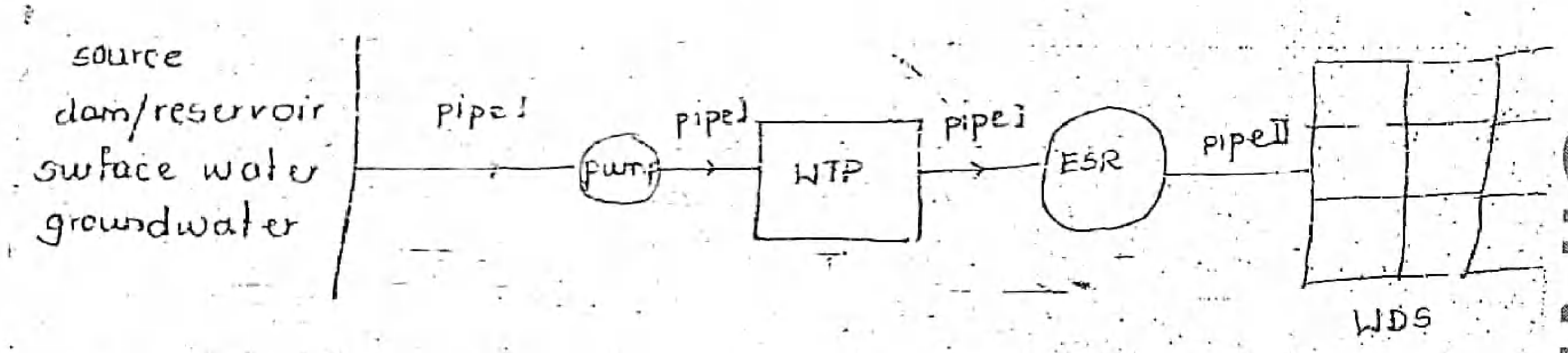
ENVIRONMENTAL ENGINEERING. 16th July

IES | GATE
paper II (40±5 marks) | 10±2%

1. Raw water engineering
2. Wastewater engineering
3. Air and noise pollution

Raw water engineering:
(Ref: Govt. of India manual) - on madeeasy website.

Water which is naturally available in Environment is Raw water on which we have not provided any treatment.



- Sources of water
- Qualities of water
- Purification of water
- Conduits required for transporting water.
- Water demand for distribution system.

Qualities of Raw water :-

The parameters which help in asserting qualities and properties of raw waters are termed as "Water quality parameters."
- Physical
- Chemical
- Biological

Physical WQ parameter:

The parameters which help in asserting physical qualities of water are termed as Physical WQ parameters.

Physical WQ parameters are those which can be felt or sensed by our senses.

1. Suspended solids
2. Turbidity
3. Colour
4. Taste and odour.
5. Temperature.

1. Suspended solids:-

It is a physical WQ parameters while dissolved solids is chemical water quality parameter.

SS in water may come from inorganic particles like clay, silt, sand, glass etc. it may come from organic particles like plant fibres, algae, plankton etc. or it may be induced in the water due to immissible liquids like oil and grease.

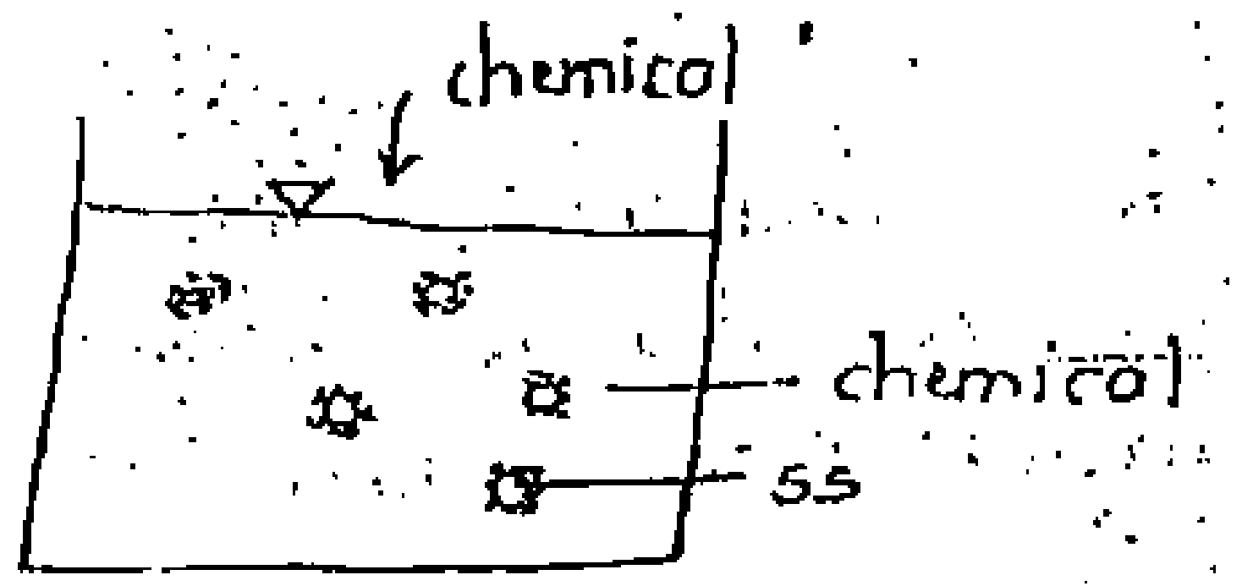
Note:
Organic solids are biologically degradable but inorganic solids are not.

Impacts of suspended solids:

i) It has a psychological effect and makes the water aesthetically displeasing.

ii) If suspended solids are biologically active (organic) they may form disease causing organics.

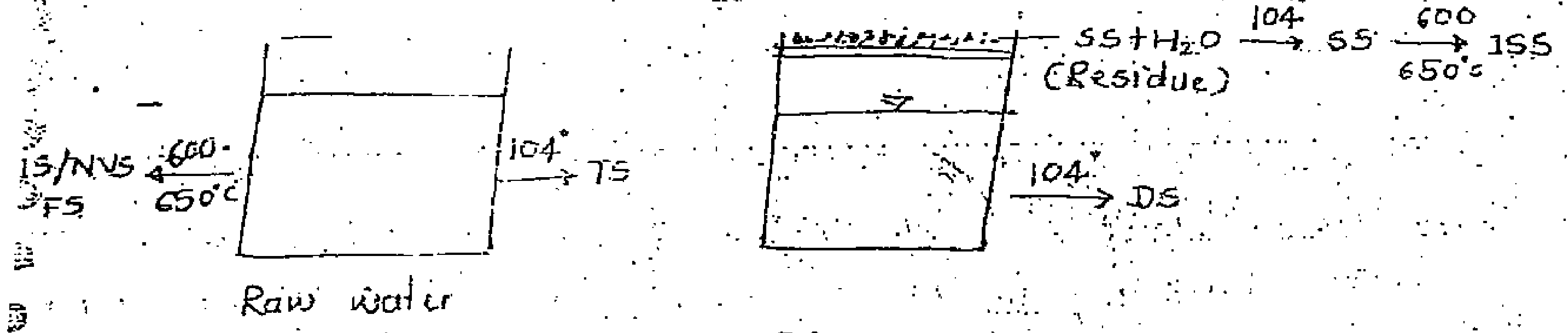
iii) Suspended solids provide adsorption sites for chemical and biological reagents thereby interfers with the further treatment of water.



iv) It partially shields the micro-organisms present in water thereby reduces efficiency of disinfection.

- Measurement of suspended solids :

Technique of measurement of weight of SS is called graviometric method.



$TS = SS + DS$

$= OS/VS + IS/NVS/FS$

$OS = TS - IS$

$OSS = SS - ISS$

i) Measurement of solids is done by graviometric method i.e. the method in which weight of solids is determined.

ii) Total solids and suspended solids can be calculated by evaporating original water samples at 104°c and by filtering the water sample thereby heating the residue left over the filter at 104°c respectively.

Dissolved Solids (DS) = Total Solids (TS) - Suspended (SS)

iii) Organic solids (both total and in suspended form) can be calculated by firing the residue and original sample at 600 to 650°c at which all the

converted into water vapours and carbon dioxide leaving behind the inorganic solids. (Fixed solids)

> Note:
> In some cases suspended solids smaller than the size of filter pores get measured as dissolved solids. To avoid this the classification of solids is done as Filtrable and non-filtrable solids.
>
> Filtrable solids correspond to dissolved solid and non-filterable solids to suspended solids.

Permissible / Acceptable limits :-

For Total solids (TS) = 500 mg/l (as per GOI manual).

Cause for rejection limit/value :-

TS = 2000 mg/L

2. Turbidity :-

It is the measure of extent to which light is either absorbed or scattered by water sample.

Turbidity is due to the presence of SS in the water but it is not the direct quantitative measure of SS. (No direct relationship between turbidity & SS - no quantitative relation but qualitative). More SS - more turbidity.

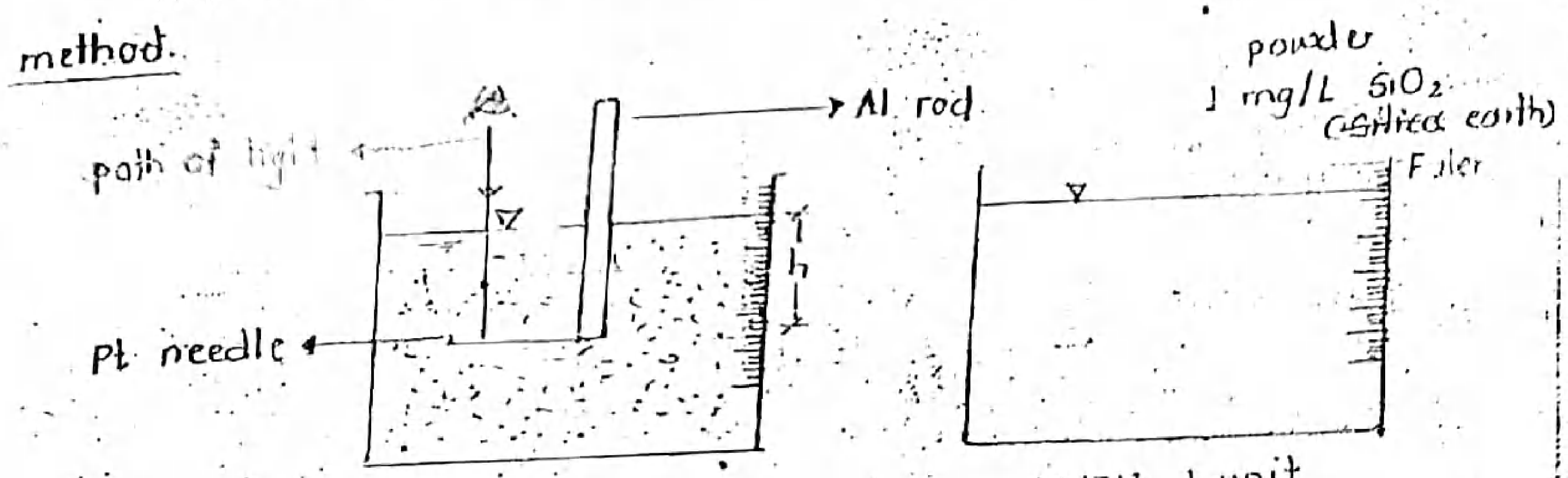
Impacts of turbidity :-

All impacts due to SS are applicable here.

i) The turbidity of water interferes with penetration of sunlight in natural waterbody. thereby affects photosynthesis which in turn inhibits the growth of aquatic life.

Measurement of turbidity :-

The first method to determine turbidity is turbidity rod method.



1 NTU. 1 unit

i) Turbidity rod method is field method in which Al rod having platinum needle at its tip is imersed in water sample whose turbidity is to be calculated.

ii) The depth of imersion of rod at which platinum needle just becomes invisible is noted and it gives the turbidity of the water sample in standard unit which is obtained by addition by 1 mg of silica in powdered form as silica oxide (Fuller's earth) in 1 litre of water (pure). This unit is ppm. mg/l or STU.

STU - silica turbidity unit.

$$1\ mg/L = 1\ ppm$$

$$\frac{10^{-3}\ gm}{10^{3}\ ml} = \frac{1}{10^{6}}$$

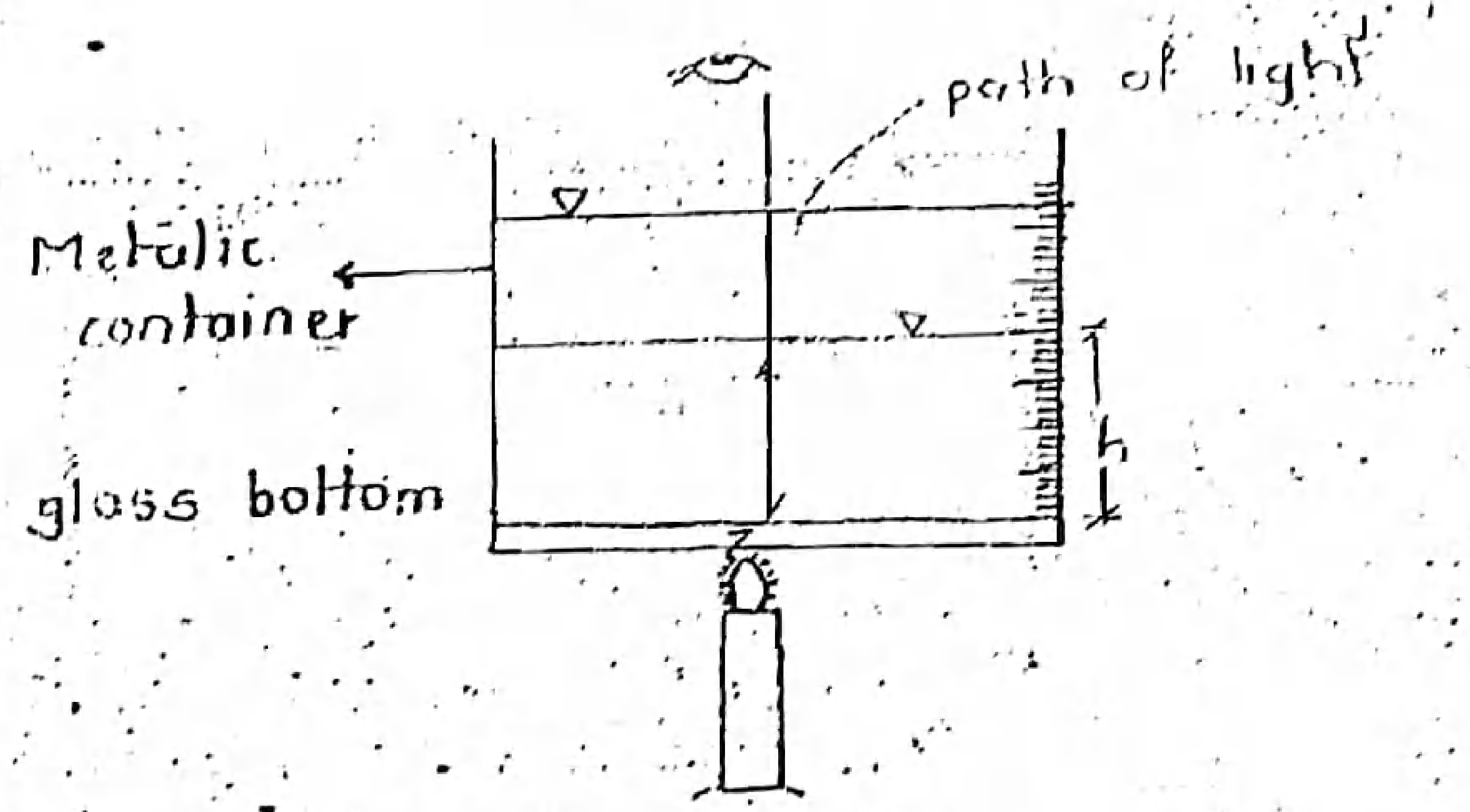
$$10^{-6} = 10^{-6}$$

Note :-

Turbidity greater than 5 units is easily detectable by naked eyes.

Jackson Turbidmeter :-

JTU - Jackson Turbidity Unit.



i) It's laboratory test which can measure the turbidity greater than 25 units. Hence it is generally not used for testing of water supplies but normally used for testing of natural water bodies.

ii) In this test level of water is increased in metalic container having glass bottom placed over ignited flame and the depth of water at which flame ceases to be seen is noted, to give turbidity of water sample in standard units.

Note :-

In both the above tests, principle involved in the measurement of turbidity is same i.e. longer is the length of path travelled by light, smaller is turbidity and vice versa.

Baylis turbidmeter & Nephlometer :-

Photometer produces current after ~~incidence of light~~ over it.

i) In Baylis turbidmeter & Nephlometer method turbidity is measured by colour matching technique.

ii) In these methods small turbidity (< 1 unit) can also be measured. Hence these methods are readily used for testing of water supplies.

iii) In this method colour produced in the test and standard sample due to their subjection to source of light is noted. (250v 50 Hz) If the colour produced in test sample is same as that of standard sample then turbidity of test sample is same as that of standard sample.

iv) To improve efficiency of test, the intensity of current is noted instead of intensity of colour by placing photometers behind the samples.

v) In Baylis turbidmeter light intensity is measured in direction of incidence of light whereas it is measured at right angles to the direction of incidence of light in Nephlometer. Hence the Nepholometer is based on scattering principle & Baylis turbidmeter is based on absorption of light.

vi) In Nephlometer turbidity is expressed in a standard unit ~~which~~ is obtained by adding 1 mg of Formazine (Hexamethylene tetramine) as base instead of silica oxide in one litre of pure water.

NTU - Nephlometer Turbidity Unit

There is no relation between Nephlometer Turbidity unit (NTU) and Jackson turbidity unit (JTU)

Acceptable limit for turbidity - 1 NTU

Cause for rejection value / limit - 10 NTU

3. Colour

True colour - due to DS

Apparent colour - due to SS + DS

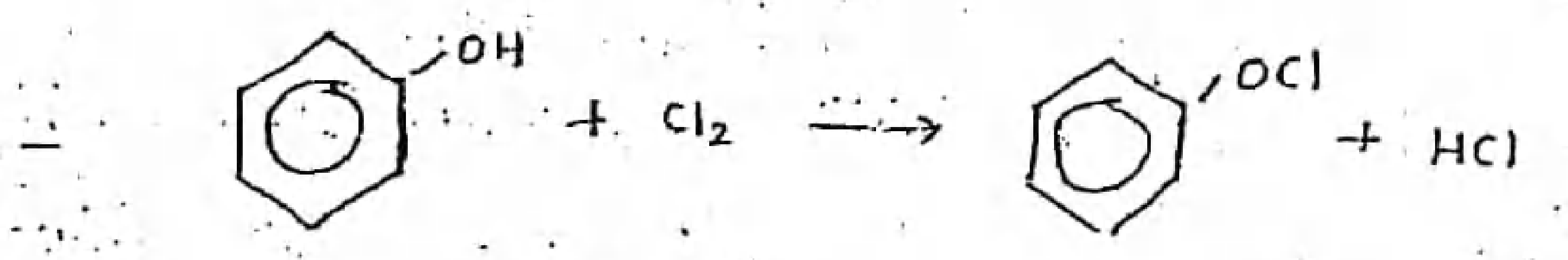
i) Colour in the water has psychological effects & makes the water aesthatically displeasing.

ii) Coloured water cannot be used for washing & dying

iii) Some colour causing organic compounds react with chlorine to form Carcinogenics (causing Cancer).

iv) Some colour causing compounds increases chlorine demand as chlorine is strong oxidizing agent it participates in Redox reaction.

v) Some colour causing organic compounds react with chlorine to induce taste & odour, like phenol.



Measurement of colour:

i) The intensity of colour in the test sample is measured on the basis of colour matching technique in which colour of the test sample is compared with the colour of standard sample.

ii) The apparatus in which test is carried out is called as Tintometer while tubes in which samples are collected are called Nessler's tubes.

iii) The intensity of colour is expressed in standard unit which is obtained by adding 1 mg of Platinum in the form of

iv) Chloroplatinate ion in 1 Litre of pure water. This unit is known as True colour unit (TCU.)

v) The testing of colour should be done within 72 hours of the collection of water samples as chemical and biological characteristic of water are changed due to storage which also cause the change in the colour of water.

Acceptable limit - 5 TCU

Cause for rejection value / limit - 25 TCU

4. Taste and odour:

i) Taste & odour in water may come from dissolved gases, dissolved organic matter and inorganic salts e.g. H_2S induce rotten egg smell to water, CO_2 induces bad taste in water.

ii) Algae secrets certain types of oils which induces bad taste in water.

2. Alkalinity :-

Alkalinity is measure of quantity of ions present in the water that are capable of neutrilising the Hydronium ion (H^+) or it may also be defined as measure of ability of water to neutrilise acids.

Alkalinity in water is due to CO_3^{--}, HCO_3^-, OH^-, HS^-, HPO_4^-, $HSiO_3^-$, HS^-.

Carbonate (CO_3^{--}), bicarbonate (HCO_3^-) & Hydroxyl (OH^-) ions are major constitutents to contribute Alkalinity and rest are minor.

CO_3^{--} - carbonate alkalinity
HCO_3^- - bicarbonate alkalinity
OH^- - caustic alkality.

Note :-

- For all the practical purposes alkalinity due to minor constituents can be neglected.

Phosphorous is mixed in water by the use of fertilizers, insecticides and detergents. It is the prime pollutant of lakes as it promots the growth of algae, which serves as its neutrient.

Alkalinity in water is produced due to anions which may be of mineral origin, which may be due to the microbial decomposition of organic matter or due to the presence of dissolved gases in water (e.g. CO_2, H_2S etc)

$$CO_2 + H_2O \rightleftharpoons H_2CO_3$$

$$H_2CO_3 \rightleftharpoons HCO_3^- + H^+$$

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

$$* \; CO_3^{--} + H_2O \rightleftharpoons HCO_3^- + OH^- \quad \text{(Weak reaction)}$$

The above reaction is very weak but if algae is present in the water, it consumes (HCO_3^-) ion due to presence of carbon (source of energy to algae) in it which drives the reaction in forward direction leading to accumulation of OH^-

Hence if algae is present in water, its pH is in the range of 8 to 10.

Impact of alkalinity:

i) The presence of alkalinity in water imparts bitter test to it.

ii) The prime objection of alkalinity is, it leads to the incrustation in pipes. (the process of deposition of precipitates in the pipe leading to the decreased discharged carrying capacity is called Incrustation)

$$Ca^{++} + CO_3^{--} \longrightarrow CaCO_3 \downarrow$$

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

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

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

> Note:-
>
> Acidity leads to corrosion of pipes where alkalinity leads to incrustation of pipes.

Measurement of alkalinity:

i) The alkalinity of water sample is measured by titrating it with acid and determining the hydrogen equivalents of alkalinity.

$$\text{No of moles} = \frac{\text{given wt. of compound}}{\text{molecular weight}}$$

$$\text{Equivalent weight} = \frac{\text{molecular weight}}{\text{valency}}$$

$$\text{No. of gm equivalents} = \frac{\text{given wt. of compound}}{\text{equivalent weight}}$$

one gram equivalent anything = one gram equivalent of any other thing one gram equivalent of anything reacts with one gram equivalent of any other thing to give one gram equivalent of resultant.

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

2 mole	1 mole	1 mole
2 gm	96 gm	98 gm
* 2 gm. eq.	2 gm. eq.	2 gm. eq.

A water sample contains 210 gm of carbonate, 122 gm of bi-carbonate ions and 68 gm of hydroxide ions. Find the alkalinity of water and express it as $CaCO_3$.

Sol^n:- Equivalents carbonate $= \frac{210}{30} = 7$

bi-carbonate $= \frac{122}{61} = 2$ — Total 13 equivalents of $CaCO_3$

hydroxide $= \frac{68}{17} = 4$

Alkalinity of water = 13 equivalents × 50.
= 650 gm as $CaCO_3$

Molarity of solution (M) $= \frac{\text{No. of moles}}{1 \text{ volume of sol}^n \text{ in lit}}$

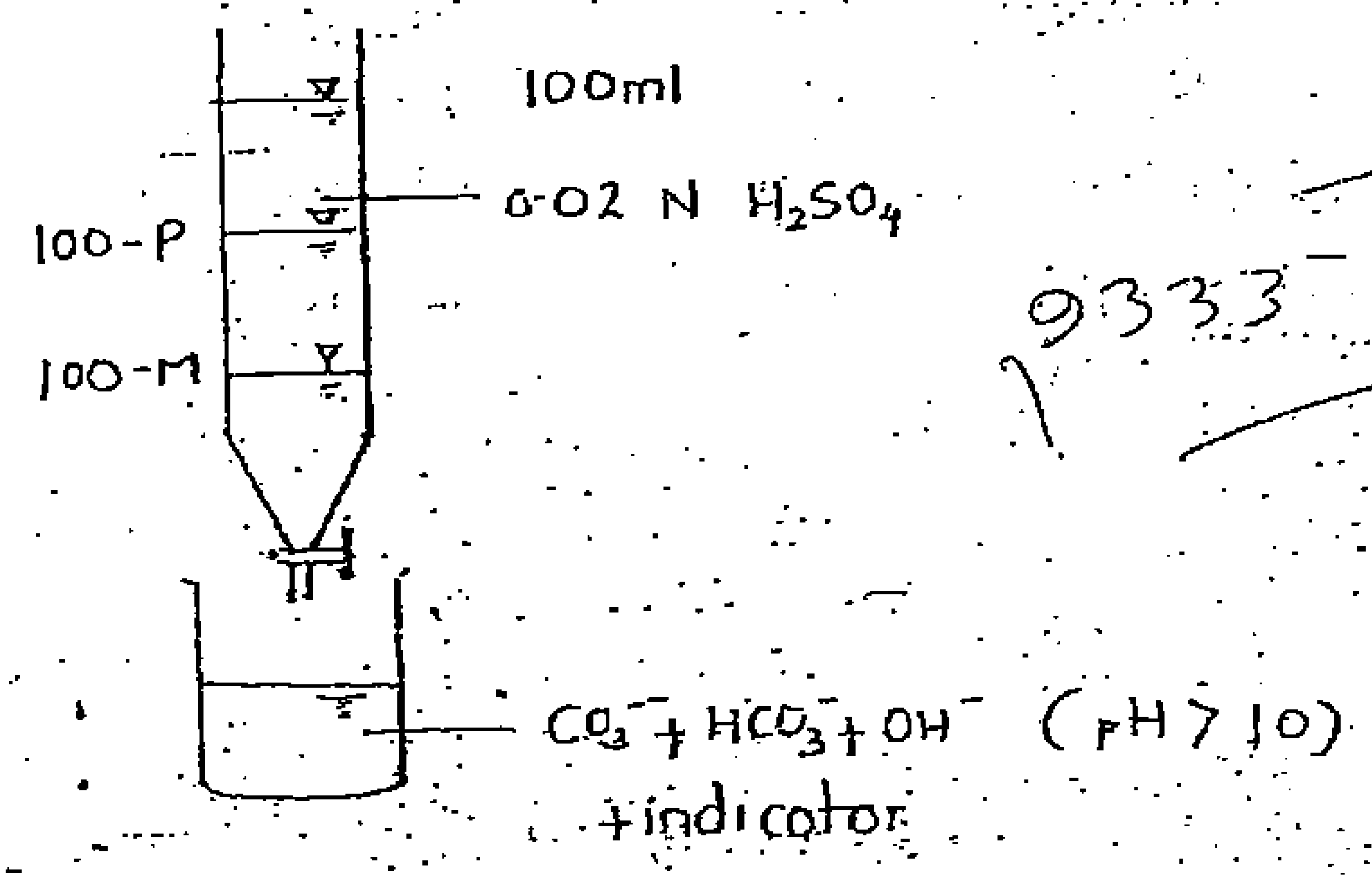
Normality of solution (N) $= \frac{\text{No. of equivalents}}{1 \text{ lit of solution}}$

Note:- If 0.02 N H_2SO_4 is used as titrant then 1 ml of this acid used corresponds to 1 mg of alkalinity as $CaCO_3$.

1 ml, 0.02 N H_2SO_4 = 1 mg of alkalinity as $CaCO_3$

2×10^{-5} gm. equivalent $= \frac{1}{50\times10^3}$ i.e. 2×10^{-5} gm. eq.

* The relative quantities of alkaline species present in water is pH dependant. (HCO_3^-, CO_3^{-}, OH^-)



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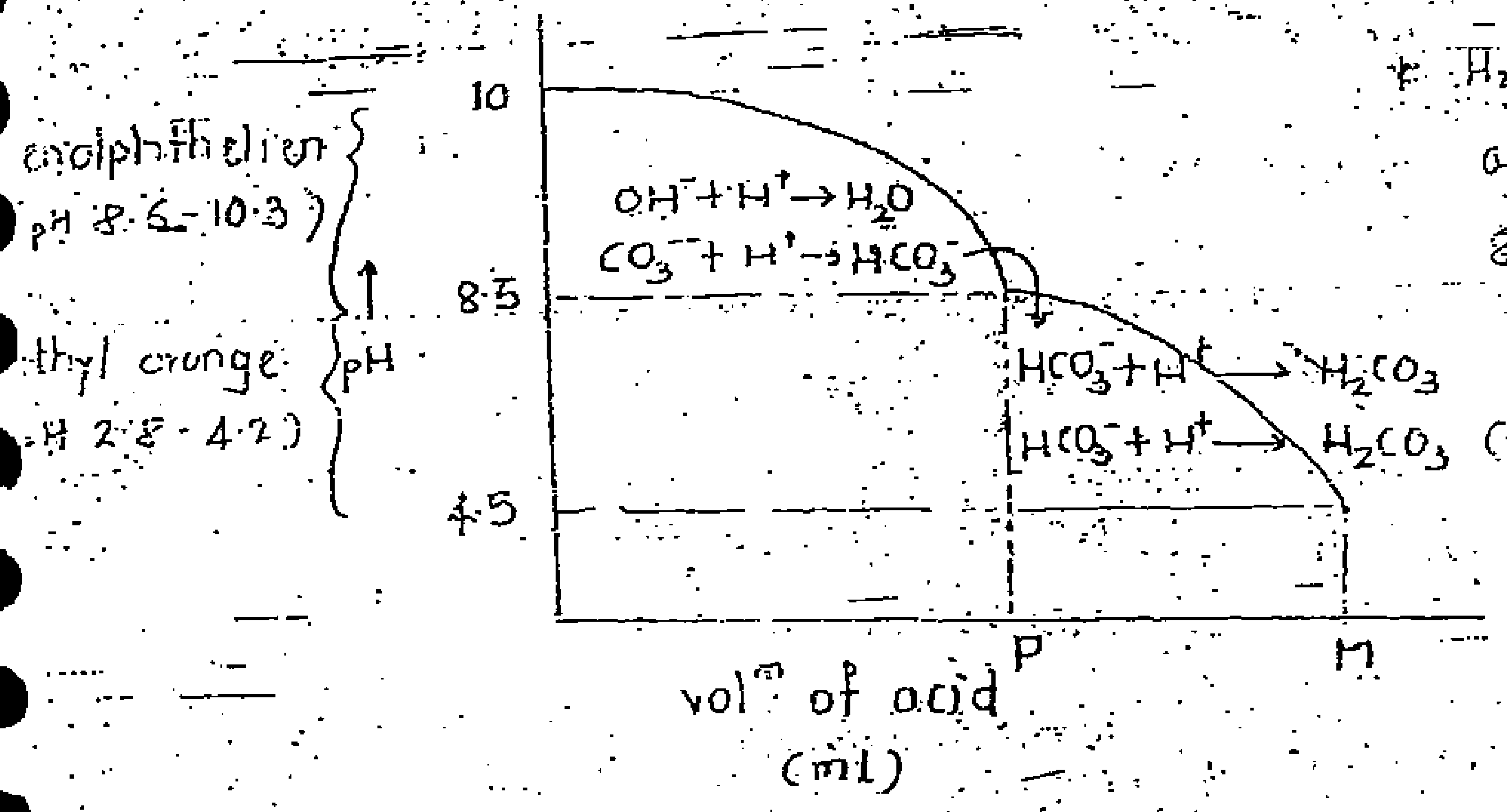


* H_2O & HCO_3^- are formed at pH 8.3 not in between 8.3 - 10

i) Conversion of carbonate to bi-carbonate is essentially completed at pH of 8.3 but this resultant bicarbonate (HCO_3^-) still requires acids to get completely neutrilized. Hence we can say that half of Carbonate alkalinity is neutrilized upto pH 8.3

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

$$HCO_3^- + H^+ \longrightarrow H_2CO_3$$

ii) Neutrilisation of hydroxyl (OH^-) is also completed upto 8.3. Therefore at pH 8.3 complete caustic & half

iii) Volume of acid used for neutrilisation of this alkalinity is 'p' ml.

iv) Indicator used in 1st stage of titration is phenolphthelin. It is basic indicator with working range of pH 8.3 to 10.6. It is originally colourless (may be yellow) and turns into pink in its working range.

v) Bicarbonate alkalinity (HCO_3^-) present in water (originally present and also converted from carbonate alkalinity neutrilisation at pH 8.3) is completely neutrilised at pH of 4.5. Therefore at pH 4.5, complete caustic, carbonate and bi-carbonate alkalinity is neutrilised. Volume of acid used for the neutrilisation of this alkalinity is 'm' ml.

vi) Indicator used in 2nd stage of titration is methyl orange. It is acidic indicator which works in pH range of 2.8 to 4.2. It is originally orange in colour and turns into yellow in its working range.

Note:-

If $p = m$, only caustic alkalinity is present (OH^-)

If $p = m/2$, only carbonate alkalinity is present (CO_3^{--})

If $p = 0$, only bicarbonate alkalinity is present (HCO_3^-)

If $p > m/2$, caustic (OH^-) & carbonate (CO_3^{--}) is more than bicarbonate (HCO_3^-) alkalinity

If $p < m/2$, bicarbonate (HCO_3^-) & carbonate (CO_3^{--}) is more than caustic alkalinity.

Hydroxyl (OH^-) and bi-carbonate (HCO_3^-) alkalinity is not present at the same pH value.

$p = m/2$

$P = x + y/2$

$x + y/2 = \frac{(x+y+z)}{2}$

$m/2 = y/2 + z$

3. pH :-

pH is a scale which is used to measure the acidity or basicity of water. This scale varies from 0 to 14. Numerically,

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

The $[H^+]$ should be in moles/lit.

Similarly, $pOH = -\log [OH^-]$ $[OH^-]$ in moles/lit

$$pH + pOH = 14.$$

For acidic water: $pH = 0$ to 7.

For basic water: $pH = 7$ to 14

If pH is less it leads to corrosion, and if it is more it leads to incrustation in pipes and causes difficulty in the chlorination.

Measurement of pH

i) Measurement of pH is done with the help of colour indicator like phenolphthelein, methyl orange, bromothymol blue. Bromothymol blue works in pH range of 6 to 7.6. Its original colour is blue and turns colourless in its working range.

ii) If bromothymol blue is used as indicator, the equipment used for measurement of pH is known as Aquascope.

iii) The pH of water sample can also be measured with the help of potentiometer also known as pH meter using Calomel electrodes.

Permissible limit for pH is 7 to 8.5 and cause for rejection is pH less than 6.5 & greater than 9.2.

1.3-10.6

2.9-4.2

6-7.6

Q. A 200 ml sample of water has initial pH of 10. 30 ml of 0·02 N H_2SO_4 is required to titrate the sample upto the pH of 4·5 and 11 ml of 0·02 N H_2SO_4 to titrate the sample upto pH of 8·3. Calculate total alkalinity of water in mg/L as $CaCO_3$. Calculate the conc. of alkaline species in mg/l as $CaCO_3$.

Given :

$M = 30$ ml

$P = 11$ ml

For 200 ml sample, 30 ml of 0·02 N H_2SO_4 is used to neutrilise HCO_3^-, CO_3^{--} and OH^- alkalinity.

1 ml sample of 0·02 N H_2SO_4 = 1 mg of alkalinity as $CaCO_3$

30 ml of 0·02 N H_2SO_4 = 30 mg.

For 1 litre.

Total alkalinity as $CaCO_3 = \frac{30}{200} \times 1000$

$= 150$ mg/l

for 200 ml, 11 ml of 0·02 N $H_2SO_4 \rightarrow \frac{1}{2} CO_3^{--}$, ~~$HCO_3^-$~~, OH^-

$(OH^- + \frac{1}{2} CO_3^{--})$ alkalinity as $CaCO_3 = \frac{11}{200} \times 1000$

$= 55$ mg/L

pH = 10. pOH = 4

$[OH^-] = 10^{-4}$ mole/lit

$= 17 \times 10^{-4}$ gm/lit

$= 10^{-4}$ ~~mole/l~~ $\frac{17 \times 10^{-4}}{17} = 10^{-4}$ gm equivalent.

$[OH^-]$ alkalinity as $CaCO_3 = 10^{-4}$ gm equivalent.

$= 10^{-4} \times (50 \times 10^{+3})$

$= 5$ mg/l as $CaCO_3$

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

$[CO_3^{--}] = 100$ mg/L

$[CO_3^{--}]$ alkalinity as $CaCO_3 = 100$ mg/l

$[HCO_3^-]$ alkalinity as $CaCO_3 = 150 - (5) - (100)$

4. Hardness :-

Hardness is the concentration of multyvalent cations present in water. It may be induced due to Ca^{++}, Mg^{++}, Fe^{++}, Fe^{+++} etc. Measure constituents are Ca & Mg while minor constituents are rest of multyvalent cations. Hence for all practical purposes hardness due to minor constituents can be neglected.

Hardness of the water can be divided into two as

1: Carbonate hardness.

2: Non-carbonate hardness

1. Carbonate hardness :-

Hardness due to carbonate (CO_3^{--}) and bicarbonate (HCO_3^-) of multyvalent cations is called carbonate hardness.

This hardness is also known as Temporary hardness as it can be removed by simple boiling or addition of lime in water.

$$Ca(HCO_3)_2 \xrightarrow{\Delta} CaCO_3 \downarrow$$

$$Mg(HCO_3)_2 \xrightarrow{\Delta} \underset{\text{(soluble)}}{MgCO_3}$$

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

or.

$$Mg(HCO_3)_2 + \underset{\text{(lime)}}{Ca(OH)_2} \longrightarrow Mg(OH)_2 \downarrow + Ca(HCO_3)_2 \$$$

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

1 mole of lime is required for removal of $MgCO_3$ and 2 mole of lime $Ca(OH)_2$ is required for removal of $Mg(HCO_3)_2$

2. Non-carbonate hardness :

Hardness due chloride, sulphate, nitrates of all the multivalent cations is known as non-carbonate hardness or permanent hardness as it can not be removed by simple boiling or addition of lime.

Special techniques are used for its removal known as softening techniques.

Impacts of hardness :-

i) It leads to increased consumption of soap due to lesser formation of leather or foam.

ii) Hardness in water leads to corrosion and incrustation in pipes.

iii) Hardness in water makes the food tasteless.

iv) Magnesium hardness due to sulphate ($MgSO_4$) induces laxative effect in the water, hence its concentration is limited to 50 mg/l

v) The hardness of water is expressed $CaCO_3$ equivalent of Ca & Mg present in water.

$$\text{Hardness as } CaCO_3 \text{ in mg/l} = \frac{[Ca^{++}] \times \text{eq.wt.of } CaCO_3 \ (50)}{\text{Eq.wt.of } Ca^{++} \ (20)} + \frac{[Mg]^{++} \times \text{eq.wt.of } CaCO_3}{\text{eq.wt. of } Mg^{++} \ (12)}$$

Measurement of hardness :-

i) The hardness of water sample is determined by titrating the water sample with 0.01 M EDTA solution. (Versandte solution)
(EDTA - Ethylene Diamine Tetra Acetic acid) using EBT as indicator (Erichrome black-T)

$$EBT + Ca^{++}/Mg^{++} \longrightarrow \underset{\text{Weak complex - wine red}}{EBT\ Ca/Mg}$$

$$EDTA + EBT\ Ca/Mg \longrightarrow \underset{\text{stable complex colourless}}{EDTA\ Ca/Mg} + \underset{\text{Blue black}}{EBT}$$

Permissible limit of hardness = 200 mg/l

Cause for rejection = 600 mg/l

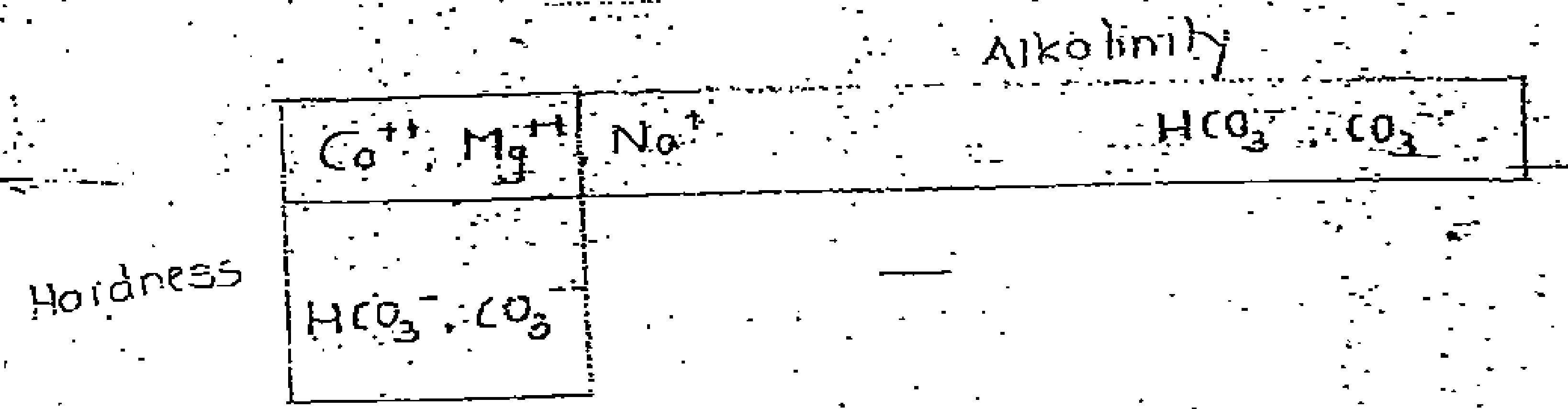
For domestic water supplies hardness should be in the range of 75 to 150 mg/l.

Degree of hardness (On the basis of $CaCO_3$)

Hardness as $CaCO_3$ (mg/l)	Degree of hardness
0 - 55	soft water
56 - 100	slightly hard
101 - 200	moderately hard
201 - 500	very hard water.

Thursday
18th July 2013

Hardness and alkalinity :



i) If Na alkalinity is absent in water ($NaHCO_3$, Na_2CO_3) then

Alkalinity = Carbonate hardness ($CaCO_3$, $Ca(HCO_3)_2$, $MgCO_3$, $Mg(HCO_3)_2$)

ii) Na alkalinity is absent only if non-carbonate hardness is present.

$$Ca/Mg \left\{ \begin{matrix} Cl^- \\ SO_4^- \\ NO_3^- \end{matrix} \right. + Na \left\{ \begin{matrix} CO_3^{--} \\ HCO_3^- \end{matrix} \right. \longrightarrow Na \left\{ \begin{matrix} Cl^- \\ SO_4^- \\ NO_3^- \end{matrix} \right. + Ca/Mg \left\{ \begin{matrix} CO_3^{--} \\ HCO_3^- \end{matrix} \right.$$

Total hardness = Carbonate hardness + non-carbonate hardness
= Alkalinity + non carbonate hardness.

Impacts of Taste & odour

i) Taste and odour causing compounds may be Carcinogenics.

ii) All impacts of SS are applicable.

Measurement of taste & odour :-

i) The intensity of taste and odour is measured in standard unit known as TON (Threshold Odour Number), which represents the dilution factor or Intensity at which taste & odour becomes hardly detectable.

ii) The testing of taste and odour is done in the apparatus known as Osmoscope.

iii) The testing of taste and odour should be done at normal temperature conditions. as change in temperature causes change in the biological activity.

$$TON = \frac{\text{final volume after addition of pure water}}{\text{initial / original volume of sample.}}$$

Acceptable limit — 1 TON (no dilution)

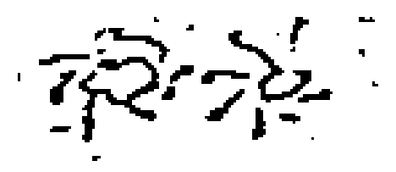
Cause for rejection value/ limit - 3 TON

5. Temperature :

i) Temperature of water affects chemical & biological quality.

ii) An approximate increase in 10°c in water almost doubles the biological activity.

Note :-

 $\xrightarrow{O_2}$ Organic matter $\longrightarrow$ Biomass

micro-organisms

~~It's~~ the process in which micro-organisms in the presence of oxygen carries out decomposition of bio-degradable organic matter (Carbon present in organic matter serves as a source of energy for micro organisms)

For water supplies temperature should be

Acceptable limit - 10°c.

cause for rejection value - 25°c

Chemical WQ parameters :-

The parameters which helps in determining the chemical character or quality of water are termed as Chemical WQ parameters.

