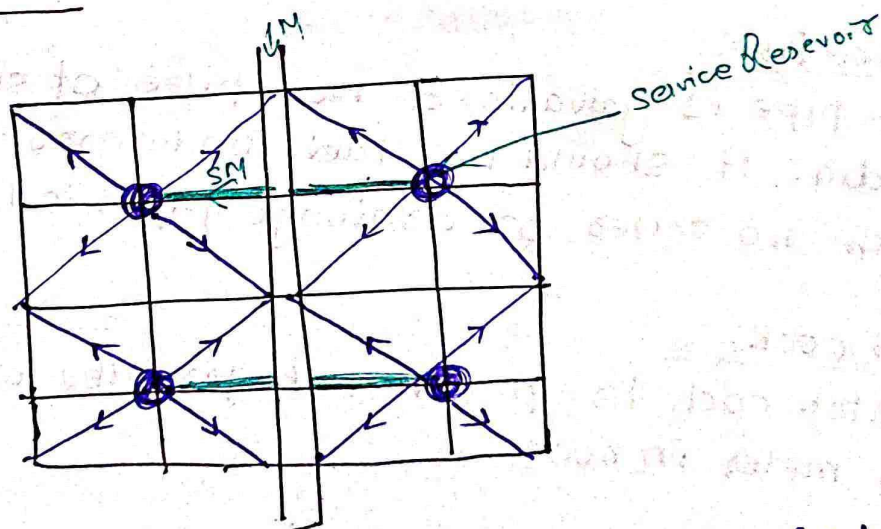
→ All the properties of the system are opposite to that of dead end system.

③ Ring system



→ It is same as that of grid iron system with only difference ~~the~~ that main pipe in this case encircles an area of high water demand so as to maintain uniform pressure throughout the system.

④ Radial system



→ Here also all the properties are same as grid iron system with only change the water is supplied in radial direction.

→ It is comparatively costly.

House Water Connection.

→ By installing a water supply plumbing system in a building, the first and the main step is to obtain a water connection from municipal water main.

→ A typical water connection, connecting the service pipe with the municipal water main is shown, that consists of following: -

(i) Ferrule:

A ferrule is a right angled sleeve made of brass or gun metal and is joined to a hole drilled in water main. Its size varies between 10 to 50mm dia.

(ii) Goose Neck:

It is a small sized curved pipe made of a flexible material and is about 75cm in length forming a flexible connection between the water main and service pipe.

(iii) Service Pipe:

Service pipe is galvanised iron pipe, of size less than 50mm dia. It should be laid underground in a trench in which no sewer or drainage pipes are laid.

(iv) Stop cock:

The stop cock is provided before the water enters the water meter in house.

(v) Water meter:

Water meter measures and records the quantity of water consumed in the house.

