

IMPORTANT FORMULAS ON ENVIRONMENTAL ENGINEERING

Water Demand

A. WATER DEMAND AND POPULATION FORECASTING

1. Population forecasting Methods

(i) Arithmetic increase method

$$P_n = P_o + n \times \bar{x}$$

Where,

P_n = Prospective or forecasted Population after n decades from the present (i.e., last known census)

P_o = Population at present (i.e., last known census)

n = Number of decades between now & future.

\bar{x} = Average (arithmetic mean) of population increases in the known decades.

(ii) Geometric Increase Method

$$P_n = P_o \left(1 + \frac{r}{100} \right)^n$$

where,

P_o = Initial Population.

P_n = Future Population after 'n' decades.

r = Assumed growth rate (%).

$$r = \sqrt[t]{\frac{P_2}{P_1}} - 1$$

where,

P_2 = Final known Population

P_1 = Initial known Population

t = Number of decades (period) between P_1 and P_2

$$r = \sqrt[t]{r_1 r_2 \dots r_t}$$

(iii) Incremental Increases Method

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

Where,

\bar{x} = Average increase of Population of known decades

\bar{y} = Average incremental increases of the known decades.

(iv) Decreasing rate of growth method

- In this method, the average decrease in the percentage increase is calculated and then subtracted from the latest percentage increase for each successive decade.
- However, this method is applicable only in cases where the rate of growth shows a downward trend.

(v) Logistic Curve Method

$$\log_e \left[\frac{P_s - P}{P} \right] - \log_e \left[\frac{P_s - P_0}{P_0} \right] = kP_s t$$

Where,

P_0 = Population of the start point.

P_s = Saturation population

P = Population at any time t from the origin.

K = Constant.

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

$$P_{sat} = \frac{2 \times (P_0 P_1 P_2) - P_1^2 (P_0 + P_2)}{P_0 P_2 - P_1^2}$$

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

$$n = \frac{1}{t_1} \log_e \frac{P_0 (P_{sat} - P_1)}{P_1 (P_{sat} - P_0)}$$

B. Water Demands

1. Fire Demand

➤ **Kutch ling's formula**

$$Q = 3182\sqrt{P}$$

Q = Quantity of water in litres per minute

P = Population in thousands

➤ **Buston formula:**

$$Q = 5663\sqrt{P}$$

Q = Quantity of water in litres per minute

P = Population in thousands

➤ **Freeman's formula:**

$$Q = 1136 \left(\frac{P}{5} + 10 \right)$$

$$F = 2.8\sqrt{P}$$

F = Number of simultaneous fire streams.

Q = Quantity of water in litres per minute

P = Population in thousands

➤ **National Board of Fire under Writers formula:**

$$Q = 4637\sqrt{P}(1 - 0.01\sqrt{P})$$

Q = Quantity of water in litres per minute

P = Population in thousands

Per capita Demand:

➤ Average Daily per Capita Demand = $\frac{\text{Quantity Required in 12 Months}}{(365 \times \text{Population})}$

➤ Maximum daily Demand = 1.8 × average daily demand

➤ Maximum hourly Demand of maximum day, i.e. Peak demand
= 2.7 × annual average hourly demand

Note:

- We will design a raw water scheme on the Demand, which has maximum fluctuation.
- To be economical, we will design a raw water scheme taking a Maximum of {Maximum Hourly Demand and Coincident Demand}.
- Coincident Demand = Maximum Daily Demand + Fire Demand

Water consumption or demands for Various Purposes:

| S.No | Types of Consumption | Normal Range (lit/capita/day) | Average | % |
|------|-----------------------------------|-------------------------------|---------|-----------|
| 1. | Domestic Consumption | 65-300 | 160 | 35 |
| 2. | Industrial and Commercial Demand | 45-450 | 135 | 30 |
| 3. | Public Uses including Fire Demand | 20-90 | 45 | 10 |
| 4. | Losses and Waste | 45-150 | 62 | 25 |

Raw Water Quality Parameters

1. CHARACTERISTICS OF WATER

1.1. Qualities of Raw Water

The parameters which help in ascertaining qualities and properties of raw water are referred as water quality parameters

i. Physical Water Quality Parameters

a. Suspended solids

- Dissolved solids (DS) = Total solids (TS) – suspended (SS)
- Acceptable limit of total solid = 500 mg/lit

b. Turbidity

- Turbidity is measured by
 - Turbidity rod -It is used for measuring the turbidity of water in the field.

- Jacksons Turbidimeter – It is used to measure turbidity when it is more than 100 p.p.m.
- Bali's Turbidimeter- – It is used to measure the turbidity of the sample when it is less than five units.

➤ Drinking water should not have a turbidity of more than 10 N.T.U

c. Colour and Temperature

- The permissible colour for domestic water is 20 ppm on the platinum cobalt scale.
- The most desirable Temperature for public supply is about 20°C. The Temperature above 35°C is unfit for public supply because it is not palatable.

d. Taste and odour

➤ **Threshold odour number**

$$\text{T.O.N} = \frac{\text{Final volume at which odour is hardly detectable}}{\text{Sample volume}}$$

where TON = Threshold odour number

$$1 \leq \text{TON} \leq 3$$

ii. Chemical characteristics

a. PH value of water

- $\text{pH} = -\log_{10}[\text{H}^+]$
- $\text{pH} + \text{pOH} = 14$
- $\text{pH} < 7$: Acidic
- $\text{pH} = 7$: Neutral
- $\text{pH} > 7$: Basic
- pH is measured by potentiometer
- **p(H) =7 to 8.5 is acceptable limit**
- **p(H) < 6.5 and p(H) > 9.2 is cause for rejection.**

iii. Total Dissolved Solids (TDS)

- Electrical conductivity in $\mu\text{Mho/cm}$ at 25°C) $\times 0.65 =$ dissolved solid content (mg/l).
- Source of Total dissolved solids
 - Major source- Na, Ca, Mg, HCO_3^- , SO_4^{2-} Cl^-
 - Minor source- Fe, K, CO_3^{2-} , NO_3^- , Fluoride Boron, silica
- According to the GOI manual, the acceptable limit of TDS (mg/l) is 500, and the cause for rejection is 2000.

iv. Alkalinity

➤ Major sources

CO_3^{2-} , HCO_3^- , OH^- .

- Minor sources

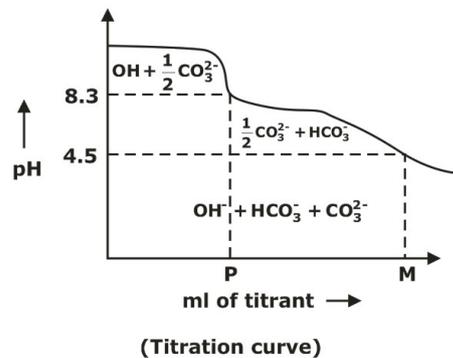


- Alkalinity measurements are done by titrating the water with acid and determining the hydrogen equivalent of alkalinity, and it is expressed in terms of mg/l as CaCO₃.

Note:

- Equivalent weight = $\frac{\text{Molecular weight}}{\text{Valency}}$
- Gram equivalent = $\frac{\text{Weight in gram}}{\text{Equivalent weight}}$
- Equivalent wt of CaCO₃ = $\frac{\text{Molecular wt}}{\text{Valency}} = \frac{100}{2} = 50$
- 1 ml of 0.02N H₂SO₄ ≡ 1 m of alkalinity expressed as CaCO₃

Relative Quantity of Alkalinity Species are pH-Dependent



- The reaction of alkalinity with hydrogen ions is shown below.
 $\text{H}^+ + \text{OH}^- = \text{H}_2\text{O}$
 $\text{CO}_3^{2-} + \text{H}^+ = \text{HCO}_3^-$
 $\text{HCO}_3^- + \text{H}^+ = \text{H}_2\text{CO}_3$
- If P = M all alkalinity is caustic alkalinity.
- If P = $\frac{M}{2}$, all alkalinity is carbonate alkalinity
- If P < $\frac{M}{2}$, predominant species are carbonate and bicarbonate
- If P > $\frac{M}{2}$, predominant species are carbonate and hydroxide
- If P = 0, total alkalinity is bicarbonate alkalinity.

v. Nitrogen Content

- Free ammonia (should not be more than 0.15 mg/l.) → indicates recent pollution
- Organic ammonia (Albuminoid) (should not be more than 0.3 mg/l) → indicates the Quantity of nitrogen before decomposition has started.
- Nitrite → indicates a partly decomposed condition
- Nitrate (should not be more than 45 mg/l) → indicates old pollution (fully oxidized)

➤ **Organic ammonia Free ammonia + organic ammonia = Kjeldahl Nitrogen Ammonia.**

- Nitrite is highly dangerous hence its permissible limit is zero.
- The colour for nitrite is developed by sulphonic acid + Napthamine
- Nitrate causes blue baby disease or Mathemoglobinemia.

vi. Hardness

➤ Total Hardness = $\frac{[Mg^{2+}] \text{ mg / l}}{\text{eq. wt of mg}} \times \text{eq. Wt of CaO}_3$

+ $\frac{[Ca^{2+}] \text{ mg / l}}{\text{eq. wt of Ca}} \times \text{eq. wt. of CaCO}_3$

- Carbonate Hardness = Minimum of(Alkalinity, Total hardness)
- Non – Carbonate Hardness = Total hardness – Alkalinity
- Acceptable limit of total hardness = 200 mg/l and cause for rejection = 600 mg/l

1.2. WATERBORNE DISEASES

- Bacteria
 - Typhoid fever (salmonella bacteria typhi)
 - Cholera (Vibrio cholerae bacteria)
 - Bacillary Dysentery (Sonnebacillus)
- Virus
 - Jaundice (Hepatitis virus)
 - Poliomyelitis or Polio (**poliovirus**)
- Protozoa
 - Amoebic dysentery

| Parameter | Desirable-Tolerable | If no alternative source is available, limit extended upto |
|---|---------------------|--|
| Physical | | |
| [Turbidity (NTU unit) | < 10 | 25 |
| Colour (Hazen scale) | < 10 | 50 |
| Taste and Odour | Un-objectionable | Un-objectionable |
| Chemical | | |
| pH | 7.0-8.5 | 6.5-9.2 |
| Total Dissolved Solids mg/l | 500-1500 | 3000 |
| Total Hardness mg/l (as CaCO ₃) | 200-300 | 600 |
| Chlorides mg/l (as Cl) | 200-250 | 1000 |
| Sulphates mg/l (as SO ₄) | 150-200 | 400 |
| Fluorides mg/l (as F) | 0.6-1.2 | 1.5 |
| Nitrates mg/l (as NO ₃) | 45 | 45 |
| Calcium mg/l (as Ca) | 75 | 200 |
| Iron mg/l (as Fe) | 0.1-0.3 | 1.0 |