1. Dissolved Solids
2. Alkalinity
3. pH
4. Hardness
5. Nitrogen content
6. Fluoride content
7. chloride content
8. Metals
9. Gases

Wednesday
17th July 2013.

1. Dissolved solids :-

i) The approximate analysis of DS can be done by finding the electrical resistivity or specific conductance of water.

$$\left(\text{Electrical conductivity / resistivity in } \mu\,\frac{\text{mho}}{\text{cm}} \text{ at } 25^\circ c.\right) \times 0.65$$

$$=$$

$$\text{Total dissolved solids in mg/l (in ionic form).}$$

ii) Electrical resistivity is measured ~~in the apparatus~~ known as Di-ionic tester. which measures the concentration of common ions present in the water. like Na^+, Ca^{++}, Mg^{++}, Cl^-, SO_4^{--}, HCO_3^-

iii) The above test is approximate test to find conc. of TDS in water. as it does not account for those organic solids which are dissolved in water but are not present in ionic form.

iii) If No alkalinity is present in water $(NaHCO_3 . Na_2CO_3)$

∴ non-carbonate hardness is absent in water

alkalinity > carbonate hardness.

Total hardness = carbonate hardness

Chloride content :

i) Chloride content is present in high conc. in water. It signifies the pollution due to industrial waste and sewage.

ii) Chloride content in water can be determined by titrating it with standard silver nitrate solution $(AgNO_3)$ using potassium chromate (K_2CrO_4) as an indicator.

$$AgNO_3 + Cl^- \longrightarrow \underset{\text{white}}{AgCl \downarrow}$$

$$AgNO_3 + K_2CrO_4 \longrightarrow \underset{\text{Red brown}}{Ag_2CrO_4 \downarrow}$$

Acceptable limit for chloride in water = 200 mg/l

Cause for rejection value = 1000 mg/l

Nitrogen content :

i) The presence of nitrogen in water indicates presence of organic matter in it.

ii) Nitrogen may be present in any of the following form.
- free amonia
- organic ammonia (Albuminoid)
- nitrite (NO_2^-)
- nitrate (NO_3^-)

Free ammonia :-

The presence of free ammonia in water indicates the recent pollution of water.

Free ammonia can be calculated by simple boiling of water and measuring the amount of ammonia gas liberated by simple distillation.

Permissible limit of free ammonia is 0·15 mg/l

Cause for rejection value / limit is also 0·15 mg/l

Organic ammonia :- (Albuminoid)

The presence of organic ammonia in water indicates the presence of nitrogen in the water before the decomposition of organic matter has started.

Organic ammonia in water can be calculated by already boiling the boiled water and calculating ammonia gas liberated using distillation process.

During this process strong alkaline agent like $KMnO_4$ (potassium permangenate) is added so as to aid liberation of ammonia gas.

Note :

Free ammonia in conjugation with ~~organic~~ ammonia is known as kjeldha's ammonia.

Kjeldha's ammonia can be calculated by adding $KMnO_4$ in the original water sample and heating it, thereby noting the ammonia gas liberated which corresponds to both free and organic ammonia.

Permissible limit for organic ammonia is 0·3 mg/l

(because this organic matter may be already decomposed thus value is more than free ammonia value)

3. Nitrite (NO_2^-) :-

The presence of nitrite in water indicates partial decomposition of organic matter. Hence it is highly dangerous and it is not permitted in water.

Nitrite content in water can be determined by colour matching technique in ~~which colour is formed~~ by Sulphonic acid and Nepthamine.

Acceptable limit of nitrite = 0 mg/l
(because it indicates presence of micro-organisms in water)

4. Nitrate (NO_3^-) :-

The presence of nitrate in water indicates complete decomposition of organic matter. Hence it signifies old pollution in water.

The presence of nitrate in water is not harmful as it is fully oxidised, but in some cases if it is present in large concentration, it affects the infants.

$$NO_3^- \xrightarrow[\text{body of infants}]{\text{Acids in}} NO_2^-$$

(Valency of N → +5) (Valency of N → +3)

$$NO_2^- + HbO_2 \longrightarrow HbNO_2$$

Haemoglobin

Nitrate is converted into nitrite due to presence of acids in the intenstine of infants which has very high affinity for haemoglobin hence replaces oxygen from it. Thereby leading to defficiency of oxygen in body. The diesease caused is known as Blue baby disease or Mathemoglobinemia.

Nitrate in water can be determined by colour matching technique and colour is formed by Phenol di-sulphonic acid, and potassium hydroxide.

Acceptable limit / cause for rejection = 45 mg/l

Note :-

The process in which ammonia gets converted into nitrate (NO_3^-) is Nitrification and process in which nitrogen is formed from nitrate is called de-nitrification.

$$NH_3 \xrightarrow{O_2} NO_2^- \xrightarrow{O_2} NO_3^-$$

N → −3 N → +3 N → +5

Nitrosomonous bacteria Nitrobector

(Nitrification - oxidation)

paracocus denitrificant

$$NO_3^- \longrightarrow N_2\uparrow$$

(Anaerobic process)
- use molecular bonded oxygen

(De-nitrification - reduction)

Fluoride content

Upto 1 mg/l of Fluorine is required for the growth of parmanent teeth and to prevent the dental cavities. If fluorine is more than 1·5 mg/l, it causes the decolourisation of teeth and name of diseases is Fluorosis.

If Fluoride content is more than 5 mg/l, it causes deformation of bones, called as Bone fluorosis. $Ca_3(PO_4)_2$ in body reacts with Fluorine.

Acceptable limit is 1 mg/l

Cause for rejection value 1·5 mg/l

Metals :-

Toxic metals - As. Pb. Cd. Hg. Ag. Ba

non-toxic metals - Na, Mg, K, Al, Ca. Mn. Cu.

Sodium (Na) :-

If sodium is present in high conc. in water, it induces bod test in it. High sodium concentration affects heart and kidney. High sodium concentration leads to the corrosion of metal surfaces and proves to be toxic for plants.

Copper (Cu) :-

High copper concentration affects the lungs and respiratory system. If $CuSO_4$ is more than 250 mg/l, it induces laxative effects.

(laxative effect is induced due to SO_4^{--} in water)

Acceptable limit is 0.05 mg/l

cause for rejection value 1.5 mg/l

Iron and Manganese (Fe and Mn) :-

High concentration of iron and manganese (0.3 and 0.5 mg/l) causes colour problem in water.

Fe - reddish brown colour

Mn - yellowish

They are present in conjugation with organic matter in water. Fe and Mn are generally found in water ~~in~~ which is devoided of oxygen.

$$\underset{\text{Ferrous ion}}{Fe^{++}} + O_2 + H_2O \longrightarrow \underset{\text{Ferric hydroxide}}{Fe(OH)_3\downarrow}$$

$$Mn^{++} + O_2 + H_2O \longrightarrow Mn(OH)_2\downarrow$$

Reddish brown colours

Fe and Mn are present ~~in~~ Groundwater and can be removed by addition of O_2

Certain type of micro-organisms utilises Fe & Mn as source of energy.

Acceptable limit of Iron is 0.1 mg/l

Cause for rejection is 1 mg/L

Gases :-

H_2S -

H_2S indicates presence of organic matter in water. It imparts rotten egg smell to water (due to sulphide ion).

CO_2 -

Presence of CO_2 indicates biological activity in water. It adds bod test to water and makes it corrosive (leading to formation of Carbonic acid.)

CH_4 -

Methane is known for its explosive tendencies. It is generally found in sewer, marshy areas and over paddy/rice fields. It is formed due to the decomposition organic matter.

O_2 -

At a particular temperature max quantity ~~of~~ ~~wa~~ O_2 present in water is known as Saturation dissolved oxygen. Any deficiency of oxygen found in water signifies the presence of biological activity in water.

Biological WQ parameters :

A water sample contains different types of organics which may biodegradable or non-bio degradable. Biodegradable organic matter is that which can be decomposed by the action of micro-organisms in presence of oxygen. This process in known as Biological activity. (Micro-organisms decomposes the organic matter due to presence of Carbon it which acts as source of energy for them.)

The reactions which takes place in the presence of oxygen are known as Aerobic reactions and it is being carried out by Aerobic micro-organisms. The end products of these reactions are highly stable. e.g. Nitrification. and these reactions are much faster in comparison with Anaerobic reactions (almost 3 times)

The reactions which takes place in absence of O_2 are called as Anaerobic reactions and carried by anaerobic bacteria. The end products are highly unstable - acids, gases and alcohols. These reactions are much slower than aerobic as in this case micro-organisms utilises molecular bonded O_2.

Aliphatic compounds are easily biologically degradable in comparison with aromatic compounds.

—C—C—C—

Aliphatic compounds

Aromatic compounds
(more bond energy)

There are certain type of micro-organisms which can act both in presence and absense of O_2 called as Facultative micro-organisms.

If the organic matter is exhausted from the system then micro-organisms starts utilising carbon either from their own cell or from their neighbourer's cell, leading to the self destruction of micro organisms. This process is known as Endogenous respiration.

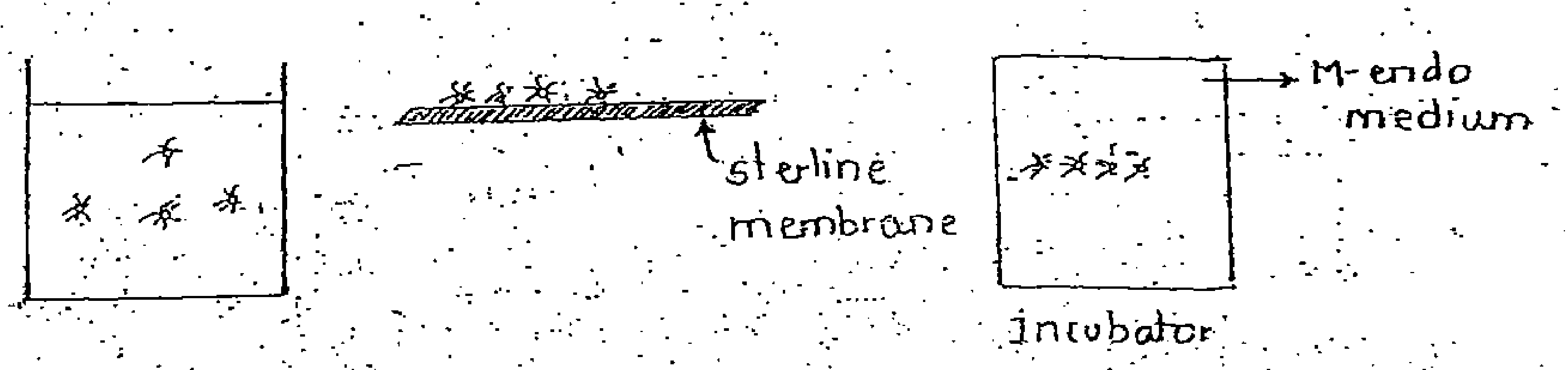
The most important micro-organisms present in water that are capable in transmitting diseases are known as pathogens.
e.g. bacteria, virus, protozoa and fungi.

The testing of pathogens in laboratory can be done but with great difficulty and testing methods involved are very costly. Hence in routine testing of pathogens is not done instead testing of Coliforms in water is done which themselves are harmless micro organisms and their absence or presence indicates the absence or presence of pathogens.

Coliforms like E-coli and B-Coli are present in intestine of all the warm blooded animals and this Coliforms survive in the water for longer durations in comparison to pathogens.

Tests for Coliforms :-

1. Membrane filter technique :

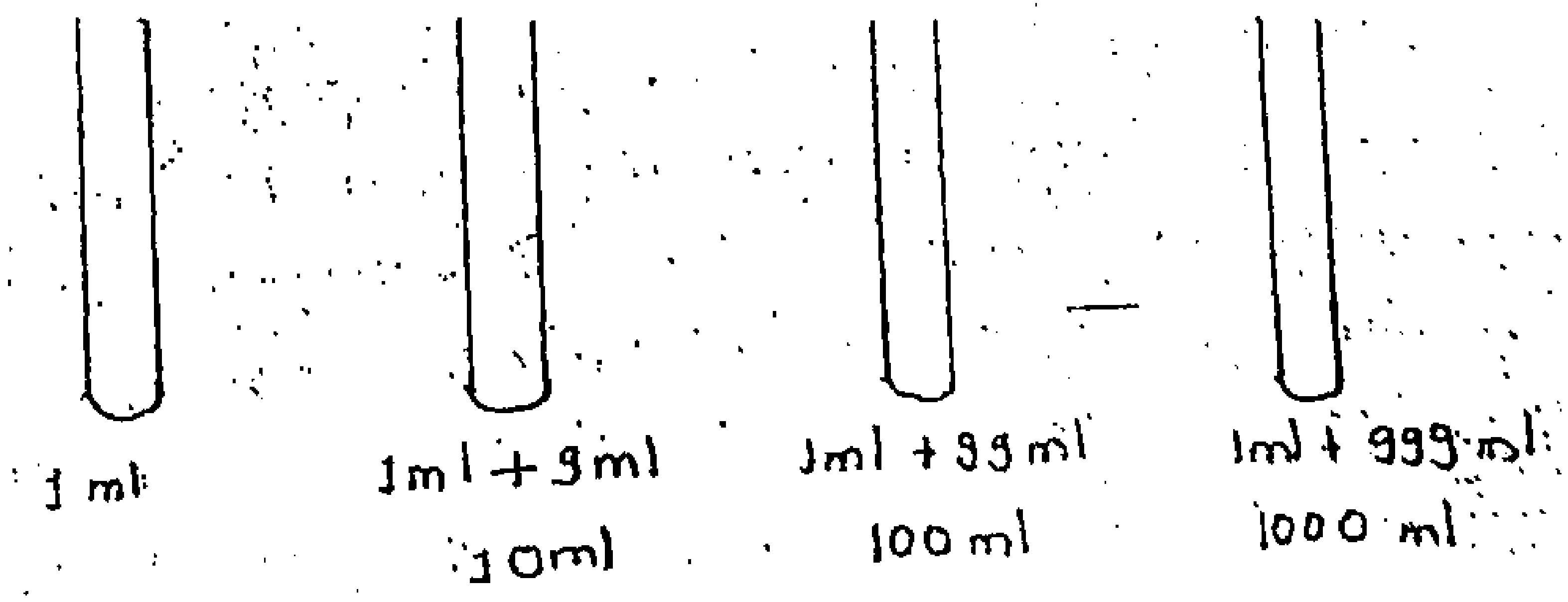


i) In this test water sample is passed through sterlie membrane over which all micro-organisms are retained.

ii) This membrane is brought in contact with nutrient (M-endo medium) which permits the growth of only Coliforms and inhibits the growth of any other type of bacteria / microbs.

iii) After incubating them at 35°C for 24 hours, the number of colonies of Coliforms are counted which are further related to colonies of pathogens.

2. Most Probable Number (MPN) Test :



itive 5 4 3 1
results tubes.

i) MPN test is multiple tube fermentation test in which we mix water samples with different dilution ratio and Lactose broth which acts as nutrient for Coliformic micro organisms.

(Coliformic micro organisms utilises Lactose broth and results in the formation of acids and CO_2)

ii) The samples are incubated at 35°C for 48 hours and after incubation these are tested for the presence of acids and CO_2. then referring to the standard table corresponding to the result observed. MPN/100ml is noted. This MPN represents the microbial density which is most likely to be present in water.

1 ml	10 ml	100 ml	MPN/100 ml
5	5	5	A
5	4	3	B
4	3	1	C

For above example the MPN is B or C/10 (max one)

3. Coliform Index test:

i) Coliform index is reciprocal of smallest quantity of the sample that gives positive B-coli test.

ii) Coliform index test and MPN test are obsolete.

Note:-

Nuissance causing bacteria:-

1. Iron bacteria:

This nuissance causing bacteria leads to pitting and tuberculation in pipes.

(pitting - localised corrosion

Tuberculation - incrustation due $Fe(OH)_3\downarrow$ known as Tubercube)

2. Sulphur bacteria:

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

Catabolysm - breakdown of nutrients in lighter ones

Anabolysm - utilisation of broken nutrients

Metabolysm - combination of above two

Diseases caused by different types of micro-organisms:

Bacteria - Typhoid, Cholera, Dysentry.

Virus - Hepititis, (Jaundice), Polio

Protozoa - Amoebic Dysentry.

Treatment of water :-

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

ii) There are various methods of water treatment as

- Screening
- Aeration
- Co-ogulation & flocculation
- Sedimentation
- Filtration
- Dis-infection
- Softening
- Fluoridisation or defluoridisation.
- de solination & defluoridisation.

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

ii) If surface water contains excessive iron, dissolved CO_2 and odourous gases then treatments given are aeration, screening, coagulation & flocculation, sedimentation, di filtration and dis-infection.

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

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

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

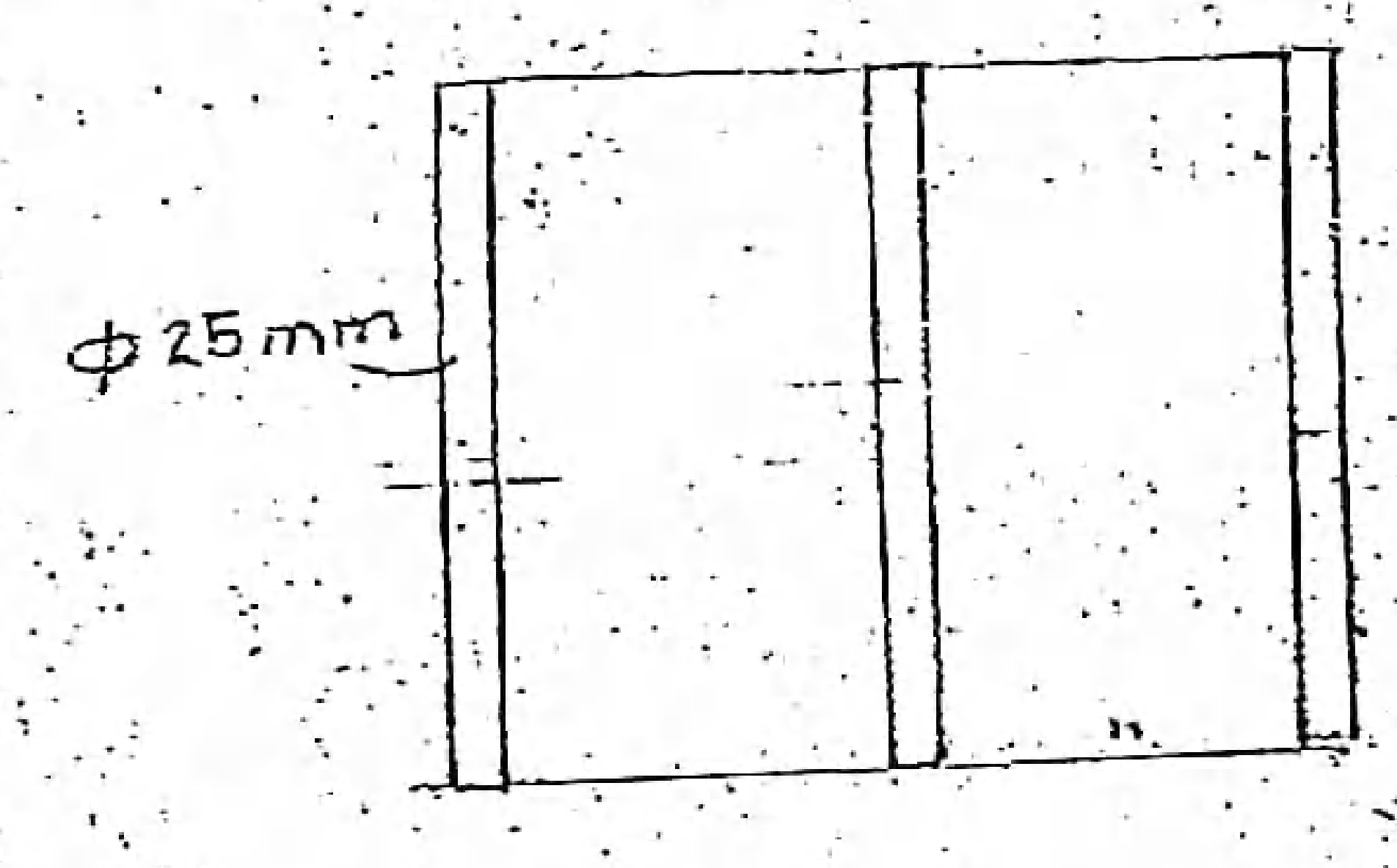
Screening :-

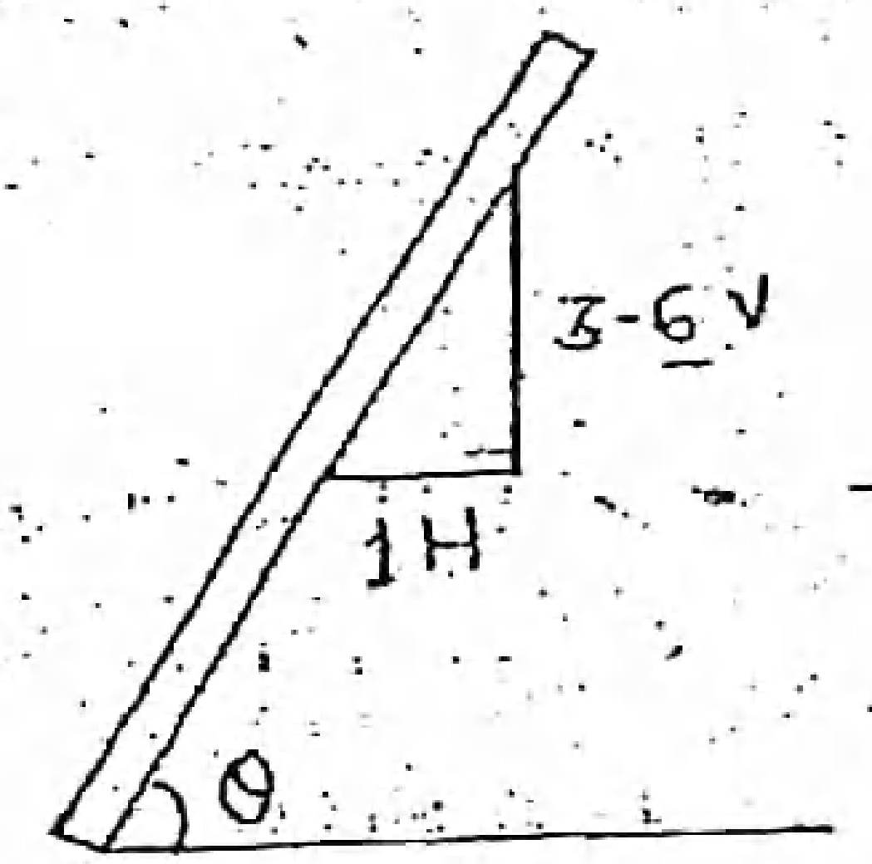
i) Screening is done to remove the heavier suspended impurities from the water like plants, trees, stones, animals etc.

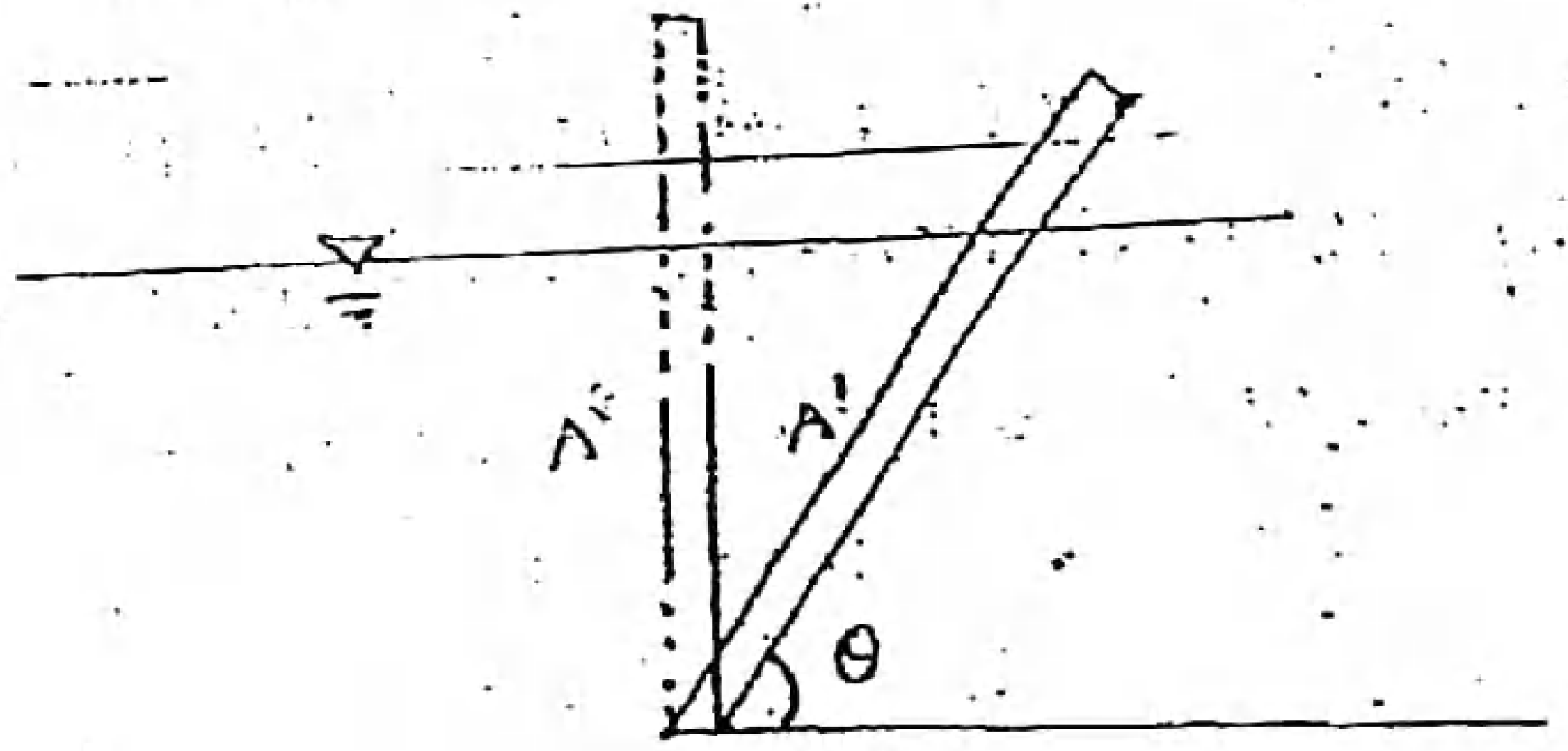
ii) Screening is generally adopted with the help of screens of two types.
- Coarse screens (Bar screens / Trash rack)
- Fine screens

Coarse screen / Trash rack :-

It is in the form of bars of dia. 25 mm and spacing of 20-100 mm is maintained between the bars. These screens are generally placed at an inclination of 3-6 V : 1 H.







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

$A' = A \sec\theta$

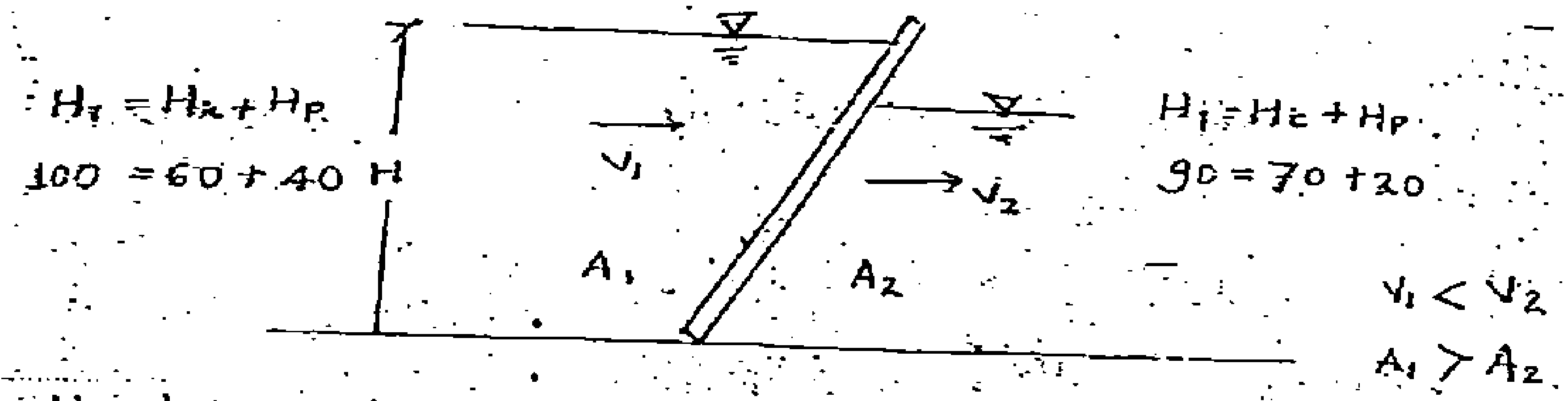
$\therefore A' > A$

$Q = AV = A'V'$

$V' < V$ $\because A' > A$.

(i) Screens are always placed at inclination, as it increases the efficiency of removal of impurities over a screen by reducing the velocity of flow giving more opportunity is provided to suspended impurities to retain over screen.

(ii) Inclination helps in better cleaning of screens by racking mechanism.



Head loss in water through ~~screen~~

$$H_L = \frac{k\,(V_2^2 - V_1^2)}{2g}$$

$$\boxed{H_L = 0.0729\,(V_2^2 - V_1^2)}$$

Fine screens :

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

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

2. Aeration :-

i) In aeration water is brought in intimate contact with air so as to remove undesirable gases like CO_2 and H_2S from water. It is also done to add oxygen in the water to carry out the oxidation of undesirable substances like organic matter and oils.

ii) It is also done to remove volatile liquids like phenol and humic acid from water.

iii) It also removes dissolved minerals like Iron and Mangese in water.

$$\left.\begin{aligned} Fe^{++} + O_2 + H_2O &\longrightarrow Fe(OH)_3\downarrow + H^+ \\ Mn^{++} + O_2 &\xrightarrow{H_2O} MnO_2\downarrow + H^+ \end{aligned}\right\}\text{oxidation}$$

This process also increases acidity of water due to H^+ ions.

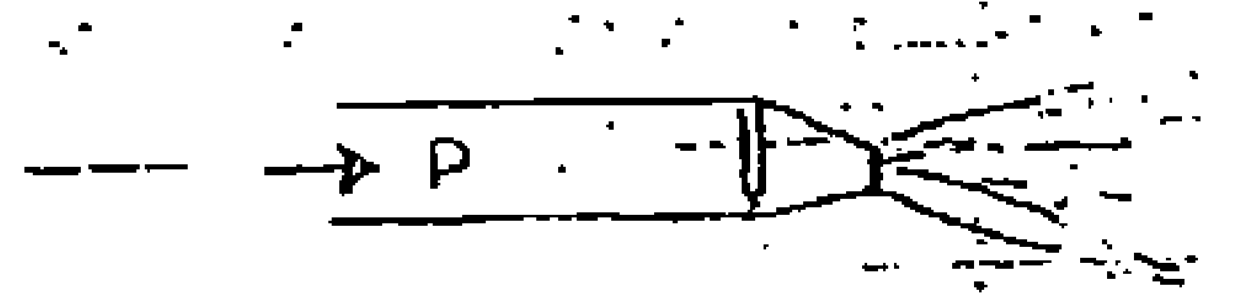
Methods of aeration.

i) Aeration is generally done for the water which is devoided of O_2 e.g. Groundwater.

ii) There are several methods to do aeration as.

- Spray nozzle
- Cascade aeration method
- Tray tower method
- Diffused air method

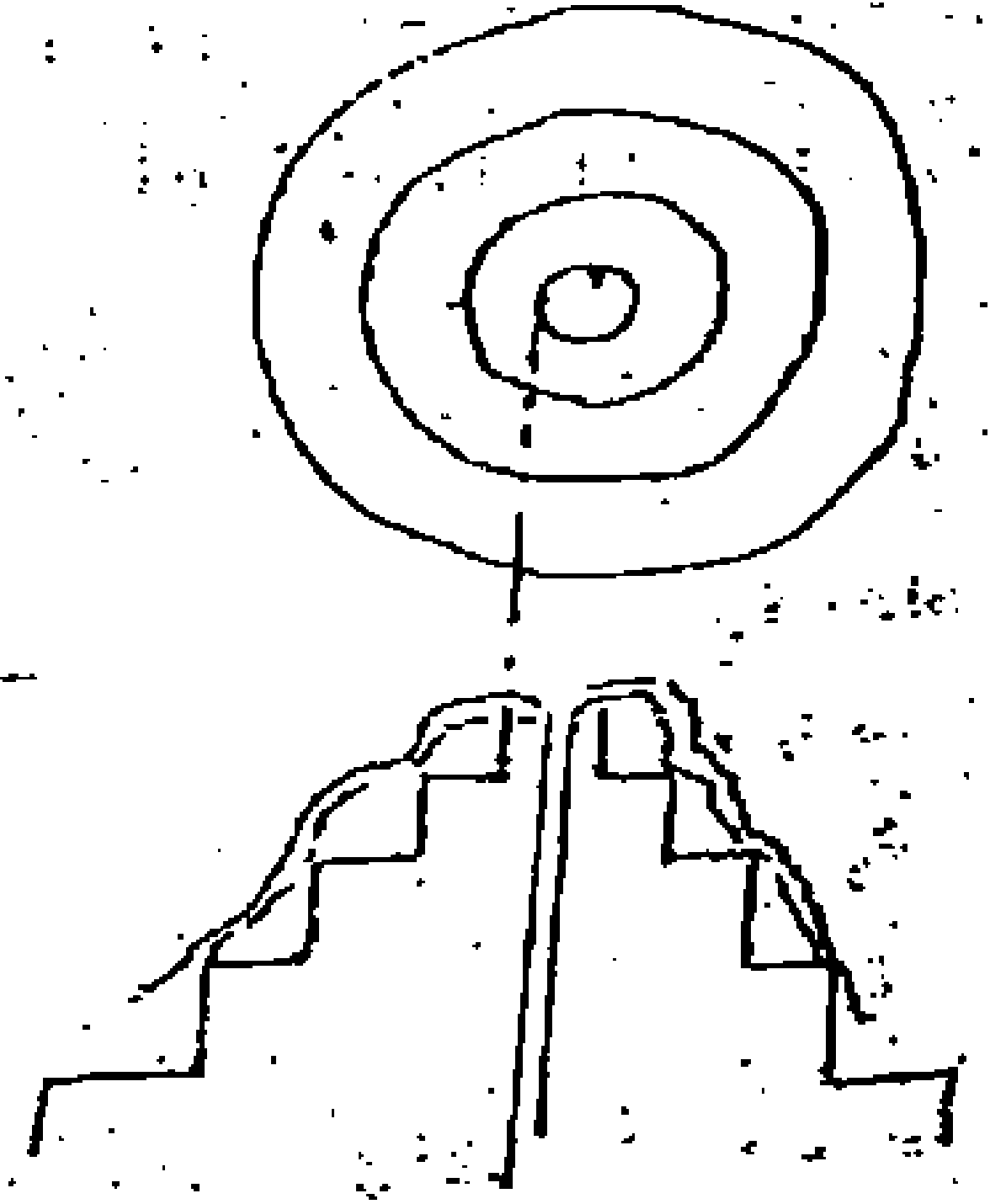
1. Spray nozzle method :-



Efficiency of this method in removal of CO_2 is 90% and in removal of H_2S is 99%.

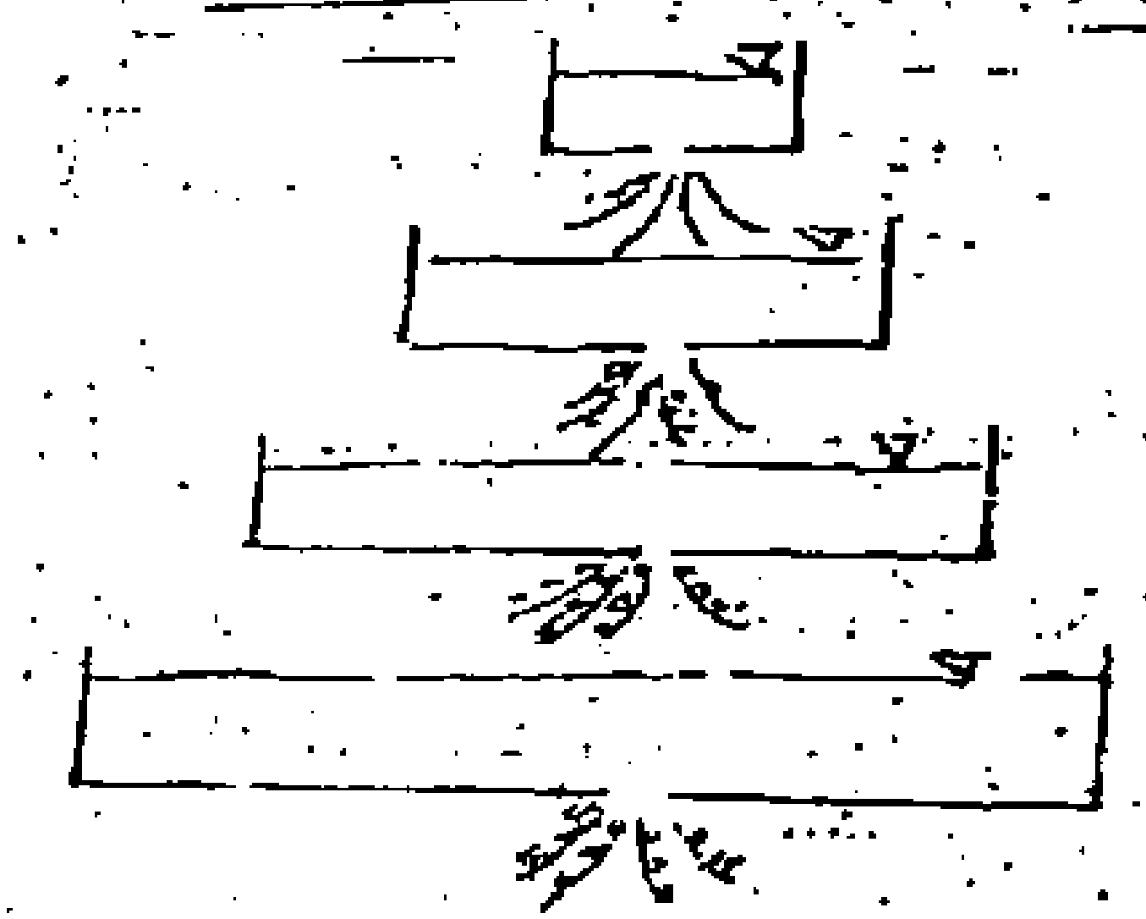
Principle is pressurised water to increase surface area of water.

2. Coscode Aerator :-



Efficiency of this method in removing CO_2 is 20 to 45% and in removing H_2S is 35%.

3. Spray tower method :-



This is the best method for removal of CO_2. It is also used for removal of Fe and Mn from water. The precipitate of Mangenese oxide is generally formed at pH of 9 hence the external alkaline agent like Potassium permangenate $KMnO_4$ is added to facilitate the formation of ppt of MnO_2.

4. Diffused air :-

3. Sedimentation :-

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

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

$$= \frac{W_s/V_s}{W_w/V_s} \qquad \because V_s = V_w$$

$$G = \frac{\gamma_s}{\gamma_w}$$

For inorganic solids. $G = 2.6$ to 2.9 (2.65)

For organic solids. $G = 1$ to 2 (1.2)

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

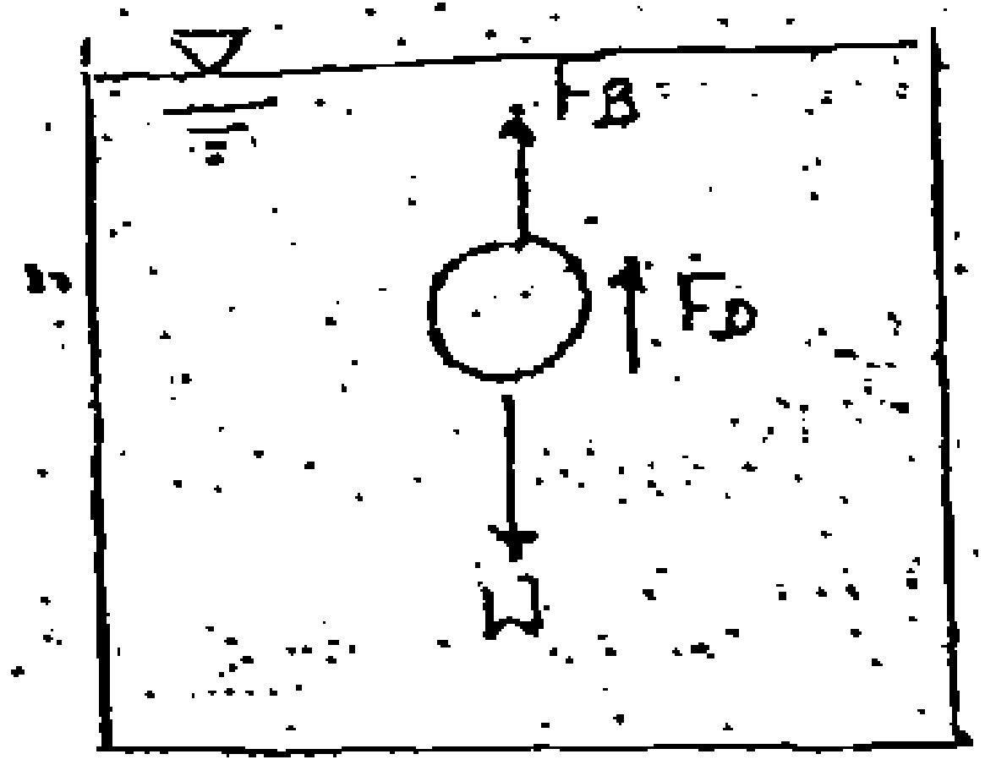
(i) Velocity of flow (~~Vt turbulance~~)

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

(ii) Viscosity of water :-

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

(iii) Size of particle :



$$F_D = \frac{1}{2} C_D \cdot \rho V_r^2 \cdot A$$

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

Force causing downward moment $= W - F_B$

Force resisting $= F_D$

As particle moves downwards it accelerates.

At certain point, $F_D = W - F_B$ ∴ Thus acceleration becomes zero and particle moves with constant velocity called as settling velocity of particle.

$$W - F_B = F_D$$

$$V_s \gamma_s - \gamma_w \cdot V_w = \tfrac{1}{2} C_D \cdot \rho \cdot A V_s^2 \qquad V_s = V_w$$

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

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

$$\frac{4}{3}\pi \left(\frac{d}{2}\right)^3 (G-1)\, g\rho = \frac{1}{2} C_D \cdot \rho \left(\frac{\pi d^2}{4}\right) V_s^2 \qquad \because \gamma_w = g\rho$$

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

For laminar flow ($R_e < 1$ and $d < 0.1$ mm)

$$C_D = \frac{24}{R_e} \qquad R_e = \frac{\rho V d}{\mu}$$

$$V_s = \sqrt{\frac{4/3\,(G-1)\cdot g \cdot d \cdot \mu\rho V_s d}{24 \cdot \rho V d\, \mu}}$$

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

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

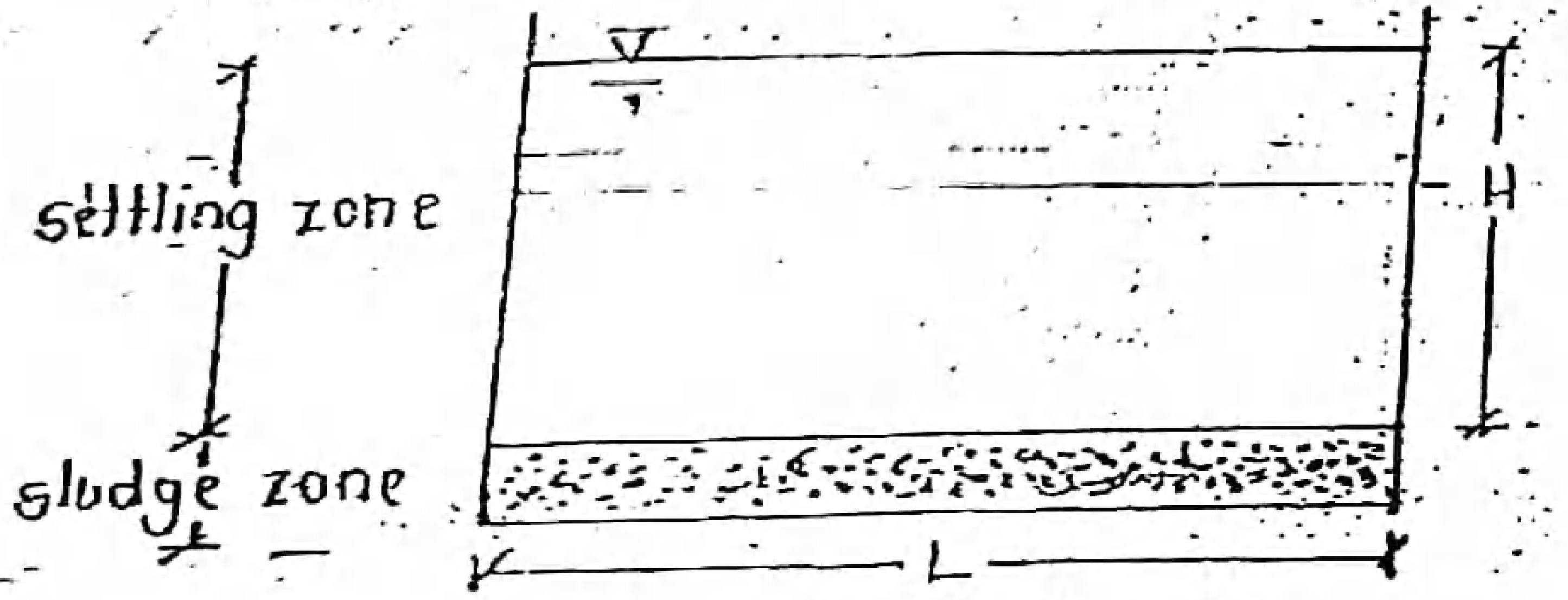
$\gamma_w = 9.8$ kN/m³

μ = N·s/m²

ν = m²/sec

Types of sedimentation tanks:

Quiscent type / Fixed type (Fill and draw type tank)



longitudinal section

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

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

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

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

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

$$Q_d = Q_{max.\,daily}$$

$$= 1.8\ Q_{avg.\,daily}$$

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

volume of tank, $V = Q_d \cdot t_d$

2. Continuous flow type tank :-

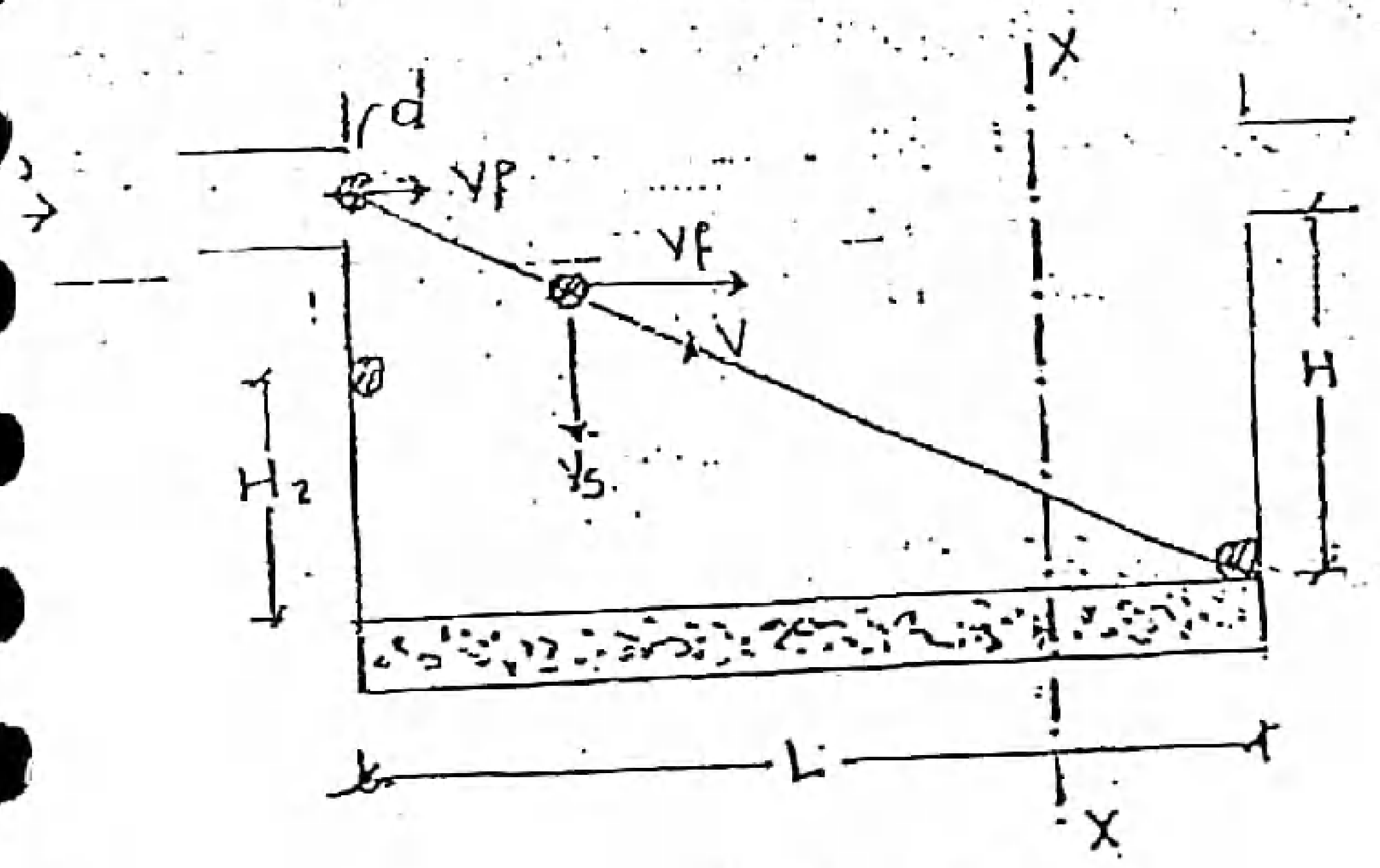
This tank is of two types.

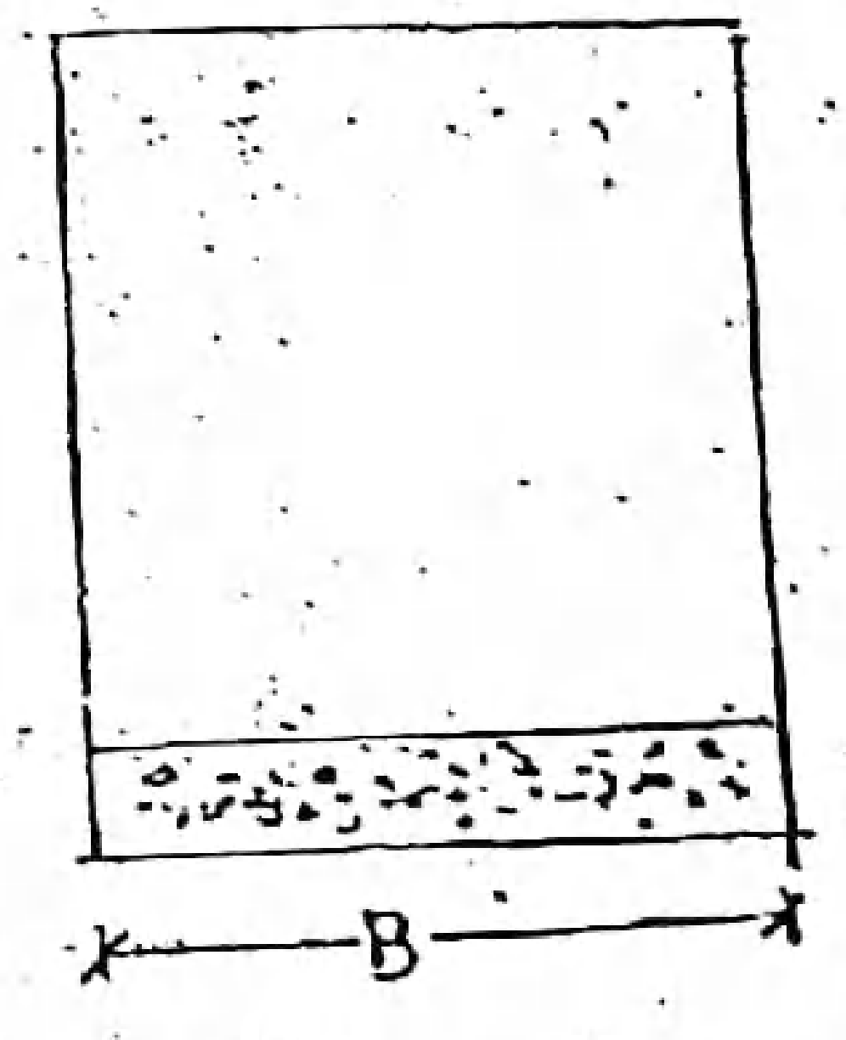
a) Horizontal flow type (square or rectangular)

b) Vertical flow type. (circular)

a) Horizontal flow type tank :-

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





section at X-X

1

Assumptions made in designing of continuous flow type tank

(i) The conc. of particles of all sizes at all points of vertical c/s at inlet is same.

(ii) A particle is assumed to be removed from the water in the tank, if it reaches the bottom of the settling zone or top of the sludge zone.

Design of tank:

$t_d = \frac{L}{V_f}$ and $t_d = \frac{H}{V_s}$ (for design particle 'd')

$A_c V_f = Q_D$ A_c - c/s area

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

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

$A_s \cdot V_s = Q_D$ A_s = plan area

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

plan area of tank $A_s = \frac{Q_D}{V_s}$

volume = $Q_D \cdot t_D$

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

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

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

$t_{d_1} < t_d$ $V_{s_1} > V_s$

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

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

$d_2 < d$

$t_{d_2} = \frac{H}{V_{s_2}}$ $\because V_{s_2} < V_s$

$t_{d_2} > t_d$

$L_2 = V_f \cdot t_{d_2}$

$L_2 > L$

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

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

$t_D = \frac{H_2}{V_{s_2}} = \frac{H}{V_s}$

$\frac{H_2}{H} = \frac{V_{s_2}}{V_s}$

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size (mm)	weight (gm)	
1	100	100%
0.8	150	
0.7	50	
0.5	200	70%
0.4	506	50%

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

$= 69\%$

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

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

overflow rate = settling velocity of design particle.

V_s = 0.35 mm/sec (for design particle)

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

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

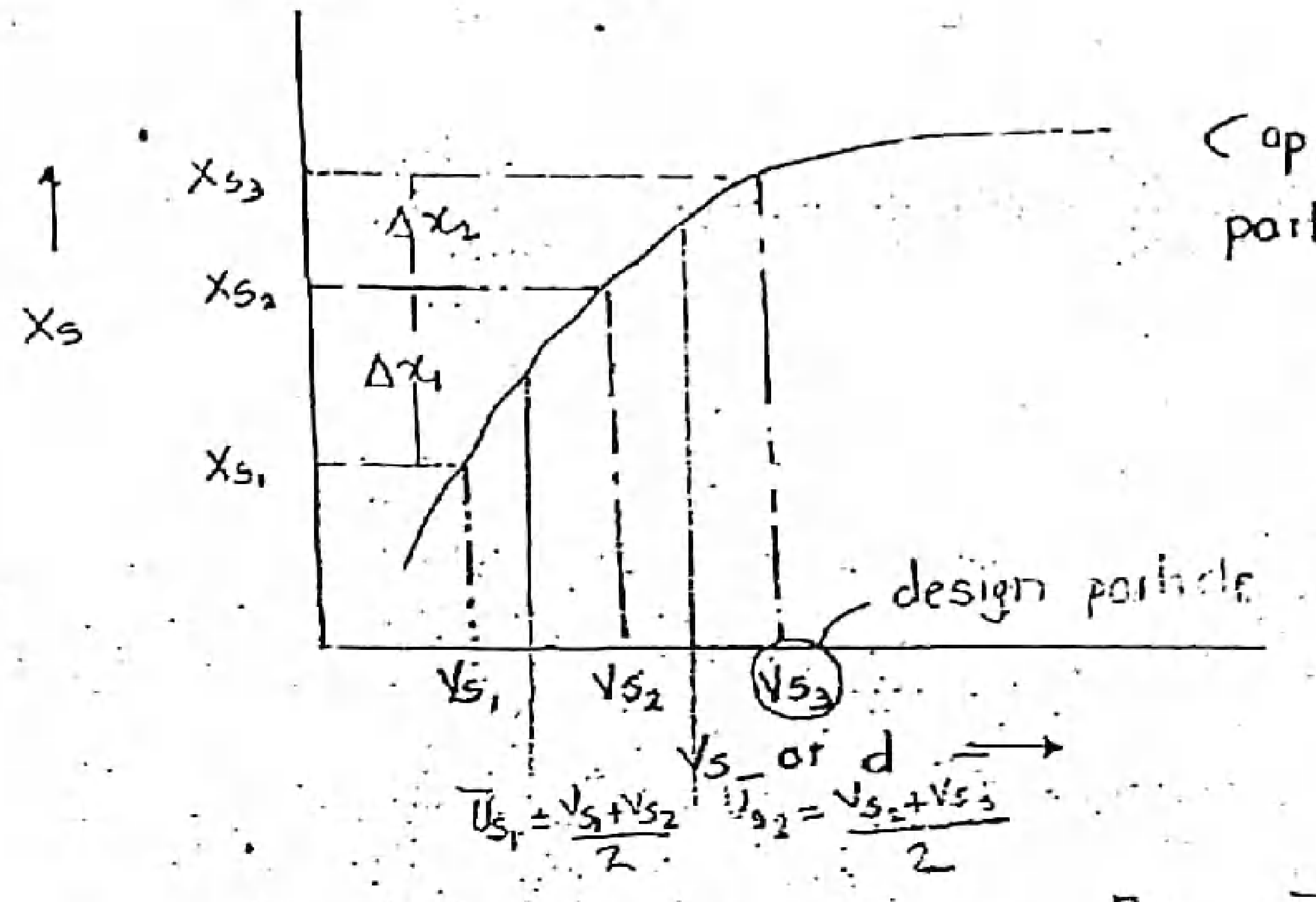
————— 0.2 ————— $= \frac{0.25}{0.35} \times 100 = 71.4\%$

————— 0.1 ————— $= \frac{0.2}{0.35} \times 100 = 57.14\%$

$$\text{Efficiency of tank } (\eta) = \frac{\left\{\left(1000\times\frac{55}{100}\right)\times\frac{100}{100} + \left(1000\times\frac{15}{100}\right)\times\frac{85.71}{100} + \left(1000\times\frac{20}{100}\right)\times\frac{71.4}{100} + \left(1000\times\frac{10}{100}\right)\times\frac{57.14}{100}\right\}100}{1000}$$

$$'' = 88.14\%$$

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



(approximate curve for particle size)

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

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

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

$$\eta = (100 - X_s) + \int_{x=0}^{X_s}\frac{\bar{U}_s}{V_s}\,dx$$

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

overflow rate = 32.6 m/day

sp. gravity = 1.2

dynamic viscosity = 1.027 centi poise

density of water = 0.997 gm/cm³

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

overflow rate = 32.6 m/day

$$= \frac{32.6\times10^{6}\text{ mm}}{1\times24\times3600}$$

$$= 0.38 \text{ mm/sec}$$

Settling velocities of particles.

$$U_s = \frac{(G-1)\gamma_w \cdot d^2}{18\mu} \quad \text{in } N/m^3 \quad Ns/m^2$$

ρ_w in N/m³ ; $\gamma_w = \rho_w \cdot g$; dia in m

$$= \frac{(1.2-1)(0.997\times10^{-3}\times10^{6}\times d^2\times 9.81\times10^{-6})\times10^{3}}{18\times1.027\times\underset{\text{centi}}{10^{-2}}\times\underset{\text{poise in SI}}{10^{-1}}}$$

Again into mm/sec

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

$$U_{s_1} = 105.8 \times (0.1)^2$$

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

particle size d (mm)	0.1	0.08	0.07	0.06	0.04	0.02	0.01
settling velocity (mm/sec)	1.058	0.67	0.51	0.38	0.16	0.04	0.01
					U_{s_3}	U_{s_2}	$\overline{U_{s_1}}$

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

$$X_s = 30\%$$

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

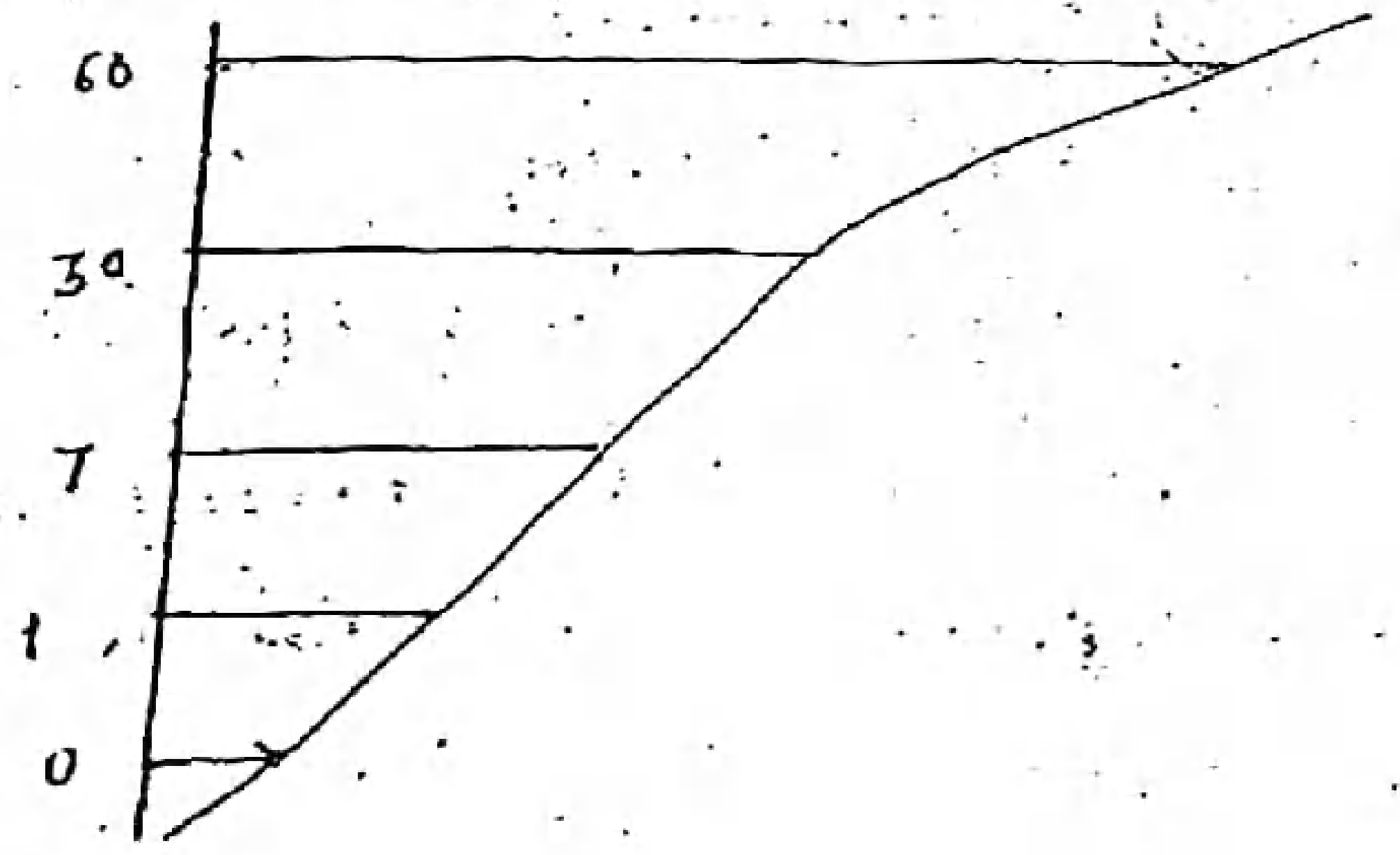
$\Delta X_1 = (1-0) = 1\%$ $\qquad \overline{U_{s_1}} = \frac{0.04+0.01}{2} = 0.025$ mm/sec

$\Delta X_2 = (7-1) = 6\%$ $\qquad \overline{U_{s_2}} = \frac{0.04+0.16}{2} = 0.1$ mm/sec

$\Delta X_3 = (30-7) = 23\%$ $\qquad \overline{U_{s_3}} = \frac{0.38+0.16}{2} = 0.27$ mm/sec

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

$$= 87.9\%$$



Design data for sedimentation tank:-

(i) overflow rate (SOR)

overflow rate is in the range of 12,000-18,000 lit/m²/day for plain sedimentation. (12-18 m³/m²/day).

It is in the range of 24000-30000 lit/m²/day for the coagulation aided sedimentation (24-30 m³/m²/day)

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

(ii) Detention time (t_D)

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

volume of tank = design discharge x detention time.

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

(iii) Velocity of flow (V_f):-

It is in the range of 0.15 to 0.9 m/~~sec~~min Normally it is taken to be 0.3 m/min.

Length of tank = velocity of flow x detion time

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

$$\therefore L/B = 4:1$$

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

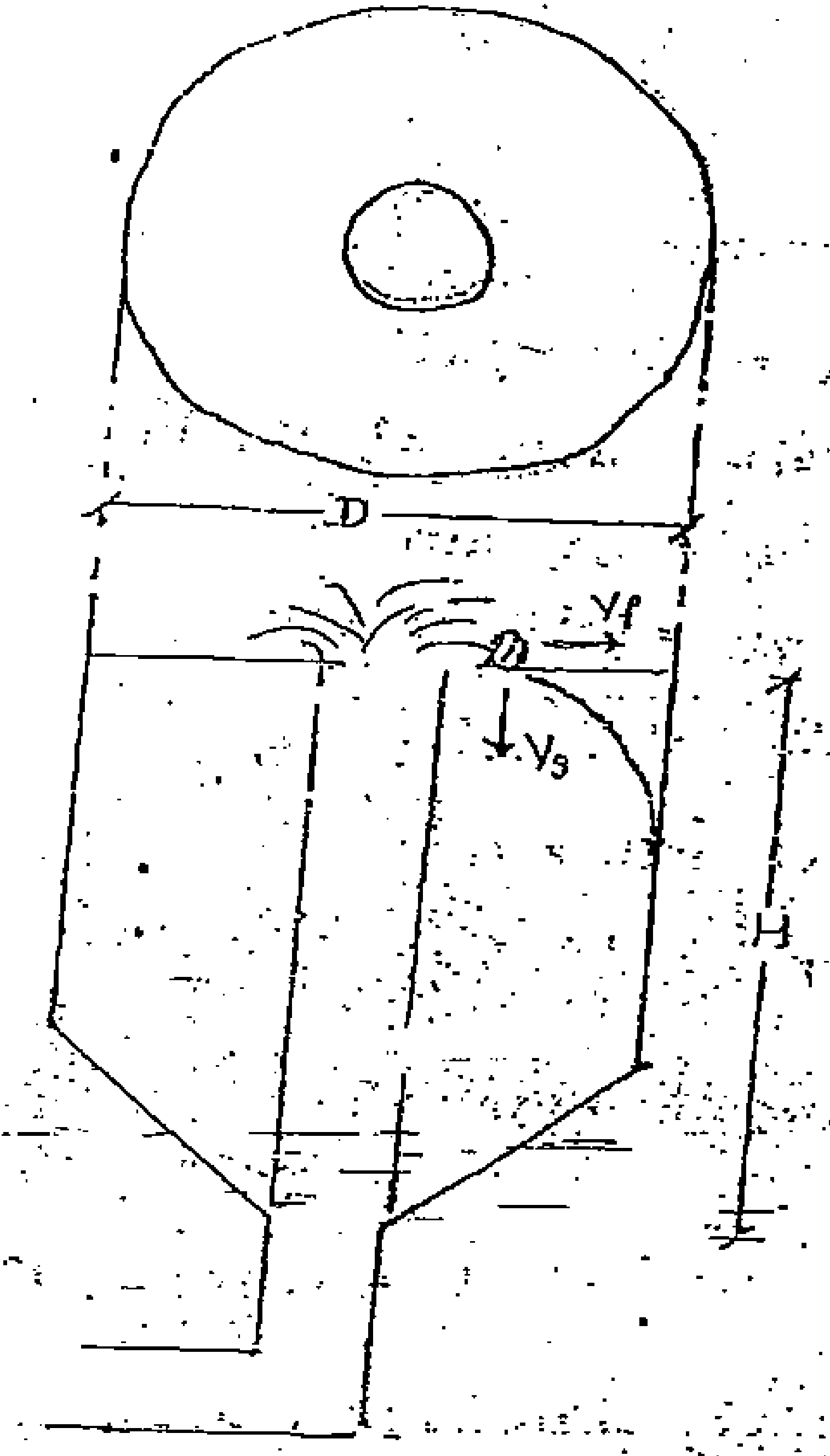
$$Q_D = Q_{max.\,daily}$$

$$= 1.8\ Q_{avg.\,daily}$$

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

(vi) If cleaning of sludge is done manually then additional volum for accumulation of sludge is provided by adding 0.8-1.2 m of depth over design depth.

b) Vertical flow type tank :-



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

volume of the tank is given by

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

$$V = Q_D \cdot t_D$$

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

Determine the surface area for settling tank for 0.5 m³/s flow using design overflow rate as 32.5 m³/m²/day. Also find the depth of tank if detention time is 95 min. Assume L/B = 2:1 to L/B = 5:1 and length of tank should not exceed 100 m.

Data:-

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

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

$t_D = 95 \text{ min}$

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

$L \ngtr 100 \text{ m}$

To find:

plan area = ?

tank depth d = ?

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

$$= 1329.2 \text{ m}^2$$

$$\text{volume of tank} = Q_D \times t_D$$

$$= 0.5 \text{ m}^3/\text{sec} \times 95 \times 60 \text{ sec}$$

$$= 2850 \text{ m}^3$$

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

$$= \frac{2850}{1329.2}$$

$$= 2.14 \text{ m}$$

Assume $L/B = 4$

$(4B) \times (B) = 1329.2$

$B = 18.2 \text{ m}$

$L = 72.9 \text{ m}$ $\ngtr 100 \text{m}$ OK.

Q. In continuous flow settling tank 3.5 deep and 65 m long, flow velocity of water is observed as 1.22 cm/sec. What size of particle of sp. gravity 2.65 may be effectively removed in this tank, if the kinematic viscosity of water is 0.01 cm²/sec.

Data :-

$H = 3.5\,m$

$L = 65\,m$

$V_f = 1.22\ cm/sec$

$G = 2.65$

$\nu = 0.01\ cm^2/sec.$

To find : dia. of particle, d = ?

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

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

$$= \frac{3.5 \times 1.22 \times 10^{1}\ mm/sec}{65}$$

$$= 0.65\ mm/sec$$

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

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

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

$$\mu = \nu \times g$$

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

$$d = 0.027\ mm < 0.1\ mm$$

4. Co-agulation aided sedimentation :-

(i) The efficiency of sedimentation is very less when water contains very fine SS, hence to improve its efficiency coagulation aided sedimentation is adopted.

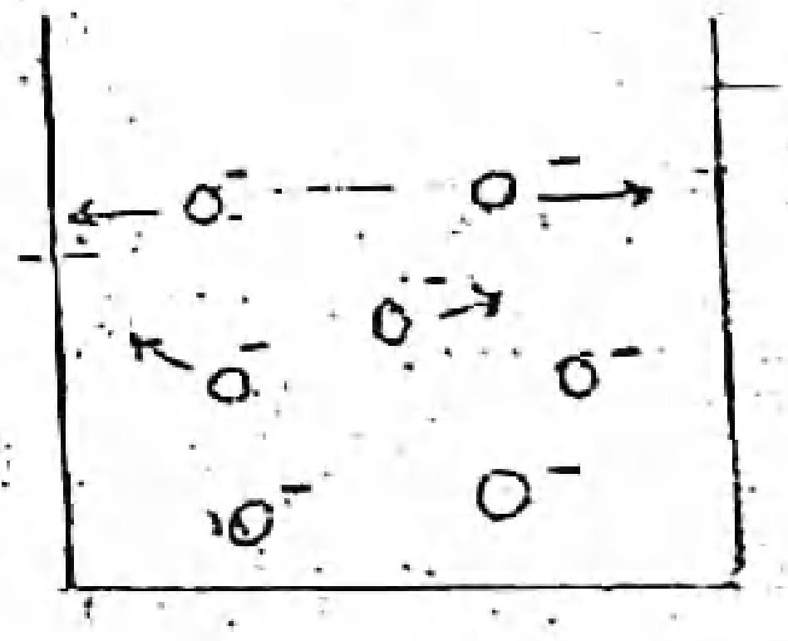
(ii) The entire process of this co-agulation is completed in three stages.

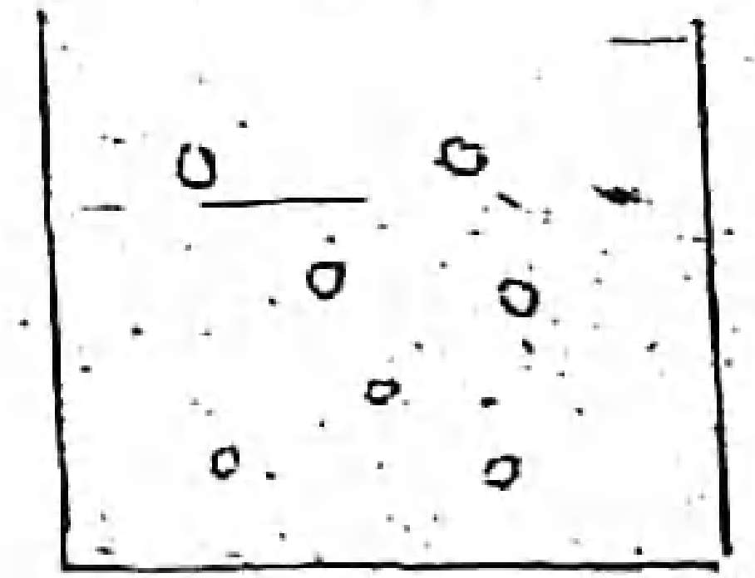
- Co-agulation (fast mixing)
- Flocculation (slow mixing)
- Sedimentation.

Coagulation :-

(i) It is the process in which certain chemicals known as coagulants are added in the water, so as to neutrilise negative protective charge over the particles in order to increase their contact opportunity in subsequent stages.

(ii) A certain minimum amount of energy is required for these neutrilisation of charge (known as Threshold energy) which is provided in the co-agulation process by inducing rapid mixing in the water.





(iii) Different types of co-agulants added in treatment of water are

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

Alum (Hydrated aluminium sulphate)

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

$$Al_2(SO_4)_3 \cdot 18H_2O + 3\,Ca(HCO_3)_2 \longrightarrow \underset{(sticky)}{2Al(OH)_3\downarrow} + 3CaSO_4 + \underset{18H_2O}{\cancel{18}H_2O} + 6CO_2$$

(ii) This process also adds permanent hardness ($CaSO_4$) in the water and also increasing the acidity of water (due to CO_2) thus making it corrosive.

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

$$\underset{\text{limestone}}{CaCO_3} \xrightarrow{\Delta} \underset{\substack{\text{quick lime}\\\text{high purity (95-96\%)}}}{CaO} \qquad \overbrace{CaO + H_2O \longrightarrow}^{\text{slaking}} \underset{\substack{\text{hydrated lime}\\\text{or}\\\text{slaked lime}}}{Ca(OH)_2} + \text{Heat}$$

Calcination

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

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

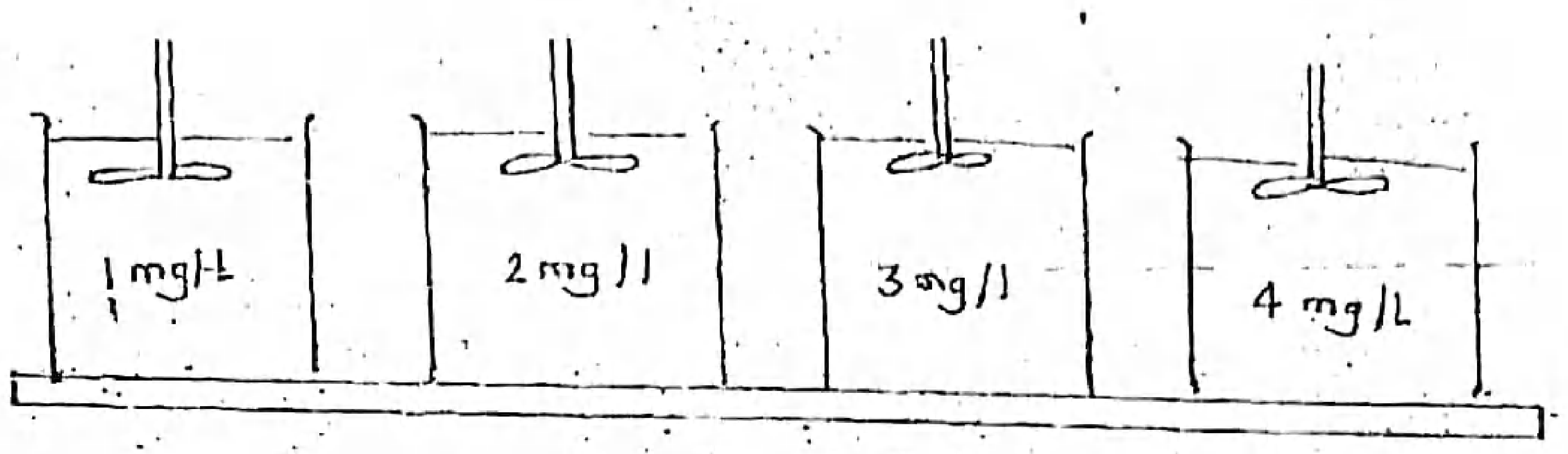
$$Al_2(SO_4)_3 \cdot 18H_2O + 3\,Ca(OH)_2 \longrightarrow 2Al(OH)_3\downarrow + 3CaSO_4 + 18H_2O$$

When lime is added acidity is not induced (CO_2) but hardness is induced in water.

$$Al_2(SO_4)_3 \cdot 18H_2O + \underset{\text{Soda ash}}{Na_2CO_3} \longrightarrow 2Al(OH)_3\downarrow + 3Na_2SO_4 + CO_2 + H_2O$$

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

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



(ii) The normal dose of alum varies between 10-30 mg/l.

(iii) Alum forms stable flocs and is capable of removing colour test and odour from the water.

(iv) It works in the pH range of 6.5 - 8.5.

(v) One mole of alum added in water gives two moles of ppt of $Al(OH)_3$

Mol. wt. of alum = $Al_2(SO_4)_3 \cdot 18H_2O$

$= (27\times2) + 3(32+16\times4) + 18(2\times1+16)$

$= 666$ gm

Mol. wt. of $Ca(HCO_3)_2$ $= (40 + 2(1+12+16\times3))\times3$

$= 486$ gm

Mol. wt. of $Al(OH)_3$ $= (27 + 3(16+1))\times2$

$= 156$ gm

$$Al_2(SO_4)_3 \cdot 18H_2O + \underset{3\text{ moles}}{3\,Ca(HCO_3)_2} \longrightarrow \underset{2\text{ moles}}{2Al(OH)_3\downarrow} + 3CaSO_4 + 18H_2O + 6CO_2$$

666 gm of alum react with 486 gm $Ca(HCO_3)_2$ → ~~156 gm Al(OH)₃~~

1 gm → $\frac{486}{666} = 0.73$ gm → $\frac{156}{666} = 0.234$ gm

OR

666 gm of alum reacts with 300 gm alkalinity as $CaCO_3$ → 156 gm $Al(OH)_3$

Copperas ($FeSO_4 \cdot 7H_2O$). Hydrated Ferrous sulphate.

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

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

$$Fe(HCO_3)_2 \xrightarrow{O_2} Fe(OH)_3 \downarrow$$

$$FeSO_4 \cdot 7H_2O + Ca(OH)_2 \longrightarrow Fe(OH)_2 + CaSO_4 + H_2O \qquad Fe(OH)_2 \xrightarrow{O_2} Fe(OH)_3 \downarrow$$

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

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

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

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

Chlorinated copperas ($Fe_2(SO_4)_3 + FeCl_3$)

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

$$FeSO_4 \cdot 7H_2O + Cl_2 \longrightarrow Fe_2(SO_4)_3 + FeCl_3$$

– (Redox reaction)

$$Fe_2(SO_4)_3 + Ca(HCO_3)_2 \longrightarrow 2Fe(OH)_3 \downarrow + CaSO_4$$

$$FeCl_3 + Ca(HCO_3)_2 \longrightarrow Fe(OH)_3 \downarrow + CaCl_2$$

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

(iii) When Ferric chloride ($FeCl_3$) is used independently it works in the pH range of 3.5 to $\overset{6.5}{8.5}$ and above 8.5.

(iv) When Ferric sulphate $Fe_2(SO_4)_3$ is used independently it works in pH range of 4 to 7 and above 9.

Sodium aluminate ($Na_2Al_2O_4$)

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

$$Na_2Al_2O_4 + Ca/Mg \left\{ \begin{matrix} CO_3^{--} \\ HCO_3^{-} \\ SO_4^{-} \\ NO_3^{-} \\ Cl^{-} \end{matrix} \right. \longrightarrow \begin{matrix} \cancel{Ca/Mg(OH)_2} \\ Ca/Mg\,Al_2O_4 \downarrow \\ \text{sticky} \end{matrix} + Na \left\{ \begin{matrix} CO_3^{--} \\ HCO_3^{-} \\ SO_4^{-} \\ NO_3^{-} \\ Cl^{-} \end{matrix} \right.$$

(ii) This process removes both temporary and permanent hardness in water hence generally used for boiler feed water treatment.

(iii) Sodium aluminate is costlier than alum.

(iv) This process does not require the presence of alkalinity in water.

(v) It works in pH range of 6 to 8.5.

Note:

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

(ii) Iron salt works in wide pH range.

(iii) Iron salts are cheaper than Aluminium salts.

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

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

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

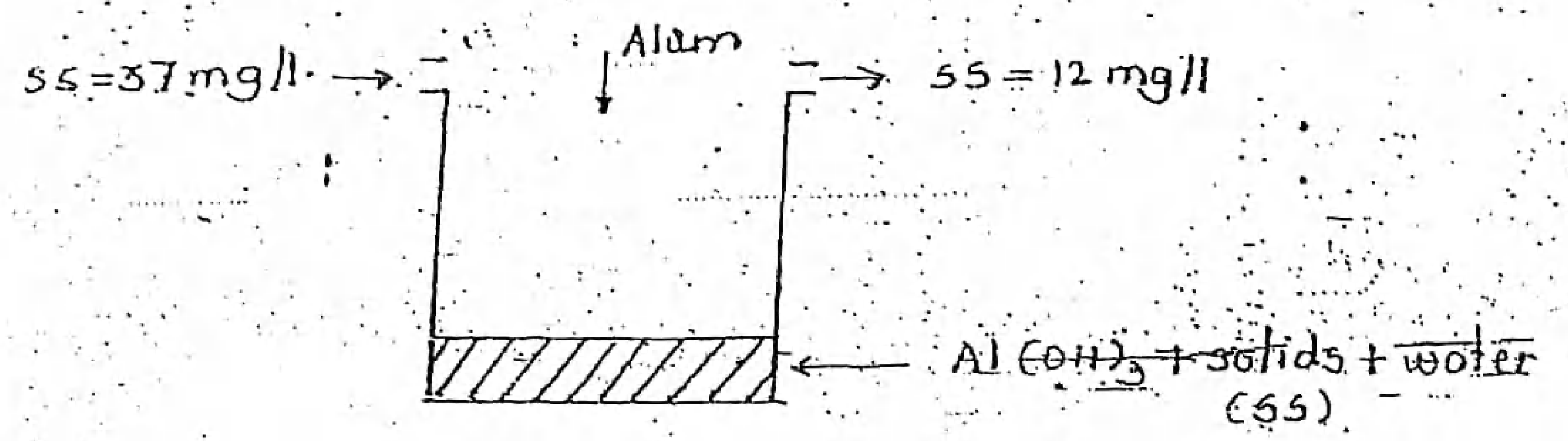
(vii) The disadvantage of Iron coagulant due to iron bacteria is overcome in sewage treatment as Concrete pipes are used in sewer constructions.

Q. A co-agulation treatment plant with flow of 0·5 m^3/sec is dosing alum at 23 mg/l. No other chemical is being added. The raw water SS concentration is 37 mg/l and effluent solid concentration is 12 mg/l. The solid content in sludge is 1%. and specific gravity of the sludge solids is 3·01. What volume of sludge must be disposed per day from this plant.

Data: Q = 0·5 m^3/sec

Alum dose = 23 mg/l

G = 3·01



settled SS in sludge = 37 − 12
= 25 mg/l

wt. of SS settled in tank in one day = (25 × 0·5 m^3/sec) × 10^3 × 86400 × 10^{-6}
day kg
= 10800 kg/day

1 gm of alum gives 0·23 gm ppt of Al $(OH)_3$

23 mgm of alum gives 0·234 × 0·23 = 0·538 mg/l of Al $(OH)_3$

wt. of ppt formed in tank = 5·58 mg/l × (0·5 × 10^3 × 86400 × 10^{-6})
= 232·4 kg/day

* Assuming 100% ppt is settled in tank,

wt. of solids in the sludge = 1080 + 232·4
= 1312·4 kg/day

1 kg of solids + 99 kg water = 100 kg of wet sludge

1312·4 kg solids + 99 × 1312·4 kg = ~~1299276~~ kg wet sludge
= 129927·6 131240

volume of sludge = volume of solids + volume of water

$$= \frac{1312 \cdot 4}{3 \cdot 01 \times 1000} + \frac{129927 \cdot 6}{1000} \qquad G = \frac{\gamma_s}{\gamma_w}$$

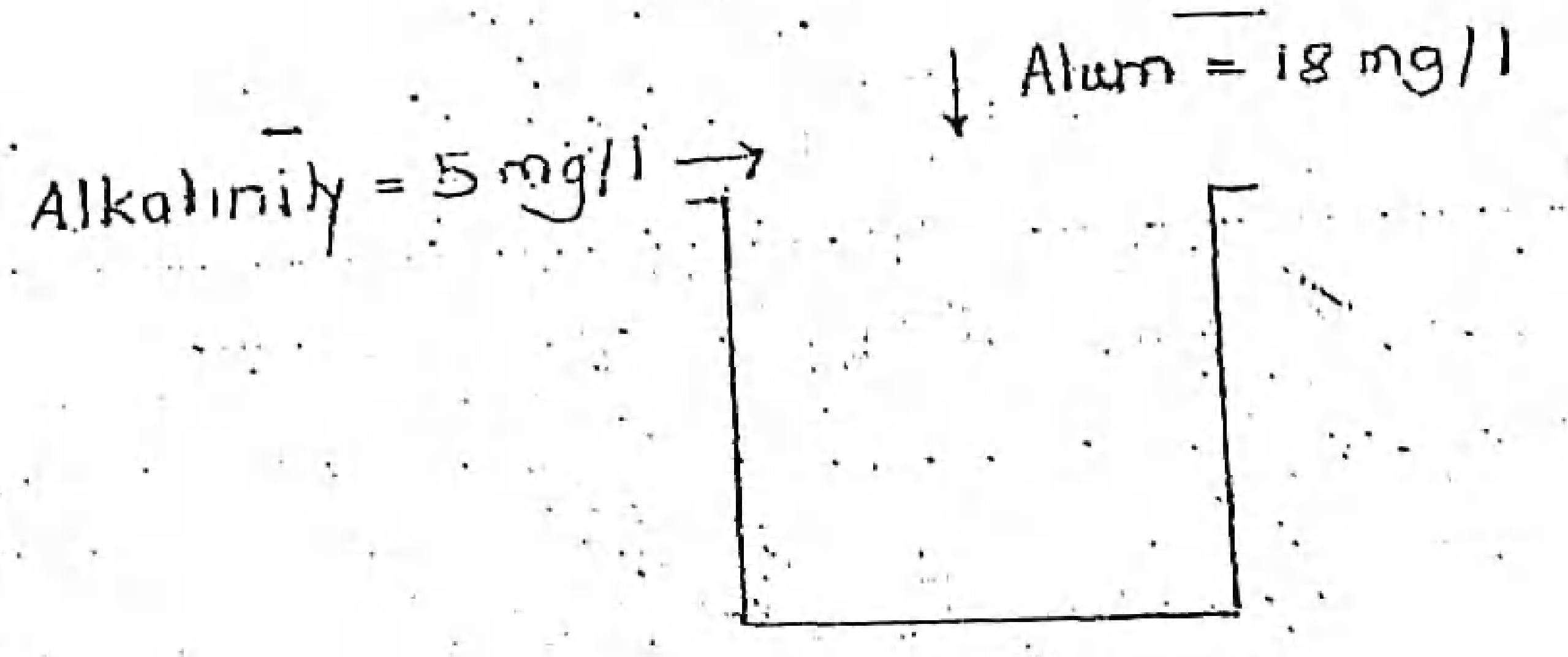
= 0·436 + 129·9

= 130·35 m^3/day.

A co-agulation sedimentation plant clarifies 40 mld of water. The quantity of filter alum required at the plant is 18 mg/l. If the raw water is having an alkalinity equivalent to 5 mg/l as $CaCO_3$, determine the quantity of filter alum and quick lime required which is 85% pure per year by the plant.