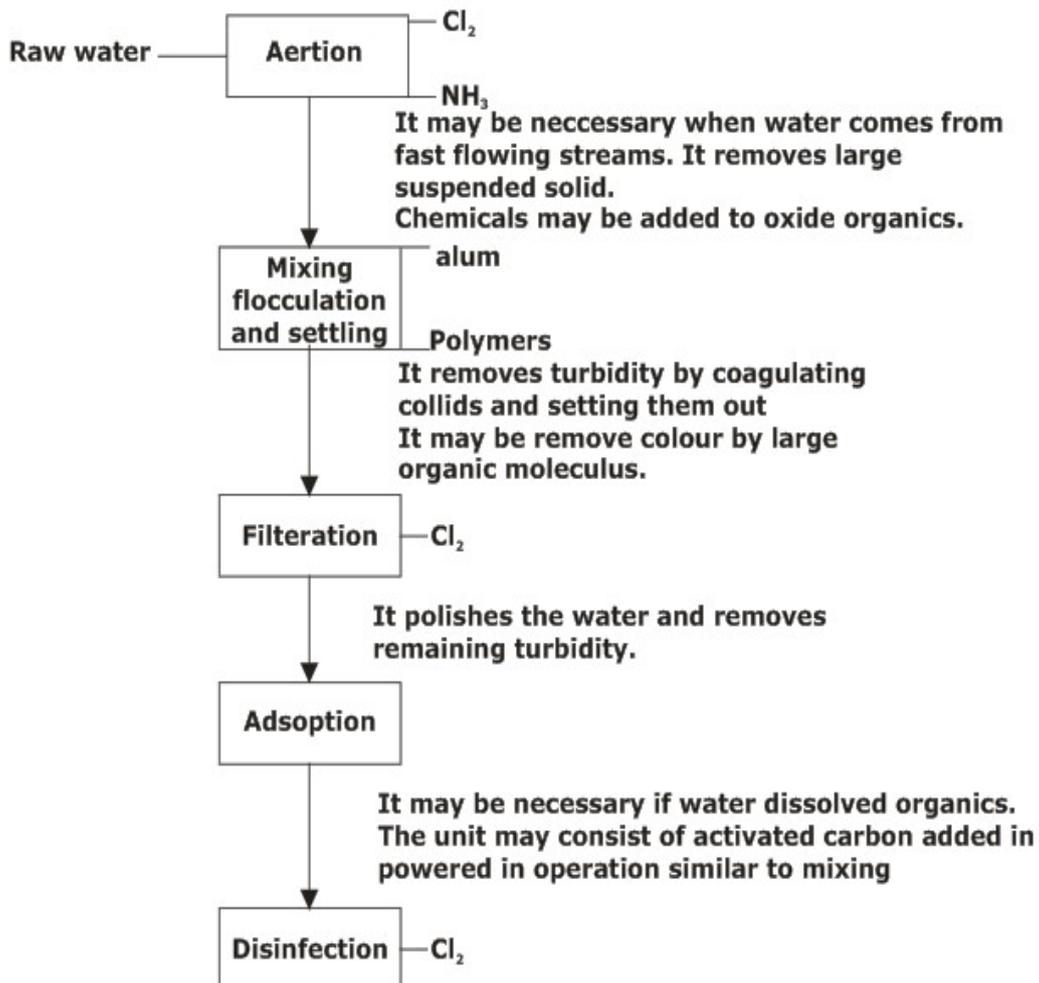
Biological WQP

Testing of Coliforms is done

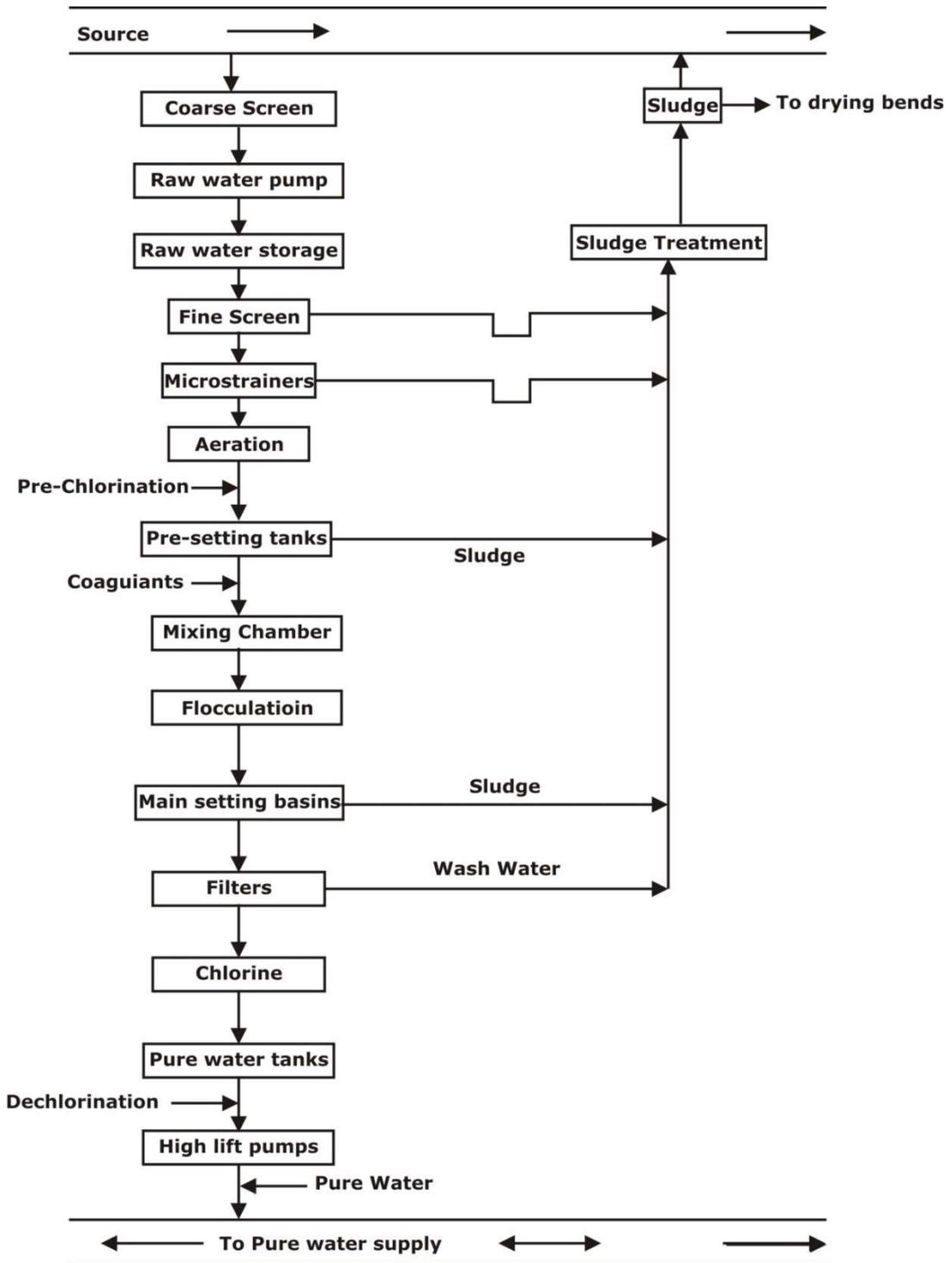
| Tests/Technique | Remarks |
|------------------------------|---|
| 1. MPN Test | i. Multiple tube Fermentation tests ii. Nutrient used: Lactose broth iii. More is the dilution of sample lesser is the possibility of getting +ve test. |
| 2. Membrane Filter Technique | i. Nutrient: M-Endo Medium ii. Coliform colonies are counted |
| 3. Coliform Index Test | Reciprocal of the smallest quantity of sample giving +ve B-coil test |

Water Treatment Plant

1. Flow of treatment

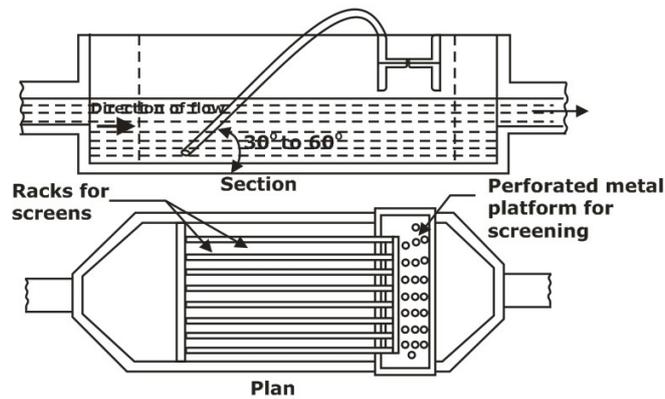


1.1. Block Diagram of various Treatment Stages



2. SCREENING

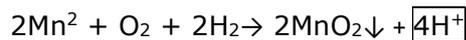
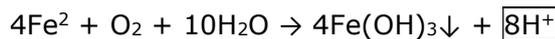
- These are classified as coarse screen and fine screen.
- Coarse screens are in the form of bars spaced at 20-100 mm c/c.
- It is kept inclined 36 V: 1 H for increasing flow area, reducing velocity.
- The coarse screen is in the form of wire mesh with an opening of less than 10 mm.



Fixed Bar Type Coarse or Medium Screen

3. AERATION

- Water is brought in intimate contact with air.
- It is used to remove undesirable gases dissolved in water like CO₂, H₂S.
- It is also used to add oxygen to oxidize undesirable substances like oils, a decomposing product of algae, etc.
- It removes iron and manganese. Iron and manganese are soluble in Fe²⁺ and Mn²⁺ form. They are oxidized to Fe(OH)₃ and MnO₂, which precipitates.



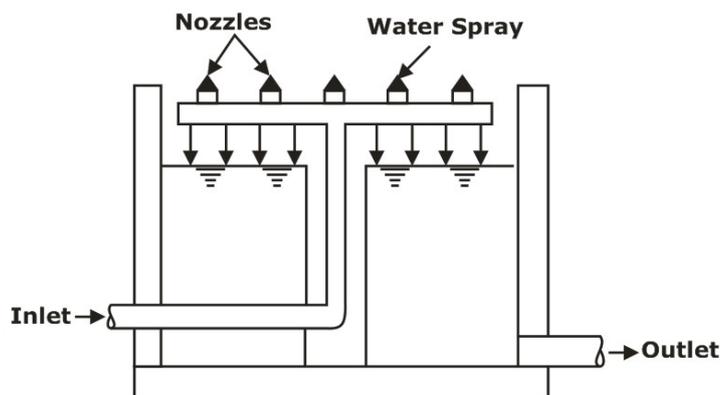
- This processes increases the acidity of water.

3.1. Gravity Aerator or Tray Tower or Trickling Bed

- It is used mostly for Fe and Mn removal.
- This method is one of the most efficient methods for the removal of CO₂.

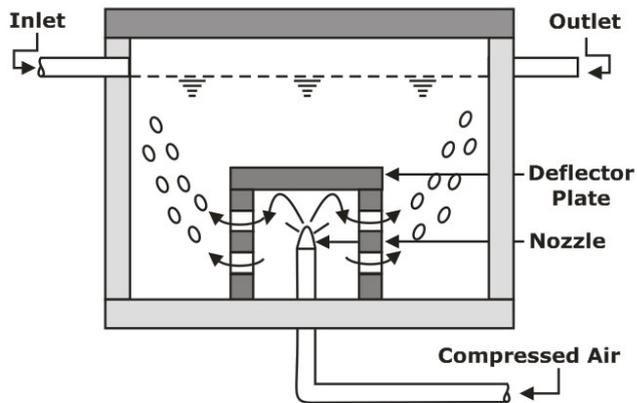
3.2. Spray Tower or Nozzle

- Nozzle type aerators are very efficient and are commonly used in the removal of carbon dioxide and iron.
- The nozzle is 2.5 to 4 cm dia and discharges about 5 to 10 l/sec.
- To aerate 10,000 m³/day, about 20 nozzles are arranged within an area of 25 m².
- Removes 90% CO₂ and 99% H₂S.



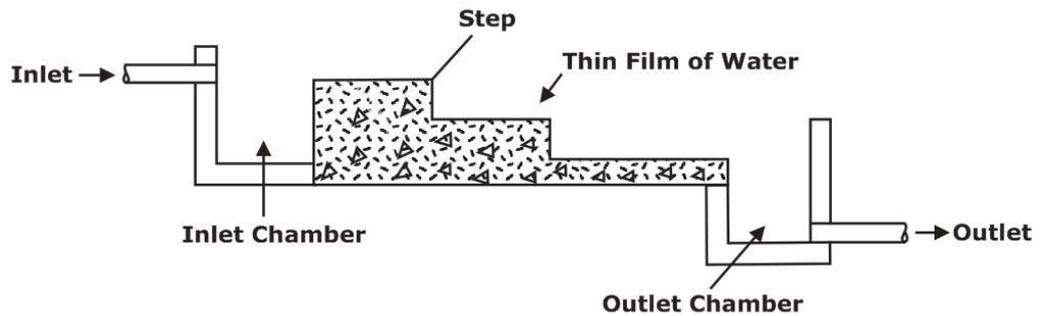
3.3. Air Diffuser

- Aeration tanks are commonly about 4 m deep and have a retention time of about 15 minutes.
- Water absorbs oxygen from compressed air, and colour, odour and taste are removed.