Data: Q = 40 mld

Alkalinity = 5 mg/l



Quantity of filter alum reqd. per year = 18 mg/l × 40 × 10^6 × 365 × 10^{-6} × 10^{-3}
1 tonne
= 262·8 tonnes/year

1 mole of alum reacts with 3 moles alkalinity as $CaCO_3$

666 gm — 300 gm —

1 gm — 0·45 gm —

18 mg/L 0·45 × 18 = 8·1 mg/l —

Additional alkalinity to be added as $CaCO_3$ = 8·1 mg/l − 5 mg/l
= 3·1 mg/l

1 mole of $CaCO_3$ alkalinity is formed by 1 mole 100% pure lime

100 gm — 56 gm —

1 gm. of $CaCO_3$ alkalinity is formed by adding $\frac{56}{100}$ mg 100% pure lime

3.1 mg ——→ $0.56 \times 3.1 = 1.73$ mg ∴ 1.736

3.1 mg of $CaCO_3$ alkalinity with 85% pure lime is formed by adding

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

Quantity of 85% pure lime in one year $= 2.04 \times 40 \times 10^6 \times 365 \times 10^{-6} \times 10^{-3}$

$= 29.7$ tonne/year.

Sunday
21th July 2013

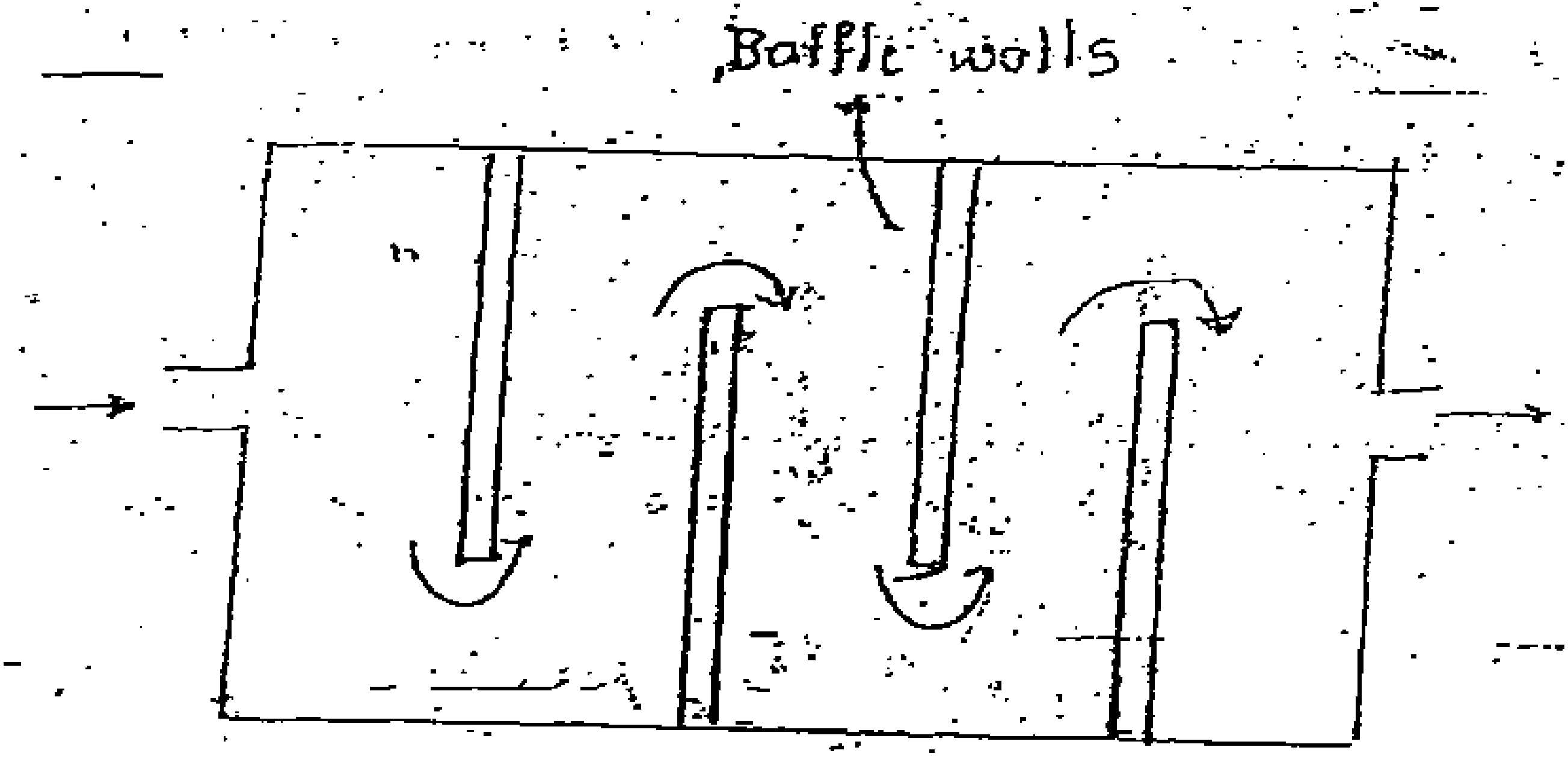
5. Coagulation:

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

Mixing basins :

(a) Round and end type
(b) Over and under type

(a) Round and end type mixing basin:



plan

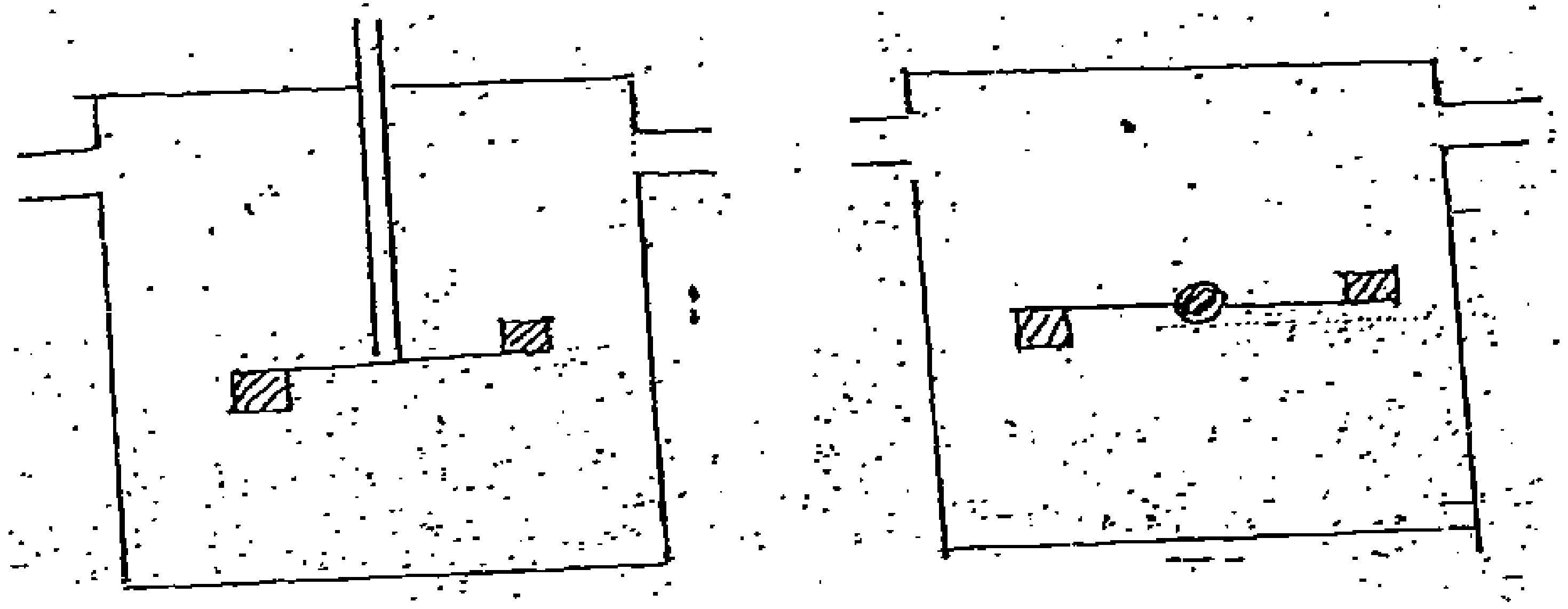
(c/s for over and under type)

In mixing basins, mixing is induced by creating the turbulance in water and this turbulance is created by rapid change in direction of flow of [illegible]

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

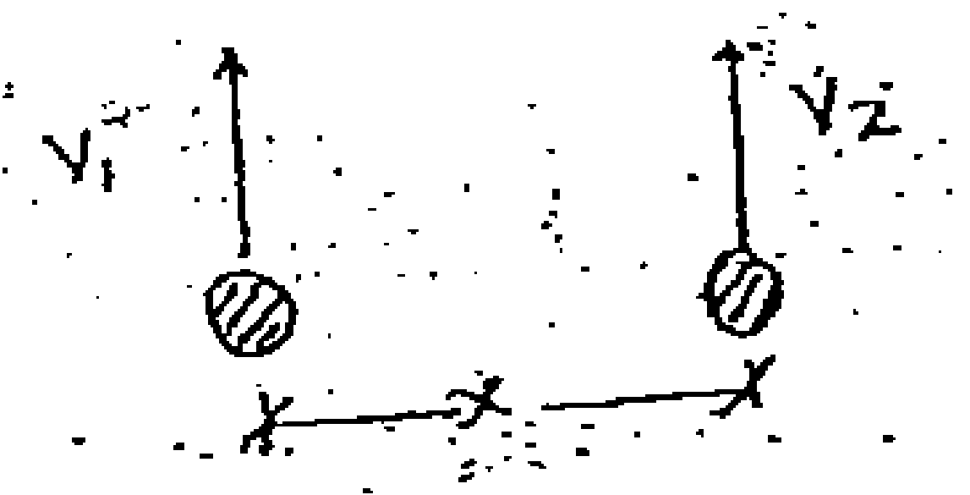
Mechanical mixers :

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



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

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



$$\therefore G = \frac{V_1 - V_2}{x} \quad /\text{sec}$$

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

Where,

P - power (Watts)
μ - dynamic viscosity (Ns/m^2)
V : volume of water (m^3)

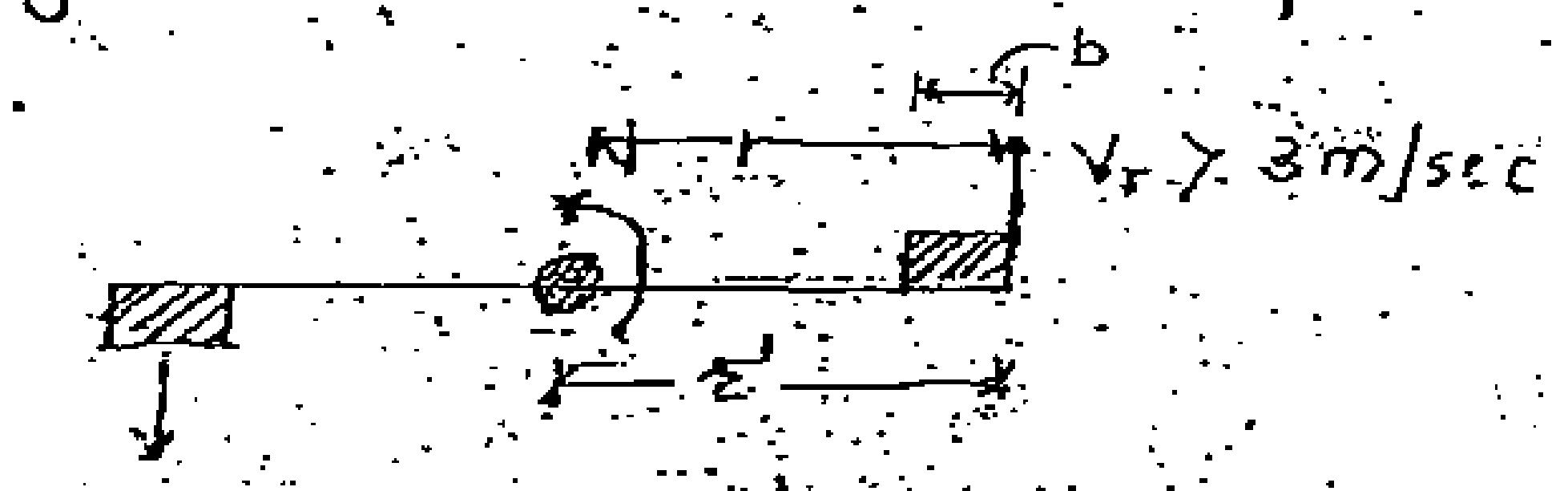
(iii) The detention time in these mixers is normally kept to be 30-60 sec.

(iv) Circular or square tanks are used in this case having depth to width ratio from 1:1 to 3:1. (H/B or H/D)

(v) The power required for the mixing is 0·1 to 3 Watts per unit discharge in m^3/hr.

(vi) The ratio of impeller diameter to tank diameter is in the range of 0·2 to 0·4.

(vii) The speed of the shaft should be such that tangential velocity of greater than 3 m/sec is imparted at the tip of the blade.



$r' = r + b/2$

$V_T = \omega r'$

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

- Flocculation :-

(i) In flocculation neutralised suspended particles are provided with sufficient contact opportunity so as to agglomerate (to combine) and grow in size and get finally removed in the following sedimentation process.

(ii) In order to promote the contact between the neutralised particle so as to form flocs greater in size than the original size of the particle, slow mixing is induced in water

(iii) The rate of flocculation depends upon the following factors.

a) Turbidity (more turbidity more is the flocculation)

b) Type and dose of coagulant (Iron coagulant will form heavy flocs in less time)

c) G (Temporal mean velocity gradient) :- depends on power of shaft.

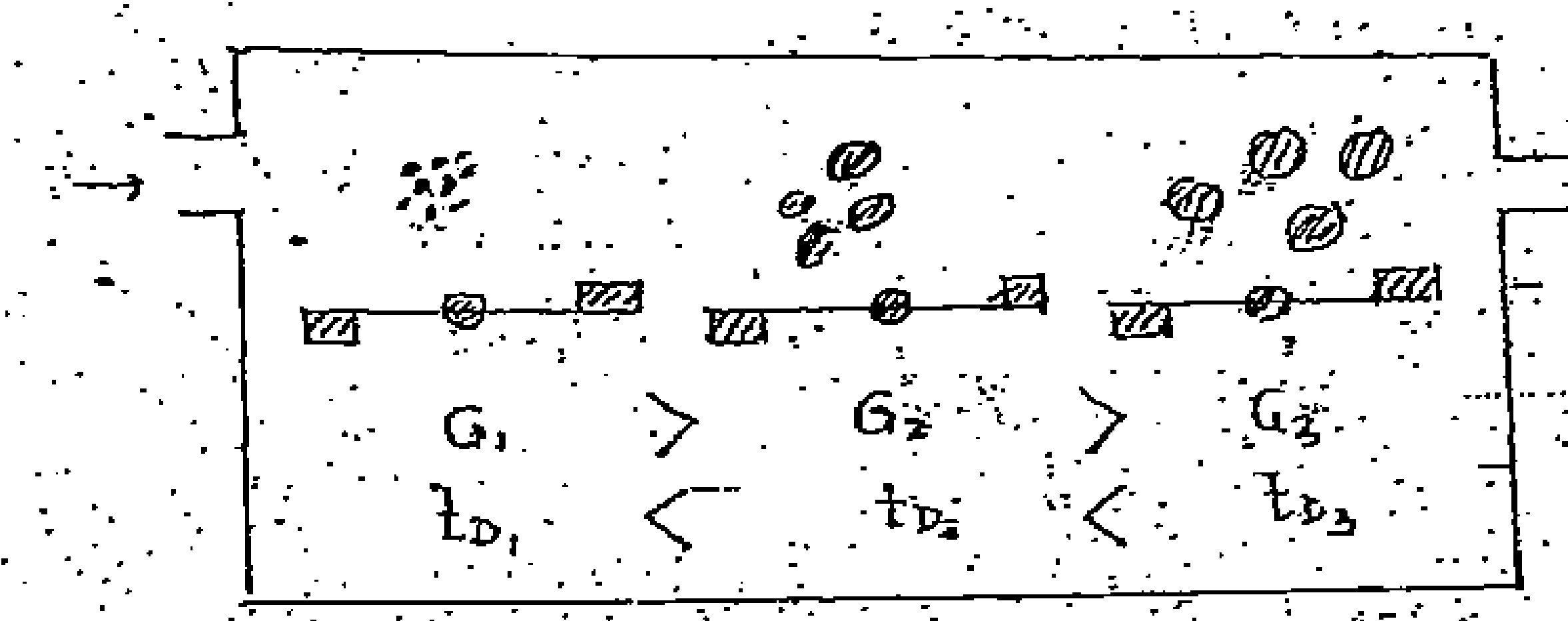
(iv) $G \cdot t_D$ is a parameter which determines the conjugation opportunity between the particles. or no of collisions i.e. no of times particles may come in contact with each other

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

$G = \frac{V_1 - V_2}{x}$ (less x)

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

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



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

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

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

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

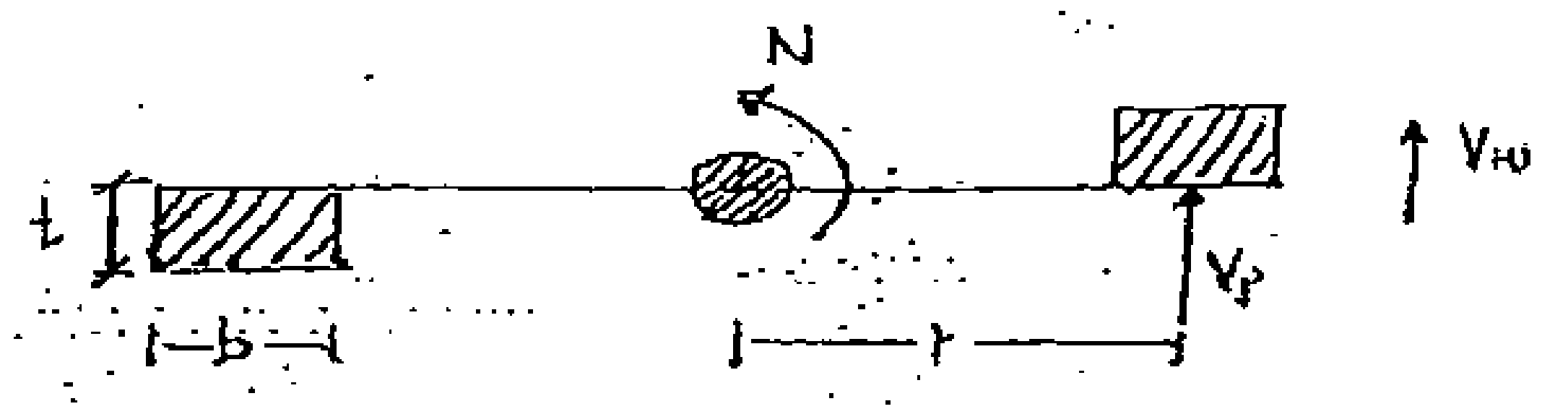
If At inlet $G = G$, $P = P$

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

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

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

Calculation of temporal mean velocity gradient (G)



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

$$P = F_D \cdot V_r$$

where.

F_D - drag force

V_r - relative velocity of wal

$$V_r = V_p - V_w$$

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

projected area of plates on which drag force will act

$A_r = n(b.l)$ for n blades.

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

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

($C_D = 1.8$ for this case)

$$G = \sqrt{\frac{1/2 \times 1.8 \times \rho_w \cdot A_r \cdot V_r^3}{\mu V}}$$

(i) The height of the tank is normally taken to be 3 - 3.5 m

(ii) Detention time for flocculation is 10 - 30 min.

(iii) The area of the plates is approximately 10 - 25 % of total area of tank. (plan area)

(iv) The velocity of flow is in the range of 0.2 - 0.8 m/sec.

(v) Temporal mean velocity gradient (G) is in the range of 10 - 75 /sec.

(vi) G-to for alum is 20,000 - 60,000 and 1 - 1.5 lacs for Iron Flocs.

Q. A flocculation chamber is 30m long ; 12m wide and 4.5m deep is to treat 75 mld of water. It is equipped with 12 m long and 0.3m wide paddles supported parallel to each other and moved by four horizontal shaft which rotate at speed of 2.5 rpm. The centreline of paddles is 1.8m from the shaft which is situated at mid depth of tank. Two paddles are mounted on each shaft opposite to each other. If the mean velocity of water is 1/4th the velocity of paddle find

(i) Power consumption

(ii) Time of flocculation

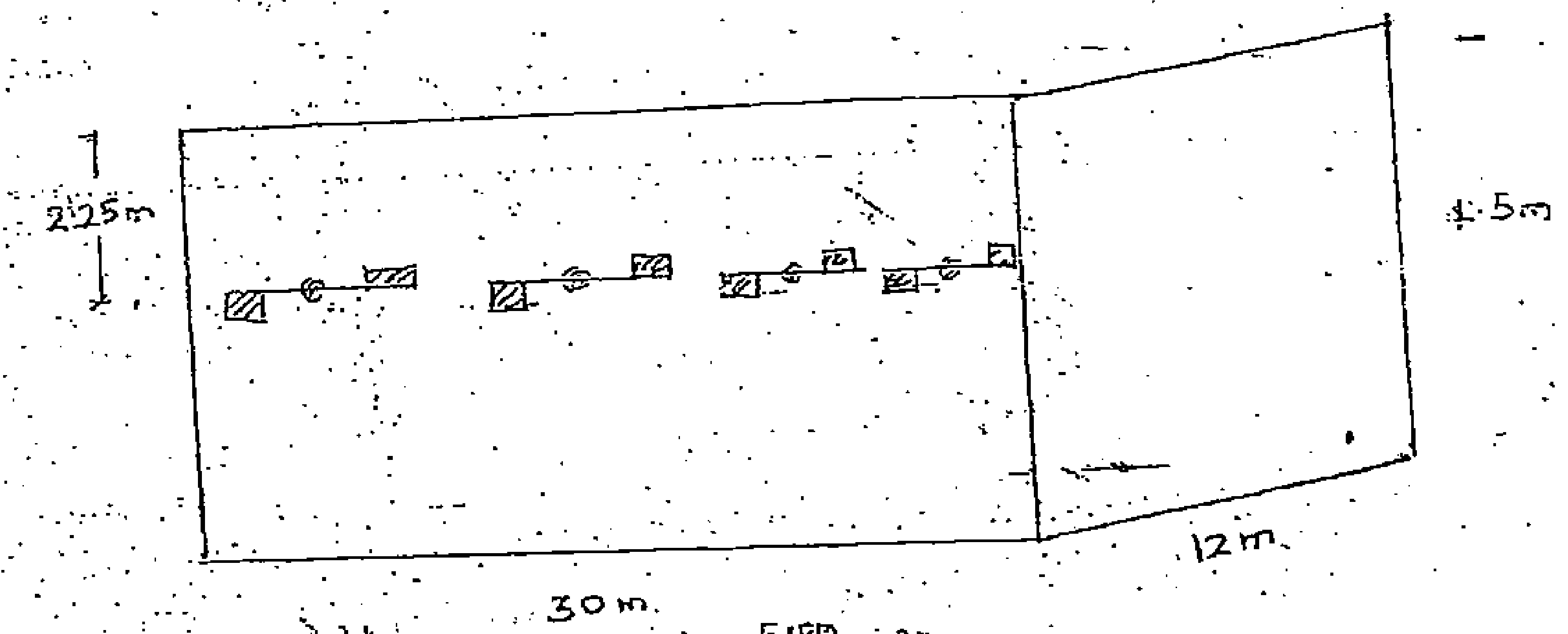
(iii) Value of G if kinematic viscosity of water is 1.31×10^{-6} m²/sec

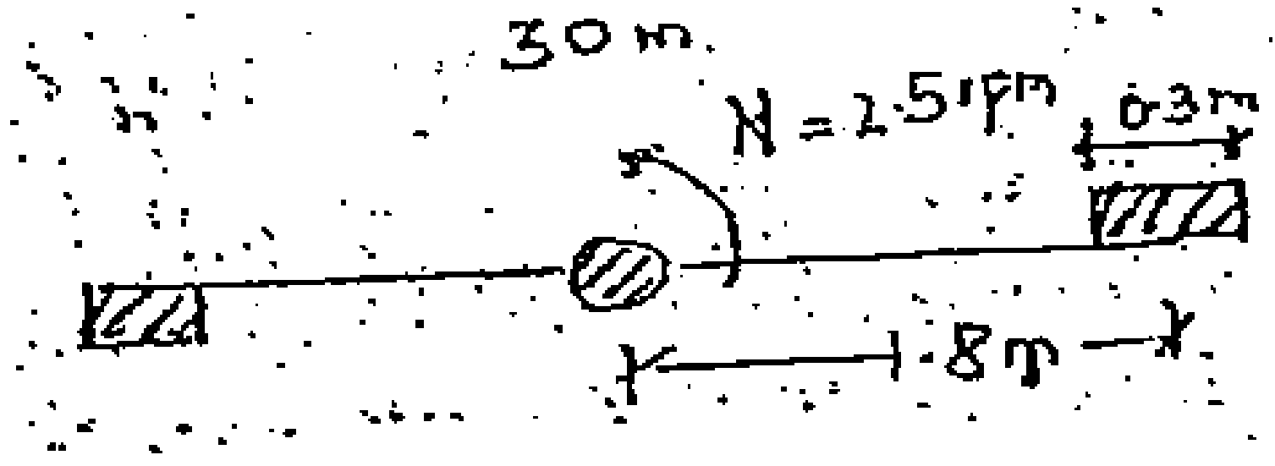
Data :-

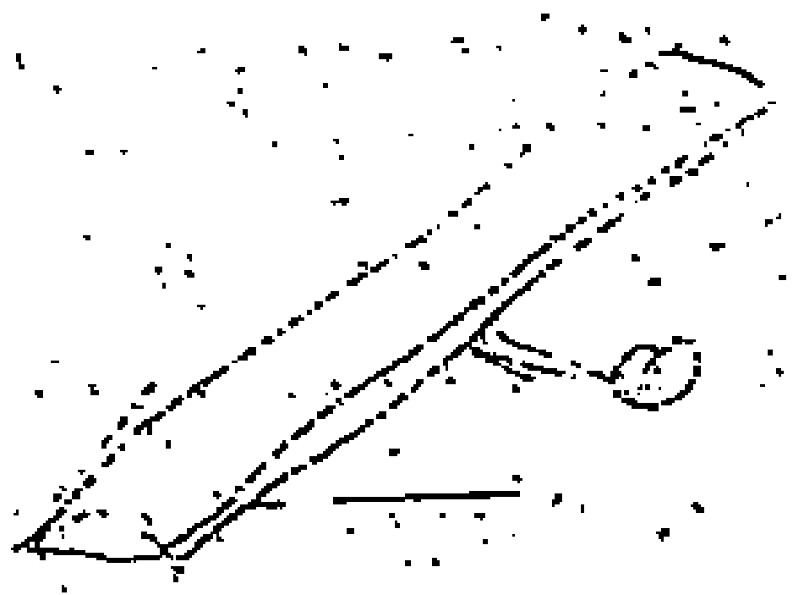
$Q = 75$ mld

$L \times B = 30 \times 12$ m

$b \times l = 0.3 \times 12$ m







$$V_r = V_p - V_w$$

$$V_w = V_p/4$$

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

$$V_p = \omega \cdot r$$

$$= \frac{2\pi N}{60} \times r$$

$$= \frac{2 \times \pi \times 2.5}{60} \times 1.8$$

$= 0.47 \text{ m/sec}$

$V_2 = \frac{3}{4} \times 0.47$

$= 0.35 \text{ m/sec}$

$F_D = \frac{1}{2} \times C_D \times \rho_w \times A_1 \times V_2^2$

$= \frac{1}{2} \times 1.8 \times 1000 \times (0.3 \times 12) \times 8 \times (0.35)^2$

"No. of blades

$= 2332.8 \text{ N} \quad (3175.2 \text{ N})$

$P = 2332.8 \times 0.35$

$= 699.84 \text{ W} \quad (1.11 \text{ kW})$

$V = Q \cdot t_D$

$t_D = \frac{\text{volume of tank}}{\text{discharge}}$

$= \frac{30 \times 12 \times 4.5 \times \cancel{24 \times 60}}{75 \times 10^6 \times 10^{-3}}$

$= 31.1 \text{ min}$

dynamic viscosity $= \nu \times \rho_w$

$\mu = \nu \times \rho_w$

$= 1.31 \times 10^{-6} \times 1000$

$\rho_w = 1000 \text{ kg/m}^3$

$= 1.31 \times 10^{-3}$

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

$= \sqrt{\frac{1.11 \times 10^3}{1310 \times (1620) \times 10^{-3}}}$

$= 22.8 \text{ sec}^{-1}$

5. Filtration:-

(i) Filtration is most often used as limiting step to remove the flocs and finer undedimented particles.

(ii) It also removes organic matter, micro organisms and the dissolved minerals from the water.

(iii) Filtration is carried out with the help of filters which are generally of two types, - Gravity filters and pressure filters.

Gravity filters:-

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

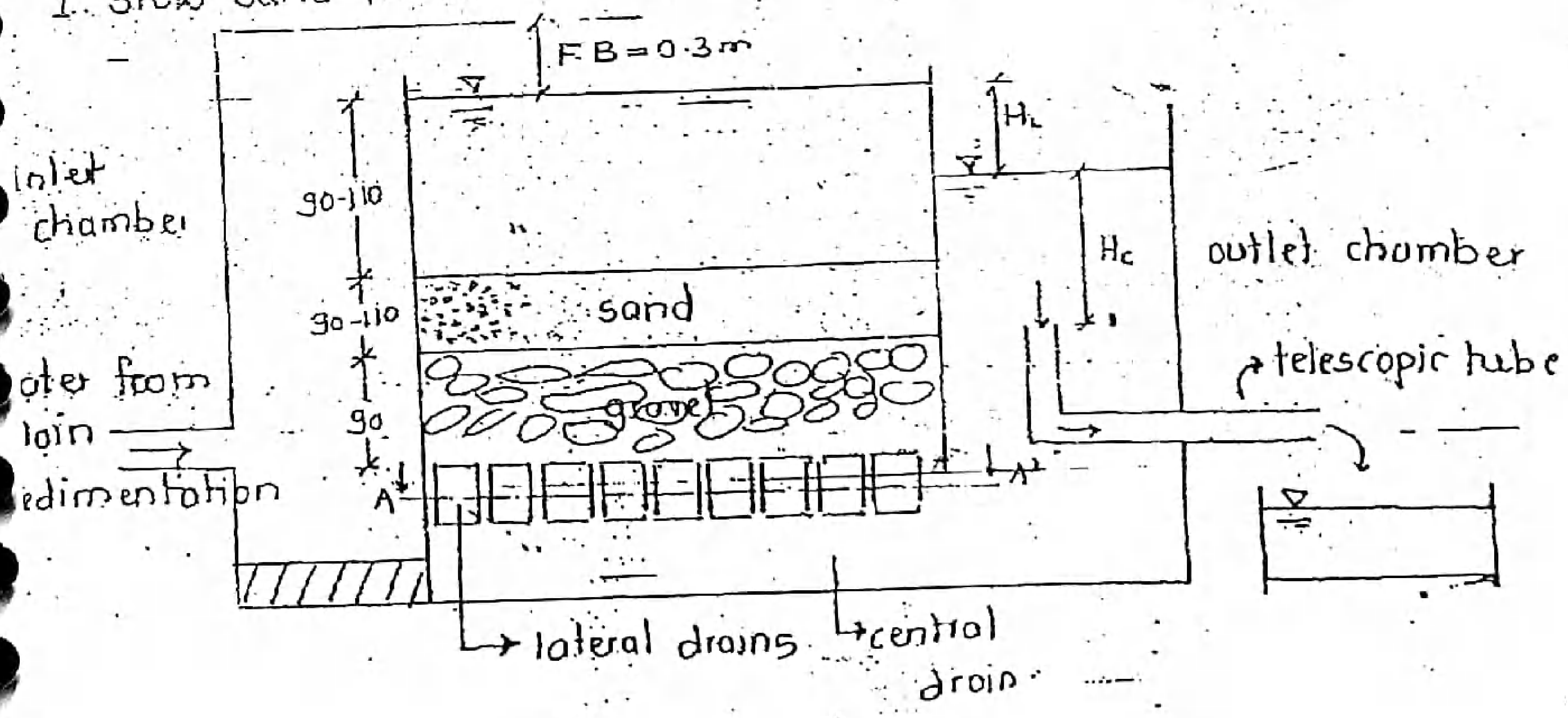
These filters are of two types -

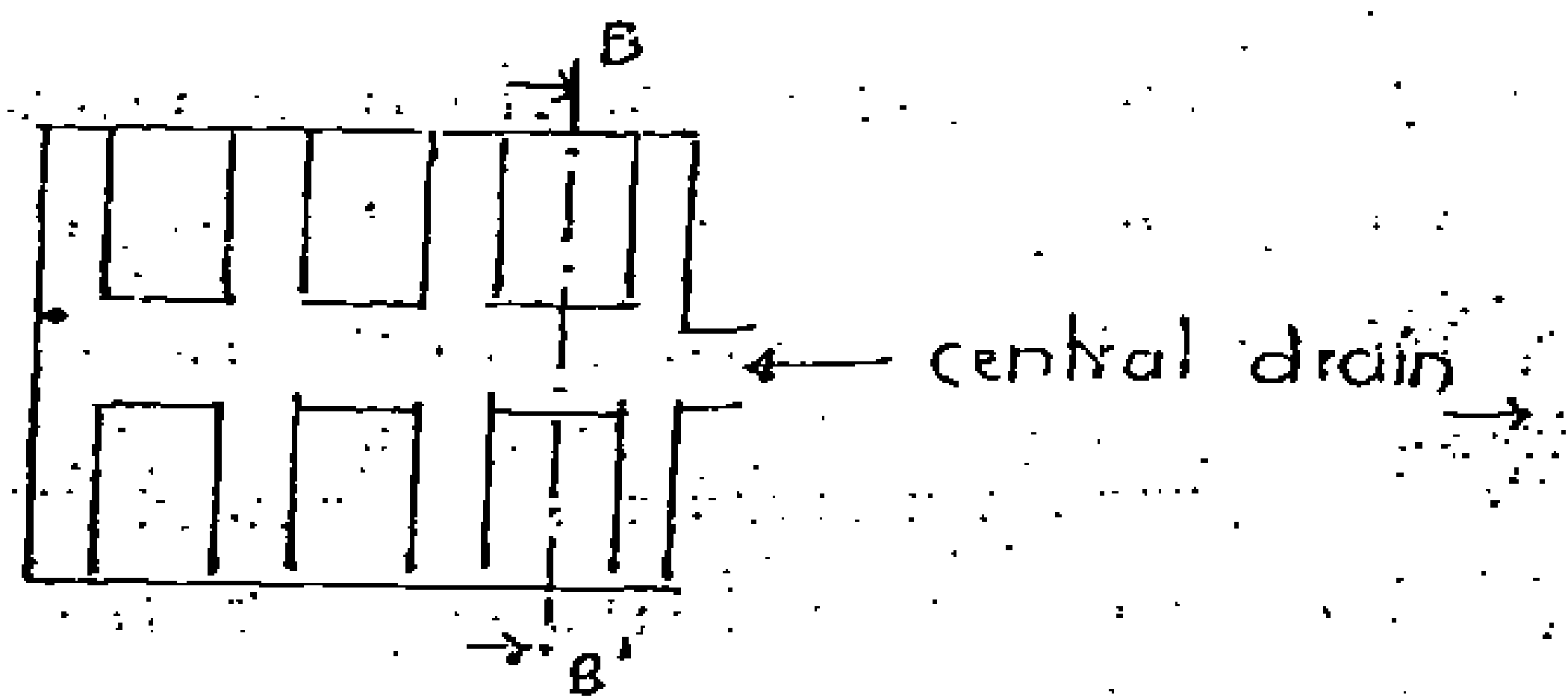
- Slow sand filters
- Rapid sand filters.

Pressure filters:-

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

I. Slow sand filters :-





section at A-A'

section at B-B'

Theory of filtration :-

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

(a) Mechanical straining :

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

(b) Sedimentation :

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

(c) Biological changes :

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

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

Over a period of time due to scarcity of organic matter, endogenous respiration starts [illegible]

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

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

(d) Electrolytic changes :

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

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

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

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

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

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

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

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

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

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

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

(x) Design life of filters is approximately 10-15 years.

(xi) In freshly cleaned filters, head loss of 10-15 cm is observed which goes on increasing with the operation of filter, hence telescopic tube is adjusted in accordance with head loss so as to obtaine constant discharge from the filter.

(xii) The ~~cleaning of the filter~~ should be done when head loss becomes 0.7 - 0.8 times depth of filter medium.

(xiii) During cleaning of filter top layer of the medium is scrabbed and 1.5 - 3 cm of sand layer is removed from the filter medium.

(xiv) The frequency of cleaning is 1-3 months.

(xv) Rate of filtration is 2400 - 4800 lit/m^2/day.

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

(xvi) After cleaning of filter it is washed with clarified water and again loaded with effluent from plain sedimentation but the filtered water is not used for next 24 to 36 hours.

(xvii) Slow sand filters are generally adopted for villages and small towns.

(xviii) The efficiency of this filter in removing micro organisms is 98 - 99 %.

(xix) The effluent of this filter are sufficiently clarified and free from nutrients like carbon, Nitrogen and Phosphorous which have reduced possibilities of aftergrowth of organisms in distribution system.

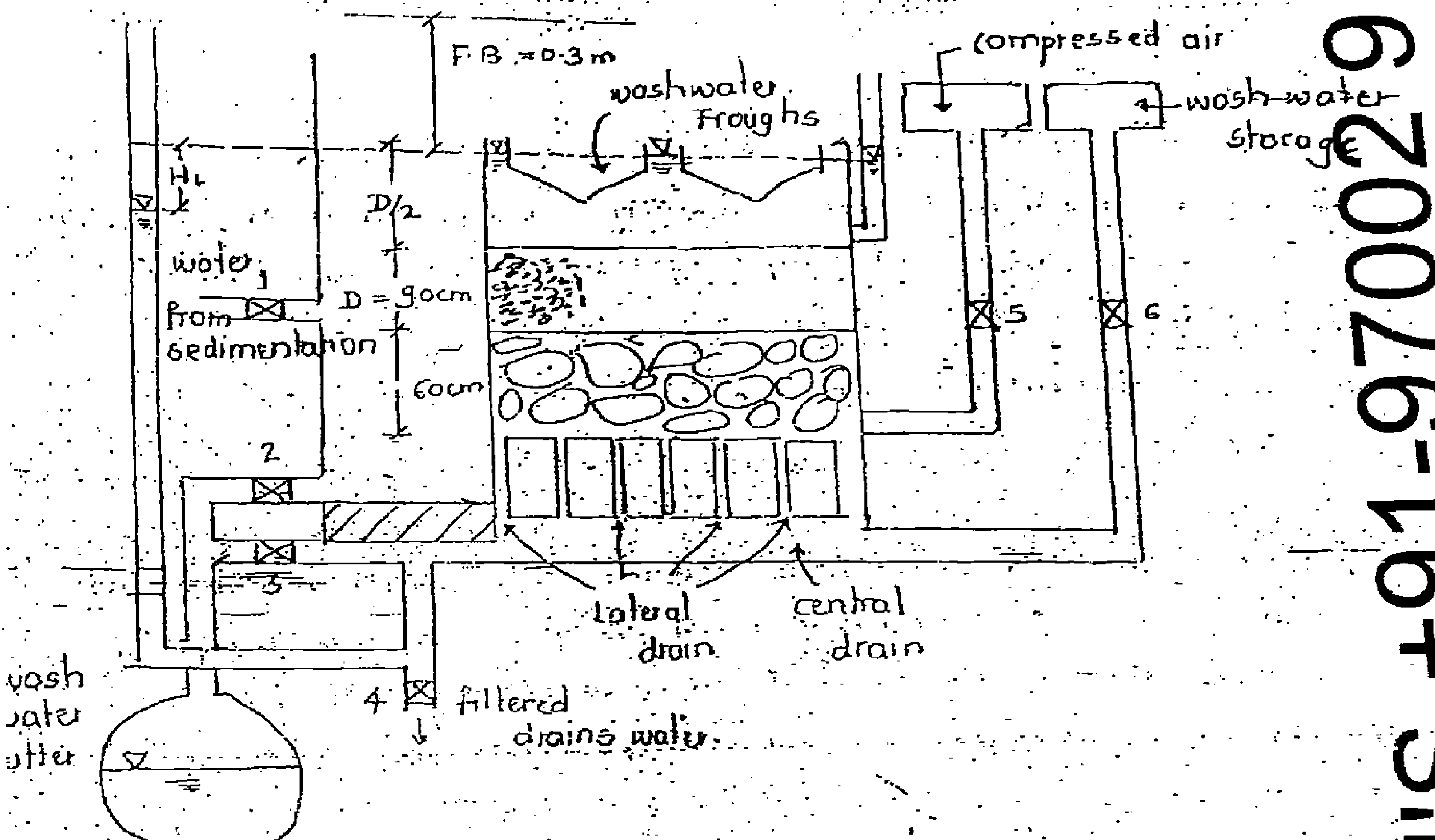
(xx) Slow sand filters are not used for water of turbidity greater than 50 NTU

(xxi) The number of filters required for treatment depends on the area of ~~treat~~ filter.

Area (m^2)	No. of filters
< 20	2 (1+1)
20 - 249	3 (2+1)
250 - 649	4 (3+1)
649 - 1200	5 (4+1)
> 1200	[illegible]

Monday
22nd Jul

2. Rapid sand filters :



(i) The operation of rapid sand filter is exactly same as that of slow sand filter. ~~During~~ During operation of filter valve no. 1 and 4 are open.

(ii) In case of RSF, the size of the particles of sand is more in comparison to that in SSF. Hence the size of the voids is comparatively more which results in deeper penetration of the impurities in filter medium. Hence in this case surface washing alone is not sufficient and is accompanied by backwashing.

(iii) In backwashing valve no. 5, 6 and 2 are opened and valve 4 are closed.

(iv) During this process, compressed air and pressurised water is passed through the medium and in backward direction (upward direction) resulting in the increase in porosity filter medium. Thereby increasing opportunity of entrapped impurities to get washed away with flowing pressurised water that is finally collected in wash water trough and disposed out of filter through valve no. 2 into washwater gutter.

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

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

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

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

(ix) Area required by this filter is 10-80 m².

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

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

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

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

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

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

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

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

(xv) The rate of filtration is 3000-6000 Lit/m²/hour.

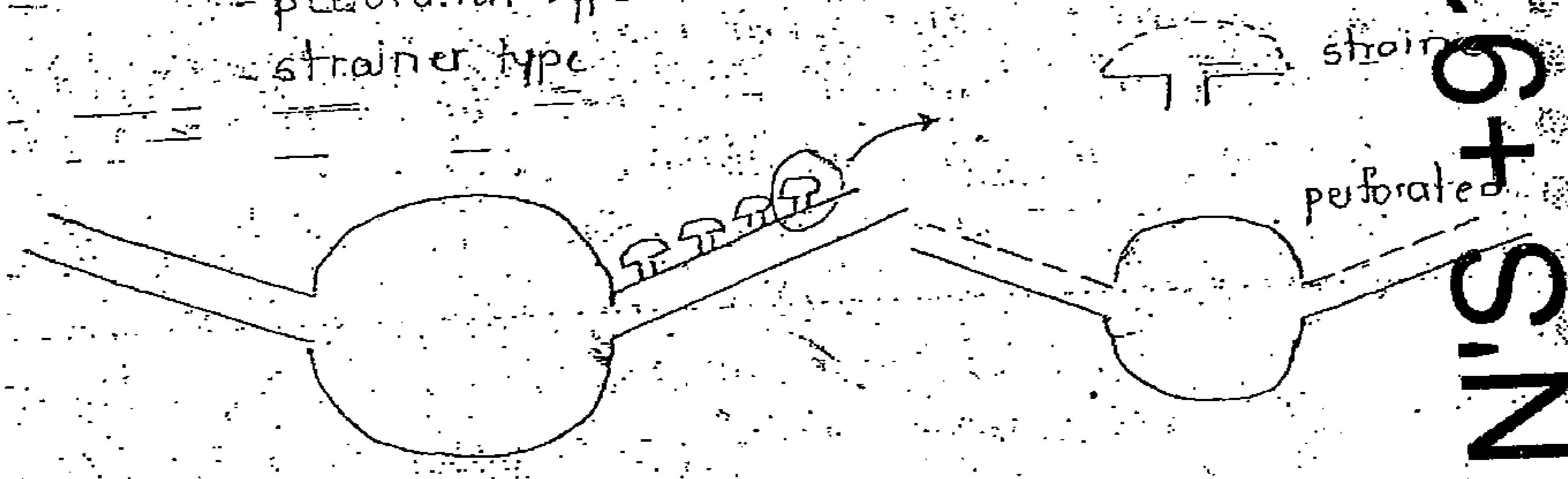
(xvi) During the filtration flow is assumed to be laminar and during backwashing it is in transition. It can be converted into turbulent if proper gradation of gravel is not provided.

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

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

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

- perforation type
- strainer type



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

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

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

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

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

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

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

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

(xxv) Factor 2 is used when size of perforation assumed is 130 mm and factor 4 is used when size of perforation is 6 mm.

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

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

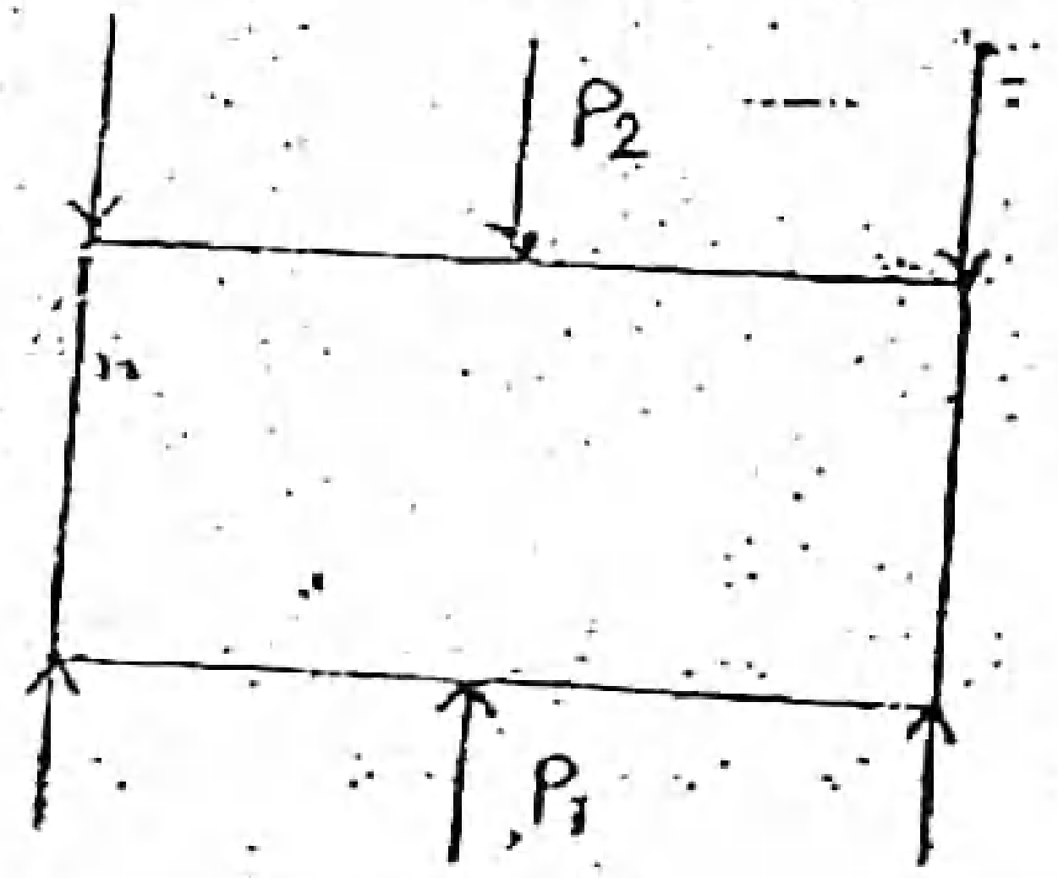
(xxvii) Length of each lateral to dia. of each lateral should not be more than 60.

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

(xxviii) The velocity of flow through central drain is limited to 1.8 - 2.4 m/s

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

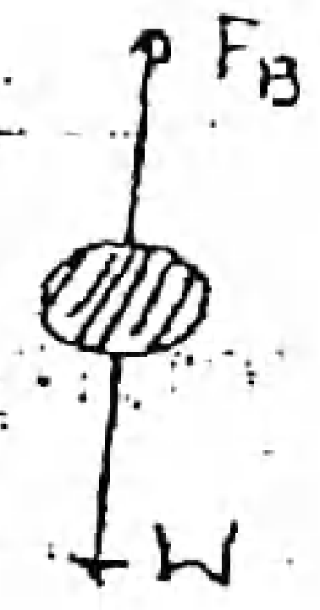
(xxx) To hydraulically expand the filter bed, head loss through the filter must be equal to the buyont weight of particles forming the filter medium.



$P_1 = \gamma_w h_1$

$P_2 = \gamma_w h_2$

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



Buyont wt. of particle $= W_B = W - F_B$

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

Buyont wt. of all the particles.

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

V_s - total volume of solids.

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

$$1-n = 1 - \frac{V_v}{V}$$
$$= \frac{V - V_v}{V}$$
$$1-n = \frac{V_s}{V}$$
$$V_s = V \cdot (1-n)$$
$$W_B = V(1-n) \cdot \gamma_w (G-1)$$
$$= (A \cdot D) \cdot (1-n) \cdot \gamma_w \cdot (G-1)$$

To convert pressure head loss into weight

$$(P_1 - P_2) A = W_B$$
$$\gamma_w \cdot H_L \cdot A = (A \cdot D) \cdot (1-n) \cdot \gamma_w \cdot (G-1)$$
$$\boxed{H_L = D(1-n)(G-1)}$$

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

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

As there is no change in the buyoant weight of particles

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

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

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

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

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

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

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

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

$C_D = 5$ to 6.

Q. A rapid sand filter proposed for a water supply treatment plant of town having population of 75,000. Avg. water supply in the town in 150 lpcd. Rate of filtration is 100 l/m²/min. Find the size and no. of filter bed required. Design the lateral and manifold under drainage system. Compute the washwater discharge required if rate of washing is 45 cm/min.

Data: population = 75,000

$R_f = 100$ l/m²/min

Avg. demand = 150 lpcd.

Design discharge $(Q_D) = 1.8 \times Q_{avg \cdot daily}$

$= 1.8 \times (75{,}000 \times 150 \times 10^{-3})$

$= 20250$ m³/day.

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

Thus effective discharge filtered by filter is.

$$Q_{eff} = \frac{20250}{23.5 \text{ hours}} \times 24$$

$= 20680.85$ m³/day

considering backwash quantity

$Q'_{eff} = 20680.85 \times 1.05$

$= 21714.89$ m³/day.

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

$= 150.79$ m²

No. of filters, $N = 1.22\sqrt{Q}$ Q in MLD

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

$= 5.6 \simeq 6$ units

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

Area of each filter $= \dfrac{150.79}{6}$

$= 25.13$ m² $\simeq 25$ sq.m

Assuming filter to be square

~~B = 5m~~

Assuming size of perforations = 6 mm

spacing between perforations = 7.5 cm c/c

laterals = 25 cm

c/s area of all the perforations = 0.2 % of filter area

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

$= 0.05$ m².

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

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

$= 42$

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

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

$d_l = 7.78$ cm

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

$$= 2 \times \left(\frac{0.05}{12}\right) \times 42 \times 10^4 \times 4$$

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

check :-

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

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

$$= 27.54 \not> 60 \quad \text{OK.}$$

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

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

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

Q. A rapid sand filter is to be provided in the water treatment plant for population of 2,75000. Water demand is 200 lpcd and rate of filtration is 15 m³/m²/hour. Allow 5% of the filtered water for storage to meet the backwash requirement. Each backwash period is 30 min. Determine the no. of filters required for allowing one standby. if available surface area is 10×4 m² for each filter. Also calculate upflow velocity and the head loss required to expand the bed to 0.66 m from the original depth of 0.60 m and porosity of 0.5. Drag coeff. for flow is 5.02. Kinematic viscosity of water is 0.10136×10^{-5} m²/sec. Backflow is in transition. Sp. gravity of particles forming filter medium is 2.5 and effective size of particles is 0.6 mm.

Data

population = 2,75,000

R_f = 15 m³/m²/hour.

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

$$\Rightarrow 106161.7 \text{ m}^3/\text{day}.$$

(considering backwash time and backwash discharge)

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

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

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

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

$$= 7.37 \simeq 8$$

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

Head loss $(h_L) = D(1-\eta)(G-1)$

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

$$= 45 \text{ cm}$$

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

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

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

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

$$\eta' = 0.545$$

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

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

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

$$V_B = (0.545)^{1/0.22} \times (0.048)$$

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

$$= 3.04 \text{ mm/sec.}$$

Operational troubles associated with RSF :

. Air binding :

(i) The initial head loss in the filter is 15-30 cm and this goes on increasing with the operation of filter as more and more impurities are entrapped in the voids of the filter medium.

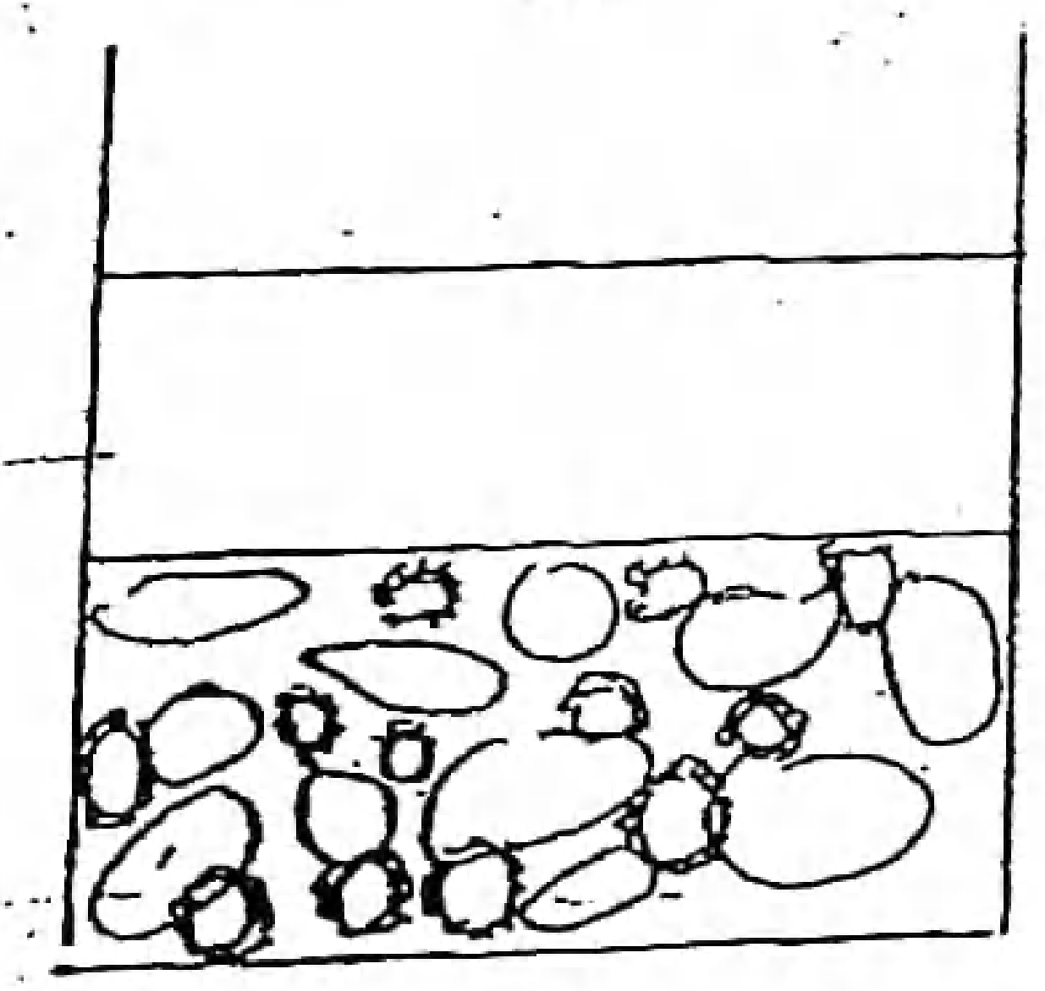
(ii) A stage comes when the frictional resistance offered by the medium particles exceeds static head of water. Due to which bottom layers of sand medium starts acting like a vaccum resulting in the release of dissolved gases present in the water.

(iii) The bubbles of these gases rises towards surface and gets stick to the medium particles thereby reducing effective area of flow through the medium. The process is known as air binding that seriously affects efficiency of filtration.

(iv) To avoid it cleaning of filter should be done when head loss becomes equal to 2.5-3.5 m and negative head equal to 1.2 m.

(v) Inverted ~~head~~ manometer is used for measurement of negative head.

. Mud ball formation :

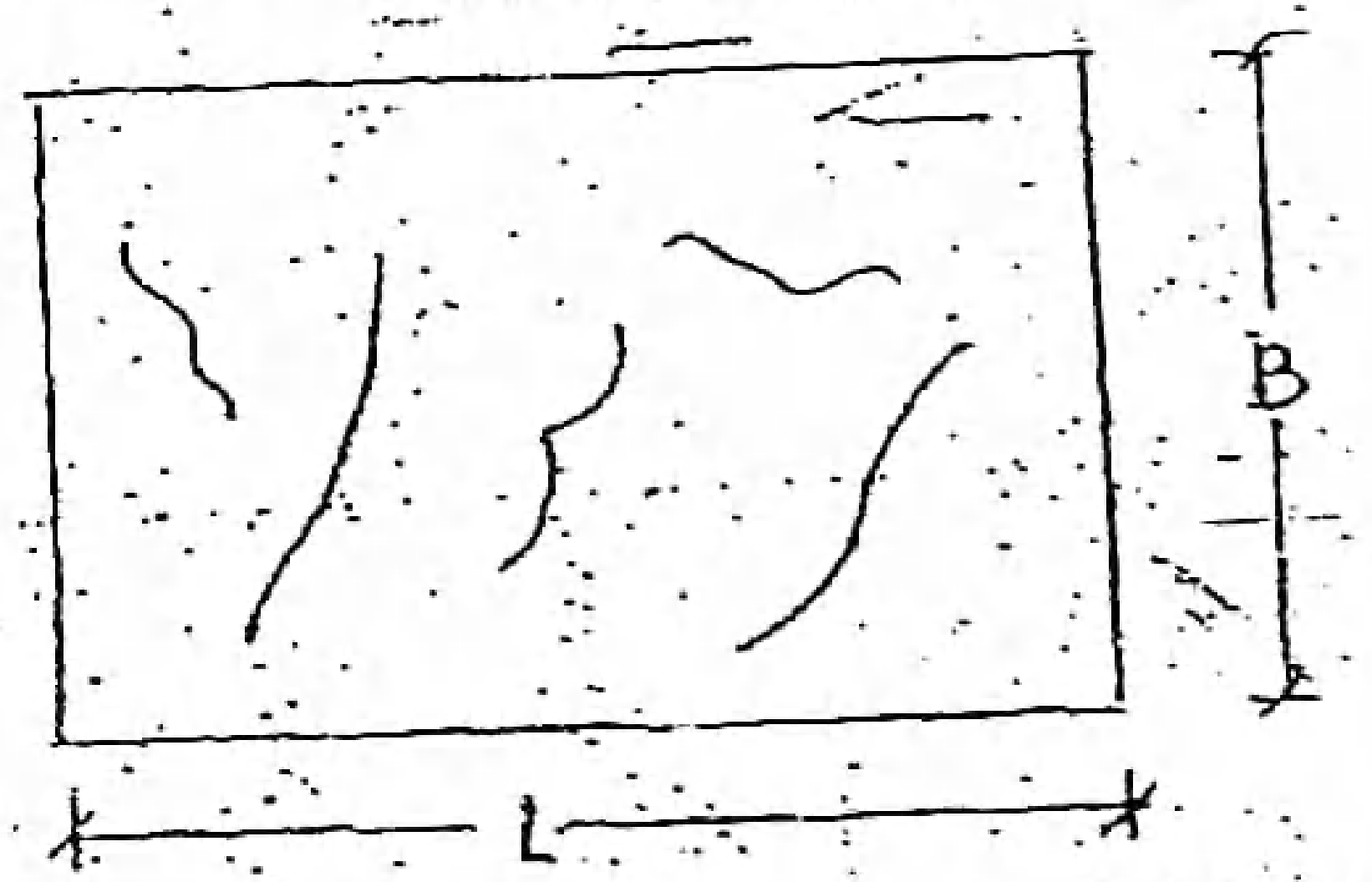


(i) Mud from atmosphere enters into filter medium and gets deposited over particles of medium. Over period of time due to inadequate washing it settles down in bottom layers of medium where it grows in size by combining with the impurities entrapped in the voids of medium, leading to the formation of mud balls.

(ii) Once these mud balls enter into gravel layer, it starts interfering with the distribution of backwash water, thereby seriously affecting efficiency of filter.

(iii) To avoid it adequate surface washing along with the use of Pottassium hydroxide (KOH) is adopted.

3. Cracking of filter :-



(i) Due to alternate wetting & drying of filter medium, the medium particles are subjected to shrinkage stresses that leads to the development of shrinkage crack over the surface.

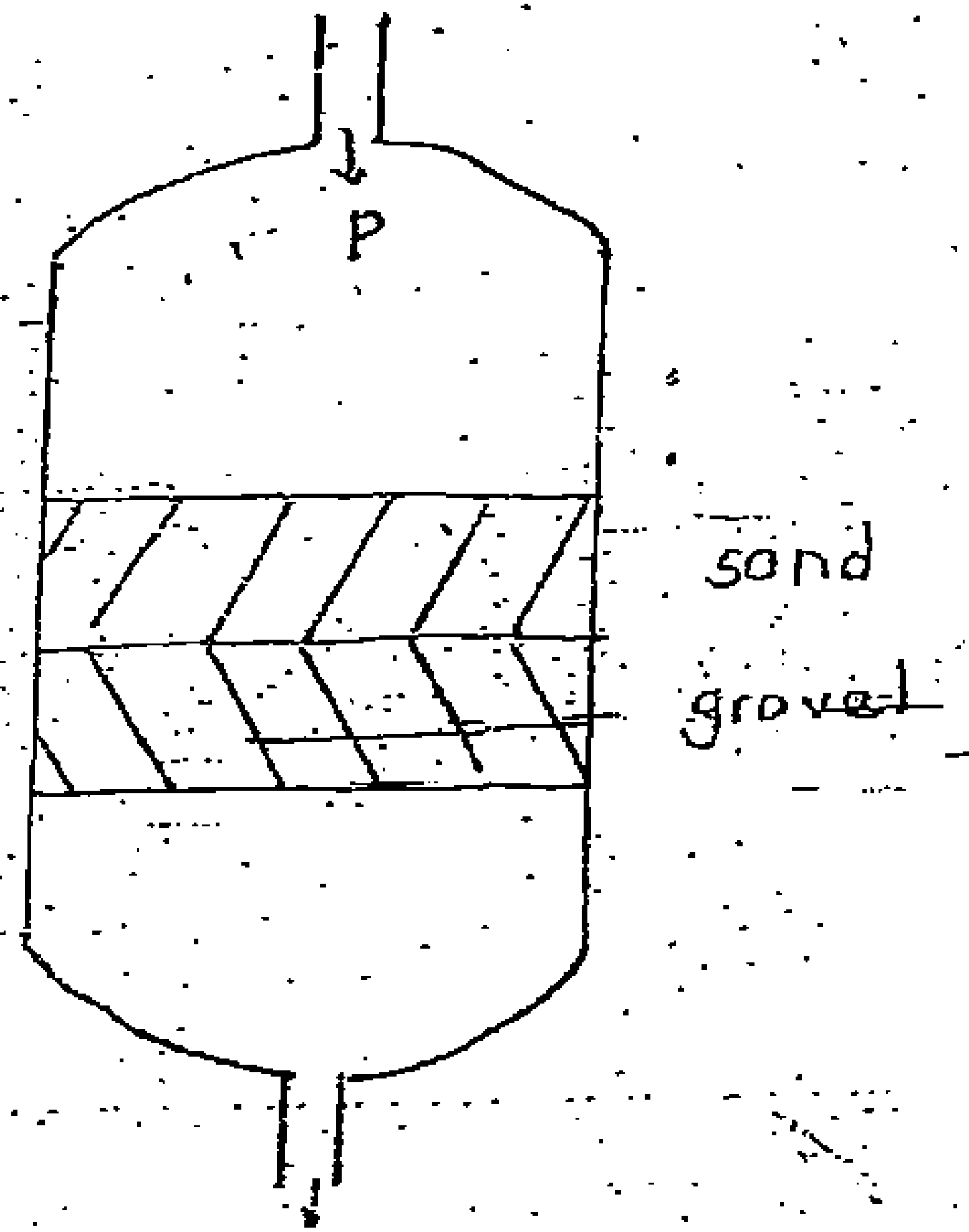
(ii) As the surface of medium is subjected to constant application of water pressure, cracks once developed increase in size resulting in deeper penetration of impurities in filter medium thereby reducing efficiency of filter.

Thursday
2-

Pressure filters :-

(i) Pressure filters the head required by water to pass through filter medium is provided by external application of pressure.

(ii) This unit is exactly similar to Rapid Sand Filter with the only difference that the entire process takes place in closed container.



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

(iv) The dia. of tank is in the range of 1·5 - 3 m. Depth of tank is 3-3·5 m. Rate of filtration is 6000 - 15000 lit/m²/hr, which

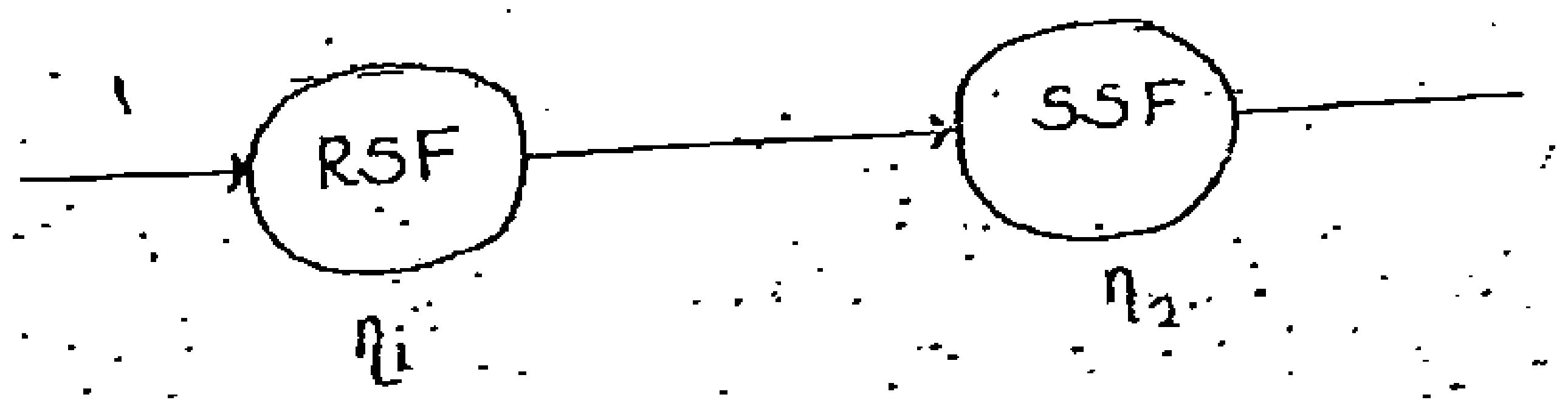
(v) is almost double of Rapid Sand filter.

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

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

Double filtration and Roughning filters :-

(i) In order to increase discharge through slow sand filters without compromising with quality of, effluent, RSF is used before slow sand filter. The process is known as Double filtration and RSF used is known as Roughning filters.



R_{F_2} - rate of filtration in SSF is more than conventional SSF. (size of sand is large)

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

6. Disinfection-:

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

Note :- The process of removal of all the micro organisms is called Sterlisation. The process of removal of only disease causing microbs is called disinfection.

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

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

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

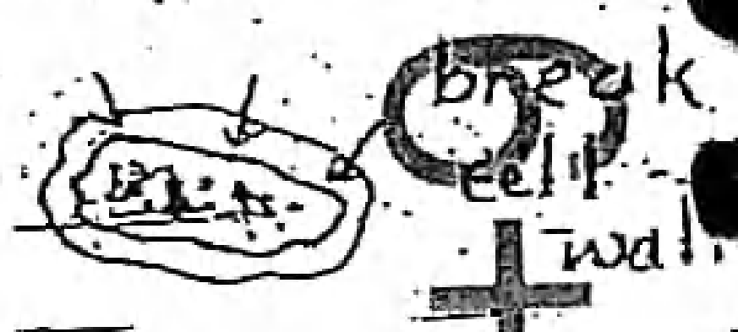
Physical disinfection	Chemical disinfection
- Boiling	- Acids and alkalies (pH < 3, pH > 11)
- UV rays	- Metallic ions (Ag^+, Hg^+)
	- Oxidising agents (Cl_2, Br_2, I_2, O_3 & $KMnO_4$)

Minor methods :-

1. Treatment with ozone (O_3)

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

$$O_3 \longrightarrow O_2 + \underset{\text{Nascent oxygen}}{O} \longrightarrow$$



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

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

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

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

Note :-

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

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

2. Treatment with Potassium permangenete ($KMnO_4$) :

(i) It is one of the most popular disinfectant used for the treatment of well water supplies which consists of lesser impurities.

(ii) Potassium permangenete also removes organic matter apart from micro-organism from water.

(iii) When $KMnO_4$ is added in the water it induces pink colour in it. If this pink colour disappears, it signifies the presence of the micro organisms and organic matter in water.

(iv) $KMnO_4$ is added further in the water upto an extent pink colour stands in it, signifying the complete removal of organic matter and micro organisms from it.

(v) The water is not used for next 48 hours till pink colour subsidize. (pshychological effect is induced ~~due~~ to colour)

(vi) Normal dose of ~~$KMnO_4$~~ is generally 1-2 mg/l having contact period of 4-6 hours.

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

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

Major method :-

Treatment with Chlorine (Cl_2) :

$$Cl_2 + H_2O \xrightleftharpoons{pH>5} \underset{\substack{\text{Highly unstable} \\ \text{Hypochlorous acid}}}{HOCl} + HCl$$

$$HOCl \xrightleftharpoons[pH<7]{pH>8} \underset{\substack{\text{Hypochlorite} \\ \text{ion}}}{OCl^-} + H^+$$

Role constants.

$$K_f = \frac{[OCl^-][H^+]}{[HOCl]} \qquad K_b = \frac{[HOCl]}{[OCl^-][H^+]}$$

(i) Chlorine reacts with water at pH of greater than 5 to form highly unstable Hypochlorous acid which further dissociates into Hypochlorite ion at pH of greater than 8 and remains in the water as Hypochlorous acid (HOCl) if pH is less than 7.

(ii) All the above these forms of chlorine (Molecular chlorine Cl_2, HOCl and OCl^-) known as freely available chlorine is responsible for the disinfection of water.

(iii) Out of these three forms of chlorine Hypochlorous acid (HOCl) is the most destructive reagent (80 times more effective than hypochloride ion).

(iv) While carrying out disinfection pH of water is kept slightly less than 7 as Hypochlorous acid will be predomident species present in water at this pH.

(v) Moreover chlorine reacts with ammonia immediately to form chloramines.

$$Cl_2 + NH_3 \xrightleftharpoons{pH > 7.5} NH_2Cl + HCl$$

Monochloramine

$$NH_2Cl + Cl_2 \xrightleftharpoons{pH\ 5-6.5} NHCl_2 + HCl$$

Dichloramine

$$NHCl_2 + Cl_2 \xrightleftharpoons{pH < 4.4} NCl_3 + HCl$$

Nitrogen tri-chloride

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

(vi) Dichloramine is predominant species present in water at pH of less than 7.

(vii) The chlorine dose added in the water should be such that residual of 0.2 mg/l after the contact period of 10 min. is left in the water.

(viii) Chlorine is assumed to inactivate the enzymes of the micro organisms required to carry out the metabolism.

Forms of Chlorine (Cl_2)

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

(i) Chlorine is generally added in liquid form to carry out the disinfection of water.

(ii) Chlorine is highly temperature susceptable. Chlorine freezes if temperature falls below 10°C, leading to choking of distribution pipe and burns the container in which it is stored above 92°C.

(iii) Optimum temperature range to carry out disinfection using chlorine is 32-48°C.

(iv) Chlorine does not form sludge while carrying out dis-infection nor its quality gets deteriorated with storage.

> Note:
> Chlorine makes explosive mixtures with Carbon monoxide (CO).

2. Hypochlorides and bleaching powder [$Ca(OCl)_2$, $CaOCl_2$].

(i) When Hypochlorites and bleaching powder are used as a disinfectant, hypochlorite ion and hypochlorous acid carries out disinfection.

(ii) For 100% pure Hypochlorite (Calcium hypochlorite) freely available chlorination ability is approximately 70%.

$$\underset{100\%}{Ca(OCl)_2} \longrightarrow \underset{70\%}{2OCl^-} + Ca^{++}$$

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

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

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

100% → 40%

127 gm → 51.5 gm

1 gm → 0.4 gm

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

5. Chloramines :-

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

4. Chlorine dioxide (ClO_2)

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

$NaClO_2 + Cl_2 \longrightarrow ClO_2 + NaCl$

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

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

It can be used for disinfection in water, if phenol is present.

stronger is the oxidising agent lesser is the stability.

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

Effectivity in disinfection:

ClO_2 > HOCl > Chloramines > Hypochlorite ion (OCl^-)

Types of chlorinations :-

- plain chlorination
- pre chlorination
- post chlorination
- double chlorination
- De-chlorination.

1. Plain Chlorination :-

When no other treatment except chlorination is given to the water, the treatment is known as plain chlorination. It removes organic matter, micro-organisms and colour from water.

It is generally adopted for water having turbidity less than 20-30 ppm. Normal dose is 0.5 mg/l.

2. Pre-chlorination :-

(i) Pre-chlorination is adopted when the conc. of micro-organisms is large in raw water.

(ii) In this method chlorine is added either at the time of co-agulation or filtration.

(iii) The normal dose of chlorine should be such that 0.1-0.5 mg/l is reached upto filtration. (If added in co-agulation, dose will be more)

(iv) Normal dose is 5-10 mg/l. (at co-agulation)

(v) Post-chlorination is ~~always~~ performed after pre-chlorination.

3. Post chlorination :-

(i) When chlorination is the last treatment given to the water, it is termed as post chlorination.

(ii) The normal dose of chlorine should be such that 0.2 mg/l of residual is left in the water after contact period of 10 min.

4. Double chlorination:

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

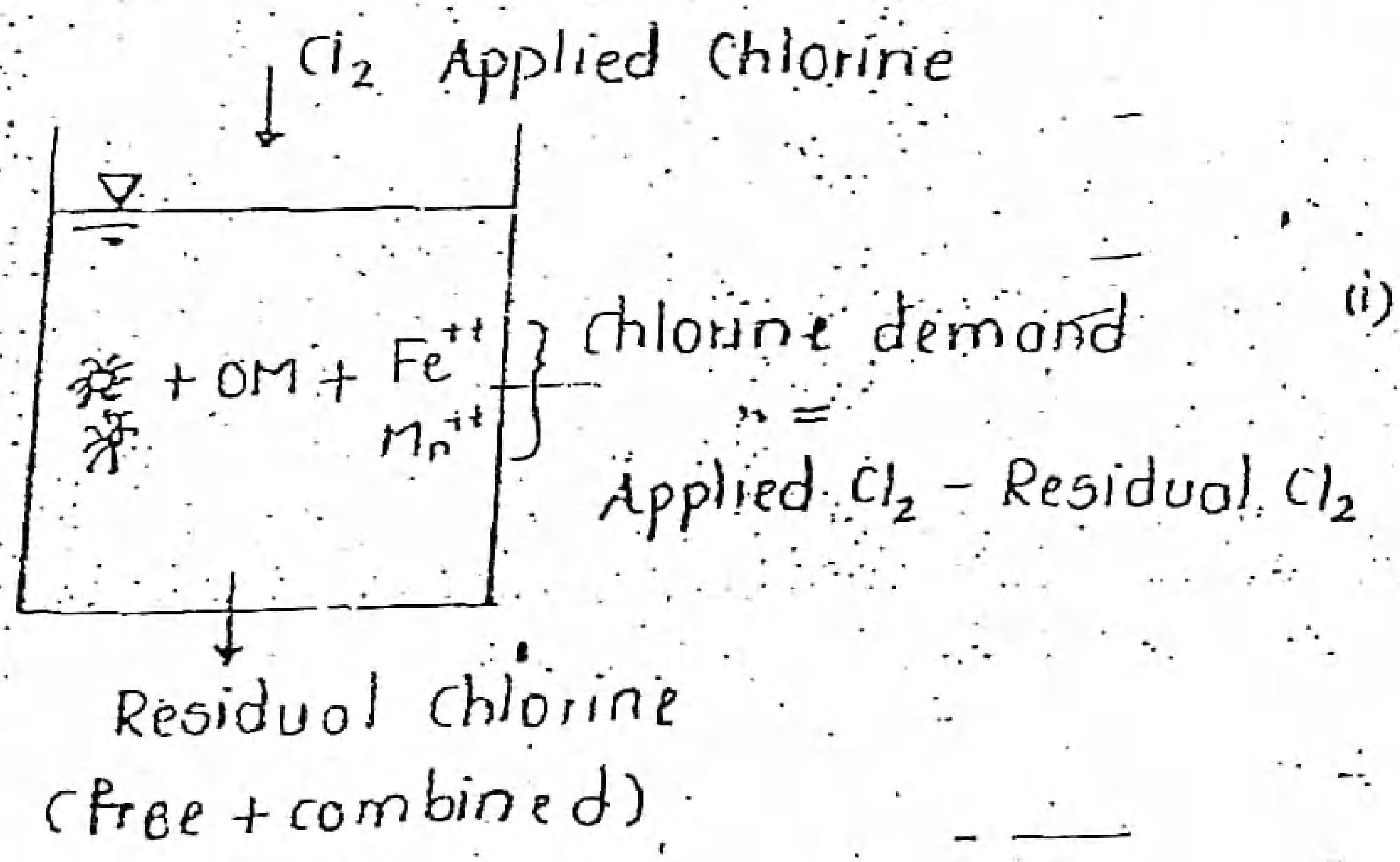
5. Super chlorination:

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

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

e.g. $Na_2S_2O_3$ (Sodium thiosulphate)
SO_2 (Sulphur dioxide)
Activated Carbon.
$Na_2S_2O_5$ (Sodium meta bisulphate)
Na_2HSO_3 (Sodium bisulphide)

6. Breakpoint chlorination:

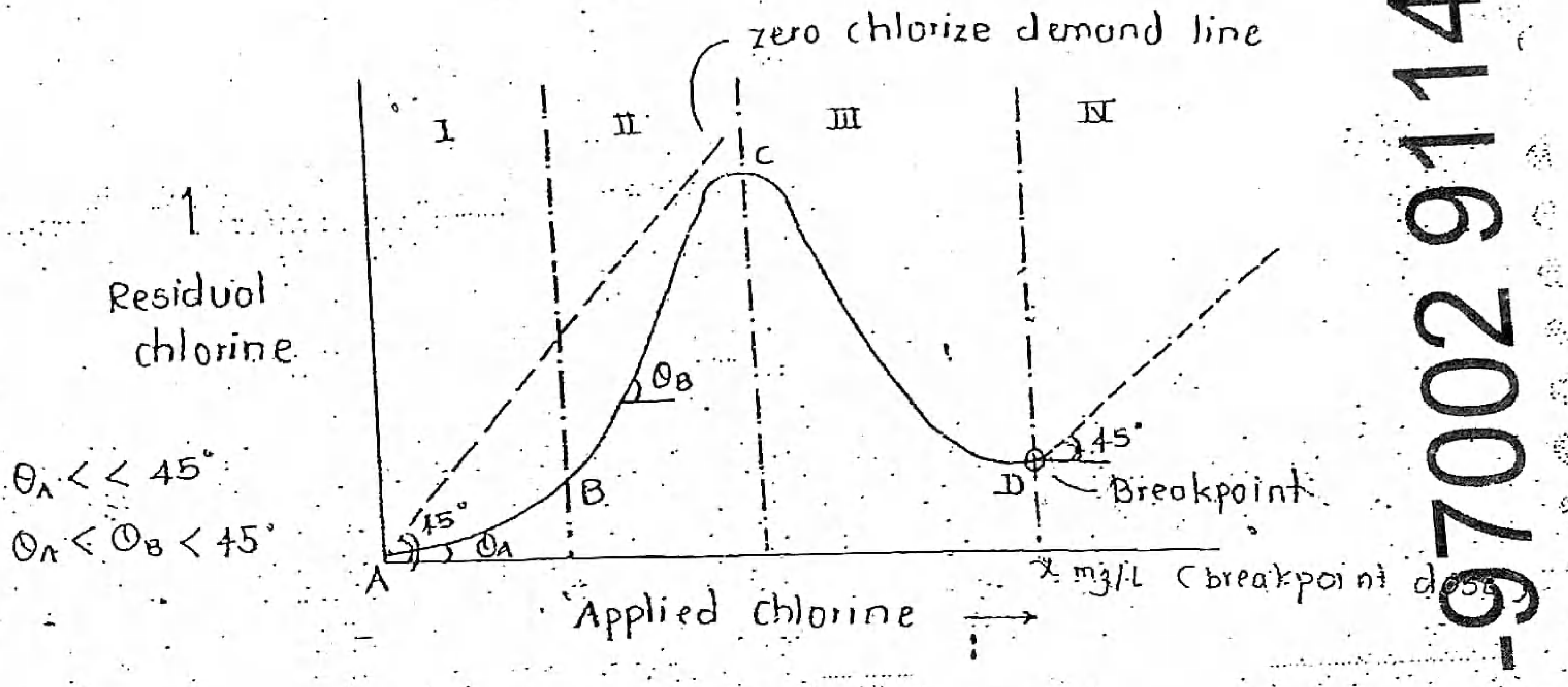


(i) Chlorine consumed for the removal of micro organisms, organic matter and Fe^{++}, Mn^{++}

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

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

$\theta_A << 45°$



(iii) During stage II: chlorine combines with ammonia & other compounds leading to the formation of chloramines & other combined forms of chlorine which appear as residual chlorine.

(iv) In stage II. as the demand for disinfection is satisfied, residual chlorine in water increases. but at no point slope is equal to 45° (as some amount of chlorine is still used for disinfection)

$\theta_A < \theta_B < 45°$

(v) At the begining of III stage (point C), bad smell starts coming out from water sample, indicating oxidation of the organic matter is started leading to increase in Chlorine demand of water and decrease in Residual chlorine of water.

(vi) In stage III, free chlorine breaks the chloramines and the chloro organic compounds into nitrogen compounds (because of increase in chlorine demand)

(vii) The end of the stage is characterized by removal of organi matter from water which is indicated by bad smell of water

(viii) After point D, whatever chlorine is added in the water bre free from the water and appears as residual chlorine, Hen point D is known as Breakpoint & chlorine dose reqd. is known as "breakpoint dose"

(ix) Theoretically, no chlorine should be added in water after point D. but practically 0·2 mg/l of residual is ensured after point D at 10 min of contact period.

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

Q. For the following information, find the breakpoint dose and the chlorine demand at dose of 1·2 mg/l.

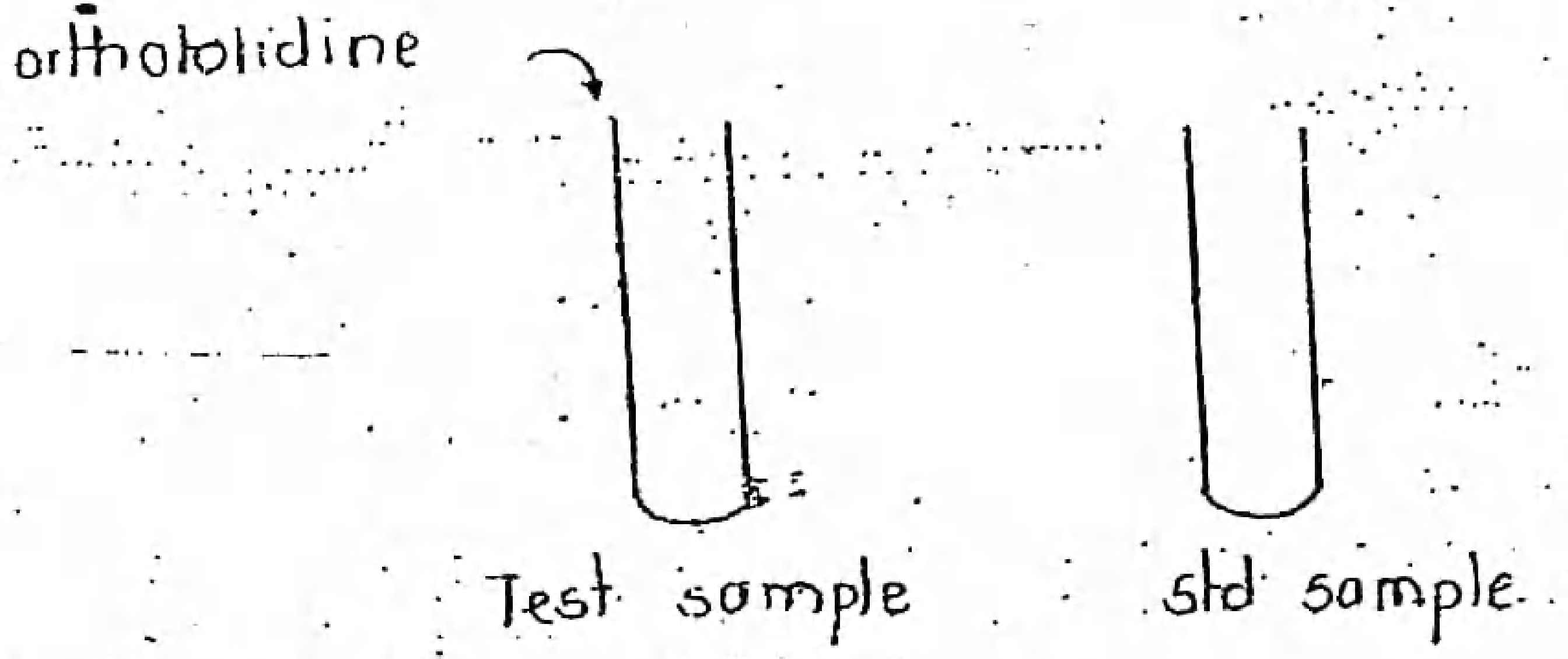
Applied Cl_2 (mg/l)		Residual Cl_2 (mg/l)	
0·2	0·2	0·19	0·17
0·4	0·2	0·36	0·14
0·6	0·2	0·50	-0·02
0·8	0·2	0·48	-0·28
1·0	0·2	0·20	-0·26
1·2	0·2	0·40	-0·20
1·4	0·2	0·60	-0·20
1·6		0·80	

Breakpoint dose is 1 mg/l of Cl_2

chlorine demand at 1·2 mg/l = 1·2 − 0·4
= 0·8 mg/l

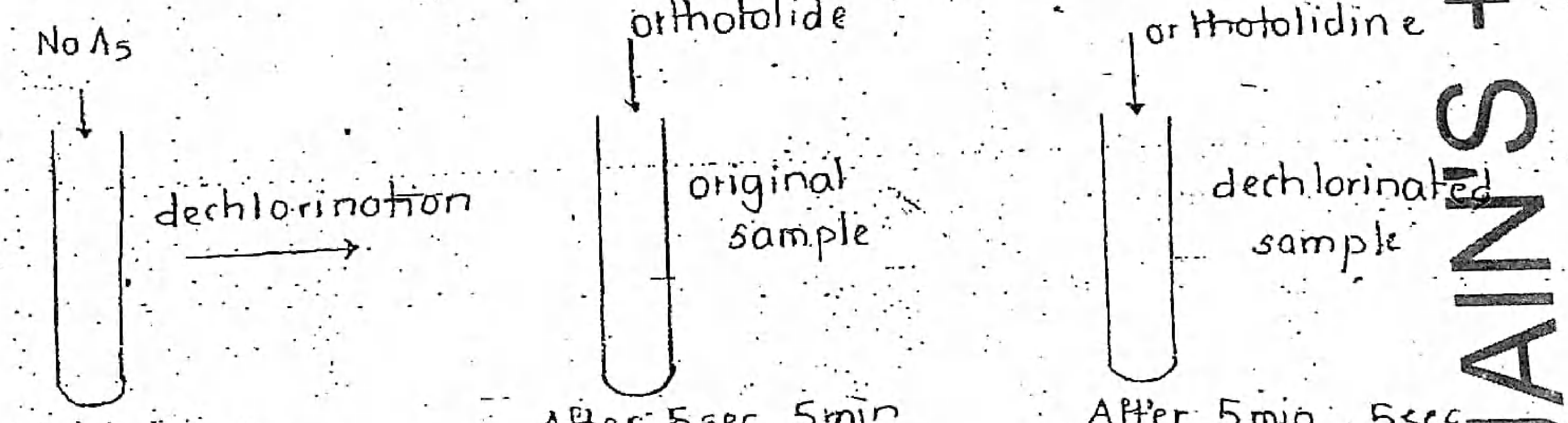
Test for chlorine residuals :-

1. Orthotolidine test :-



Residual chlorine $(R)_1$ = Free chlorine (after 5 sec)
$(R)_2$ = Free chlorine + combined chlorine (after 5 min)

combined chlorine = $R_2 - R_1$



Free chlorine +
combined chlorine
+
Fe, Mn + NO_2^-

After 5 sec, 5 min
R_1 = Free Cl_2 + Fe/Mn + NO_2^-
R_2 = Free Cl_2 + Fe/Mn + NO_2^- + combined Cl_2

After 5 min, 5 sec
$R_1 = Fe^{++}, Mn^{++} + NO_2^-$

(i) In this test 10 ml of chlorinated water sample is mixed with 0·1 ml of orthotolidine solution which results in development of yellow colour. This colour is compared with the colour of the standard solution. If the colour of std. solution is same as that of test sample, residual chlorine conc. in test sample, is same as that of std. sample.