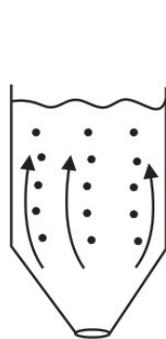
3.4. Cascade Aerator

- Cascade type aerators depend on the turbulence created in a thin stream of water flowing swiftly down an incline and impinging against fixed upstate.
- Removes 20-45% CO₂ and 35% H₂S.
- Cheap compared to others.

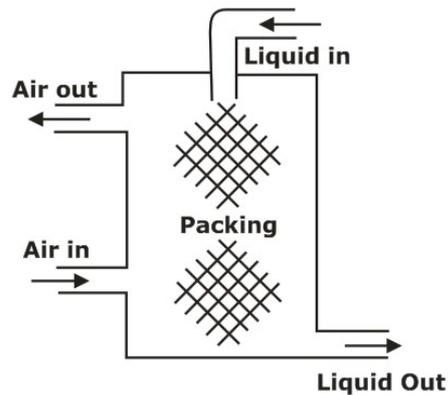


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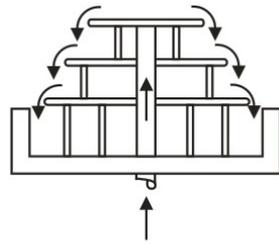
Various Water Treatment Aeration Devices



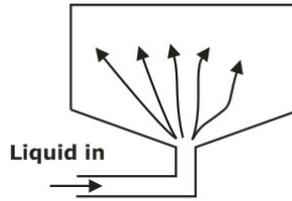
(a) Gas Dispenser



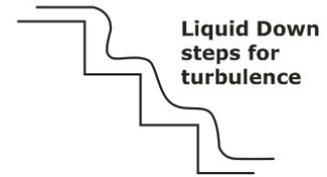
(b) Packed Column



(c) Multiple Platform Aerator



(d) Spray Aerator



(e) Cascade Aerator

4. Sedimentation

- It is a natural process by which solids with higher density than the fluid in which they are suspended, settling under gravity's action.
- The purpose of sedimentation tank is to remove suspended solids.
- Sedimentation is classified into two categories:
 - Plain sedimentation, and
 - Sedimentation with coagulation
- Settling velocity will be calculated by stoke's law unless and otherwise given.

- $V_s = \frac{(G - 1)\gamma_w d^2}{18\mu}$... applicable for $Re < 1$ (stoke's law)

- $V_s = 418 \times (G - 1) \times d^2 \times \left[\frac{3T + 70}{100} \right]$ applicable for $Re < 1$ (stoke's law)

G= specific gravity of the particle

V_s = Settling velocity

d= diameter of particle

μ = Dynamic viscosity

T = Temperature of water in °C

- $V_s = \left[\frac{\frac{4}{3}gd(G-1)}{C_D} \right]^{1/2}$

- $C_D = 0.4$ for $Re \geq 10^4$

- $C_D = \frac{24}{Re}$ for $Re \leq 0.5$

- $C_D = \frac{24}{Re} + \frac{3}{\sqrt{Re}} + 0.34$ for $0.5 \leq Re \leq 10^4$

- $V_s = \left[\frac{\frac{4}{3}gd(G-1)}{C_D} \right]^{1/2}$

- $V_s = 1.8\sqrt{gd(G-1)}$ for $d \geq 0.1$ mm

- $V_s = 418 \times (G - 1) \times d \times \left[\frac{3T + 70}{100} \right]$ for $0.1 \leq d \leq 1$ mm

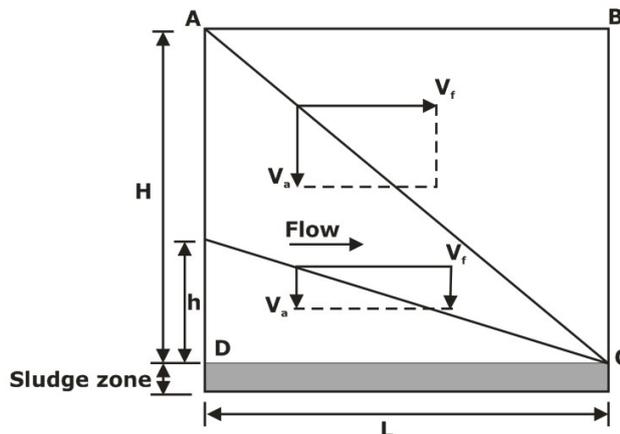
- Sedimentation Tanks are classified as Quiescent type (fill and draw-type) and continuous type.
- The quiescent type tank will have detention time = 24 hr and period of cleaning = 8-12 hr.
- Three nos of the Tank are required in quiescent type. The Tank will be designed for max daily flow and max daily flow = 1.8 x avg. Daily flow.

4.1. Continuous Flow Type

- a) Horizontal flow tank → Rectangular.
- b) Vertical flow tank → Circular.

4.1.1. HORIZONTAL FLOW TANK

- The design is based on the assumption that:
 - the Concentration of suspended particles of each size is the same at all points of the vertical cross-section at the inlet end.
 - A particle is removed when it reaches the bottom of the settling zone.



- Time of horizontal flow

$$\text{Time of horizontal flow} = \frac{\text{Length of tank}}{\text{Velocity of flow}} = \frac{L}{V_f}$$

- The velocity of flow (V_f)

$$V_f = \frac{Q}{BH}$$

- Time of horizontal flow or (Detention time)

$$\text{Time of horizontal flow or (Detention time)} = \frac{L}{Q/BH} = \frac{LBH}{Q} = \frac{\text{Volume of tank}}{\text{Discharge}}$$

- Time of falling through height = $\frac{H}{V_s}$

- If a particle of settling velocity V₂ is introduced at the topmost inlet point, it will be assumed to be removed if the time of falling through H is detention time.

$$\Rightarrow \frac{H}{V_s} = \frac{LBH}{Q} \Rightarrow \boxed{V_s = \frac{Q}{BL}}$$

- This Quantity $V_s = \frac{Q}{BL}$ is called overflow rate.

- The overflow rate for plain sedimentation is 12000 to 18000 lit/m²/day.

- The overflow rate for sedimentation with coagulation is 24000 to 30000 lit/m²/day.

- Particles with a size greater than the particle for which settling velocity equals the overflow rate are 100% removed.
- A particle having a settling velocity less than the overflow rate will not get 100% removed.
- The percentage removal of these particles will be given by $\left(\frac{h}{H}\right)$

$$\frac{h}{V_s} = \text{Detention time} = \frac{H}{\text{overflow rate}} = \frac{H}{V_s}$$

$$\Rightarrow \frac{h}{H} = \frac{V_s}{V_s}$$

- The percentage removal of a particle having settling velocity $V_s = \frac{V_s}{V_s} \times 100$

Note:

- If the overflow rate is increased, the efficiency of the sedimentation tank is decreased and vice versa.
- Theoretically, depth does not have any effect on the efficiency of the Tank.
- For tube settlers detention time is < or = 10 minutes.

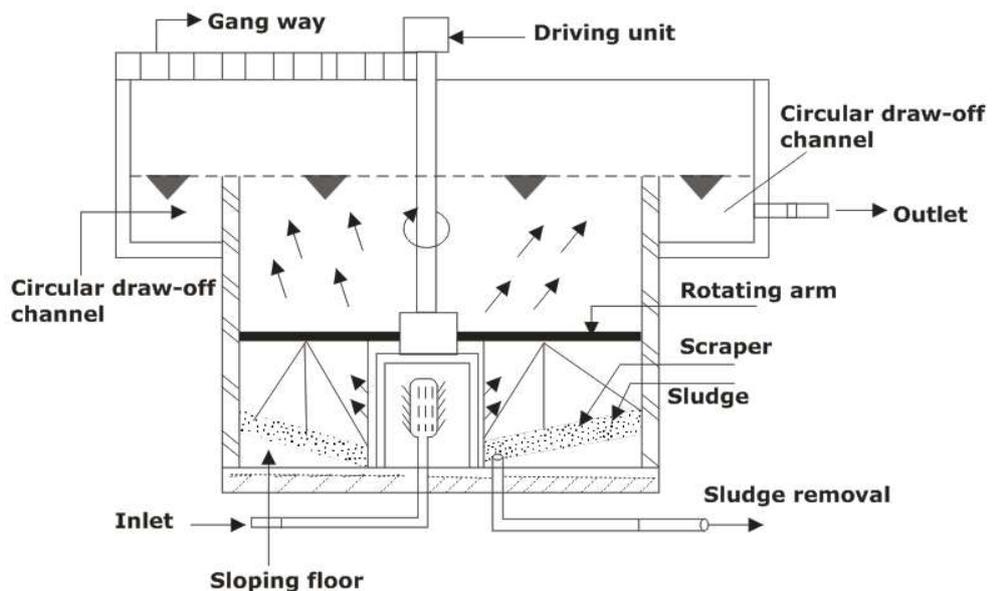
SHORT-CIRCUITING IN SEDIMENTATION TANK

- The degree of short-circuiting is the derivation of the actual flow pattern to the ideal flow pattern. Hence, Displacement efficiency

$$(n_d) = \frac{\text{Flowthrough period}}{\text{Theoretical Detention Time}} \times 100$$

- Generally $(n_d) > 30\%$

4.1.2. CIRCULAR SEDIMENTATION TANK



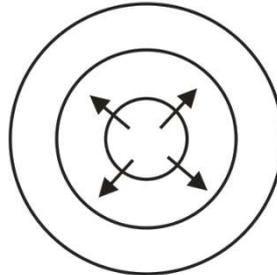
- In a circular tank, the horizontal flow velocity of water continuously decreases at a distance from the centre increases.

- Hence particle path will be parabolic as opposed to the straight-line path in the case of the horizontal flow tank.
- Volume of circular tank is given by

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

$$Q \times t_d = \text{volume}$$

$$\text{Over flow rate} = \frac{Q}{\pi d^2}$$



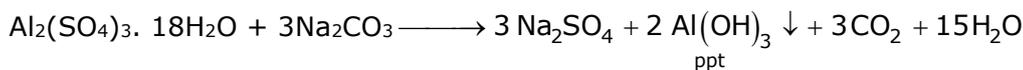
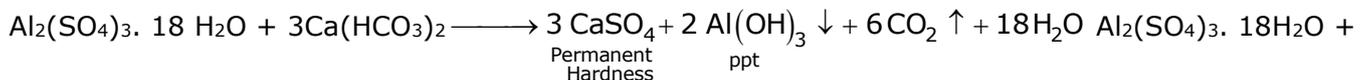
- Weir loading Rate = $\frac{Q}{\pi D}$
- weir loading rate affects lighter particle (flocs) clarification.
- Weir loading Rate is normally taken as 300 m³/d/m, but when the Tank is property design, its value goes up to 1500 m³/d/m

5. SEDIMENTATION WITH COAGULATION

The common coagulants added in water:

(a) Alum

- The chemical formula of alum is Al₂(SO₄)₃. 18 H₂O



- Normal alum dose is 10-30 mg/litre of water and is very effective in the pH range of 6.5 to 8.5.
- Alum coagulant is cheap and the flocs formed are very stable.
- Alum also reduces colour, taste and odour, but the only disadvantage is that it is difficult to dewater the sludge formed.

NOTE: ·

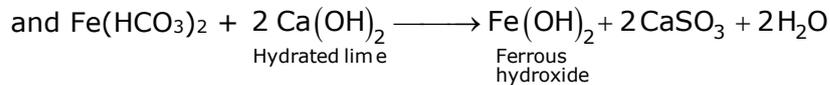
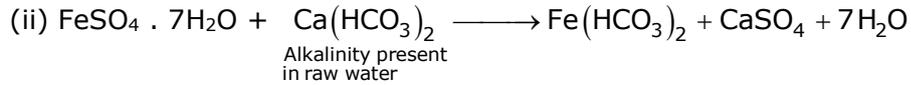
- A mole of alum gives 2 moles of Al(OH)₃.
- 666 gm of alum gives 2 × 78 gm of Al(OH)₃.