(i) The conc. of free and combined chlorine in residual form is to be calculated seperately. then colour is noted after 5 sec and 5 min successively.

(ii) If water sample contains colour causing compounds like Fe^{++}, Mn^{++} and NO_2^-, false colour is induced in the water sample during this test, resulting in the increased conc. of residual chlorine than actually present.

(iii) Hence if colour causing compounds are present in water Arsenate-orthotolide test is performed in which Sodium Arsenate is added in original water sample, leading to the dechlorination of water sample. Orthotolidine is added to this dechlorinated sample and colour is noted.

(v) Orthotolidine is also added to the original chlorinated water sample and colour is noted after 5 sec and 5 min successively.

2. Starch-Iodide test:

1 L of water sample + 10 ml KI + 5 ml starch ⟶ blue coloured complex

$$\text{blue coloured complex} \xrightarrow[\text{0.01 N } Na_2S_2O_3 \text{ or } Na_2S_2O_3]{\text{titrate with}} \text{vol}^m \text{ of complex i.e. gm. eqt. of } Cl_2$$

to decolourise.

Residual chlorine = 0.3546 × volume of 0.01 N $Na_2S_2O_3$ used to decolourise blue solution.

(i) This test is generally used when ~~water~~ sample consist of colour causing compounds making orthotolidine test difficult.

(ii) Precise results are obtained from this test if residual chlorine conc. is more than 1 mg/l.

3. DPD and chlorotex test:

(DPD - di-ethyl phylene di-amine)

These reagents are developed by British drug house (BDH). This test is based on colour matching technique and both these reagents are used to produce colour.

Factors affecting efficiency of chlorination:

1. Turbidity:

More is the turbidity less will be the efficiency of chlorination. Because SS partially shield the micro organisms and consumes more amount of chlorine.

2. Fe^{++}/Mn^{++}:

If colour causing compounds (Fe^{++}/Mn^{++}) are present in water, chlorine oxidises them thus diverting from its main objective of dis-infection and thus reducing efficiency.

3. Ammonia

chlorine reacts with amine to produce chloramines which are less reactive agents than free chlorine. Thus the efficiency of dis-infection is brought down.

4. pH:

Chlorination is pH dependant process as Hypochlorous acid and Hypochlorite ion (which are responsible for carryi disinfection) are formed in particular pH range. The pH of 5-7 is maintained for effective disinfection of water supp

5. Temperature:

As the temperature decreases, efficiency of chlorin reduces because of reduction in reactivity of chlorine in w Moreover 28-32°C is effective range of temperature for chlorination.

Friday
26th July

7. Softening :-

(i) Softening is done to remove the hardness from wate both temporary and permanent:

Removal of temporary hardness

(i) $$Ca(HCO_3)_2 \xrightarrow{\Delta} CaCO_3 \downarrow$$

or

$$Ca(HCO_3)_2 + Ca(OH)_2 \xrightarrow{\Delta} 2\,CaCO_3 \downarrow$$

(ii) $$Mg(HCO_3)_2 \xrightarrow{\Delta} MgCO_3 \downarrow$$

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

(ii) $$Mg(HCO_3)_2 + Ca(OH)_2 \longrightarrow Mg(OH)_2 \downarrow + Ca(HCO_3) \downarrow$$

$$Ca(HCO_3)_2 \xrightarrow{\Delta} CaCO_3 \downarrow$$

Removal of permonent hardness:

1. Lime soda process:

(i) Lime removes all the carbonate hardness present in the water (α)

$$Ca(HCO_3)_2 + Ca(OH)_2 \longrightarrow 2\,CaCO_3 \downarrow$$

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

$$Mg(HCO_3)_2 + 2\,Ca(OH)_2 \longrightarrow Mg(OH)_2 \downarrow + 2\,CaCO_3 \downarrow$$

(ii) Lime converts non-carbonate hardness of magnesium into the non-carbonate hardness of Calcium. (β)

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

(iii) Lime also reacts with CO_2 in water to form ppt of $CaCO_3$ (γ).

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

(iv) Soda ash removes all the Non-carbonate hardness of Ca (originally present and also converted from non-carbonate hardness of Mg by addition of lime) (δ)

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

(v) Precipitate of $CaCO_3$ is formed at pH of 9-9.5 and the ppt of $Mg(OH)_2$ is formed at pH of 11 or more.

(vi) The pH of the water, if is not in the above mentioned range, it is increased artificially by addition of lime or soda ash. (This will not remove any hardness

(vii) Availability of lime is reduced in water if CO_2 is present in it. Hence if the conc. of CO_2 is more than 10 mg/l, it is to be first removed by the aeration process before performing lime-soda process.

(viii) Under normal conditions in the treatment of water complete removal of hardness is not possible by the use of chemical reagents.

(ix) Normally 40 mg/l of $CaCO_3$ and 10 mg/l of $Mg(OH)_2$ is left in softened water and not removed completely by any of the settling process. Thereby leading to the incrustation of pipes. Hence these ppt are again made soluble in water by the process known as Re-carbonation.

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

$$Mg(OH)_2 \downarrow + CO_2 \xrightarrow{H_2O} MgCO_3$$

(x) Recarbonation generally occurs at pH of 9.5. Hence pH of the water is to be reduced by the addition of acids before adding CO_2 by recarbonation.

Lime + soda at coagulation flocculation

Raw water → Screen → Aeration → Sedimentation → filter → disinfecti

↑ Recarbonation.

(xi) Lime-soda process is generally adopted in conjugation with coagulation-flocculation as overall quality of effluent is much better and load over filter is reduced. Again efficiency of coagulati flocculation will increase.

(xii) Lime soda process helps in reducing corrosion of pipes by increasing alkalinity of water.

(xiii) This process also helps in removing micro-organisms in water (increase in pH upto 11 will not be suitable for suitable survival of microbs)

(xiv)

Note :-

(i) 1 mg of Ca-Hardness removed as $CaCO_3$ leads to the formation of 1 mg of sludge of $CaCO_3$ (as $CaCO_3$)

(ii) 1 mg of Mg-Hardness removed as $CaCO_3$ leads to the formation of 0.58 mg of sludge (of $Mg(OH)_2$) as $CaCO_3$.

(iii) 1 mg of lime added in the water leads to the formation of 1 mg of sludge as $CaCO_3$.

$$Ca^{++} \longrightarrow CaCO_3 \downarrow$$

1 mg of $CaCO_3$ hardness due to Ca^{++} → 1 mg of $CaCO_3$ sludge

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

1 mg of $CaCO_3$ hardness due to Mg^{++} → 0.58 mg of $CaCO_3$ sludge.

$$\text{Dry sludge as } CaCO_3 \text{ in mg/l} = \left[\begin{matrix} Ca^{++}\text{ hardness} \\ \text{removed} \end{matrix} + \begin{matrix} 0.58\, Mg^{++}\text{ hardness} \\ \text{removed} \end{matrix} + \begin{matrix} \text{lime} \\ \text{added} \end{matrix} \right]$$

(xv) If lime-soda process is clubed with coagulation-flocculation then additional solids formed due to formation of ppt of coagulant and SS removed are considered also.

(xvi) Lime soda process reduces the hardness of water upto 50 mg/l i.e. it does not produce zero hardness. (This remained hardness is induced by us only)

A raw water contains following constituents.

i) $CaCO_3$ alkalinity = 200 ppm
ii) $Mg(HCO_3)_2$ = 120 ppm
iii) $CaSO_4$ = 100 ppm
iv) Fe_2O_3 = 40 ppm
v) $MgCl_2$ = 150 ppm
vi) $MgSO_4$ = 100 ppm
vii) NaCl = 25 ppm
viii) SiO_2 = 30 ppm

Calculate the annual requirement of lime and soda required for treating 80,000 litre of water per day, in ~~one~~ year.

	Lime required	Soda ash required
$CaCO_3$ alkalinity	1 mole	
$Mg(HCO_3)_2$	2 moles	
$CaSO_4$		1 mole
Fe_2O_3	-	-
$MgCl_2$	1 mole	1 mole
$MgSO_4$	1 mole	1 mole
NaCl	-	-
SiO_2	-	-
	4 moles	3 moles

i) Annual requirement of lime for removal of $CaCO_3$ alkalinity = moles of $CaCO_3$ × mol. wt of lime

ii) Annual requirement of lime for removal of $Mg(HCO_3)_2$ = moles of $Mg(HCO_3)_2$ × mol. wt. of ~~$Mg(HCO_3)_2$~~ lime

Similarly,

Total wt. of lime required annualy

$$= \left\{\left(\frac{200}{100}\right)\cancel{\times 74 +} \left(\frac{120}{146}\right)\times 2 \times 74 + \left(\frac{150}{95}\right)\times 74 + \left(\frac{100}{120}\right)\times 74\right\}$$

$$\times 80{,}000 \times 365 \times \underbrace{10^{-6}}_{kg} \times \underbrace{10^{-3}}_{tonnes}$$

$$= 13.08 \text{ tonnes}$$

Total wt. of soda ash required annualy.

$$= \left\{\left(\frac{100}{136}\right)\times 106 + \left(\frac{150}{95}\right)\times 106 + \left(\frac{100}{120}\right)\times 106\right\} \times 80{,}000 \times 365 \times 10^{-6} \times 10^{-3}$$

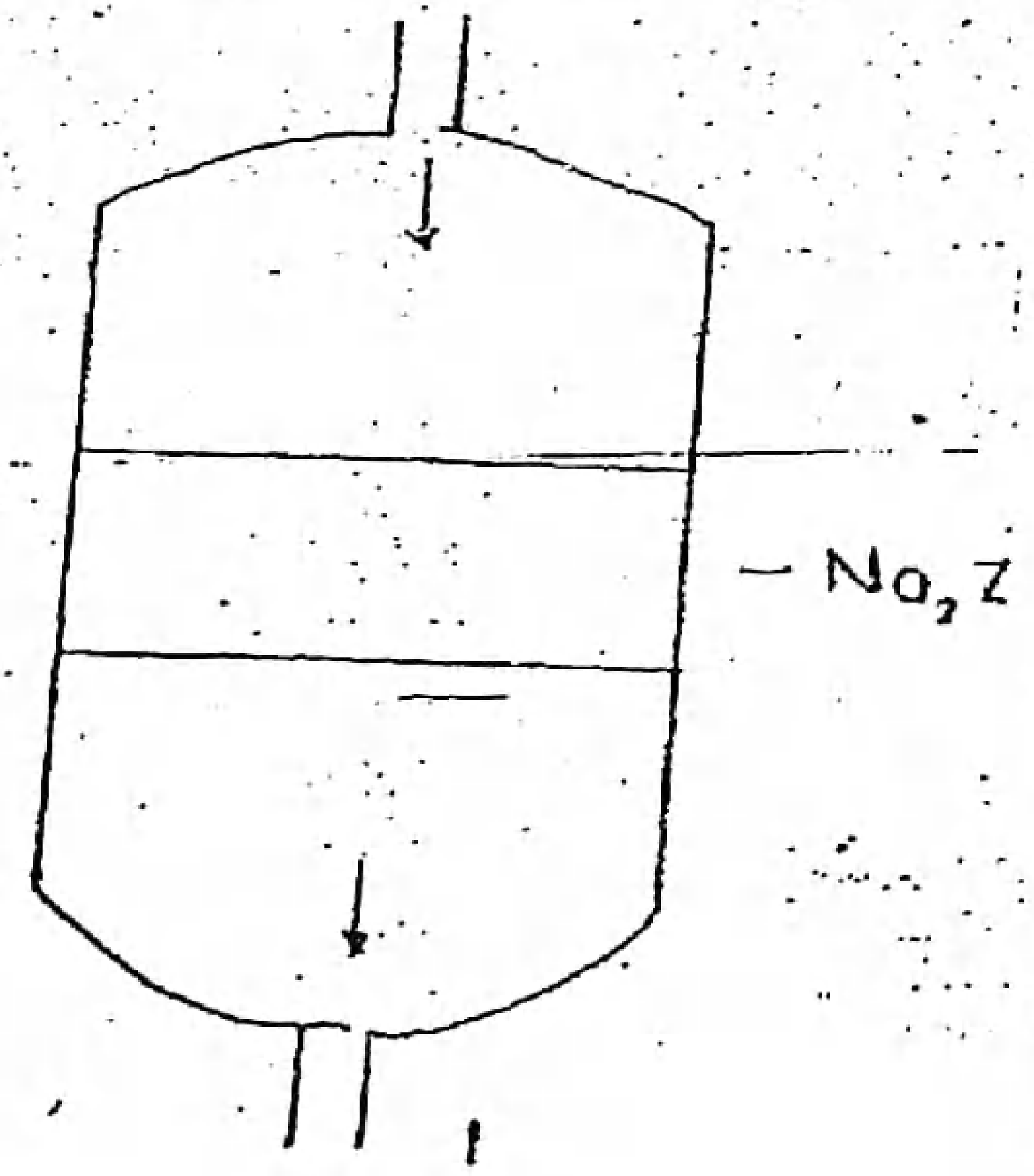
$$=$$

2. Zeolite process (Base exchange process):

(i) The equipment used in zeolite process is in the form of pressure filter.

(ii) Zeolites are natural or synthetic, cations or base exchange hydrated silicate of sodium and aluminium.

e.g. $Na_2O \cdot (Al_2O_3, x \cdot SiO_2, y \cdot H_2O)$ i.e. Na_2Z



$$Ca/Mg\left\{\begin{matrix} CO_3^{--} \\ MgHCO_3^{-} \\ Cl^{-} \\ SO_4^{--} \\ NO_3^{-} \end{matrix}\right. + Na_2Z \longrightarrow \underset{\text{Exhausted bed}}{Ca/Mg \cdot Z} + Na\left\{\begin{matrix} CO_3^{--} \\ MgHCO_3^{-} \\ Cl^{-} \\ SO_4^{--} \\ NO_3^{-} \end{matrix}\right.$$

$$Ca/Mg \cdot Z + \underset{\text{5-10\% brine solution}}{NaCl} \longrightarrow \underset{\text{Regenerated bed}}{Na_2Z} + Ca/Mg \cdot Cl$$

(iii) In this method hard water is passed over zeolite bed which removes all the hardness causing cations present in it and gets itself converted into exhausted bed.

(iv) This exhausted bed is regenerated before its further use by addition of 5-10% brine solution in the filter.

(v) This method does not produces sludge hence incrustation of pipes is avoided by use of this method.

(vi) This method is capable of producing zero hardness water.

(vii) This method proves to be costlier if iron and mangenese are present in water as Fe/Mg zeolite can not be regenerated back to sodium zeolite.

3. De-mineralisation

(i) In this method minerals are removed from water and the complete removal is achieved by first passing the water throug cation exchange resins and then through anion exchange resin

(ii) In the first stage water is passed through cation exchange resins which removes minerals from water but acids are formed in this process.

$$Ca/Mg\left\{\begin{matrix} CO_3^{--} \\ HCO_3^{-} \\ Cl^{-} \\ SO_4^{--} \\ NO_3^{-} \end{matrix}\right. + H_2R \longrightarrow Ca/MgR + \left\{\begin{matrix} H_2CO_3 \\ HCl \\ H_2SO_4 \\ HNO_3 \end{matrix}\right.$$

Acids

(iii) These acids are removed from water by passing it through anion exchange resins.

$$\left.\begin{array}{l} H_2CO_3 \\ HCl \\ H_2SO_4 \\ HNO_3 \end{array}\right\} + ROH \longrightarrow R\left\{\begin{array}{l} CO_3^{--} \\ Cl^- \\ SO_4^{--} \\ NO_3^- \end{array}\right. + H_2O$$

Exhausted bed.

(iv) Cation exchange resins can be regenerated by passing acids through it.

$$Ca/Mg\,R + \begin{array}{l} HCl \\ H_2SO_4 \\ H_2CO_3 \\ HNO_3, \end{array} \longrightarrow H_2R + Ca/Mg\left\{\begin{array}{l} Cl^- \\ SO_4^{--} \\ CO_3^{--} \\ HNO_3 \end{array}\right.$$

(v). Anion exchange resins can be regenerated by passing soda ash through it.

$$R\left\{\begin{array}{l} CO_3^{--} \\ Cl^- \\ SO_4^{--} \\ NO_3^- \end{array}\right. + Na_2CO_3 \xrightarrow[H_2O]{} ROH + Na\left\{\begin{array}{l} Cl^- \\ SO_4^{--} \\ CO_3^{--} \\ NO_3^- \end{array}\right.$$

(vi) This method also gives zero hardness water and does not lead to the formation of sludge.

(vii) Mor costlier and time consuming than zeolite process. Presence of Iron and Mangenese will not affect this process.

8. Minor treatments:

1. Activated carbon:

(i) Due to its strong adsorption properties it removes, taste, colour, odour, phenol, excess chlorine, dissolved minerals, micro-organics and organic matter.

(ii) Activated carbon is added generally at the time of co-agulation flocculation and filtration.

(iii) If it is added in water partially at the time of coagulation; partially at filtration, it is known as split method of application

(iv) Normal dose of activated carbon varies between 5-20 mg/l.

2. Treatment with $CuSO_4$.

(i) Due to its strong alguecidal property it is used to remove the algae from the water system.

(ii) Normal dose of $CuSO_4$ is 0.5 - 0.6 mg/l.

(iii) $CuSO_4$ is added in the water just before it is being passed in the distribution system. (after disinfection)

3. Removal of Iron and Mangenese:

(i) Iron and Mangenese is generally present in combination with the organic matter in water hence for removal of Fe & Mn, the bond between organic matter and Fe/Mn is to be broken. which is done by addition of oxidising agent (chlorine or Potassium pumangenate, lime) followed by aeration process.

$$OM + Fe/Mn \longrightarrow Fe^{++}/Mn^{++} \xrightarrow[\text{Aeration}]{} Fe(OH)_3,\ MnOH_2$$

4. Defluoridisation:

(i) The process of removal of Fluorine from water if its concentration is more than 1.5 mg/l.

(ii) Chemicals added to remove Fluorine are:
- Activated Carbon
- Lime
- Bone charcoal
- $Ca_3(PO_4)_2$

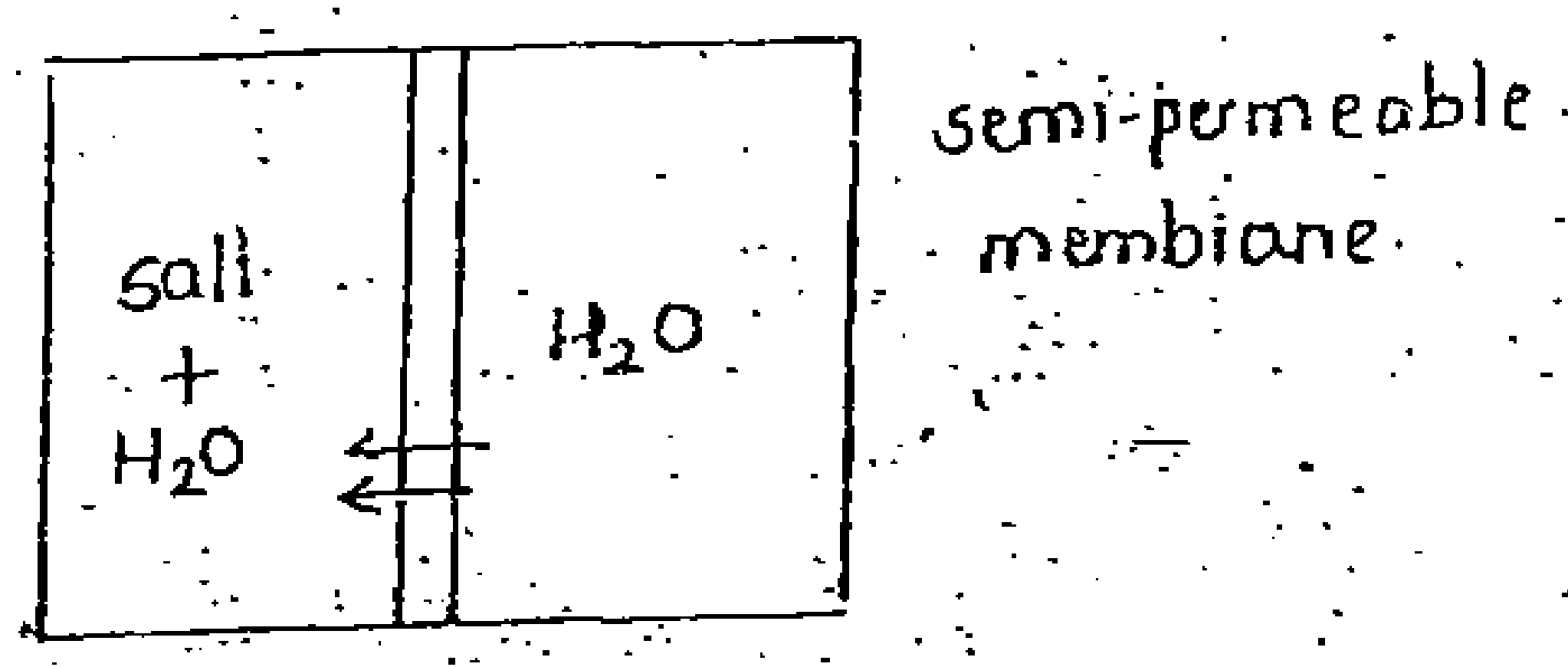
5. Fluoridation:

(i) The process of addition of fluorine in water if its conc. is less than 1 mg/l is known as fluoridation.

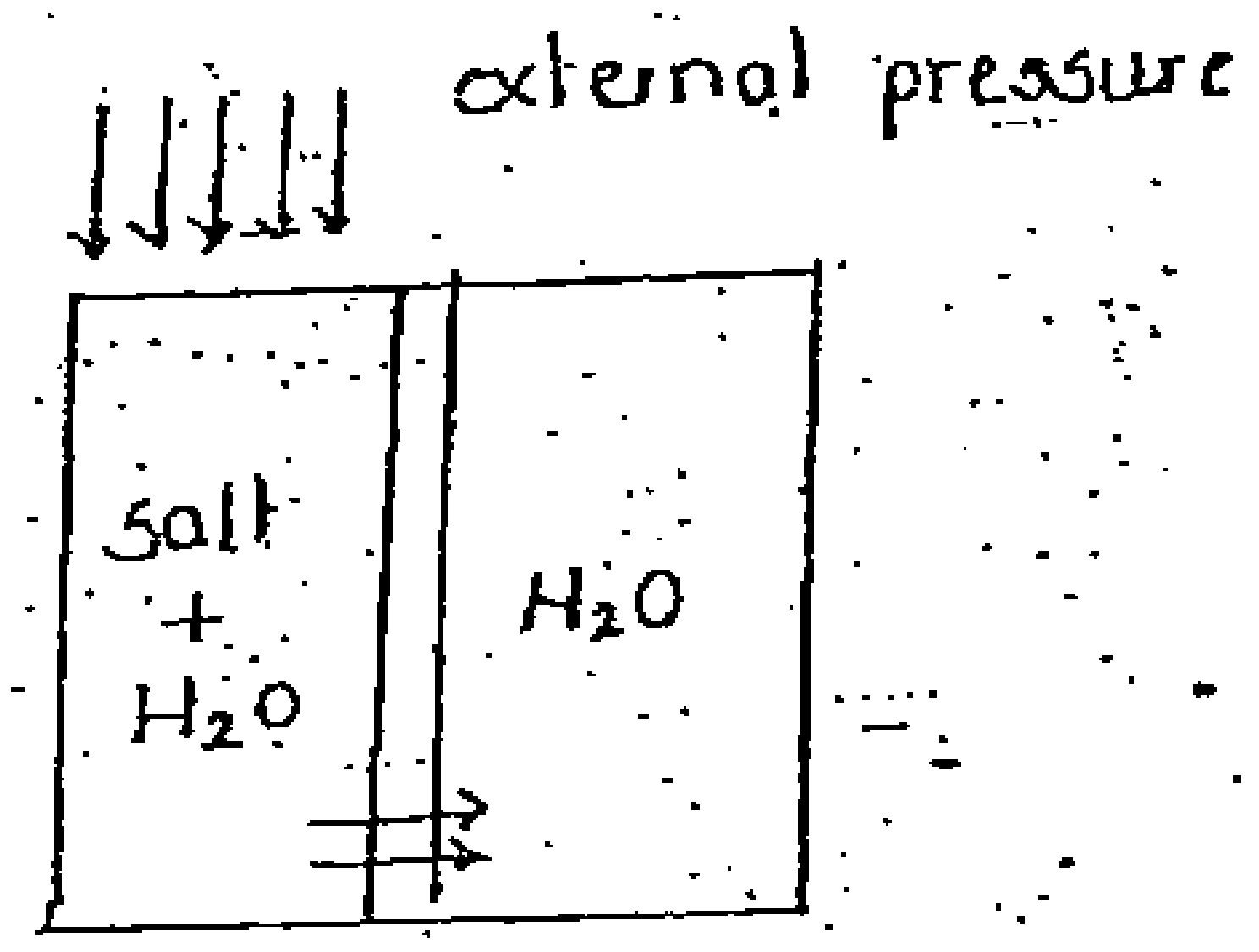
(ii) Chemicals added to cause Fluoridation are

NaF - Sodium Fluoride
Na_2SiF_6 - Sodium Silico Fluoride
H_2SiF_6 - Hydro Silic Acid.

6. De-salination :-
- Reverse osmosis
- Evaporation and distillation
- Freezing
- Electrolytic method.



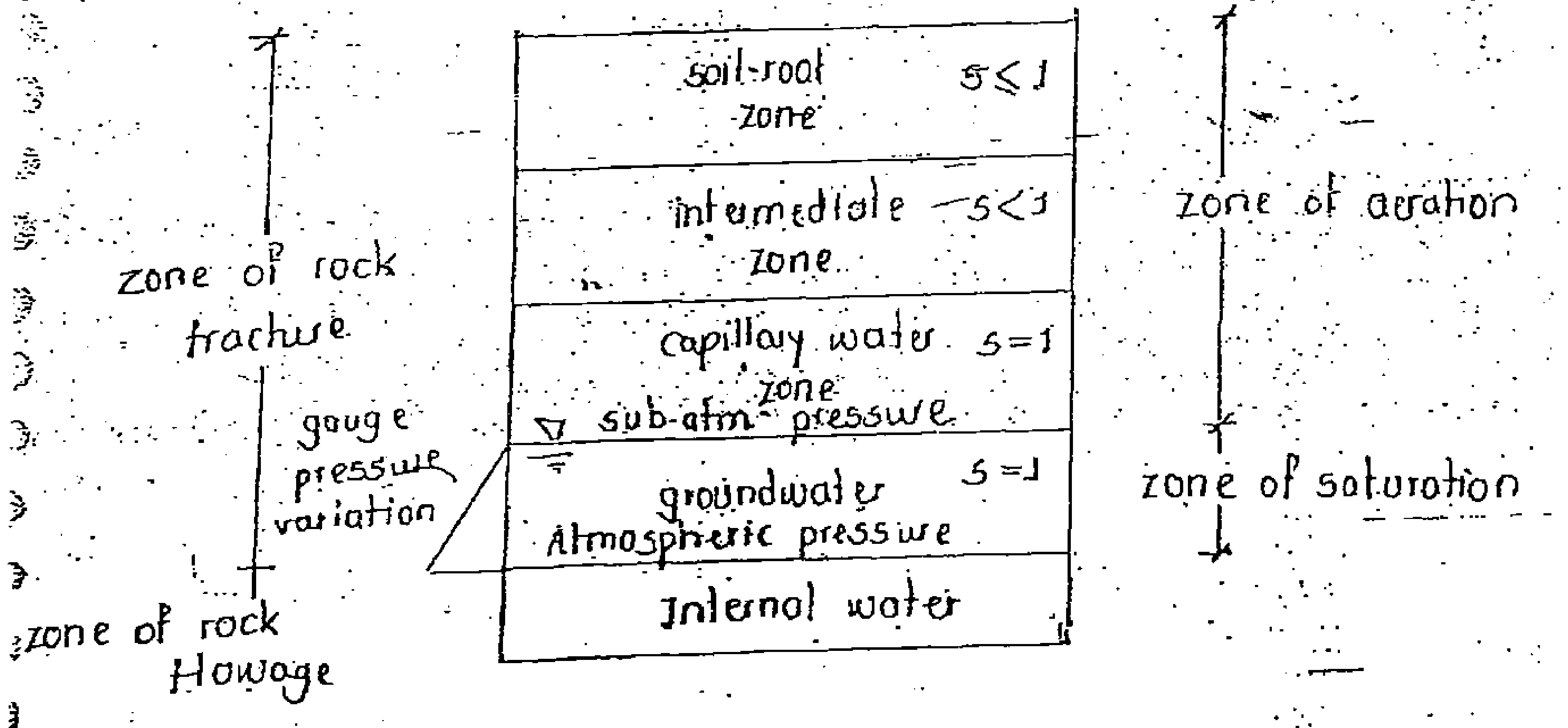
Osmosis



Reverse osmosis

Development of Groundwater :-

(i) With regards to existance of water below earth crust at different depths, earth crust can be divided into following zones :



The soil particles will take and transfer the pressure & not the water in voids, thus pressure at zone of saturation is atmospheric pressure.

1. Zone of Rock flowage :

(i) It is the zone in which rock undergoes permanent deformation and is subjected to plastic stresses.

(ii) Water present in this zone is known as internal water as it exists between internal cavities of rock.

(iii) This water cannot be extracted by any means hence this zone is of no engineering use.

(iv) The depth of this zone varies in km depending upon the topography.

2. Zone of rock fractures :

(i) It is the zone in which medium is subjected to elastic stresses. Water in this zone is stored in the voids of the medium whose quantity is dependant on the porosity of th medium.

(ii) This zone is further divided into zone of saturation and zone of Aeration.

The possibility of occurance of groundwater depends upon ~~the~~ following properties of medium.
- porosity
- permeability.

1. Porosity :

(i) The porosity of the medium represents the % voids in it.

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

(ii) The amount of water which can be stored in medium is dependant on the volume of voids in medium which in turn is dependant on porosity of medium.

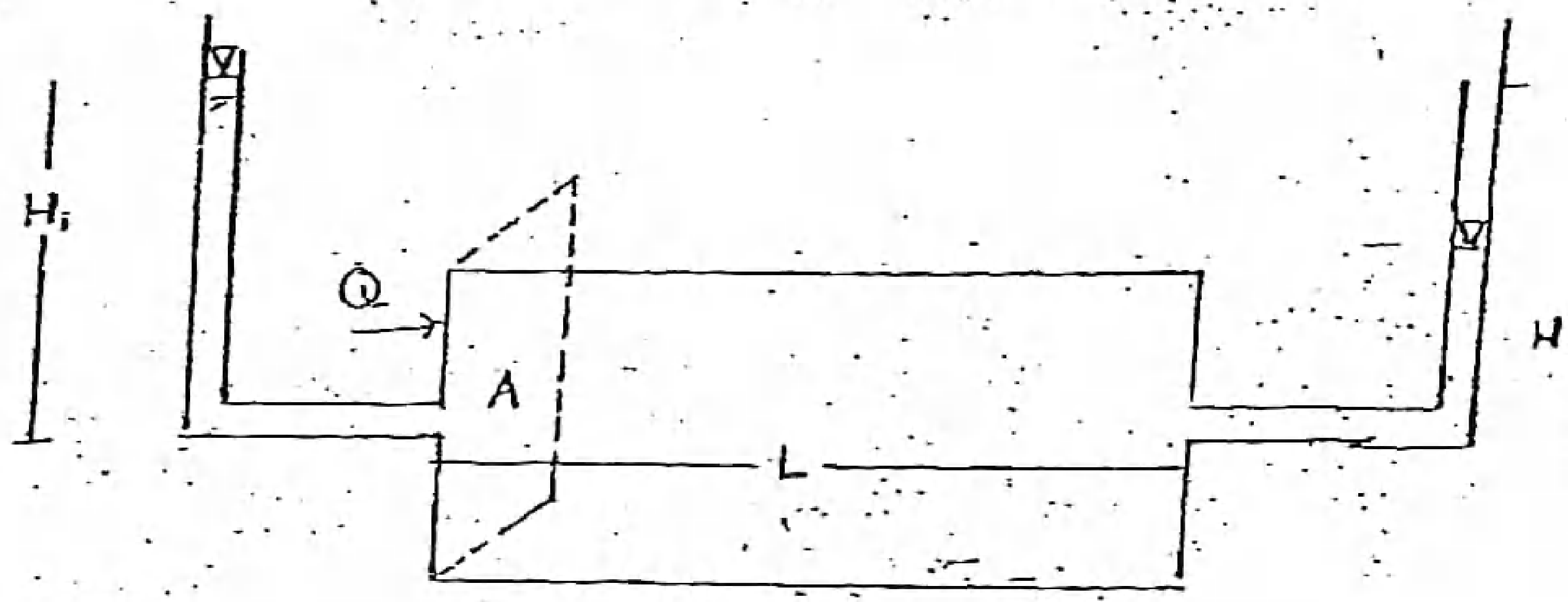
2. Permeability :

(i) It is the property of medium which permits flow of medium fluid through its interconnecting voids or it may also be defi as velocity of flow through medium under unit hydraulic gradien

According to Darcy's law.

$$v \propto i$$

$$v = k \cdot i$$



$$i = \frac{H_L}{L} = \frac{H_1 - H_2}{L}$$

$$v = \left(\frac{H_1 - H_2}{L}\right) \cdot k$$

$$Q = A \cdot V$$

But water flows through voids only

$$A_v < A$$

A_v - area of voids

$$Q = A \cdot V = A_v \cdot V_a$$

V_a - actual velocity of flow

$$V_a > V$$

$$V_a \propto i$$

$$V_a = K_p \cdot i$$

where K_p - coefficient of percolation

$$\frac{V_a}{V} = \frac{A}{A_v}$$

$L = V_A$

(Assuming length of path travelled by water particle is equal to length of specimen) practically $L_A > L$

$$\frac{V_a}{V} = \frac{A \cdot L}{A_v \cdot L}$$

$$= \frac{A \cdot L}{A_v \cdot L_A} = \frac{V}{V_v}$$

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

$$\frac{V_a}{V} = \frac{K_p \cdot i}{K\ i}$$

$$\frac{K_p}{K} = \frac{V_a}{V} = \frac{1}{\eta}$$

$$K_p = \frac{k}{\eta}$$

$\therefore K_p > k$ as $\eta < 1$ always

coefficient of permeability. $k = f$ (medium properties, fluid properties)

k in m/sec

$$= \left(D^2 \cdot e^K \cdot \frac{1}{C_1} \cdot \frac{1}{C_2}\right) \left(\frac{\gamma_w}{\mu}\right)$$

e - void ratio is more appropriate parameter to compare voids of two medium as it depends on volume of solids

$$e = \frac{V_v}{V_s}$$

C_1 - conc of foreign medium impurities

C_2 - entrapped gases conc. in medium voids

coefficient of intrinsic permeability (K_o)

$$K_o = f \text{ (medium properties)}$$

$$= k \left(\frac{\mu}{\gamma}\right)$$

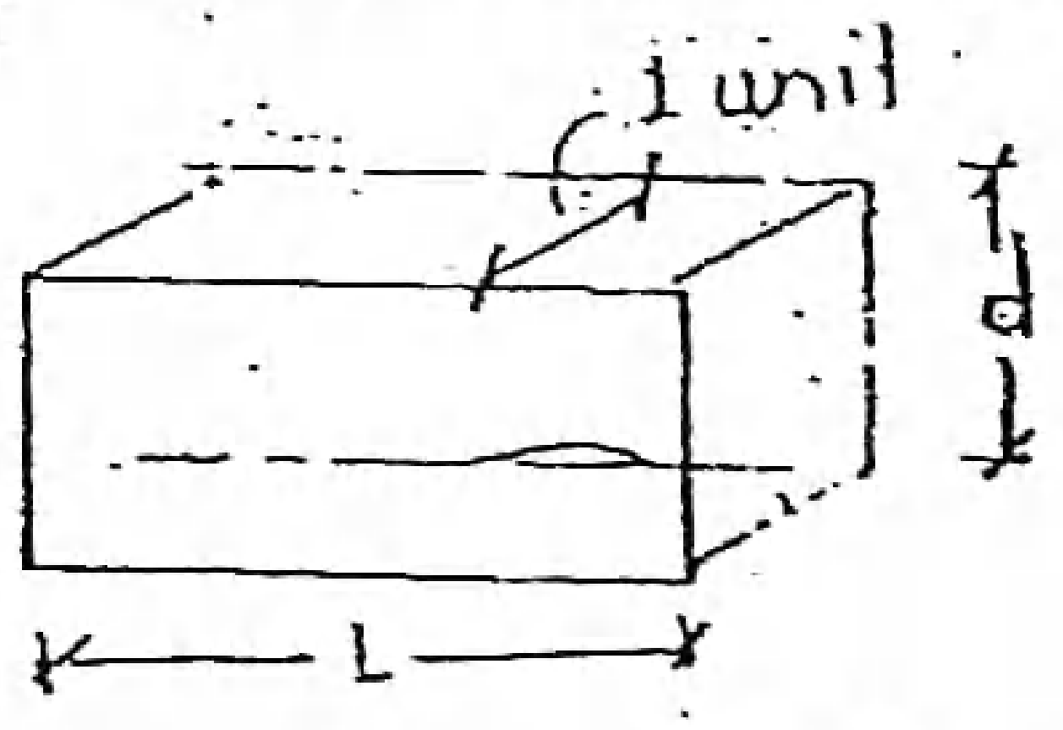
K_o - in m^2

Is also darcy

$1 \text{ darcy} = 9.87 \times 10^{-13}\ m^2$

Note :

The rate of flow of water through the entire medium of unit width and under unit hydraulic gradient is termed as coefficient of transmissibility (T)



$$T = K(A)$$

$\because A = d \times 1$

$$T = K \cdot d \quad m^3/m/sec$$

k - coeff. of permeability.

Groundwater yield :

Saturday
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Specific yield (Y_s)

(i) Water present in the voids of the medium cannot be completely extracted as some of the water is adsorbed over surface of the medium particles known as Pelicular water.

(ii) The quantity of this water drained or retained is represented in terms of specific yield or specific retaintion.

$$\text{specific yield } (Y_s) = \frac{\text{Amt. (vol. or wt.) of water drained by gravity}}{\text{Amt. (vol. or wt.) of soil retained}}$$

Specific Retention (Y_R) / Field capacity :

$$\text{specific retention} = \frac{\text{Amt. of water retained against gravity}}{\text{Amt. of soil drained}}$$

V_D - volume of water drained

V_R - volume of water retained

$$Y_s + Y_R = \frac{V_D}{V} + \frac{V_R}{V} = \frac{V_D + V_R}{V}$$

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

If $V_v = V_w$, i.e. degree of saturation $S = 1$

$$\therefore Y_s + Y_R = \frac{V_v}{V} = \eta \quad \text{i.e. porosity}$$

specific yield will be more for sand while specific relention will be more for clay medium.

Geological formations and their types :

1. Aquifers :

(i) These geological formations posses both permeability and porosity. Hence appricialble yield can be extracted from them.

(ii) These formations are generally of three types :

- confined aquifers
- unconfined aquifers
- perched aquifers

(a) Unconfined aquifers :

(i) These are the formations in which water table serves as the topmost layer of the saturation.

(ii) These formations are not being subjected to any confining pressure due to presence of impervious layer over them.

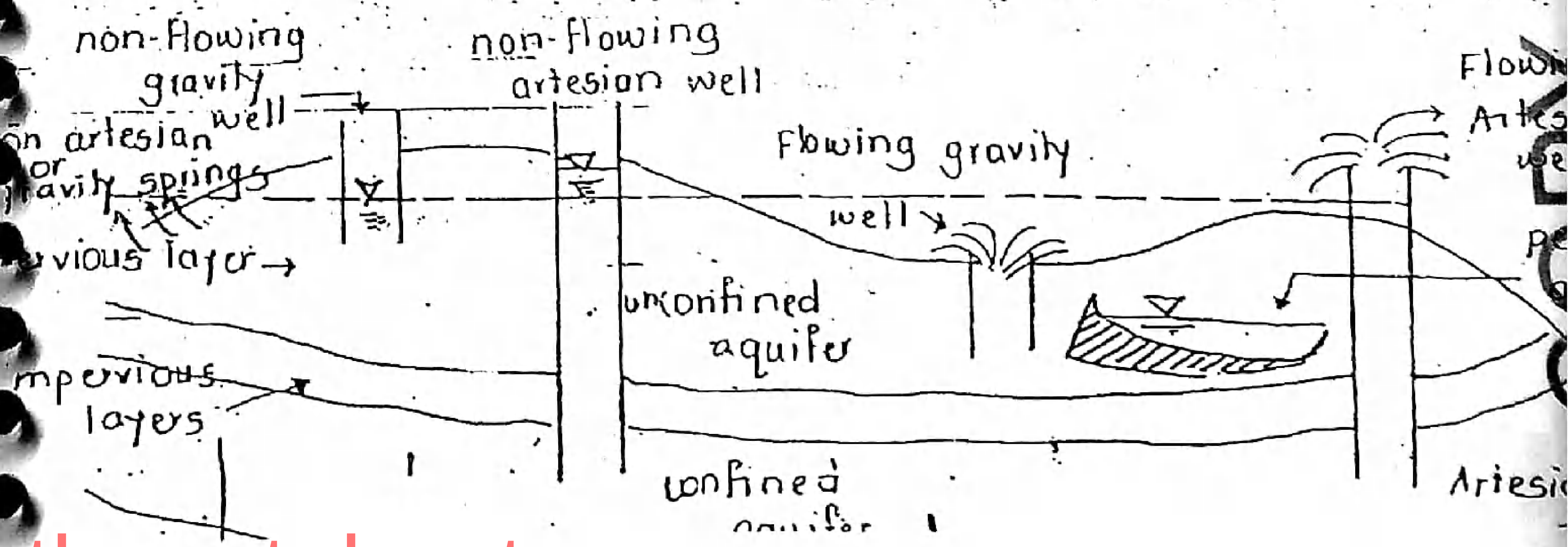
(b) Confined aquifers :

(i) These are the formations which are subjected to confining pressure due to presence of impurvious layer over them.

(ii) Water present in the voids of these formations posses the superatmospheric pressure.

(c) Perched aquifer :

(i) In some cases impervious layer exist in between the pervious layer carrying saturated soil mass that serves as the source of water termed as Perched aquifer.



2. Aquiclude :-

(i) These are the formations which are highly porous but non-permeable. Hence water cannot be extracted from these types of geological formations e.g. clay.

3. Aquitard :-

These are the geological formations which are porous but posses very less permeability. Hence water does not readily flow out of these formations but instead seeps out. e.g. sandy clay.

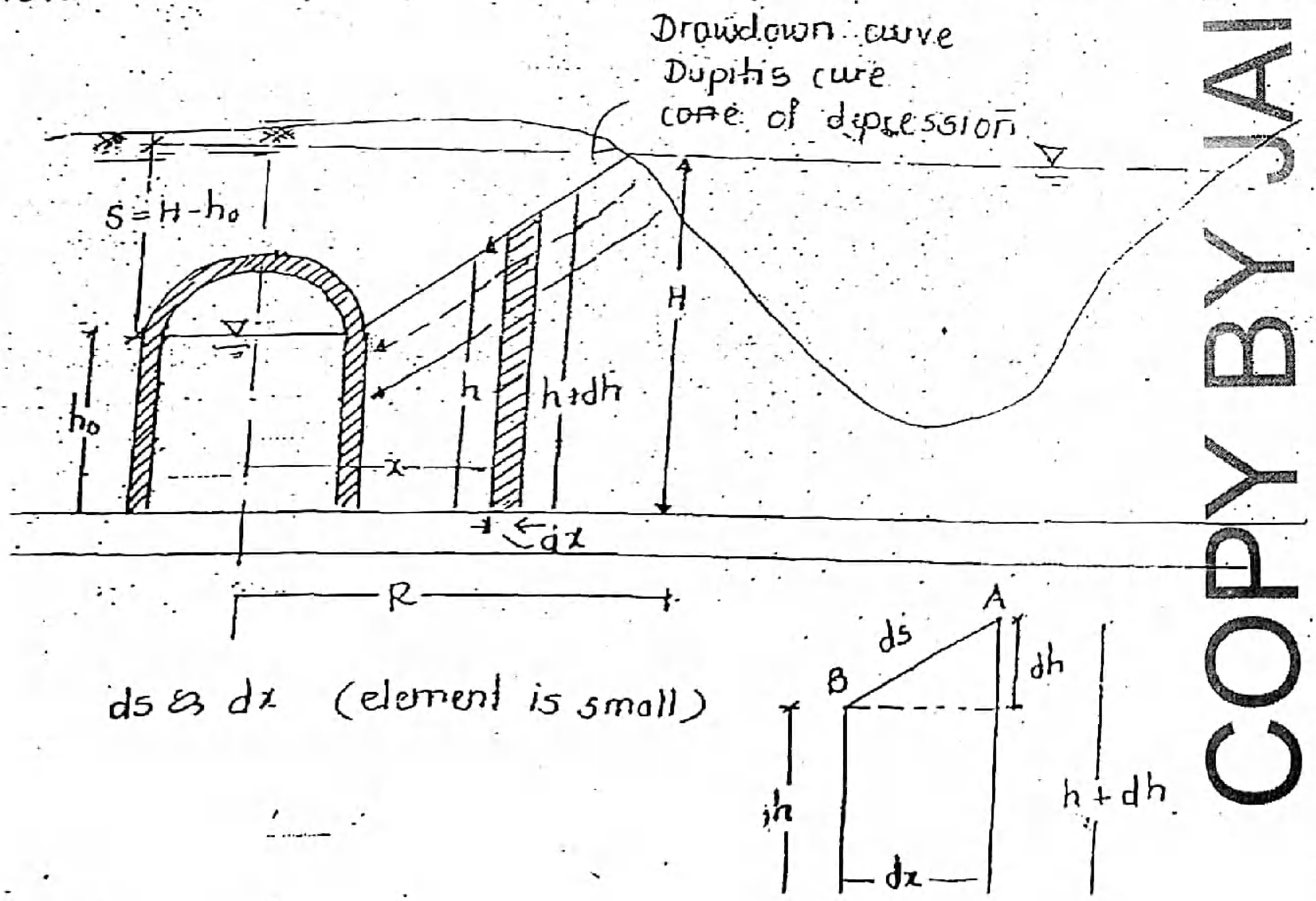
4. Aquifuge :-

These are the geological formations which are neither porous nor permeable e.g. Granite.

Different forms in which water may be extracted from ground :-

1. Infiltration galleries :-

(i) These are nearly horizontal tunnels constructed at the shallow depth of the 3-5 m along the bank of river in water bearing strata.



discharge through element.

$$q_x = a_x \cdot V_x$$

$$= (h \cdot L) \cdot K \cdot i_x \qquad \because V = k \cdot i_x \text{ Darcy's law}$$

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

Total discharge

$$Q = \int q_x$$

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

$$Q \int_{x=0}^{x=R} dx = K \cdot L \int_{h=h_o}^{h=H} h \cdot dh$$

$$Q [x]_0^R = K \cdot L \left[h^2/2\right]_{h_o}^{H}$$

$$Q = \frac{KL(H^2 - h_o^2)}{2R}$$

$$= \frac{K \cdot L (H - h_o)(H + h_o)}{2R}$$

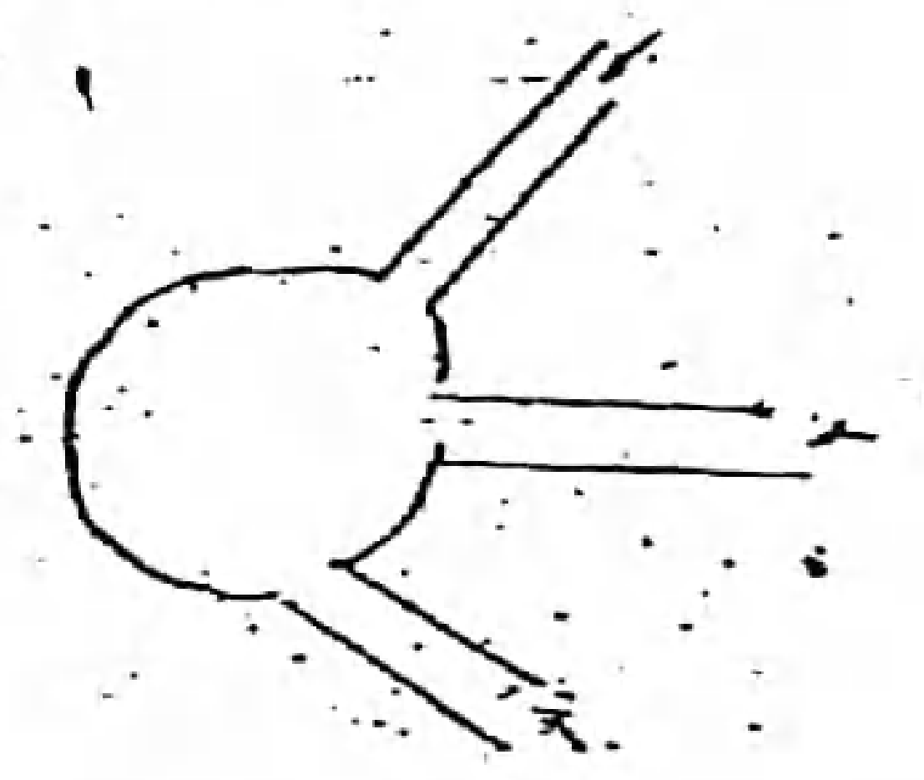
It can also be directly calculated as taking avg. area through which discharge passes

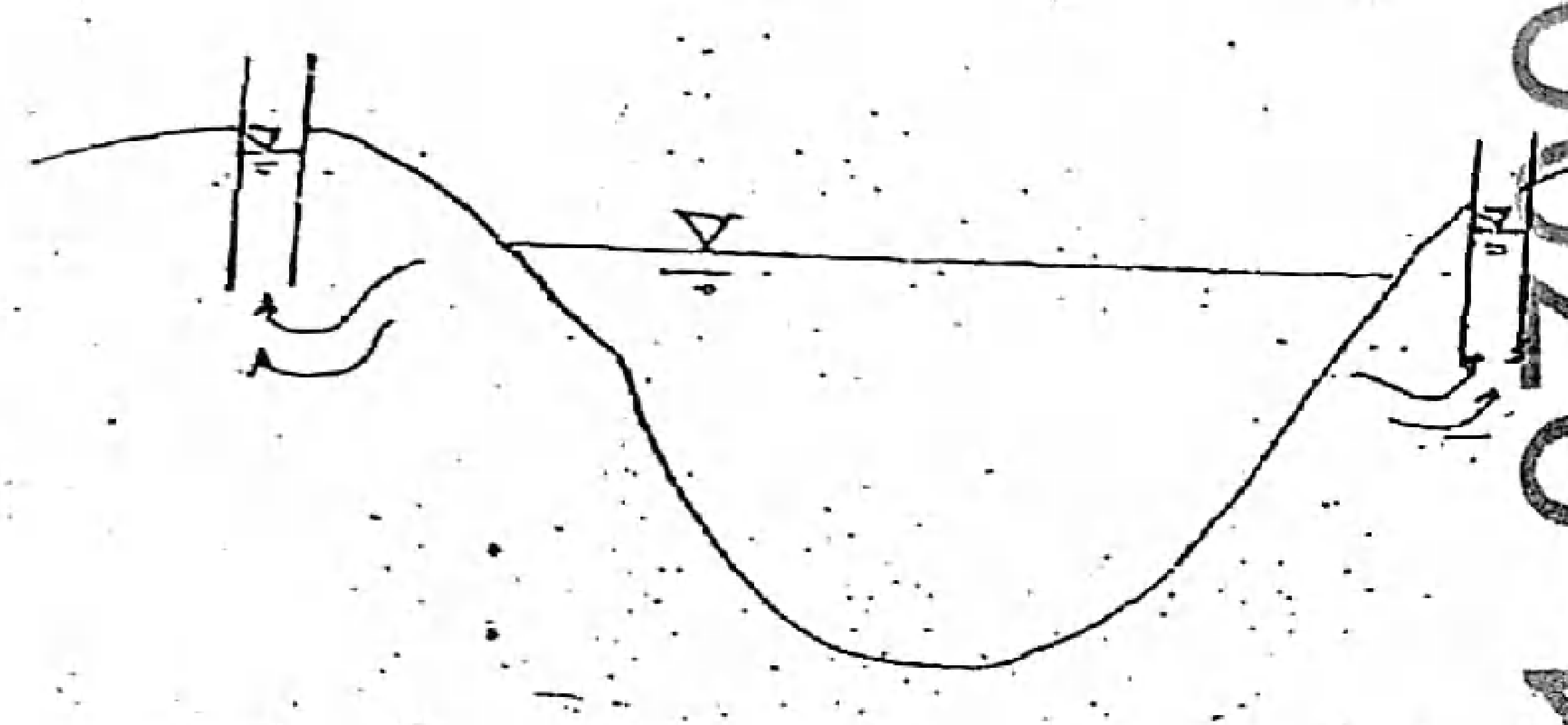
$$Q = \frac{(H - h_o)}{2}$$

2. Infiltration wells :

(i) These are discontinuous structures constructed along bank of river in which water is collected through seepage from bottom.

(ii) All such wells are connected through a common well known as Jack well from which water is pumped to treatment plant.





3. Artesian springs.

Non-artesian springs are not considered as potential source of raw water, as groundwater table may drop in future and the discharge may get reduced.

4. Wells :

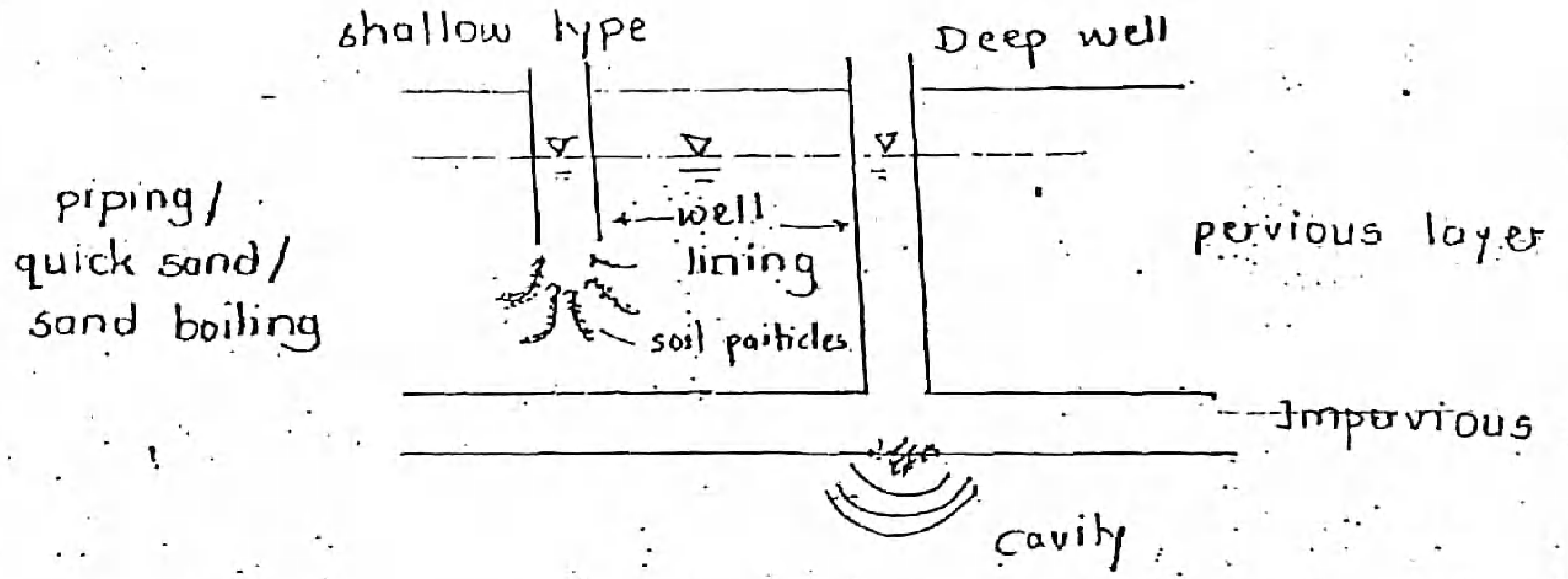
These are generally of two types :

- Wells
  - open wells
    - shallow wells
    - ~~tube~~ deep wells
  - tube wells
    - screen type
      - strainer type
      - gravel pack type
    - cavity type

(a) ~~Open wells~~ :

(i) In shallow wells water is drawn from topmost water bearing strata which is liable to be contaminated.

(ii) Large quantity of discharge can not be extracted through shallow wells as with increase in discharge, velocity of flow through well increases and if this velocity exceeds critical velocity (velocity ... which medium



lead to the destabilisation of well lining and finally resulting in sinking (this process is known as piping probl sinking of well is consequence of piping)

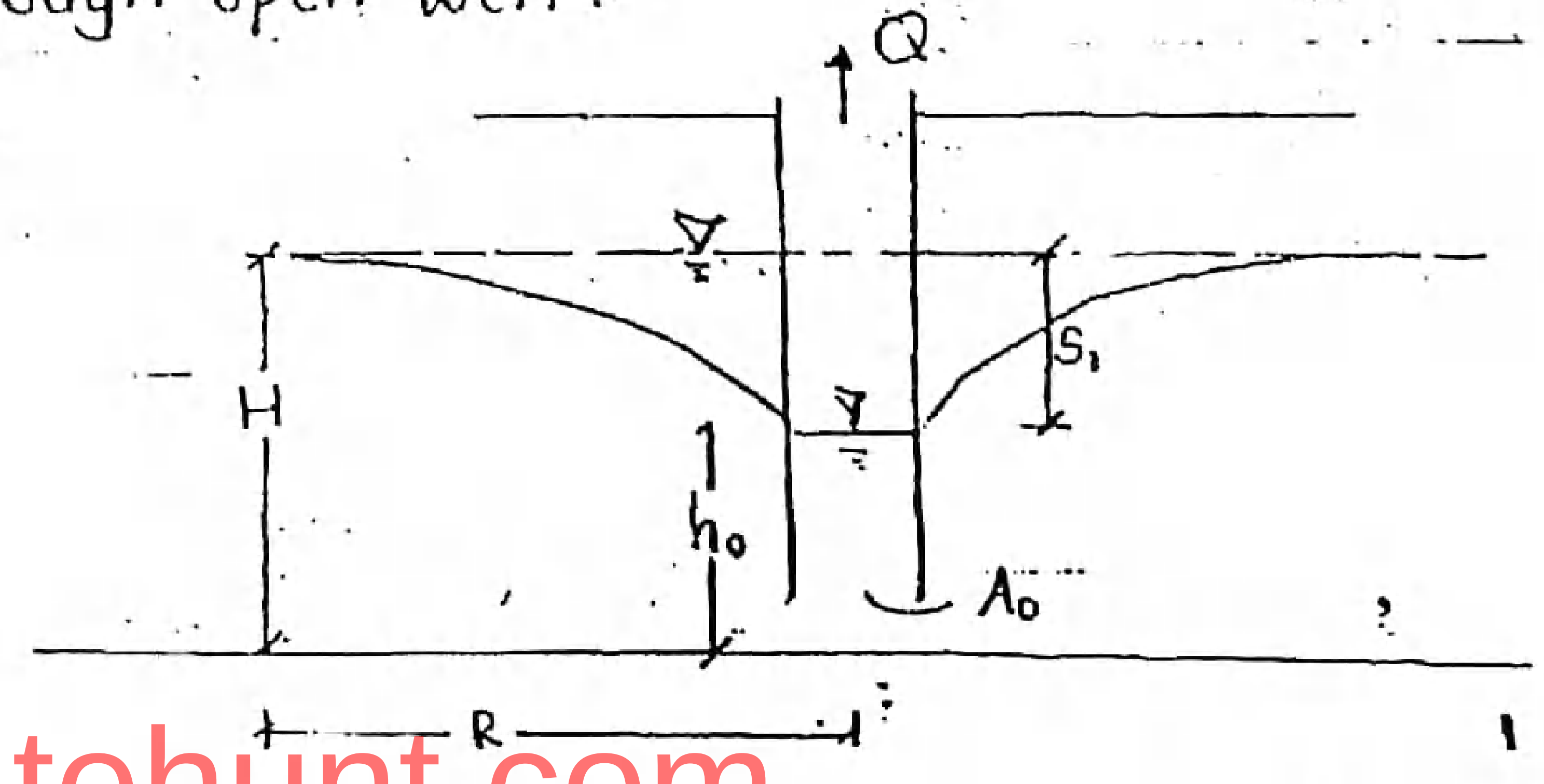
(iii) This problem doesn't occur in deep well as with increase in discharge when velocity through well increases resulting in movement of medium particles from bottom of the well leading to the increased area of flow from bottom. (This process is known as cavity formation)

(iv) Due to this increased area of flow, velocity through the well again decreases which finally results in no movement of medium particles along with the water.

Note :

In case of deep wells, destabilisation of well lining does not take place even after piping occurs as well lining is bei supported by impervious layer.

Yield through open well :



$$Q_1 = A_a V$$
$$= A_a K \cdot i$$
$$= A_f K \cdot \frac{S_1}{R} \qquad \frac{S_1}{R} = \text{head loss per unit length.}$$
$$= \frac{K}{R} \cdot A_f S_1$$
$$Q_1 = C \cdot A_f S_1$$
$$C = \frac{Q_1}{A_f S_1} \qquad m^3/m^2/m/sec$$

For drawdown S,

$$Q = C \cdot A \cdot S$$
$$= \left(\frac{Q_1}{A_f S_1}\right) \cdot A \cdot S$$

C' is termed as discharge per unit drawndown per unit area and called as Specific yield.

(i) The above expression is valid only when height of water in the well is constant (h_0) i.e. drawdown should be constant.

(ii) The velocity at which piping starts in the medium is termed as critical velocity and the drawdown at which this critical velocity is achieved through well is known as Critical depression Head.

$$i = \frac{S_1}{R}$$

V depends on S_1

(iii) Under normal conditions, working head is kept to be 1/3rd of critical depression Head (Factor of safety of 3 against piping).

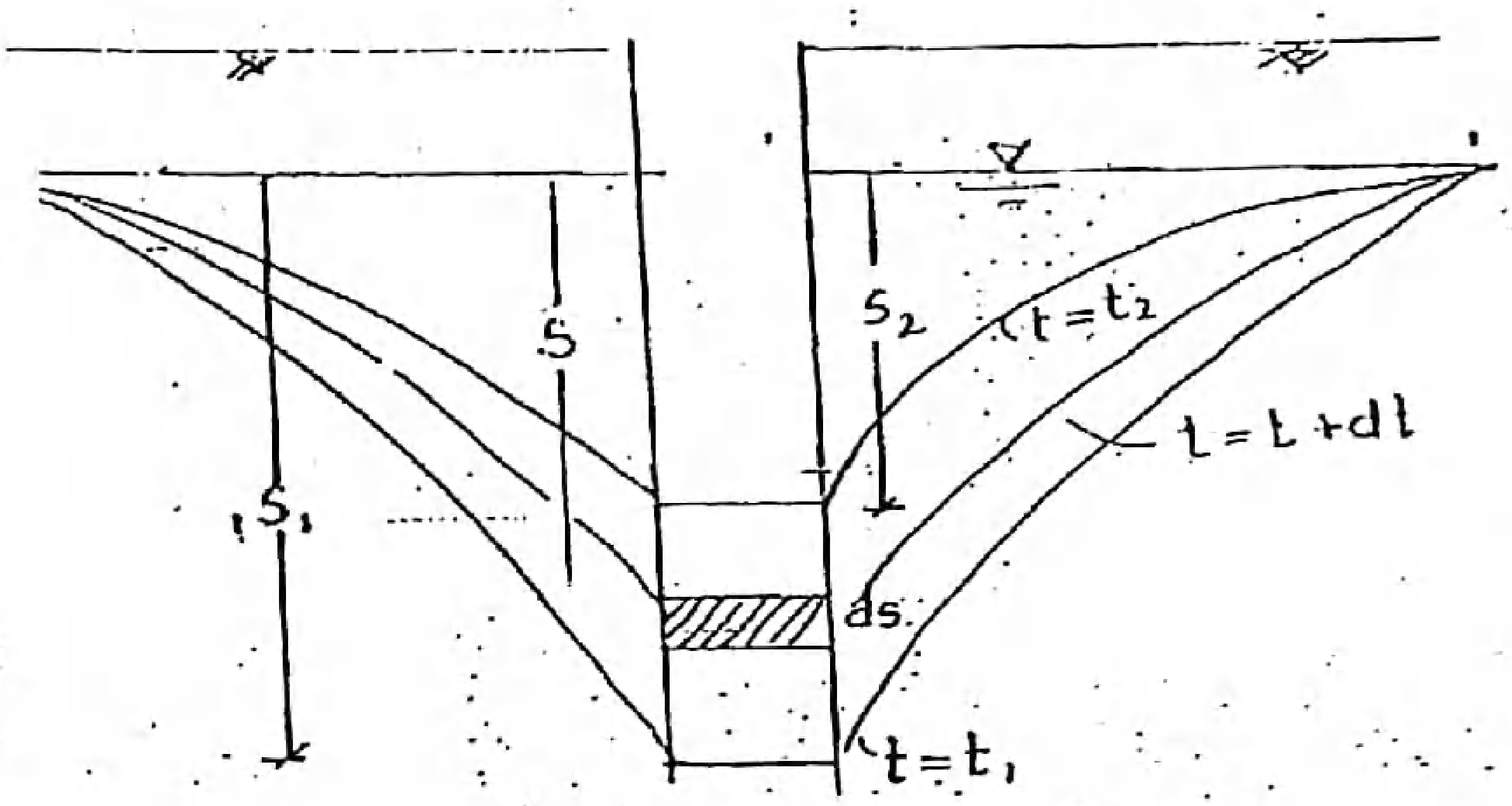
(iv) The area of flow considered while computing discharge through well is 4/3 times of actual area of well.

$$A = \frac{4}{3} A_0$$

(Factor of safety against piping cavity is 4/3)

$$FOS_{(T)} = FOS_p \times FOS_c$$
$$= 3 \times 4/3$$
$$= 4$$

Recuperation test:



volume of water recuperated in 'dt' time interval

$$dv = Q \cdot dt$$
$$= -A \cdot ds \qquad \text{Cos drowdown is decreasing}$$
$$-A \cdot ds = CA \cdot S \cdot dt$$
$$-\frac{ds}{S} = C \cdot dt$$
$$-\int_{S=S_1}^{S=S_2} \frac{ds}{S} = C \int_{t=t_1}^{t=t_2} dt$$
$$-\ln[S]_{S_1}^{S_2} = C[t]_{t_1}^{t_2}$$
$$\ln\left[\frac{S_1}{S_2}\right] = C(t_2 - t_1)$$
$$2.303 \log_{10}\left(\frac{S_1}{S_2}\right) = C \cdot t \qquad t = (t_2 - t_1)$$
$$C = \frac{2.303}{t} \log_{10}\left(\frac{S_1}{S_2}\right) \qquad m^3/m^2/m/se$$

(i) As it is very difficult to maintain constant drawdown during pumping. Recuperation test is performed to find value of specific yield.

(ii) In this test time required by water to recuperate known distance is noted to find sp. yield of medium.

Q. During a recuperation test water in an open well was found depressed by pumping upto 2.1 m and it recuperated 1.6 m in 90 min. Find the diameter of well to yield 10 lit/sec. under the depression head of 2 m.

Data: $S_1 = 2.1$ m

recuperation, $S_2 = 1.6$ m i.e. $(S_1 - S_2) = 0.5$.

$t = 90$ min

$C = 10$ lit/sec

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

$$= \frac{2.303}{90 \times 60} \log_{10}\left(\frac{2.1}{0.5}\right)$$

$$= 2.65 \times 10^{-4} \ m^3/m^2/m/sec$$

$$Q = C \cdot A \cdot S$$

$$10 \times 10^{-3} \ m^3/s = 2.65 \times 10^{-4} \times \left(\frac{\pi D^2}{4}\right) \times 2$$

$$D = 4.89 \ m$$

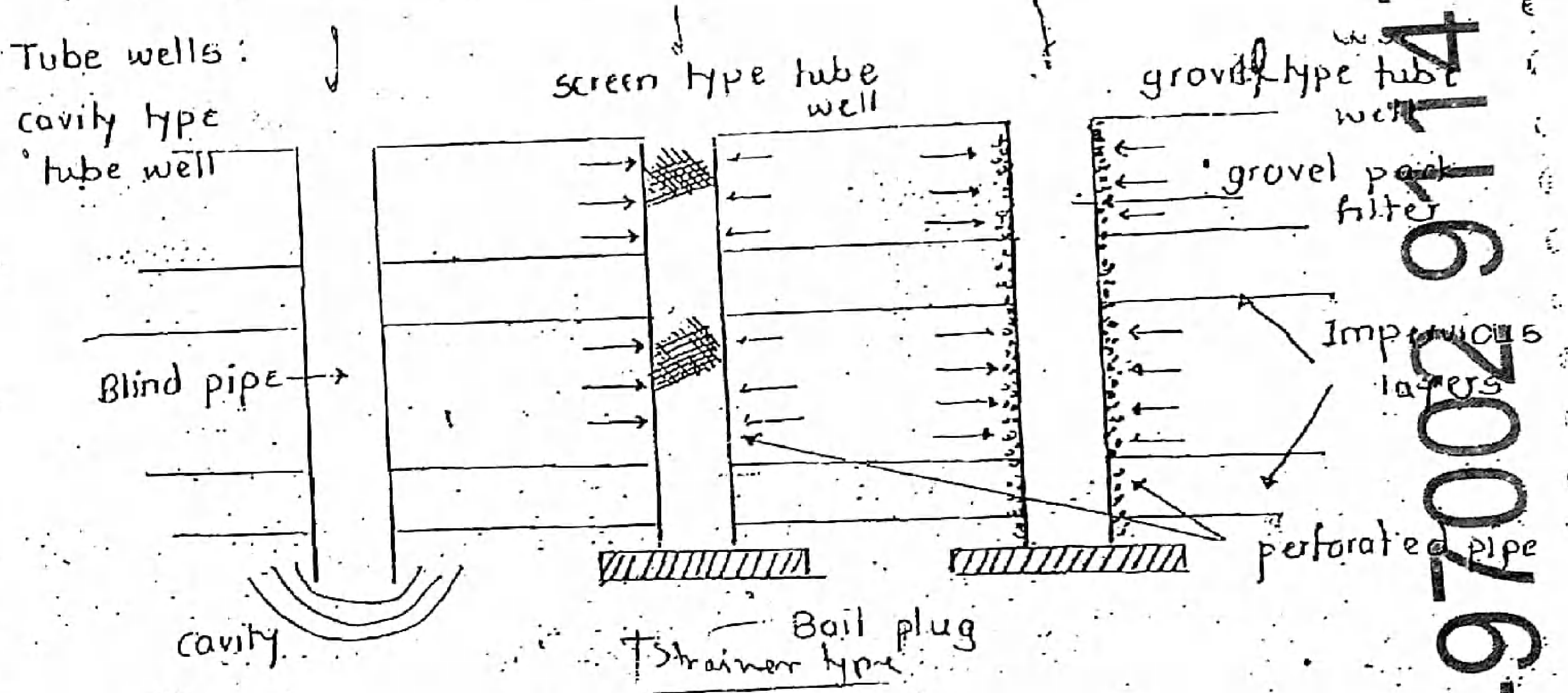
Q. In a fine sand aquifer reagion design a well to get 10 Lit/sec yield under the depression head of 2.5 m. Value of C'/A in m^3/hr per sq.m of area per unit drawdown may be taken as 0.6.

$$Q = C \cdot A \cdot S$$

$$10 \times 10^{-3} \ m^3/s = \frac{0.6}{60 \times 60} \times \left(\frac{\pi D^2}{4}\right) \times 2.5$$

$$D = 5.52 \ m$$

(b) Tube wells:



Bail plug is used for avoiding dissipation of pressure applied to pump water from screen type and gravel type tube well.

Yield through tube well:

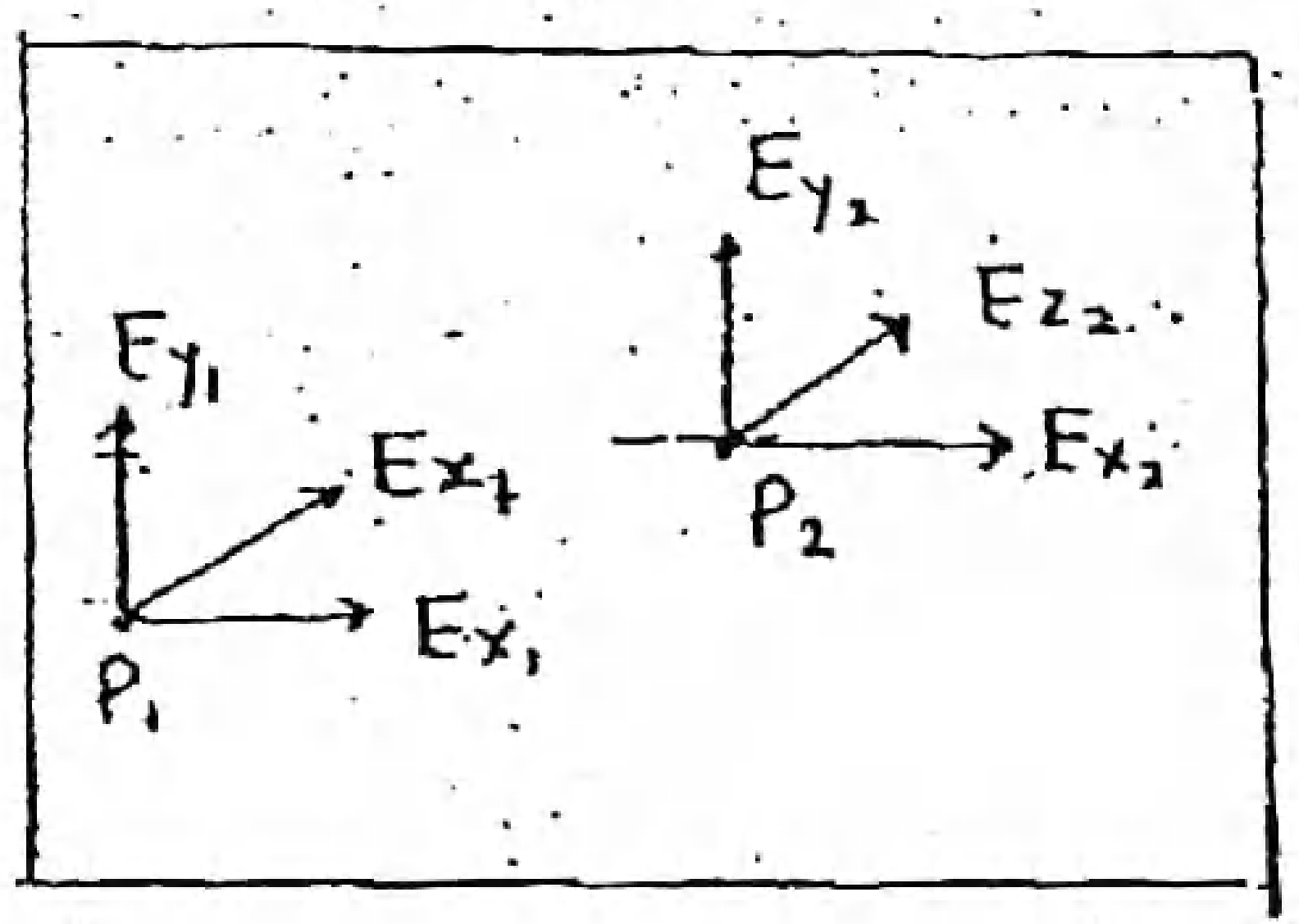
I) Yield through tube well in unconfined aquifer

- Dupits Theory
- Theim's Theory

Dupits Theory

Assumptions

(i) Medium is assumed to be homogenous isotropic and infinit



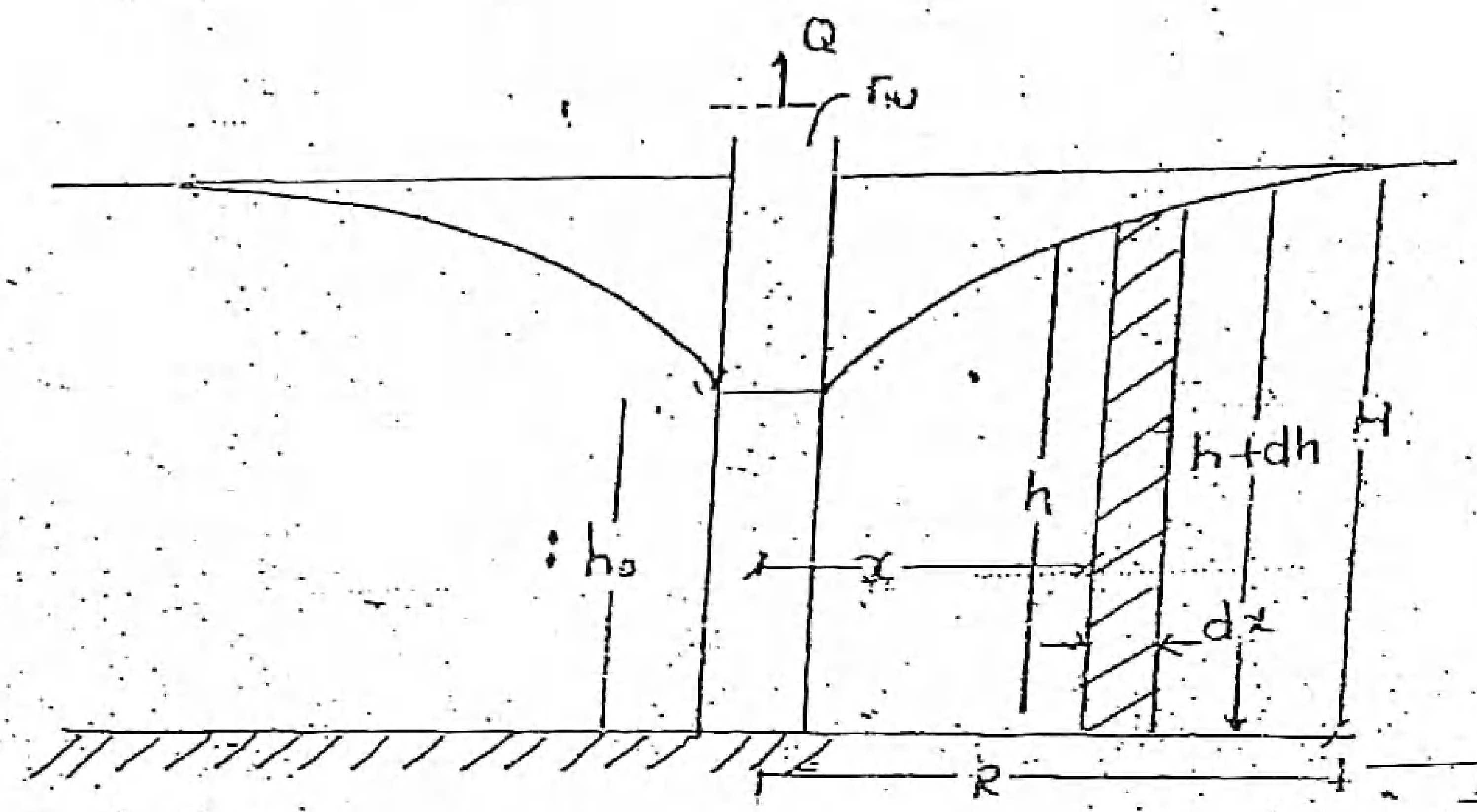
For particles P_1 & P_2

$E_{x_1} = E_{y_1} = E_{z_1}$ & $E_{x_2} = E_{y_2} = E_{z_2}$ is Isotropy

& $E_{x_1} = E_{x_2}$ $E_{y_1} = E_{y_2}$ $E_{z_1} = E_{z_2}$ is Homogenity.

(ii) The flow of the water in the vicinity of well is radial horizontal and laminar.

(iii) The loss of head is directly proportional to tangent of hydraulic gradient $\left(\frac{dh}{dx}\right)$ instead of its sine $\left(\frac{dh}{ds}\right)$



A. For unconfined aquifer

discharge through cylindrical section of thickness dx.

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

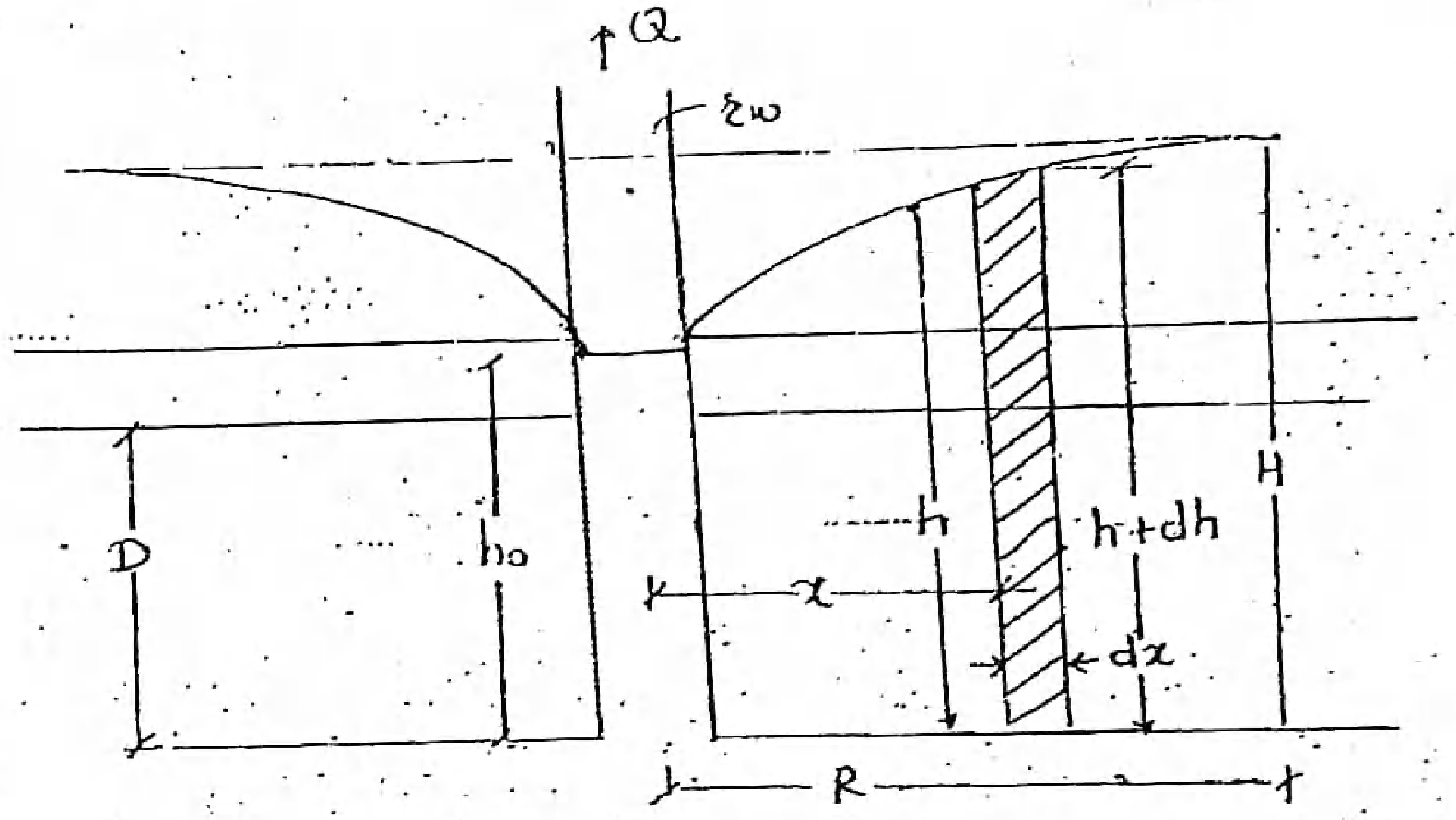
Total discharge through well,

$$Q = \int q_x$$
$$= \int 2\pi x \cdot h \cdot k \cdot \frac{dh}{dx}$$
$$Q \int_{x=r_w}^{x=R} \frac{dx}{x} = 2\pi k \int_{h=h_0}^{h=H} h \cdot dh$$
$$Q \ln [x]_{r_w}^{R} = 2\pi k \left[\frac{h^2}{2}\right]_{h_0}^{H}$$

discharge through unconfined aquifer

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

$(H_2^2 - h_0^2) = s(H + h_0)$



discharge through section at distance x

$$q_x = a_x \cdot V_x$$
$$= (2\pi x \cdot D) \cdot k \cdot i_x$$
$$= 2\pi x \cdot D \cdot k \cdot \frac{dh}{dx}$$

Total discharge. $Q = \int q_x$

$$Q = \int 2\pi x \cdot D k \cdot \frac{dh}{dx}$$
$$Q \int_{x=r_w}^{x=R} \frac{dx}{x} = 2\pi k D \int_{h=h_0}^{h=H} dh$$
$$Q \ln [x]_{r_w}^{R} = 2\pi k \cdot D \, [h]_{h_0}^{H}$$
$$Q = \frac{2\pi D k \cdot (H - h_0)}{2.306 \log_{10} (R/r_w)}$$

$H - h_0 = s$

Workbook (Quality control of water supply)

Q.6.

$$HOCl \underset{pH<7}{\overset{pH>8}{\rightleftharpoons}} H^+ + OCl^-$$

At pH = 7

$$K = 2.5\times10^{-8} \text{ mole/l at } 20°C$$

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

$$[H^+] = 10^{-7} \text{ moles/lit.}$$

$$[HOCl] = \frac{[OCl^-]\times10^{-7}}{2.5\times10^{-8}}$$

$$[HOCl] = 4[OCl^-]$$

$$\text{Fraction of } HOCl = \frac{[HOCl]}{[HOCl]+[OCl^-]}$$

$$= \frac{4\,OCl^-}{5[OCl^-]}$$

$$= 0.8$$

Q. 15 Only clay is present in ppt form

Q. 18. Carbonate hardness $= \frac{190}{61}\times50$ $[HCO_3^-] = 190$ mg/l

$$= 155.76 \text{ mg/L}$$

Q. 21.

$$\%\ \text{removal} = \frac{10^6-10^2}{10^6}\times100$$

$$\log \text{ removal} = \log 10^6 - \log 10^2$$

Q. 20

$$[OH^-] = 10^{-5.6}\times10^{-3} \text{ — for m mol/L}$$

$$= 10^{-8.6} \text{ mole/L}$$

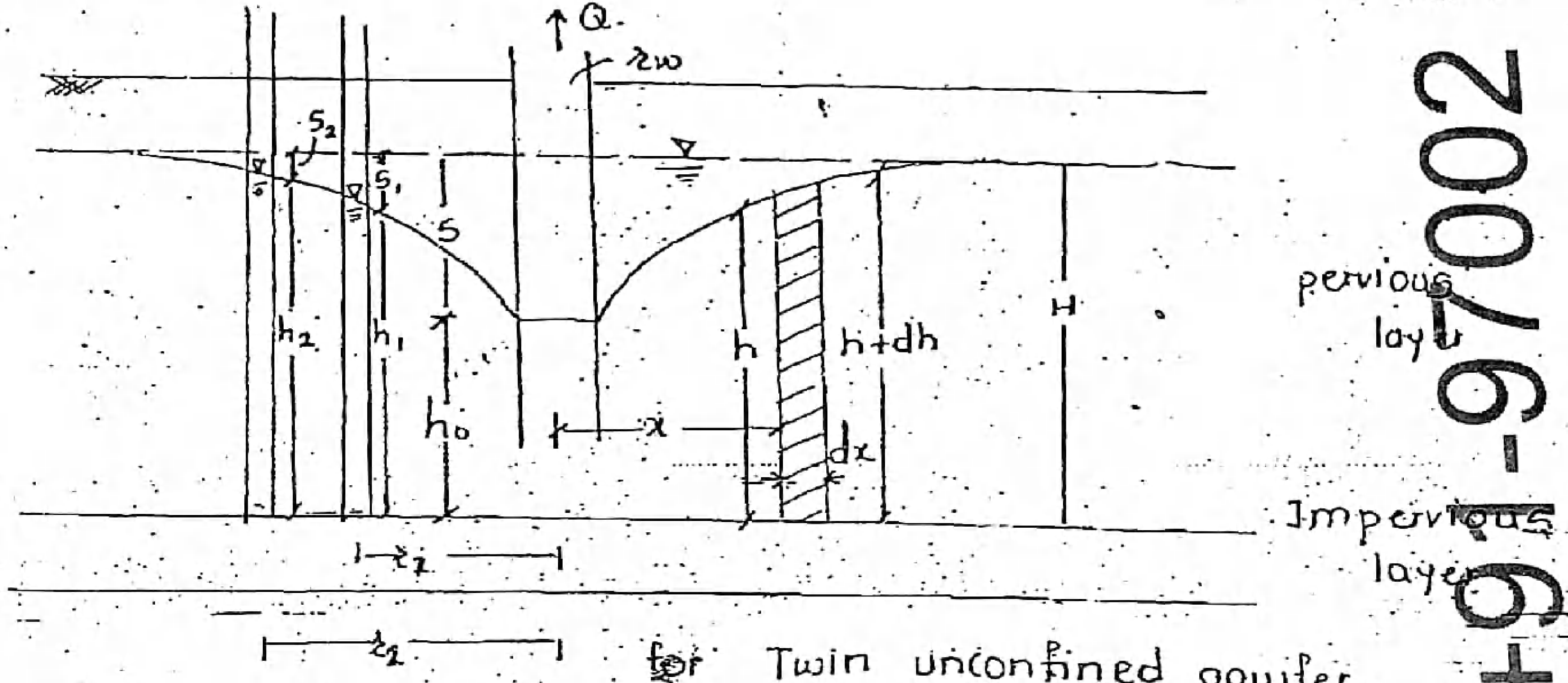
Q-26.