(b) Copper: Copperas

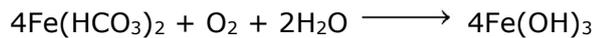
- Its chemical formula is FeSO₄.7 H₂O.
- When lime is added first, the following reaction takes



- Similarly, when copperas is added earlier to lime, the reaction that takes place is



- The ferrous hydroxide formed in either case further gets oxidized, forming hydroxide as below:



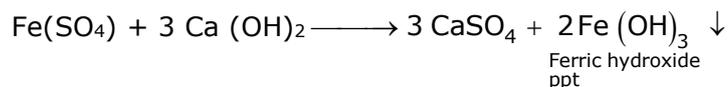
- Copperas is extensively used as a coagulant for raw waters that are not coloured.
- It is generally cheaper than alum and functions effectively in the pH range of 8.5 and above.
- For coloured raw waters, it is, however, not used, as it does not give satisfactory results

(c) Use of Chlorinated Copperas as Coagulant:

- When chlorine is added to a solution of copperas (i.e. ferrous sulphate), the two react chemically to form ferric sulphate and ferric chloride.
- The chemical equation is as follows:



- The resultant combination of ferric sulphate and ferric chloride is known as chlorinated copperas. It is a valuable coagulant for removing colours, especially whose raw water has a low pH value.
- Both the chlorinated copperas and lime constituents are effective coagulants, and their combination is often quite effective.
- The chemical reactions that take place are given below:

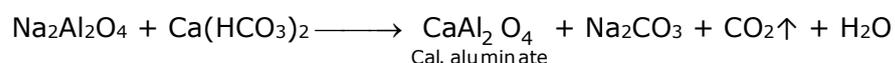


The resulting ferric hydroxide forms the floc and helps in sedimentation.

- Ferric sulphate is quite effective in the pH range of 4 to 7 and above 9, whereas ferric chloride is quite effective in the pH range of 3.5 to 6.5 and above 8.5.

(d) Use of Sodium Aluminate as a Coagulant:

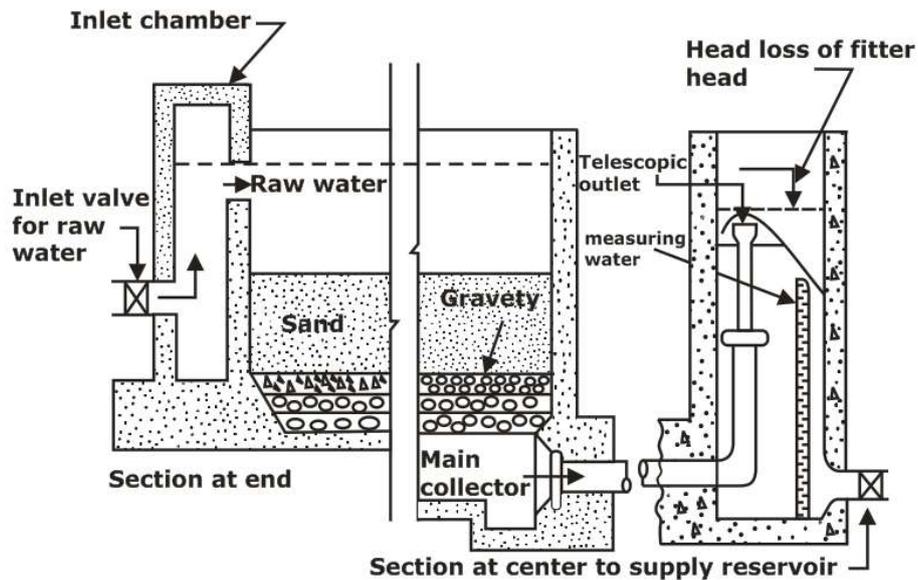
- Besides alum and iron salts, sodium aluminate (Na₂Al₂O₄) is also sometimes used as a coagulant.
- The chemical reactions that are involved are



6. FILTRATION

- Filters are classified as:
 - (a) Slow sand filter
 - (b) Rapid sand filter
 - (c) Pressure filter
 } gravity filters
- The slow sand filter removes a larger percentage of impurities as compared to rapid sand filters.
- Slow sand filters have a very slow rate of filtration about $\frac{1}{30}$ th of the rapid sand filter.

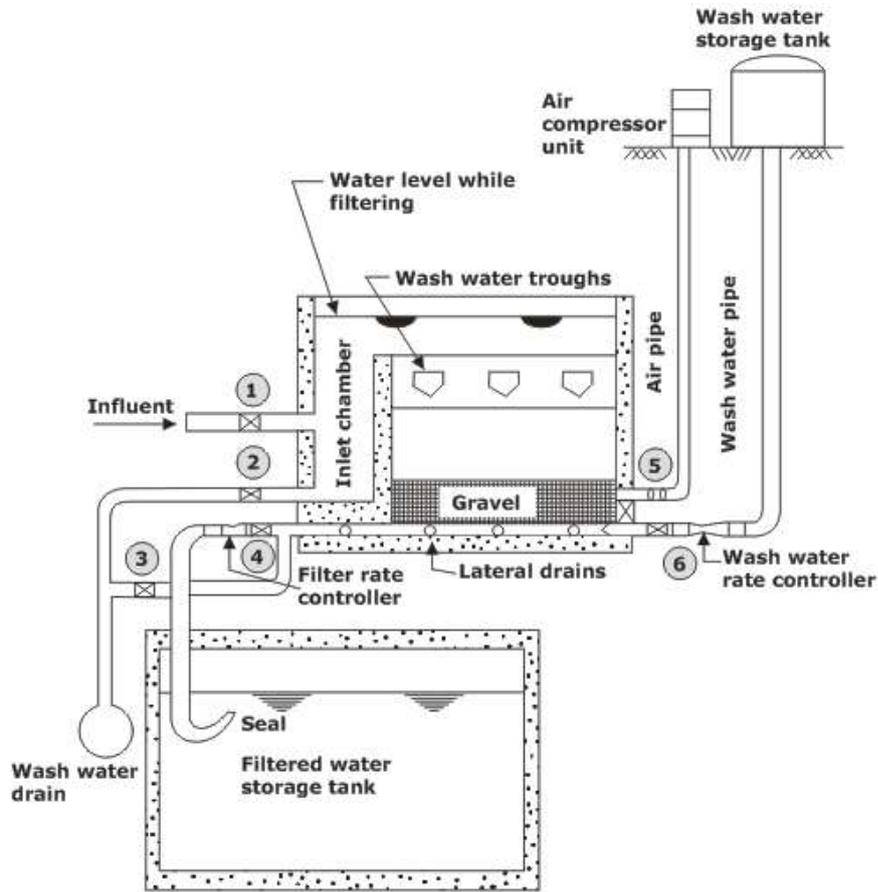
6.1. SLOW SAND FILTER



- It utilizes the effluent from a plain sedimentation tank only, which are relatively clearer.
- The depth of the Tank is 2.5 to 3.5 m.
- Plan area required is 100-2000 m².
- Filter medium is sand or anthracite or garnet
- D_{10} (of filter medium) = 0.2 - 0.3 mm.
- $\frac{D_{60}}{D_{10}}$ = uniformity coefficient = 5 (as per GOI manual).
- The depth of sand is 90-110 cm \approx 1 m
- The depth of water over the sand medium would be approximately the same as the depth of the sand medium.
- The top 15 cm of the sand layer would be finer remaining may be of uniform size.
- The material is gravel provided in 3 layers.
 - Top layer size – 3 - 6 mm
 - Middle layer – 20-40 mm
 - Bottom layer – 40 - 65 mm
- The design period of a slow sand filter is ten years.

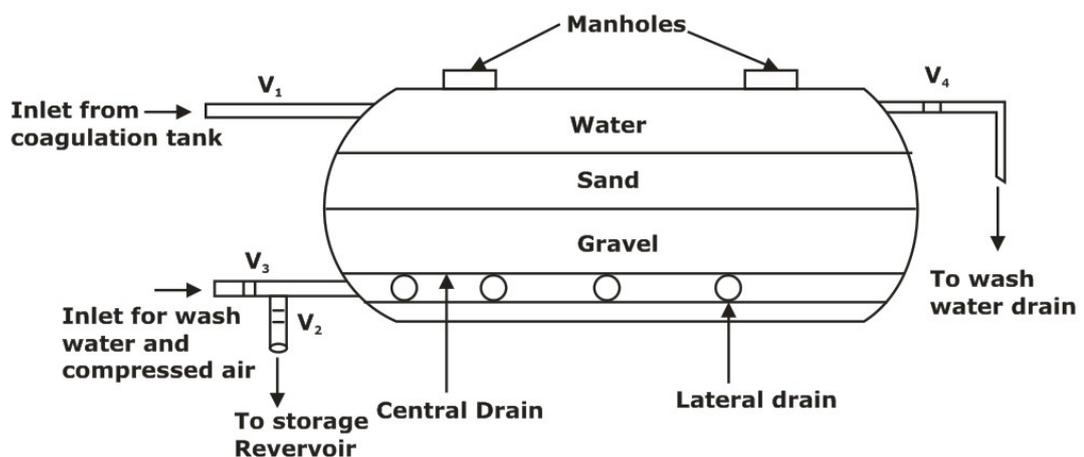
6.2. RAPID GRAVITY FILTER

- Particles more than and less than one μm diameter are efficiently removed.
- Removes suspended & colloidal matter
- It also removes microorganisms (i.e. bacteria and helminth etc.)

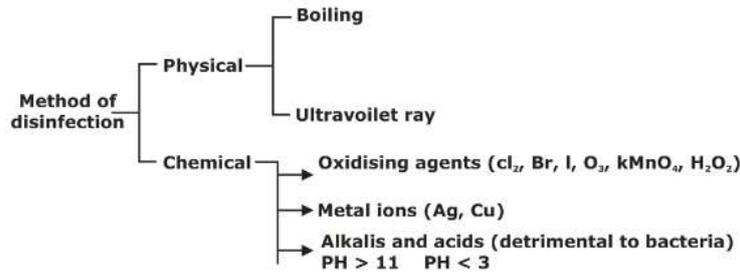


6.3. PRESSURE FILTERS

- The diameter of the Tank is 1.5 - 3.0 m.
- Height or length is 3.5 to 8.0 m.
- It is operated as a rapid gravity filter except that raw water is neither flocculated nor sedimented before it enters the filter.



7. DISINFECTION



- Out of various methods, Chlorination is most commonly adopted.

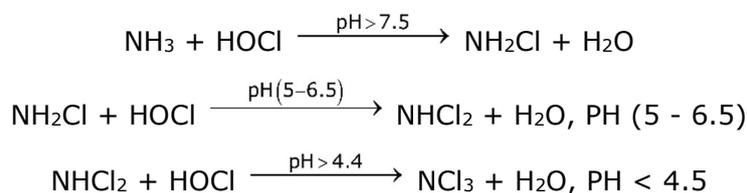
7.1. MINOR METHODS

- Treatment with excess lime
- Treatment with ozone
- Treatment with F and Br
- Treatment with $KMnO_4$

7.2. MAJOR METHODS

7.2.1. Chlorination

- At $pH < 5$, chlorine does not react with water and remains as free chlorine.
- $(HOCl + OCl^- \text{ and } Cl_2)$ are combined called freely available chlorine.
- pH of water should be maintained slightly below 7.
- Moreover, the chlorine will immediately react with ammonia present in water to form chloramines.



- Chloramines are a combined form of chlorine.
- It is less effective than free chlorine (25 times lesser). But they are stable and remain in the water for a greater duration.
- Doses of chlorine should be sufficient to leave a residue of 0.2 mg per litre after 10 minutes of the contact period. This dose is called chlorine demand of water.
- The residual chlorine is tested by DPD (Diethyl-Paraphenylene diamine) test.

7.3. FORMS IN WHICH CHLORINE IS ADDED

- Free chlorine (liquid or gaseous form)
- Hypochlorite's (Bleaching powder)
- Chloramines (ammonia + chlorine)
- Chlorine dioxide (ClO_2)

7.4. TYPES OF CHLORINATION

a) Plain Chlorination

- Only Chlorination and no other treatment is given to water.

- It removes bacteria, organic matter and colour.
- It is used for clean water, i.e. turbidity between 20- 30 mg/l.
- The dose is 0.5 mg/l.

b) Pre Chlorination

- In this case, chlorine is added before filtration or rather before sedimentation and coagulation.

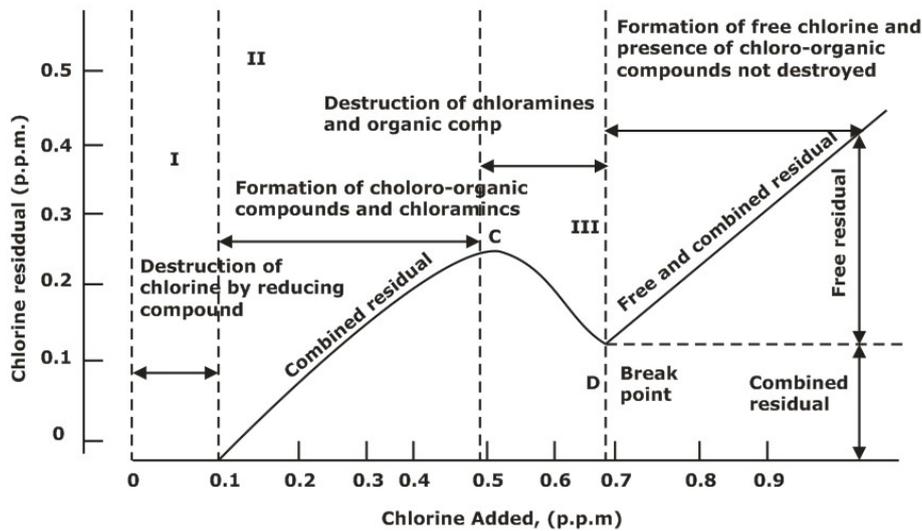
c) Post Chlorination

- Applying chlorine at the end when all treatment is complete is called post chlorination.
- The dose should be such that 0.1 to 0.2 mg/lit should be left after a contact period of 20 mm.

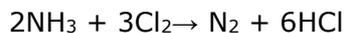
d) Double Chlorination

- Prechlorination & post-chlorination combinedly is called double Chlorination.

e) Breakpoint Chlorination



- During the disinfection process amount of residual chlorine is less in the beginning (stage I), during which iron, nitrite etc., are oxidized.
- In stage II, chloramines and combined residual chlorine form. Combined residual chlorine will gradually increase as demand for disinfection is satisfied.
- Chlorine residue is tested by the DPD test, which measures both combined and free chlorine.
- At point C, a bad smell started coming out. It is because oxidation of organic matter starts at point C. Hence the residual decreases.
- In stage III, free chlorine breaks down chloramines into nitrogen compounds.



- Chloro-organic compounds are also destroyed. At this point, D bad smell suddenly disappears. It implies that organic matter oxidation is complete.
- Any further chlorine addition simply appears as free chlorine, i.e. chlorine breaks away from water. Thus point D is called a breakpoint.
- In general practice, chlorine is added beyond breakpoint to ensure a 0.2 - 0.3 mg/litre residual as free chlorine.
- The difference between applied chlorine and residual chlorine is called the chlorine demand.

$$\text{Chlorine demand} = \text{Applied chlorine} - \text{Residual chlorine}$$

f) Super Chlorination

- When excess chlorine (5 to 15 mg/l) is added to water during the epidemic, it gives a residual of 1 to 2 mg/lit beyond breakpoint is called super Chlorination.

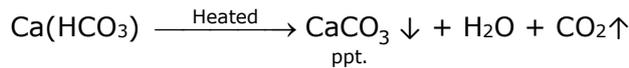
Note: The various dechlorinating agents are:

- Sodium thiosulphate (Na₂S₂O₃) - cheapest of all
- Activated carbon
- Sulphur dioxide (SO₂)

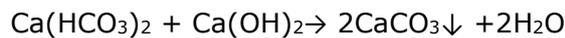
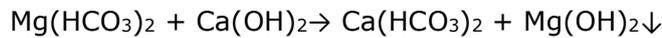
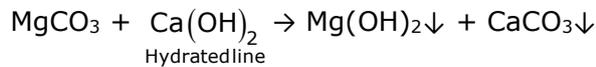
8. WATER SOFTENING

8.1. REMOVAL OF TEMPORARY HARDNESS

8.1.1. By simple boiling



- The boiling does not remove temporary hardness due to magnesium because MgCO₃ is fairly soluble in water. Hence this hardness is removed by the addition of lime.

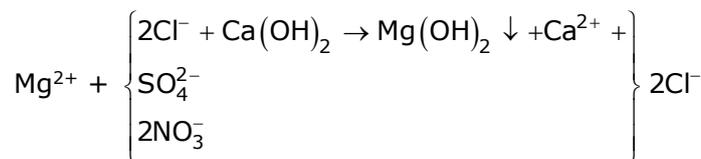
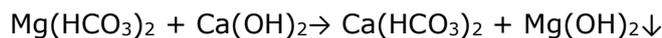
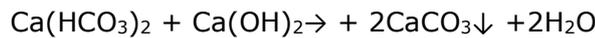


- 1 mole of MgCO₃ requires a mole of hydrated lime, whereas a mole of Mg(HCO₃)₂ requires 2 moles of lime.

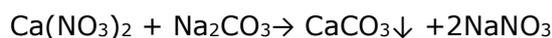
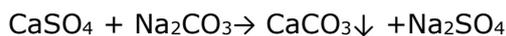
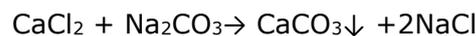
8.2. REMOVAL OF PERMANENT HARDNESS (WATER SOFTENING)

- Lime soda process
- Base exchange process
- Demineralization process

8.2.1. LIME SODA PROCES



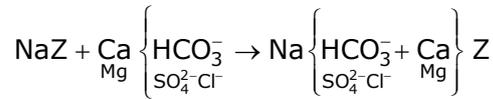
Further



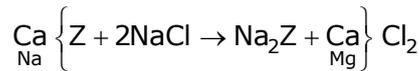
- Lime removes entire carbonate hardness.
- The lime reacts with the non-carbonate hardness of magnesium to convert it to the non-carbonate hardness of calcium.
- The non-carbonates hardness of calcium is finally removed by soda ash.

8.3. BASE EXCHANGE PROCESS (CATION EXCHANGE PROCESS)

- Zeolite is a natural or synthetic cation or base exchange hydrated silicates of sodium and aluminium. It is called green sand.
- Zeolite → Na₂O.Al₂O₃.xSiO₃.yH₂O(greensand)



- Thus water will have zero hardness. But sodium zeolite can be regenerated from calcium and magnesium zeolites.
- Regeneration from CaZ₂ and Na Z is done by using 5 - 10% solution (brine solution).



Water Distribution System

1. TYPES OF DISTRIBUTION SYSTEM

There are three types of distribution systems

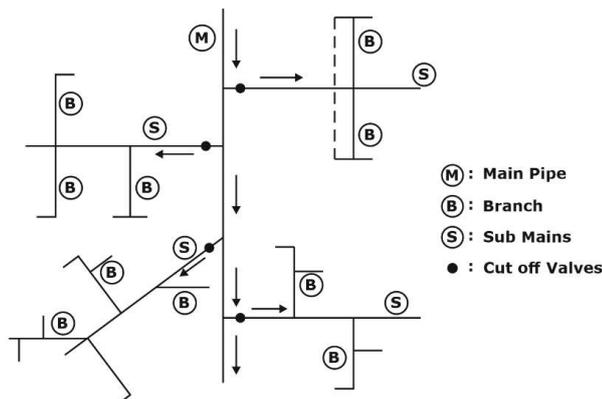
- (i) Gravity Distribution System:** In this system, water is distributed from high to lower by the simple action of gravity.
- (ii) Direct Pumping System:** Treated water is directly pumped to the distribution mains in this type of system.
- (iii) Combined System:** In this type of distribution system, treated water is pumped and transported to an elevated reservoir and then fed to the distribution system under the action of gravity only.

2. LAYOUT OF DISTRIBUTION SYSTEM

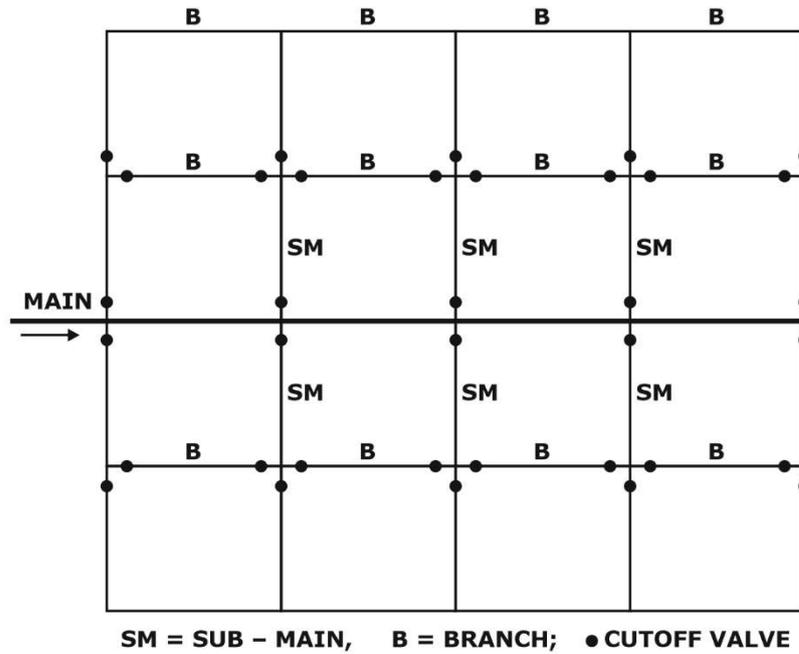
There are four methods of laying out of distribution system

- (a) Dead end system
- (b) Grid system
- (c) Circular system
- (d) Radial system

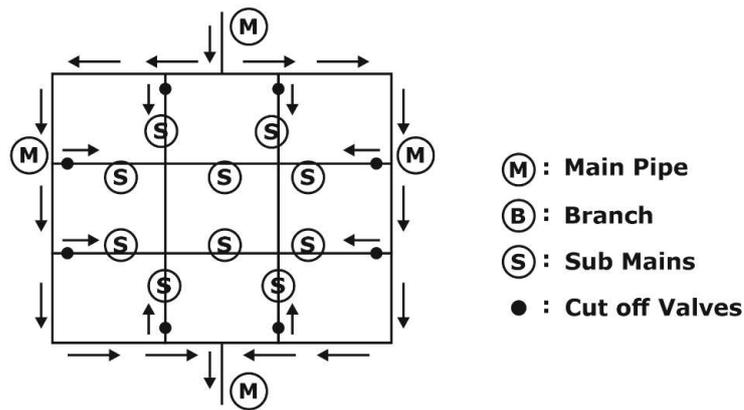
(a) Dead End System:



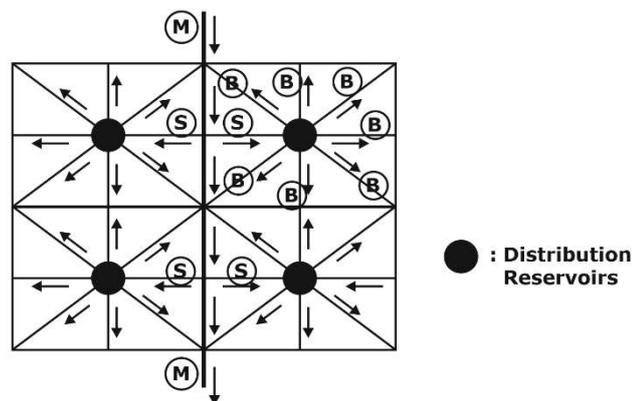
(b) Grid System:



(c) Circular/ Ring System:



(d) Radial System:



3. ANALYSIS OF NETWORK OF PIPES

There are two major methods for the analysis of a network of pipes:

(a) Equivalent Pipe Method: In this system, a complex system of pipes is replaced by a single hydraulically equivalent pipe, causing the same head loss and having the same discharge. This method is used for a large network of pipes.