Contaminant	Cause for rejection
Mercury	0.001
Nickel	0.02
Zinc	~~5~~ 15

28th July

Theims Theory :

All assumptions are same as that of Dupit's Theory.



for Twin unconfined aquifer.

discharge through section dx.

$$q_x = a_x \cdot V_x$$

$$= (2\pi x \cdot h)\cdot k \cdot i_x$$

$$= 2\pi x \cdot h \cdot k \cdot \frac{dh}{dx}$$

Total discharge.

$$Q = \int q_x$$

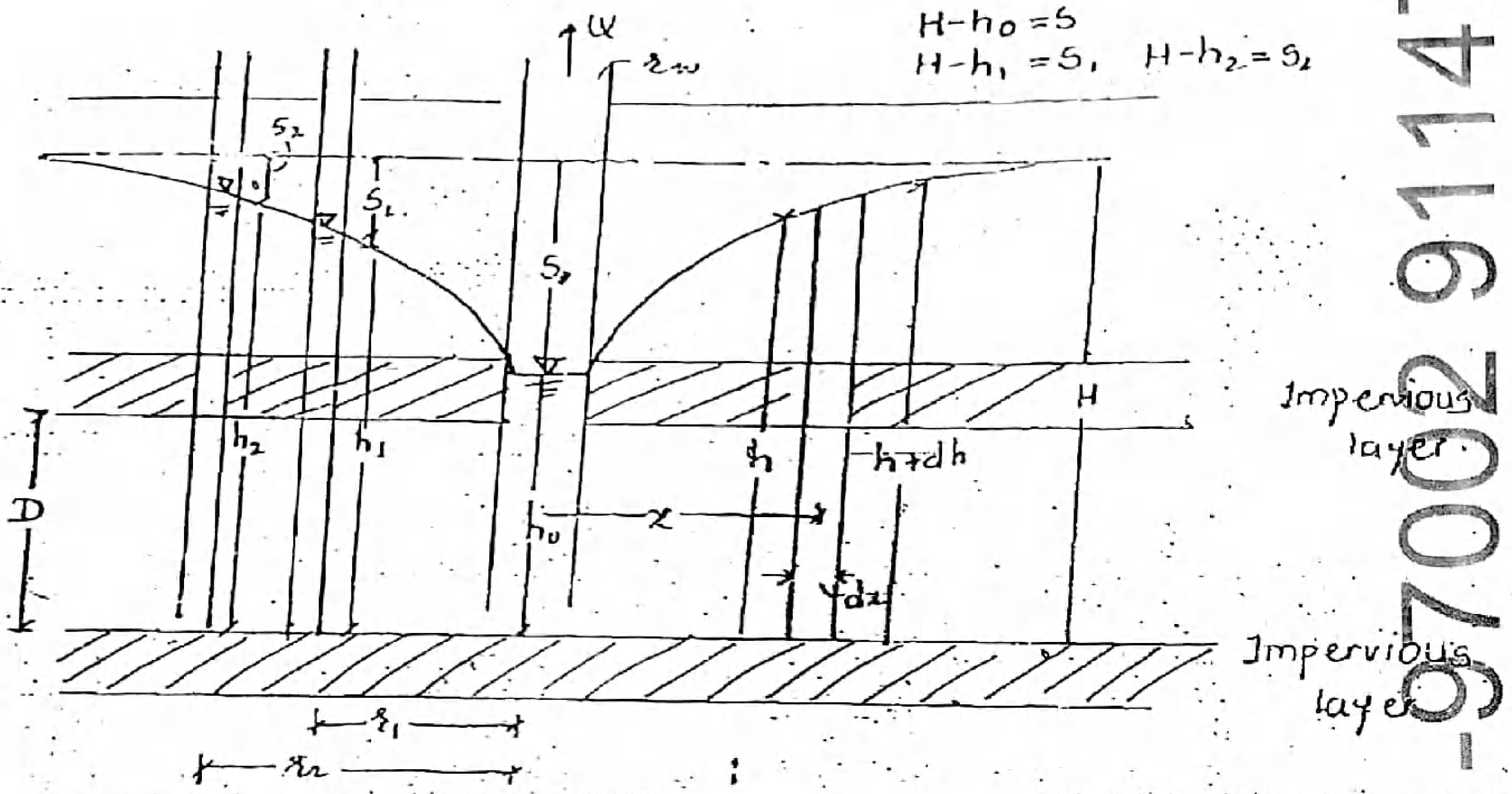
$$Q \int_{x=r_w}^{x=r_1} \frac{dx}{x} = 2\pi k \int_{h=h_0}^{h=h_1} h \cdot dh$$

$$Q \ln\left(\frac{r_1}{r_w}\right) = \pi k (h_1^2 - h_0^2)$$

$$Q = \frac{\pi k (h_1^2-h_0^2)}{2.303 \log_{10}\left(\frac{r_1}{r_w}\right)} \quad \text{or}$$

$$Q = \frac{\pi k [h_2^2-h_1^2]}{2.306 \log_{10}\left(\frac{r_2}{r_1}\right)} \quad \text{or}$$

$$Q = \frac{\pi k (h_2^2-h_0^2)}{}$$



discharge through section dx.

$$q_x = a_x \cdot \cancel{d}x$$
$$= (2\pi x D) \cdot K \cdot i_x$$
$$= 2\pi x \cdot D \cdot K \cdot \frac{dh}{dx}$$

Total discharge

$$Q \int_{x=r_w}^{x=r_1} \frac{dx}{x} = 2\pi K \cdot D \int_{h=h_0}^{h=h_1} dh$$

$$Q \ln\left(\frac{r_1}{r_w}\right) = 2\pi K \cdot D \; (h_1 - h_0) \qquad (h_1 - h_0 = S)$$

$$Q = \frac{2\pi K D \, (h_1 - h_0)}{2.303 \log_{10}\left(\frac{r_1}{r_w}\right)}$$

or

$$Q = \frac{2\pi K D \, (h_2 - h_0)}{2.303 \log_{10}\left(\frac{r_2}{r_w}\right)}$$

for limits
$x = r_w$ to $x = r_2$
$h = h_{0}$ to $h = h_w$

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

$x = r_1$ to r_2
$h = h_1$ to $h = h_2$

Well loss :

(i) The loss which takes place due to movement of water in well is Well loss.

In general, head loss due to friction is

$$h_f = \frac{4 f L V^2}{2 g D}$$

i.e.

$$S_f = \frac{4 f L \cdot Q^2}{2 g D A^2}$$

$$S_f = K_1 \cdot Q^2 \qquad \text{where } K_1 = \frac{4 f L}{2 g D}$$

Aquifer loss.

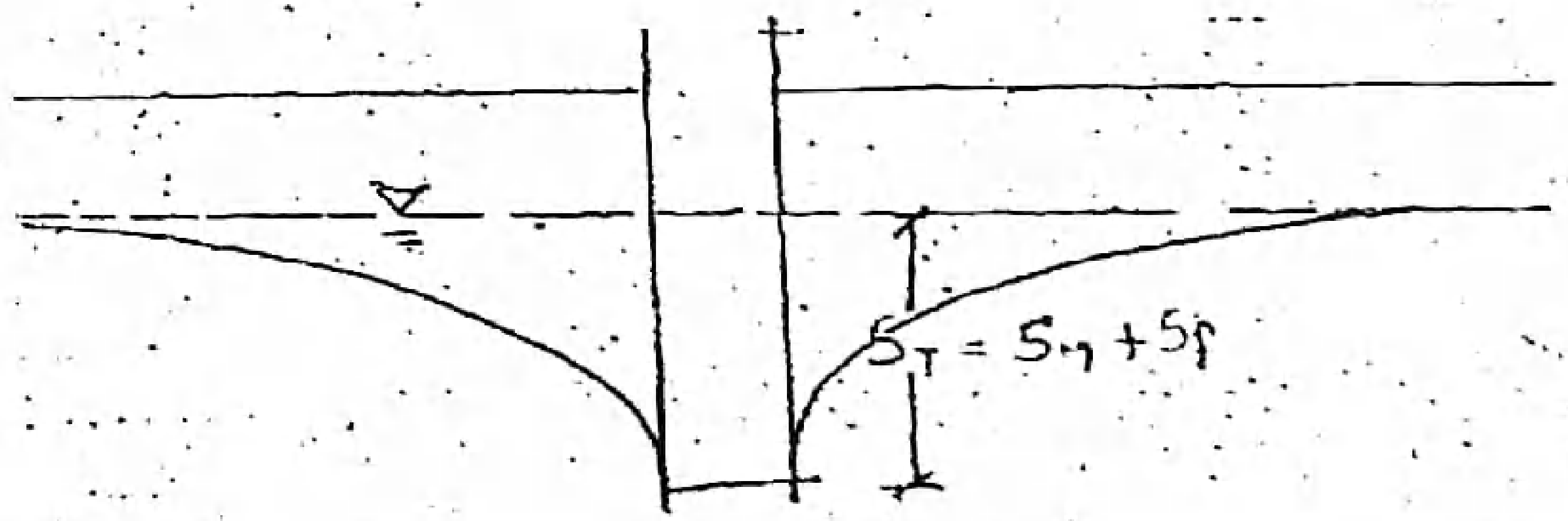
$$S_n = \frac{Q}{CA} = K_2 \cdot Q \quad \text{due to medium}$$

$$K_2 = \frac{1}{CA}$$

$$Q = CA \cdot S$$

Total loss = well loss + aquifer loss

$$S_T = K_1 Q^2 + K_2 Q$$



Specific capacity :

(i) It is defined as rate of flow through the well per unit drawdown that has determined for initial first meter fall ~~of water~~ in well.

(ii) This flow is not same for all the drawdowns.

$$\text{specific capacity} = \frac{Q}{K_1 Q^2 + K_2 Q}$$

Q. A tube well of 15 cm dia. penetrates fully an artisian aquifer 27 m thick. Determine the permeability of aquifer if the steady discharge of 30 Lit/sec is obtained from well under drawdown of 3 m at the well face. Radius of influence is 200 m.

JES 2012

Data :

$D = 27\ m$

dia. of well $r_w = 15/2\ cm = 7.5'$

$Q = 30\ Lit./sec$

$R = 200m$

$S = 3m$

To find : $k = ?$

Solution :

$$Q = \frac{2\pi K D (H - h_o)}{2.303 \log (R/r_w)}$$

By Dupits theory confined aquifer

$$30\times10^{-3}\ m^3/s = \frac{2\pi \times k \times 27 \times 3}{2.303 \log_{10}\left(200/7.5\times10^{-2}\right)}$$

$k = 4.65\times10^{-4}\ m/sec$

A 60 cm dia. well is being pumped at the rate of 1360 Lit/min. Measurements in the nearby test well were made at some time as follows.

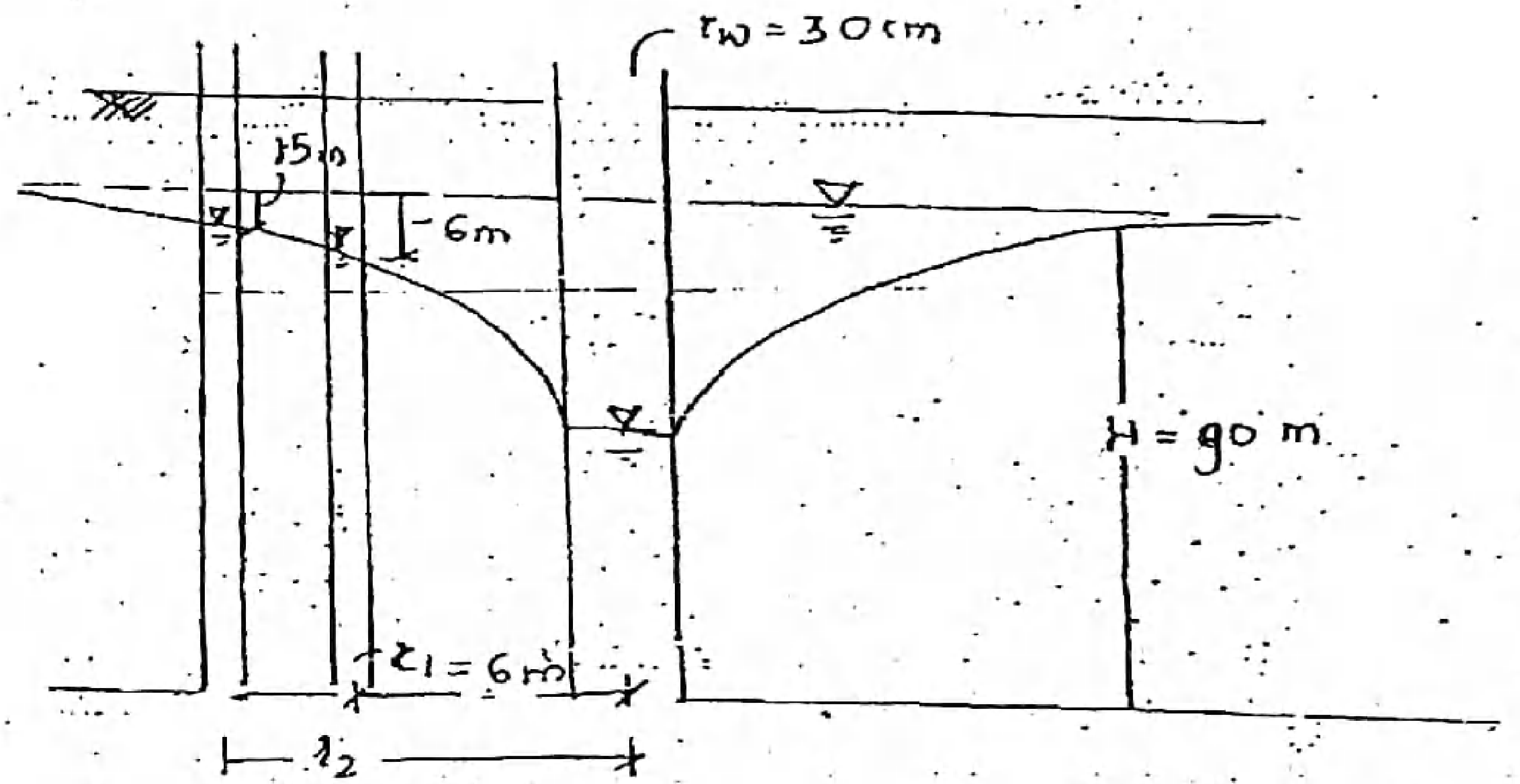
At a distance of 6m from well being pumped drawdown 6m and at 15 m drawdown was 1.5m. The bottom of the well is 90m below the ground water table. Find out coefficient of permeability. If all the observed points were on Dupit's curve, what was the rawdown in well during pumping? What is specific capacity of ell? What is max. rate at which water can be drawn from this ell.

ata : dia. of well = 60 cm ∴ $r_w = 30\ cm$

$Q = 1360\ Lit/min$

$r_1 = 6\,m.$ $S_1 = 6\,m$

$r_2 = 15\,m$ $S_2 = 1.5\,m$



Discharge as per Theims theory in unconfined aquifer.

$$Q = \frac{\pi k (h_2^2 - h_1^2)}{2.306 \log_{10}\left(\frac{r_2}{r_1}\right)}$$

$h_1 = H - S_1 = 90 - 6 = 84$

$h_2 = H - S_2 = 90 - 1.5 = 88.5$

$$\frac{1360\times10^{-3}}{60} = \frac{\pi \times k \times (88.5^2 - 84^2)}{2.303\times\log_{10}\left(\frac{15}{6}\right)}$$

$k = 8.5\times10^{-6}\ m/sec$

To find drawdown (h_o)

$$Q = \frac{\pi k (h_1^2 - h_o^2)}{2.306 \log_{10}\left(\frac{r_1}{r_w}\right)}$$

$$\frac{1360\times10^{-3}}{60} = \frac{\cancel{\pi\times 8.5\times10^{-6}}\,(84^2 - h_o^2)}{2.306\times\log_{10}\left(\frac{6}{0.3}\right)}$$

$2546627.7 = 84^2 - h_o^2$

$h_o = 67.2\ m$

$S = H - h_o$

$= 90 - 67.2$

For specific capacity,

$s = 1. \quad H = 90\ m$

$h_0 = 90 - 1 = 89\ m$

$$\text{specific capacity} = \cancel{\frac{\pi k (H_2^2 - h_0^2)}{2.303 \log_{10}\left(\frac{r_2}{r_w}\right)}}$$

can not be used

$$= \cancel{\pi \times 8.5 \times 10^{-6} \times (88.5^2 -}$$

- Using Dupits theory.

$$SC = \frac{\pi k (H^2 - h_0^2)}{2.306 \log (R/r_w)}$$

To find R use Dupits theory

$$Q = \frac{\pi k (H^2 - h_0^2)}{2.306 \log_{10}\left(\frac{R}{r_w}\right)}$$

$$\frac{1360 \times 10^{-3}}{60} = \frac{\pi \times 8.5 \times 10^{-6} \times (90^2 - 67.2^2)}{2.306 \log_{10} (R/30 \times 10^{-2})}$$

$$0.552 = \frac{1}{\log_{10} (R/30 \times 10^{-2})}$$

$$\log_{10}\left(\frac{R}{30 \times 10^{-2}}\right) = 1.8104$$

$$R = 20.5$$

$$SC = \frac{\pi k \cdot 8.5 \times 10^{-6} \times (90^2 - 89^2)}{2.306 \times \log \left(\frac{20.5}{30 \times 10^{-2}}\right)}$$

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

$$= 67.8\ L/min$$

To find max. discharge. $h_0 = 0$

By Dupits theory

$$Q_{max} = \frac{\pi \cdot k (H^2)}{2.306 \log_{10} (R/r_w)}$$

$$= \frac{\pi \times 8.5 \times 10^{-6} \times 90^2}{2.306 \log (20.5/0.3)}$$

$$= 0.051\ m^3/se$$

Non-equilibrium formula :

(i) As the drawdown in the well is very difficult to be kept constant, i.e. it varies with time non-equilibrium relation is used to find the discharge through well at different drawdown observed at different time measured in observation well.

(ii) As per this relation drawdown ~~at any time t~~ in observation well can be computed.

$$S = \frac{Q}{4\pi T}\left[\ln \frac{4Tt}{r^2 A} - 0.5772\right]$$

where

T - coefficient of transmissibility

r - distance of observation well from test well

A - coefficient of storage

t - time at which drawdown s is observed in the observation well.

For same observation well,

if $s = s_1 \longrightarrow t = t_1$

$s = s_2 \longrightarrow t = t_2$

$$s_1 = \frac{Q}{4\pi T}\left[\ln \frac{4Tt_1}{r^2 A} - 0.5772\right]$$

$$s_2 = \frac{Q}{4\pi T}\left[\ln \frac{4Tt_2}{r^2 A} - 0.5772\right]$$

$$s_2 - s_1 = \frac{Q}{4\pi T}\left(\ln \frac{t_2}{t_1}\right)$$

T same as k i.e coefficient of permeabili

For two observation wells,

for first well $s_1 = s$ & $t = t_1$

for second well $s_2 = s$ & $t = t_2$

for first well, $$s = \frac{Q}{4\pi T}\left[\ln \frac{4Tt_1}{r_1^2 A} - 0.5772\right]$$

for second well, $$s = \frac{Q}{4\pi T}\left[\ln \frac{4Tt_2}{r_2^2 A} - 0.5772\right]$$

$$s-s = \frac{Q}{4\pi T}\left[\ln \frac{t_1}{r_1^2} \cdot \frac{r_2^2}{t_2}\right]$$

$$\ln\left(\frac{t_1}{r_1^2} \cdot \frac{r_2^2}{t_2}\right) = 0$$

$$\frac{t_1}{r_1^2} \cdot \frac{r_2^2}{t_2} = 1$$

$$\frac{t_1}{r_1^2} = \frac{t_2}{r_2^2} \qquad \text{i.e. } \frac{t}{r^2} \text{ is constant.}$$

Water Demand :

(i) The designing of treatment plant is done on the basis of water demand. Hence following quantities must be assessed before designing ~~the treatment~~ plant.

1. Total volume of water required in a year = V
   (annual draft/ demand)
2. Total volume of water required in a day = $\frac{V}{365}$
   (annual average daily draft/ demand)
3. Total volume of water required in a day by each individual = $\frac{V}{365P}$ P - pop^n^ at end of design life.
   (annual average per capita daily draft)

(ii) Total water demand comprises of following demands :

a) Domestic water demand:

(i) It is the amount of water required for all domestic activities viz drinking, cooking, washing, bathing etc.

(ii) For city having full flushing system it varies between 135 - 225 Lit/capita/day. normally taken 200 lpcd.

b) Industrial water demand :

(i) It is amount of water required for all industrial activities in the city.

(ii) It varies between 50 - 450 lpcd.

c) Institutional water demand :

(i) The amount of water required to meet all requirements of institutes of all locality i.e. schools, hospitals, hotels

(ii) It varies between 20 - 50 lpcd.

d) Water for public use :

(i) It is the quantity of water required to meet the publi requirements like cleaning of roads, gardening.

(ii) It is normally taken to be 10 lpcd.

e) Fire demand :

(i) It's the quantity of water required for fire hazards in the city.

(ii) It is taken as 1 lpcd or it may be also computed a

$$Q_{Fire} = 100\sqrt{P} \quad \text{kilo Lit.}$$

→ P in thousands

(as per GOI manual)

Total water demand is addition of all above demand it is taken to 250-350 lpcd.

Factors affecting water demand :

1. Size of city :
   Larger city will include more industrial, institutiona public usage & fire demand.
2. Climate :
   In summer more water will be used.

3. Industrial activity:
More industrial activities more is water demand.

4. Sewerage system:
If sewerage system is water carrying water demand will be more in the city & for conservatory system, it is less

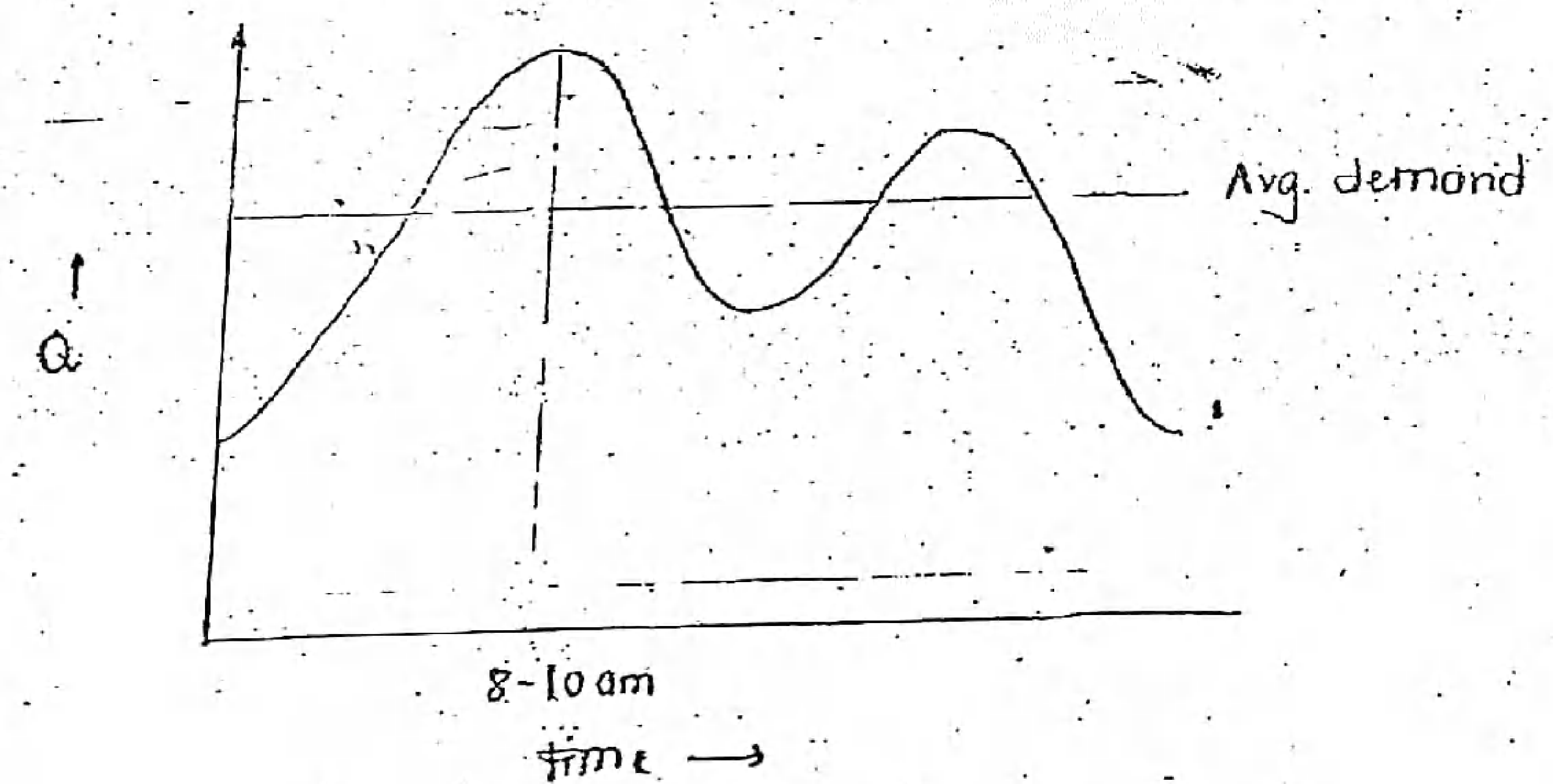
5. Quality:
Better will be the quality more is the water demand.

6. Cost of water:
More cost of water will reduce water demand. i.e. the water will be used more judicially.

7. Type of distribution system:
Wastage of water will be more in continuous system than intermittant water supply system. Thus water demand is more due to more usage in Continuous system.

8. Pressure
More pressure. more losses and thus more water demand.

Fluctuation in water demand:



variation of water demand in a day

Max-daily demand = 1.8 × Avg daily demand.

Max. hourly demand = 1.5 × (Avg. demand of max-day)

$$\text{max. hourly demand} = 1.5\left(1.8 \times \frac{\text{avg daily demand}}{24}\right)$$

$$= 2.7 \text{ (avg. hourly demand of } \text{max- day.)}$$

Note:
In general ratio of max. demand to avg. demand for a particular duration can be calculated using Good Rich equation

$$P = \frac{\text{max. demand}}{\text{avg. demand}} = 180 \cdot t^{-10}$$

where t - time duration in days.

6 5

8 ~~$\bar{x} = 12.9$~~ 68 $\bar{x} = 36.5$ $\bar{\sigma} = 3$

52 ~~$\bar{\sigma} = 100.06$~~

63

$$\bar{\sigma} = \sqrt{\frac{(x - x_i)^2}{n}}$$

Coincident draft/ demand:
It is max. of the max. hourly demand or aggregate of max. daily demand and fire demand.

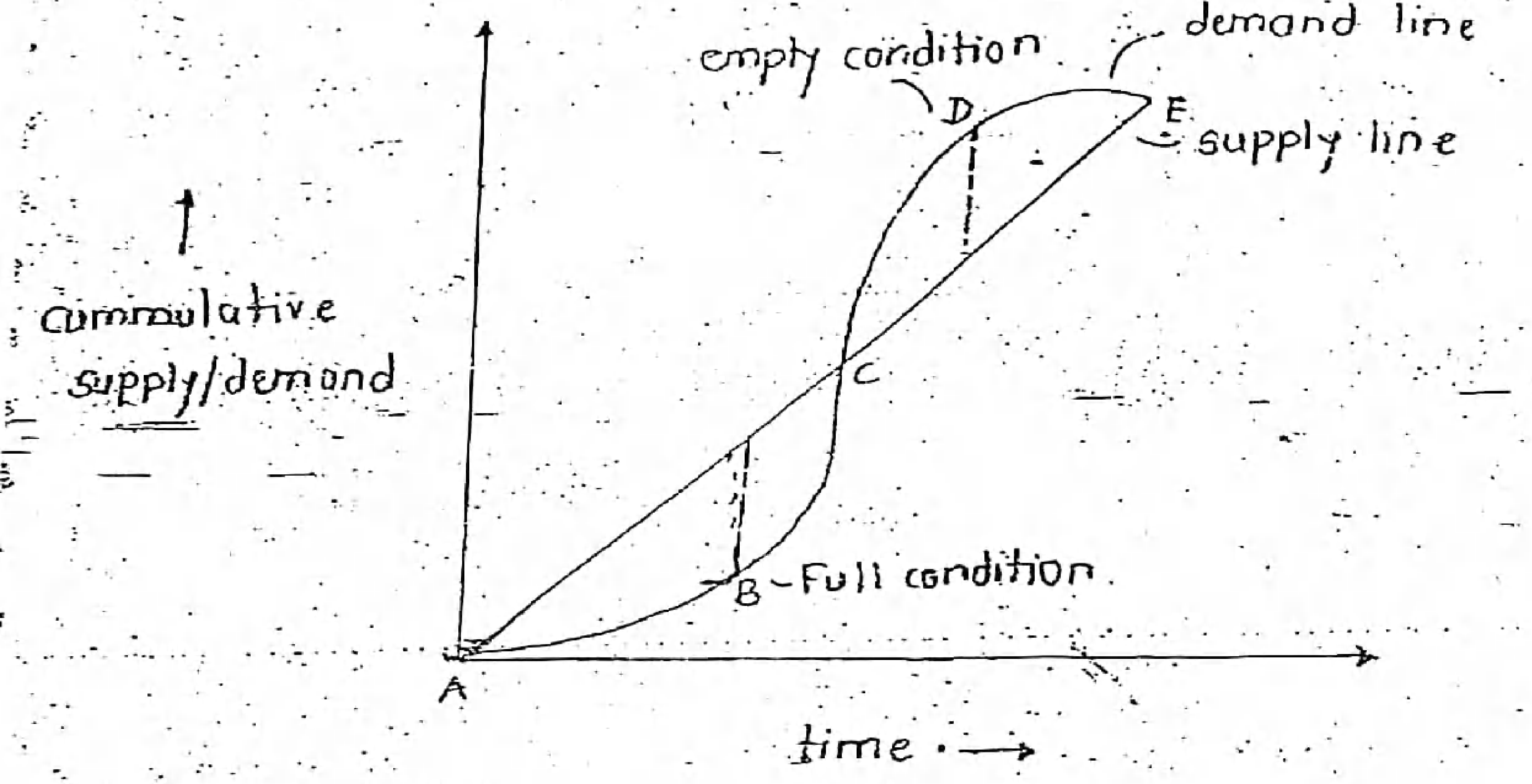
Design life and design discharge of treatment units:

Component	Design life	Design discharge
source-dam/reservoir	50	Q_{MD}
pumping mains (before treatment)	30	Q_{MD}
pumps	15	Q_{MD}^*
Water treatment plant	15	Q_{MD}
Gravity mains (after treatment)	30	Q_{MH}
service reservoirs	15	Q = (Balancing storage + fire + emergency storage)
Distribution system	30	Q

Storage capacity of Service Reservoir :

(i) The designing of service reservoir is done for the storage of water required in meeting following demads.

- Balancing storage
- Fire demand
- Emergency storage.



(i) Initially rate of supply is more than rate of demand (from slope) which means water is stored in reservoir (accumulated)

(ii) The max. accumulation occurs at point B (full reservoir).

(iii) After point B, slope of demand line is increasing i.e. demand rate is more than rate of supply - water level is depleting.

(iv) At point C. the quantity of water supplied is equal to water demand quantity. still water level is depleting beyond this pt.

(v) A point D signifies the empty condition of reservoir

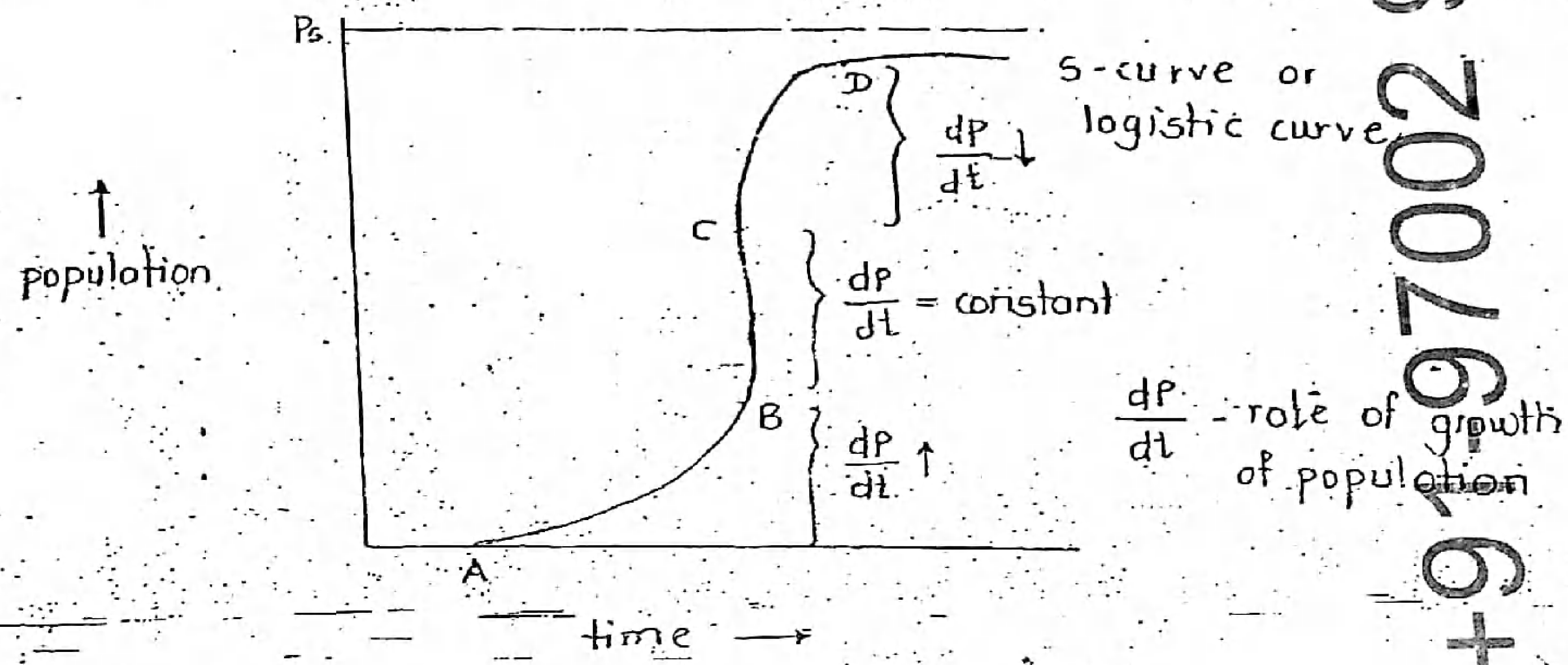
(vi) At point E water level reaches initial water level in the the reservoir.

Balancing storage = A + B

A - max. diff bet^n cum. demand

29th July

Population forecasting :-



1. Arithmetic increase method :

(i) In this method rate of growth of population is assumed to be constant i.e. for reagion BC.

$$\frac{dP}{dt} = \text{constant}$$

(ii) If $\bar{x}$ is the average increase in the population over the given duration

At $t = t_0$ — $P = P_0$

$t = t_1$ — $P = P_0 + \bar{x}$

$t = t_2$ — $P = P_0 + 2\bar{x}$

After n years, $P_n = P_{n-1} + \bar{x} = P_0 + n\bar{x}$

2. Geometric increase method (compound / uniform increase meth

(i) In this method, rate of growth of population is assume constant but population is compounded for this given (r to compute population in future.

(ii) If r% is constant rate of compounding.

At $t = t_0$ $\quad P = P_0$

$t = t_1$ $\quad P_1 = P_0 + P_0 \cdot r = P_0(1+r)$

$t = t_2$ $\quad P_2 = P_1 + P_1 \cdot r = P_1(1+r) = P_0(1+r)^2$

After n years, $t = t_n$ $\quad P_n = P_{n-1} + P_{n-1} \cdot r$

$$= P_{n-1}(1+r) = P_0(1+r)^n$$

To find r (arithmatic average/mean)

$$r\% = \frac{r_1 + r_2 + r_3 + r_4 + \dots r_n}{n}$$

Geometric mean :

$$r\% = \sqrt[n]{r_1 \cdot r_2 \cdot r_3 \cdots r_n}$$

AM > GM

Since arithmatic mean is more than geometric mean, it is advisible to compute the rate of growth by Arithmatic mean method to be on safer side.

3. Increamental increase method:

(i) In this method, rate of growth of population is not assumed to be constant. Rate of growth of population may increase or decrease

(ii) In this method, avg. increamental increase in increase of population is also considered

After n years,

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

$\bar{x}$ - avg. increase in population

$\bar{y}$ - avg. increamental increase in population

At $t = t_0$ $\quad P = P_0$

$t = t_1$ $\quad P_1 = P_0 + \bar{x}$

$t = t_2$ $\quad P_2 = P_0 + 2\bar{x} + \bar{y}$

$$P_3 = (P_0 + 2\bar{x} + \bar{y}) + \bar{x} + \bar{y} = P_0 + 3\bar{x} + 2\bar{y}$$

Note:

(i) Population forecasted by geometric increase method is max. in comparison to that comupuled by arithmatic or increament increase method.

(ii) Population forecasted with arithmatic increase method is minimum in comparison to geometric increase method & increamental increase method.

(iii) G.O.I. manual recommends geometric increase method for forecasting method.

(iv) Geometric increase method is generally recommeded for young cities and arithmatic increase method for old ones.

4. logistic method:

Population after n years,

$$P(t) = \frac{P_s}{1 + m\, l_n^{-1}(nt)} \qquad \text{(equ}^n\text{ of curve)}$$

where, m, n - constants.

P_s - saturation population.

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

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

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

At $t = t_0$ $\quad P = P_0$

$t = t_1$ $\quad P = P_1$

Q Using the given information find the population of city in 2010 using Geometric increase, Arithmatic increase and Increamental increase method.

Year	Population (in thousands)
2000	26
2001	29
2002	35
2003	43
2004	47

(i) Arithmatic increase method.

~~Year~~	Pop^n (thousands)	Increase
2000	26	3
2001	29	6
2002	35	8
2003	43	4
2004	47	
		$\bar{x} = 5.25$

Population in 2010

$P_{2010} = P_{2004} + 6\bar{x}$

$= 47.000 + 6\ (5250)$

$= 78.900$

(ii) Geometric increase method.

Year	Pop^n (thous.)	Increase	Growth rate
2000	26	3	$3/26 \times 100 = 11.5$
2001	29	6	$6/29 \times 100 = 20.68$
2002	35	8	$8/35 \times 100 = 22.8$
2003	43	4	$4/43 \times 100 = 9.3$
→ 2004	47		
			$\bar{r} = 16.07\%$ after 2004

Population in 2010

$P_{2010} = P^{*}_{2004} + (1+r)^{c}$

(iii) Increamental increase method:

Year	Pop^n (thousand)	Increase	Increase in Increase
2000	26	3	
2001	29	6	3
2002	35	8	2
2003	43	4	-4
2004	47		
			$\bar{y} = 0.33$

population in 2010.

$P_{2010} = P_{2004} + 6\bar{x} + \frac{6(6+1)}{2}\bar{y}$

$= 47 + 6 \times (5.25) + 21\ (0.33)$

$= 85.43$

Different type of volves in pipes:

1. Gate / Sluice valve :-

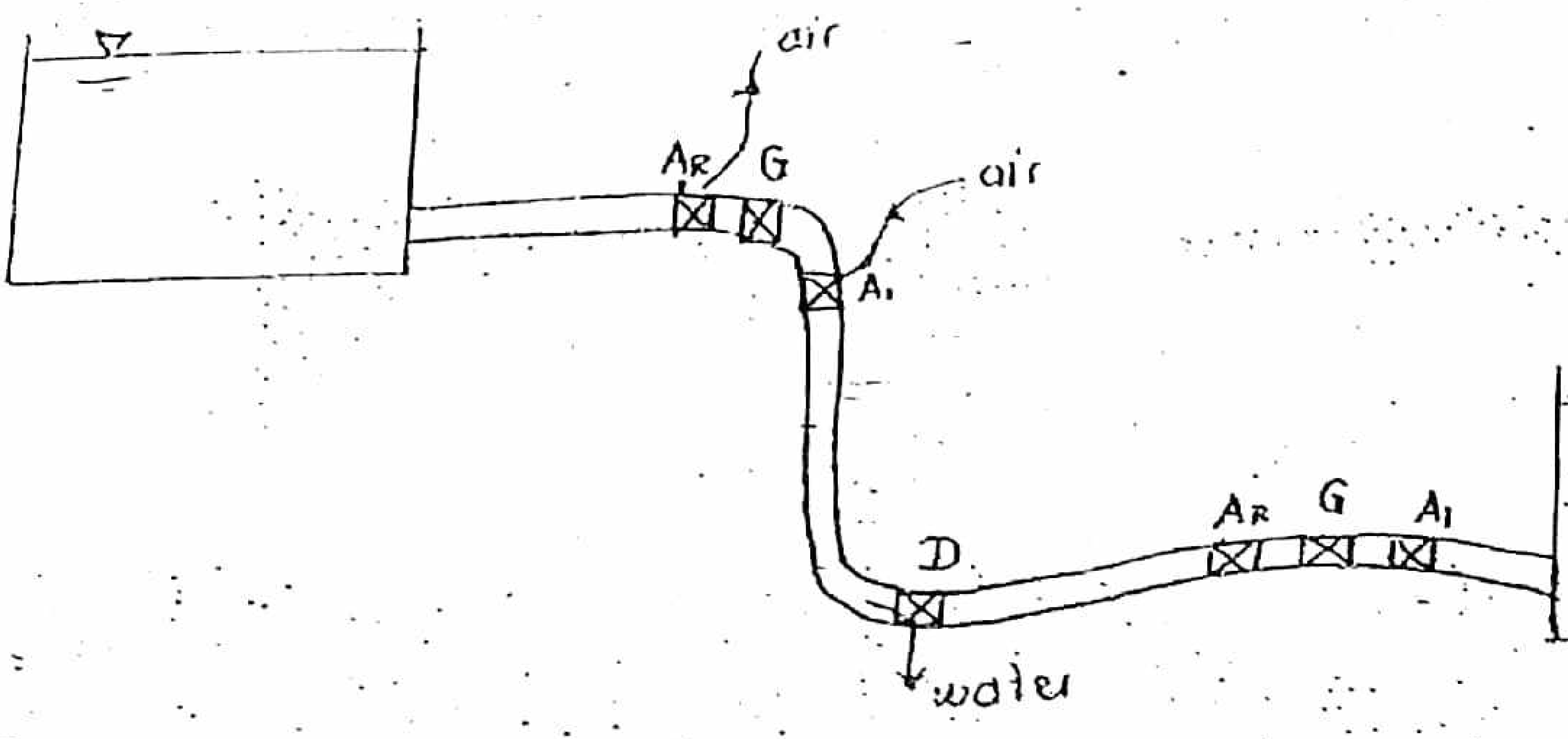
(i) This volve is used to regulate flow of water in pipe by dividing it into the number of sections.

(ii) These volves are generally used at summit point of the pipe system as at summit point pressure available in water system is minimum, hence strength of material required for construction of these volves can be subsidized.

2. Air volves.

- Air inlet
- Air relief

(i) Air volves are used in pipe for safety of pipe. These are generally placed at summit points on either sides of gate volves.



3. Drain / Scour off valves:

(i) These valves are provided to remove the water out of pipe system in case of activities like maintainance of pipes.

(ii) These valves are generally provided on lowermost point in pipe system in order to ensure gravity drainage of water.

Reflux valves or Non-return valves or check valves:

(i) These valves are used to prevent the back flow of water in reverse direction. These are generally used on the delievery side of the pumps.