(b) Hardy Cross Method:

- It is a trial and error method in which flow in the system of pipes is assumed, and resulting head losses are balanced.
- Successive corrections are applied until the network is hydraulically balanced. Two conditions must be satisfied in the analysis:
 - (i) At any junction, the quantity of water entering the junction should equal the quantity of water leaving the junction.
 - (ii) In any closed loop, the algebraic sum of pressure drops should be equal to zero.

Steps for Calculation:

- (i) Divide the network into several closed loops.
- (ii) Assume flow in each pipe satisfying continuity equation. Take clockwise flow as positive and anticlockwise flow as negative.
- (iii) Calculate the head loss in each pipe using the formula $h_f = rQ^n$
- (iv) The modification in discharge is given by the following formula:

$$\Delta Q = \frac{-\sum rQ^n}{\sum |nrQ^{n-1}|}$$

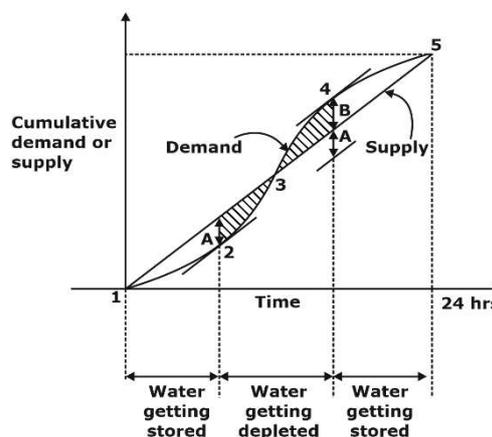
- (v) ΔQ is added to the assumed discharge algebraically.

4. DESIGN OF BALANCING RESERVOIR

A reservoir is designed such that it can meet the fluctuating demand with a constant rate of supply. The storage capacity of the reservoir can be determined using the mass curve method.

4.1. Mass Curve Method

- A mass curve is a plot of accumulated supply/demand with time.
- The amount of balancing storage is determined by adding the maximum ordinate between the demand and supply line.



- In region 1-2 – Demand rate < Supply rate (Accumulation of water)
- In region 2-4 – Demand rate > Supply Rate (Depletion of water)
- In region 4-5 – Demand Rate < Supply rate (Accumulation of water)

- Maximum water that can be stored in the reservoir = A + B

Design of Sewer

1. HYDRAULIC DESIGN OF SEWERS

1.1. Important Formulas for Determining Flow Velocity

Following formulas used to determine flow velocities in sewers:

(i) Manning's formula: The flow velocity is given by

$$v = \frac{1}{n} R^{2/3} \sqrt{S}$$

Where,

R = Hydraulic radius = A/P

A = Cross sectional area of sewer

P = Wetted Perimeter

S = Ground slope

n = manning's constant

(ii) Chezy's formula: The flow velocity is given by

$$v = C\sqrt{RS}$$

Where,

C = Chezy's constant

1.2. Design Data

- Sewage should be designed for maximum hourly discharge, and it should be ensured that flow velocity will always be greater than self-cleansing velocity.
- To avoid clogging, sufficient velocity known as '**Self-cleansing velocity**' needs to be maintained in the system.
- Maximum hourly discharge = 3 × Average daily discharge
- Maximum daily discharge = 2 × Average daily discharge
- It is assumed that 80% of the water supply goes to sewers.
- The self-cleansing velocity can be calculated using Shield's formula

$$v = \frac{1}{n} R^{1/6} (K(G - 1)d_p)^{1/2}$$

Where,

G = Specific gravity of the particle

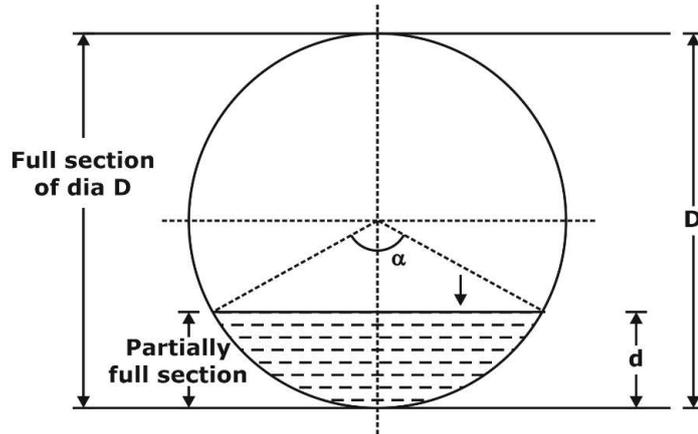
d_p = Size of particle

K = A constant

R = Hydraulic radius of sewer

n = manning coefficient

3. Circular Sewer running Partially Full



When the sewage is running partially full at depth d such that,

- Proportional depth

$$d = \frac{D}{2} \left(1 - \cos \frac{\alpha}{2} \right)$$

- Area of flow

$$\frac{d}{D} = \frac{1}{2} \left(1 - \cos \frac{\alpha}{2} \right)$$

- Proportional area

$$a = \frac{\pi D^2}{4} \left(\frac{\alpha}{360} - \frac{\sin \alpha}{2\pi} \right)$$

$$\frac{a}{A} = \left(\frac{\alpha}{360} - \frac{\sin \alpha}{2\pi} \right)$$

- Wetted Perimeter

$$p = \pi D \frac{\alpha}{360}$$

- Proportional wetted perimeter

$$\frac{p}{P} = \frac{\alpha}{360}$$

- Hydraulic radius

$$r = \frac{a}{p} = \frac{D}{4} \left(1 - \frac{360 \sin \alpha}{2\pi \alpha} \right)$$

- Proportional hydraulic radius

$$\frac{r}{R} = 1 - \frac{360 \sin \alpha}{2\pi \alpha}$$

- Proportional velocity of flow

$$\frac{v}{V} = \frac{N}{n} \left(\frac{r}{R} \right)^{2/3} \quad (\text{Using manning's formula})$$

- Proportional discharge

$$\frac{q}{Q} = \frac{a \times v}{A \times V}$$

Note:

- For the constant value of manning's coefficient, the velocity will be maximum when d = 0.81D
- For a constant value of manning's coefficient, the discharge will be maximum when d = 0.95D

4. Equal Degree of Self Cleansing:

- For equal self-cleansing, the drag force under partial flow should be the same as that under full flow.

$$\begin{aligned} \gamma_w rs &= \gamma_w RS \\ \Rightarrow rs &= RS \\ \Rightarrow \frac{v}{V} &= \frac{N}{n} \left(\frac{r}{R}\right)^{\frac{1}{6}} \end{aligned}$$

Quality Parameters of Sewage

1. INTRODUCTION

- Wastewaters are usually classified as industrial wastewater or municipal wastewater.
- Industrial wastewater with characteristics compatible with municipal wastewater is often discharged to the municipal sewers.

1.1. Important Waste Water Contaminates

| Sl. No | Contaminant | Sources | Environmental Significance |
|--------|-----------------------|------------------------------------|---|
| 1. | Suspended solids | Domestic use, Industrial wastes | Causes sludge deposits and Anaerobic condition in Aquatic environment |
| 2. | Biodegradable Organic | Domestic use, Industrial wastes | Cause biological degradation |
| 3. | Pathogens | Domestic water | Transmit communicable diseases |
| 4. | Nutrients | Domestic and Industrial waste | Cause eutrophication |
| 5. | Refractory Organics | Industrial waste | Causes taste and odour Problems |

2. PHYSICAL CHARACTERISTICS

The most important physical characteristics of water are its turbidity, colour, odour and Temperature.

2.1. TURBIDITY

Wastewater is normally turbid, containing wastes from baths, faecal matter, pieces of paper, greases, vegetable debris, fruit skins, etc.

2.2. COLOUR

- Fresh wastewater is usually grey or light brown. However, as bacteria break down organic compounds, the dissolved oxygen in the wastewater is reduced to zero and colour changes to black. This condition of waste is said to be septic or stale.
- The common method of colour removal is by coagulation followed by sedimentation.

2.3. ODOUR

- Odours in wastewater usually are caused by gases produced by the decomposition of organic matter.
- The most characteristic odour of stale or septic wastewater is hydrogen sulphide produced by anaerobic microorganisms that reduce sulphates to sulphides.

2.4. TEMPERATURE

The average Temperature of sewage in India is 20°C, near the ideal Temperature for biological activities.

3. CHEMICAL CHARACTERISTICS

Important chemical characteristics of wastewater are listed below:

- Total solids, suspended solids and Settleable solids.
- pH value.
- Chloride content
- Nitrogen content
- Presence of fats, greases, and oils.
- Sulphides, sulphates and H₂S gas.
- Dissolved oxygen.
- Chemical oxygen demand (COD).
- Bio-chemical oxygen demand (BOD).
- Total organic carbon (TOC).
- Theoretical oxygen demand (ThOD).

3.1. TOTAL SOLIDS, SUSPENDED SOLIDS AND SETTLEABLE SOLIDS

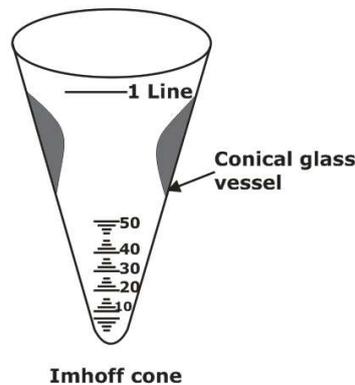
Solids present in wastewater may be in four forms: suspended solids, dissolved solids, colloidal solids and settlement solids.

- Suspended solids are those which remain floating in the water.
- Dissolved solids are those which dissolve in wastewater.
- Colloidal solids are finely divided solids remaining either in suspension or in solution.
- Settleable solid is that portion of solid matter that settle out if the wastewater is allowed to remain undisturbed for 2 hours.
- Inorganic matter consists of sand, gravel, debris, chlorides, sulphates etc., whereas organic matter consists of:
 - Carbohydrates such as cellulose, cotton, fibre, sugar etc.
 - fats and oils from kitchens, garages, shops etc.
 - nitrogenous compounds like proteins, urea, fatty acids etc.

The number of various kinds of solids present in wastewater can be determined as follow:

- Total solids can be determined by evaporating a known value of the wastewater sample and weighing the dry residue left. The mass of residue left divide by the volume of the sample is total solids in mg/l.
- The suspended solids, also called non-filterable solids. Thus weighting the dry residue left and dividing by volume of sample filtered will give suspended solids in mg/l.

c) The quantity of settlement solids can be determined using the Imhoff cone (figure). Wastewater is allowed to stand in the cone for two hours, and the quantity of solids settled down in the bottom is directly readout, which gives an approximate amount of settlement solids.



3.2. pH VALUE

- The alkalinity of the fresh wastewater samples is alkaline, but as time passes, it becomes acidic because of the bacteria action in anaerobic or nitrification processes.

3.3. CHLORIDE CONTENT

- These are derived from kitchen wastes, human faeces, and urinary discharges etc.
- The chloride content can be measured by titrating the waste sample with a standard silver nitrate solution, using potassium chromate as an indicator.

3.4. NITROGEN CONTENT

- The presence of nitrogen in wastewater indicates the presence of organic matter, and may be found, in the following forms:
 - a) Free ammonia or ammonia nitrogen (indicates recent pollution)
 - b) Albuminoid nitrogen or organic nitrogen (indicates quantity of nitrogen before decomposition has started).
 - c) Nitrites (indicates partly decomposed condition).
 - d) Nitrates [indicates old pollution (fully oxidized)]

3.5. PRESENCE OF FATS, OILS AND GREASES

- The amount of facts and greases in the wastewater sample can be determined by evaporating and mixing the residual solids with ether (hexane). The solution is then poured off and evaporated, leaving the facts and greases as residue, easily weighted.

3.6. SULPHIDES, SULPHATES AND HYDROGEN SULPHIDE GAS

- Sulphides and sulphates are formed due to the decomposition of various sulphur-containing substances present in the wastewater.
- This decomposition also leads to the evolution of hydrogen sulphide gas, causing bad smells and odours, besides causing corrosion of concrete sewer pipes.
- The initial decompositions is associated with the formation of H_2S gas, which also ultimately gets oxidized to form sulphates ions.

3.7. DISSOLVED OXYGEN

- Dissolved oxygen is required for the respiration of aerobic microorganisms and all other aerobic life forms.
- The dissolved oxygen in fresh wastewater depends upon Temperature.

If the Temperature of sewage is more, the D.O. content will be less. Max quantity of D.O. that can remain mixed in water at a particular temperature is called Saturation Dissolved Oxygen.

- Dissolved oxygen less than four ppm is detrimental to the survival of fish.
- The D.O content of wastewater is determined by Winkler's Method.

3.8. CHEMICAL OXYGEN DEMAND (COD)

- The COD test is used to measure the organic matter content of wastewater, both biodegradable and non-biodegradable.
- The oxygen equivalent of organic matter that can be oxidized is measured using a strong chemical oxidizing agent in an acidic medium.
- Potassium dichromate is excellent for this purpose.
- This test is also sometimes called the dichromate-oxygen demand test.
- $(\text{COD} - \text{BOD}_u) = \text{Non-biodegradable organics}$.

3.9. THEORETICAL OXYGEN DEMAND (ThOD)

- If the chemical formula and the quantity of all organic matter present in the sewage are known, the exact amount of oxygen required to oxidize can be calculated stoichiometrically. It is called theoretical oxygen demand (ThOD).
- For most practical cases, $\text{COD} = \text{ThOD}(\text{taken})$, However, generally $\text{ThOD} > \text{COD} > \text{BOD} > \text{TOC}$

3.10. TOTAL ORGANIC CARBON (TOC)

- It is another method of expressing organic matter in terms of carbon content.

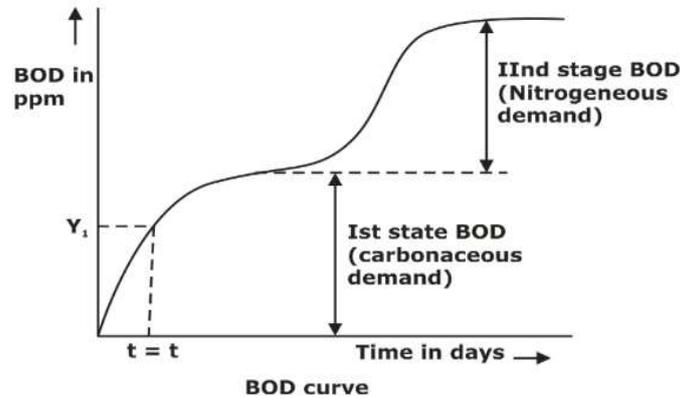
3.11. BIOCHEMICAL OXYGEN DEMAND

- Biochemical oxygen demand is used to measure the quantity of oxygen required for oxidation of biodegradable organic matter present in a water sample by aerobic biochemical action.
- Three classes of materials exert oxygen demand of wastewater:
 - a) Carbonaceous organic materials.
 - b) Oxidisable nitrogen derived from nitrite, ammonia, and other organic nitrogen compounds serve as food for specific bacteria (Nitrosomonas and Nitrobacter).
 - c) Chemical reducing compounds, e.g. Fe^{2+} , SO_3^{2-} (sulphites), SO^{2-} (sulfide) which are oxidized by dissolved oxygen.
- For domestic sewage, nearly all oxygen demand is due to carbonaceous organic material and is determined by the BOD test.

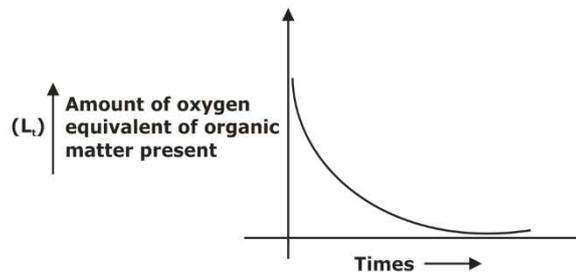
$$\text{BOD}_5 = \text{D.O. consumed in the test by diluted sample} \times \frac{\text{Vol. of the diluted sample}}{\text{Vol. of the undiluted sewage sample}}$$

The above factor in the bracket is called the dilution factor.

- The first demand occurs due to oxidation of organic matter and is called carbonaceous demand or first stage demand. Later demand occurs due to biological oxidation of ammonia and is called nitrogenous demand or second stage demand.
- However, the term BOD usually means the first stage BOD.



3.11.1. REACTION KINETICS



L_t = amount of organic matter present at time t

$$BOD_t = L_0(1 - 10^{-k_D t})$$

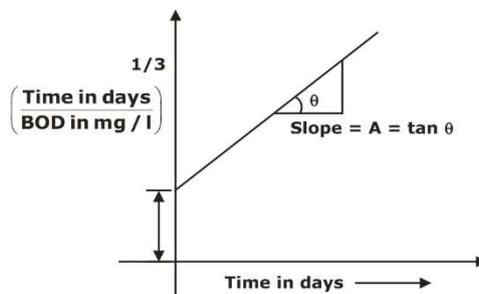
- Unit of K_D is in terms of per day, and it is temperature-dependent.

$$K_{D_{(T^2)}} = K_{D_{20^0}} [1.047]^{T-20^0}$$

(Vanhoff-Arrhenius equation)

- K_D is also sometimes called deoxygenating constant

3.11.2. ESTIMATION OF K_D



$$K_D = 2.61 \frac{A}{B} \text{ Per day}$$

B. POPULATION EQUIVALENT

- Average standard BOD of domestic sewage is 80 gms per person per day.
- The Number of people who produce the amount of BOD at the rate of 80 gms per person per day equal to that produced by industrial sewage is called the population equivalent of industrial sewage.

C. RELATIVE STABILITY (S)

- Relative stability (S) is calculated as

$$S = \frac{O_2 \text{ available in effluent}}{\text{Total } O_2 \text{ required for 1st stage BOD}}$$

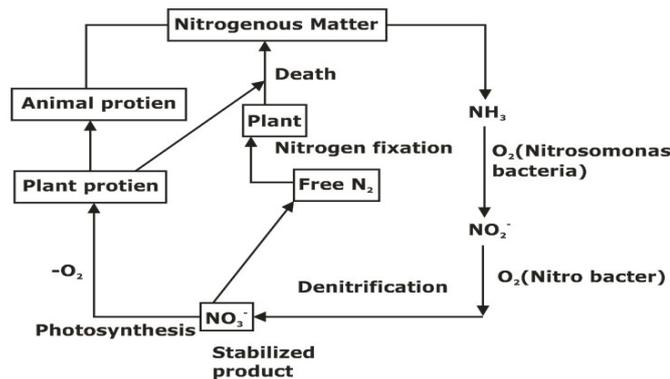
$$S = 100 \left[1 - (0.794)^{t_{20}} \right] = 100 \left[1 - (0.63)^{t_{37}} \right]$$

T_{20}/t_{37} = time in days for a sewage sample to decolourise a sample of methylene blue solution when incubated at 20° and 37°, respectively.

. VARIOUS NATURAL CYCLES

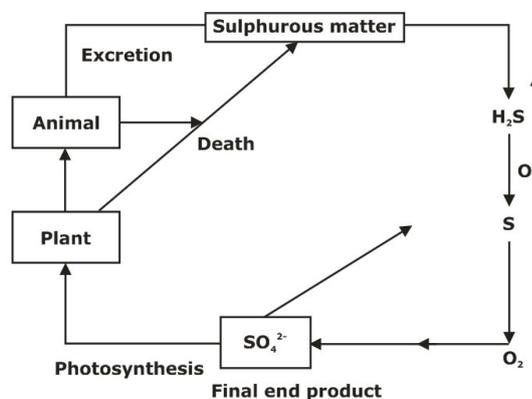
- (1) Aerobic cycle
- (2) Anaerobic cycle

(i) Aerobic cycle



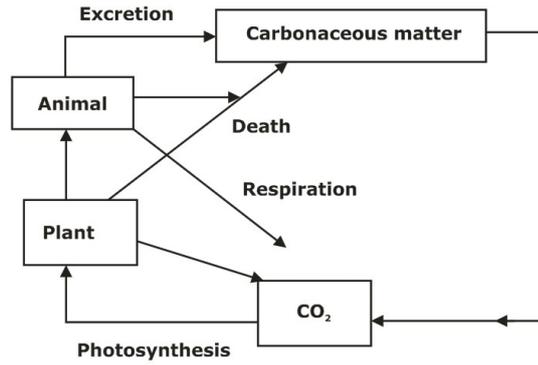
Nitrogen cycle (Aerobic cycle)

(ii) Sulphur Cycle



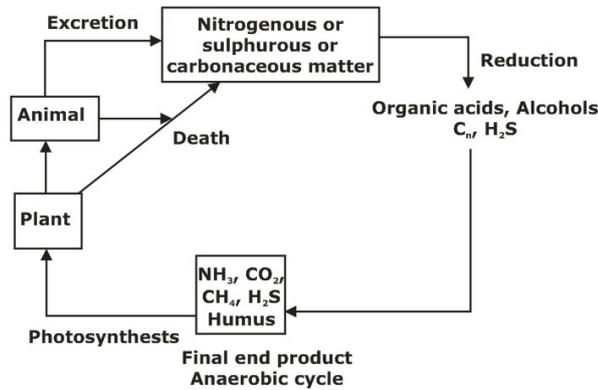
Sulphur Aerobic cycle

(iii) Carbon Cycle



Carbon Aerobic cycle

(iv) Anaerobic Cycle



**Final end product
Anaerobic cycle**

Sewage Treatment Plant

Sedimentation Tank

Settling Velocity

(i) $V_s = \frac{(G - 1)\gamma_w d^2}{18\mu}$ for $d < 0.1 \text{ mm}$

$V_s = 418 \times (G - 1) \times d^2 \times \left[\frac{3T + 70}{100} \right]$ applicable for $Re < 1$ (stoke's law)

G= specific gravity of the particle

V_s = Settling velocity

d= diameter of particle

μ = Dynamic viscosity

T = Temperature of water in °C

(ii) $V_s = \left[\frac{\frac{4}{3}gd(G - 1)}{C_D} \right]^{1/2}$

Where

- $C_D = 0.4$ for $Re \geq 10^4$
- $C_D = \frac{24}{Re}$ for $Re \leq 0.5$
- $C_D = \frac{24}{Re} + \frac{3}{\sqrt{Re}} + 0.34$ for $0.5 \leq Re \leq 10^4$

(iii) $V_s = \left[\frac{g \times (G - 1) d^{1.6}}{13.88 v^{0.6}} \right]^{0.714}$ $0.1 \leq d \leq 1 \text{ mm}$

(iv) Newtons Equation for Turbulent Settling

$$V_s = 1.8 \sqrt{gd(G - 1)} \text{ for } d \geq 0.1 \text{ mm}$$

(v) Modified Hazen's Equation for Transition Zone

$$V_s = 60.6d(G_s - 1) \left(\frac{3T + 70}{100} \right) \text{ For } d \text{ between } 0.1 \text{ to } 1 \text{ mm}$$

For inorganic solid $V_s = d(3T + 70)$

For organic solid, $V_s = 0.12d(3T + 70)$

Where T = Temperature in °C.

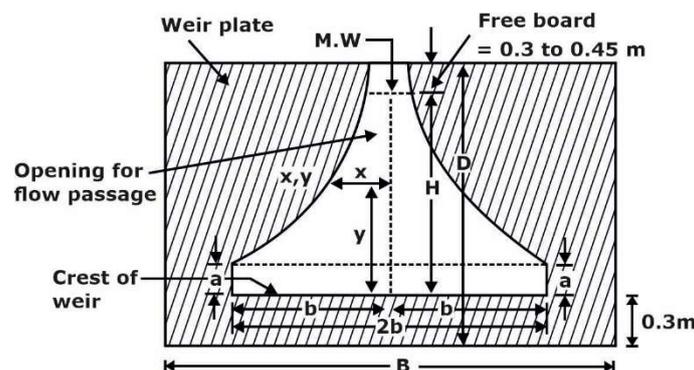
Grit Chamber

- Grit chamber removes particles of size $\geq 0.2 \text{ mm}$
- The specific gravity of the frit is usually in the range of 2.4 to 2.65, but for design adopt ≈ 2.65 .
- For 0.2 mm particle setting velocity = 0.025 m/s
- Surface over flow rate (Q/A) = 2160 m³/m²/day.
- Stokes law cannot be applied to grit chamber settling because the particle size is $\geq 0.2 \text{ mm}$
- Detention time is 40 – 60 sec.
- Depth is 1 – 1.5 m.
- The freeboard is 0.3 m.

Critical Scour Velocity in Constant Velocity Horizontal Flow Grit Chamber (V_H)

$$V_H = 3 \text{ to } 4.5 \sqrt{gd(G - 1)}$$

PROPORTIONAL FLOW WEIR



$$x = \frac{2BV_h}{C_d \cdot \sqrt{2g} \cdot \pi \sqrt{y}}$$

$$b = 1.467BV_h$$

Where,

B= Width of the channel.

V_h= Horizontal flow velocity.

C_d= Coefficient of discharge.

x and y are coordinates on the weir profile.

Parabolically or V-Shaped Grit Chamber Provided with a Parshall Flume

(i) Parshall Flume

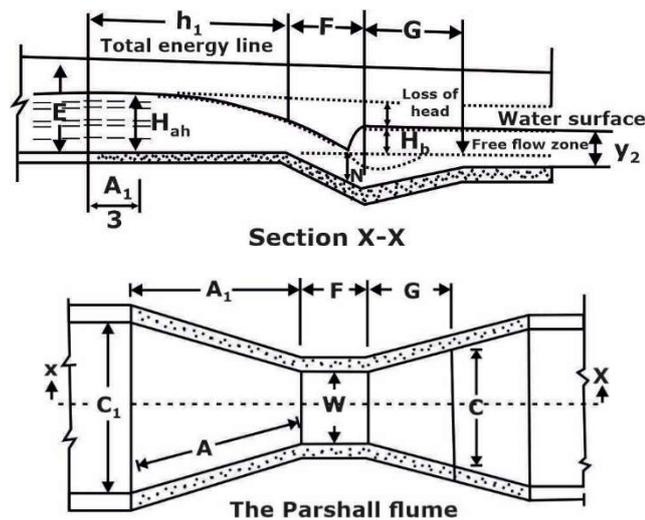
$$Q = 2.264W(H_a)^{3/2}$$

Where,

W= Width of the throat in the meter.

Q= Flow in (m³/sec) through Parshall flume.

H_a= Depth of flow in the upstream leg of a flume of one-third portion in the meter.



(ii) Parabolic Grit Channel

$$Q = C_1 \int_{y=0}^{y-y} xdy$$

where $V_h = \frac{Q}{\int_0^y xdy} = C_1$

$$Q = C_2 y^n \quad Y^{n-1} = \frac{C_1}{C_2} \cdot X$$

n = Discharge coefficients of the control section.

= 1.5 for partial flume.

= 1 for proportional flow weir.

- Aerated Grit Channels

$$\text{Detention period} = 3 \text{ minutes}$$

- Detritus Tank

$$\text{Detention period} = 3 \text{ to } 4 \text{ minutes}$$

Skimming Tank

- (i) Detention Period = 3 to 5 minutes.
- (ii) Amount of compressed air required = 300 to 6000 m³ per million litres of sewage.
- (iii) Surface Area,

$$A = 0.00622 \frac{q}{V_r}$$

Where q= rate of flow of sewage in m³/day.

V_r= Min. rising velocity of greasy material to be removed in m/min
= 0.25 m/min mostly.

Sedimentation Tank

- (i) Overflow rate
= 40000 to 50000 lit/m² day for plain sedimentation.
= 50000 to 60000 lit/m² day for sedimentation with coagulation.
= 25000 to 35000 lit/m² day for secondary sedimentation tank
- (ii) Depth ~ 2.4 to 3.6 m.
- (iii) Detention time = 1 to 2 hour.
- (iv) width = 6.0 m
- (v) Length = 4 to 5 times width.
- (vi) Velocity of flow V_f = 0.3 m/min.

$$(vii) V = \frac{Q}{BH}$$

Where,

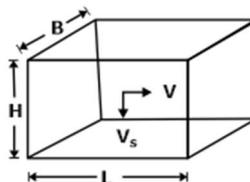
V = Flow velocity

B = Width of the Basin

H = Depth of sewage in the tank.

$$(viii) \frac{V}{V_s} = \frac{L}{H}$$

$$(ix) V_s = \frac{Q}{BL}$$



Detention Time

- (a) For rectangular Tank

$$t = \frac{BLH}{Q}$$

(b) for circular Tank

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

$$Q = \frac{V}{t}$$

Where

d = Dia of the Tank

H = Vertical depth of wall or side depth

Displacement Efficiency (η)

| |
|--|
| $\eta = \frac{\text{Flowing through period}}{\text{Detention period}}$ |
|--|

Trickling Filter

COMPARISON OF CONVENTIONAL AND HIGH RATE TRICKLING FILTER

| | Characteristics | Conventional or standard rate filters | High rate filter |
|----|--|--|---|
| 1. | Depth of filter media | Varies between 1.6 to 2 Am | Vanes between 1.2 to 1.8m2 |
| 2. | Size of filter media | 25 to 75 mm More | 5 to 60 mm Less |
| | Land required | land area is required as filter loading is less | land area is required as the filter loading as more |
| | Cost of operation | More for treating equal quality of sewage | Less for treating equal quality of sewage |
| | Method of operation | Continues application less flexible required less skilled supervision | Continues application more flexible and more stalled operation is required |
| | Type of effluent produced | Effluent is highly nitrified and stabilized with B.O.D in effluent 20 ppm or so. | Effluent is nitrified up to nitrate stage only and is thus less stable, and hence it is of slightly inferior quality BOD in effluents |
| | Dosing interval | Generally, vanes between 3 to 10 minutes Vary loading 20 to 44 m.L per ha/day | 20 ≥ 20 ppm or so |
| | Filter loading values i) Hydraulic loading ii) Organic loading | Varies between 900 to 2200 kg of BOD per ham of filter media per day | Varies between 110 to 330 mL per hectare per day Varies between 6000 to 18000 of BOD per ham of filter media per day |

(a) Conventional Trickling Filter or Low Rate Trickling Filter

$$n = \frac{100}{1 + 0.0044\sqrt{u}}$$

Where,

n = The efficiency of the filter and its secondary clarifier, in terms of % of applied BOD

u = Organic loading in kg/ha-m/day applied to the filter (called unit organic loading)

(b) High Rate Trickling Filter

(i)

$$F = \frac{1 + \frac{R}{I}}{\left(1 + 0.1 \frac{R}{I}\right)^2}$$

Where F = Recirculation factor

R/I = Recirculation ratio

(ii)
$$n = \frac{100}{1 + 0.0044\sqrt{\frac{Y}{VF}}}$$

Where,

Y = Total organic loading in kg/day applied to the filter, i.e. the total BOD in kg.

$\frac{Y}{VF}$ Unit organic loading in kg/Ha-m/day

V = Filter volume in Ha-m.

% efficiency of single-stage high rate trickling filter.

(iii)
$$n = \frac{100}{1 + \frac{0.0044}{1 - n'} \sqrt{\frac{Y'}{V'F'}}}$$

Where,

n' = Final efficiency in the two-stage filter.

Y' = Total BOD in the effluent from the first stage in kg/day.

F' = Recirculation factor for second stage filter

V' = volume in second stage filter in ha-m.

Sludge Digestion Tank

- The diameter of the tank is normally 6 – 38 m
- The depth of the tank is approximately 6 – 12 m
- The lower slope is 1 to 1 : 3.
- Dia/depth = 1.5 – 4.

(i) When the change during digestion is linear.

(a)
$$V = \left(\frac{V_1 + V_2}{2}\right)t$$

Where,

V = The volume of digestion in m^3 .

V_1 = Raw sludge added per day (m^3/day)

Equivalent digested sludge produced per day on completion of digestion, m^3/day .

Digestion period in the day.

(b) $V = \left(\frac{V_1 + V_2}{2} \right) t + V_2 T$ with monsoon storage

Where,

T = number of days for which digested sludge V_2 is stored (monsoon) storage)

(ii) When the change during digestion is parabolic

(a) $V = \left[V_1 - \frac{2}{3}(V_1 - V_2) \right] t$ without monsoon storage

(b) $V = \left[V_1 - \frac{2}{3}(V_1 - V_2) \right] t + V_2 T$ without monsoon storage

Destruction and Removal Efficiency (DRE)

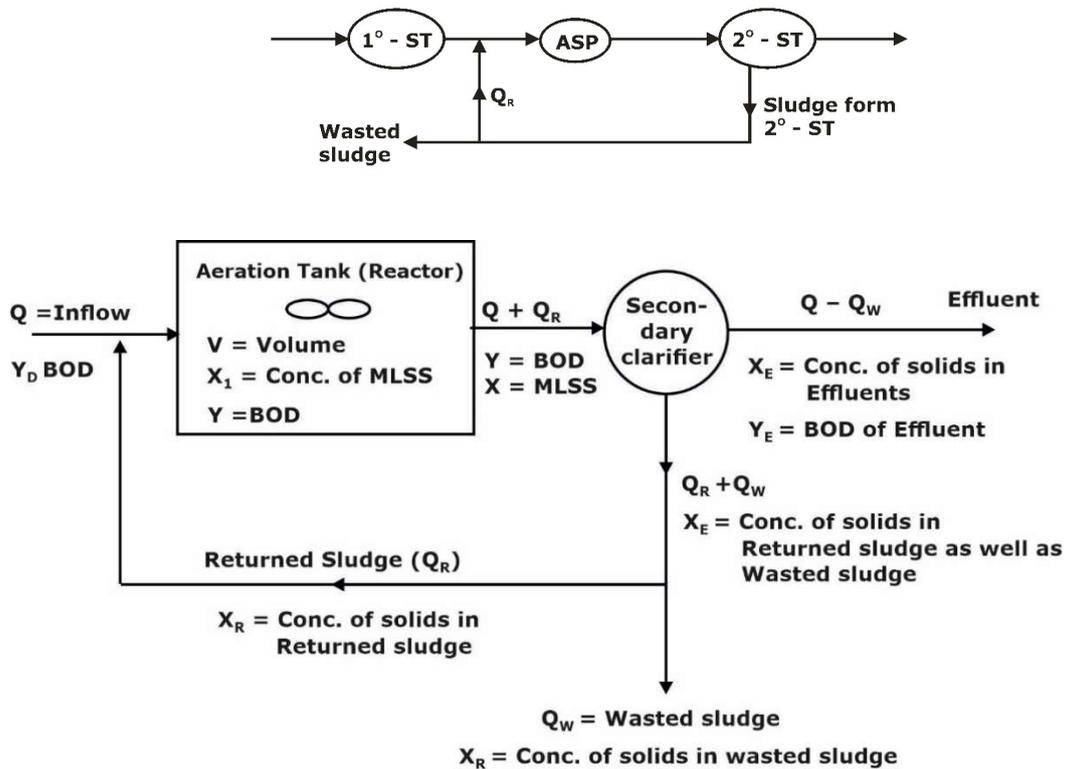
$$DRE = \frac{W_{in} - W_{out}}{W_{in}} \times 100$$

Where,

W_{in} = The mass fill rate of one POHC (Principal organic Hazardous constituent) in the waste stream.

W_{out} = Mass emission rate of the same POHC present in the exhaust emission before releasing to the atmosphere.

Aeration Tank (ASP)



(i) Detention period, $t = \frac{V}{Q}$

Where

V = volume of the Tank in m³.

Q = Quantity of wastewater flow into the aeration tank excluding the quantity of recycled sludge (m³/day)

(ii) Volumetric BOD Loading or Organic Loading, (U)

$$u = \frac{QY_0}{V}$$

Where,

QY₀ = Mass of BOD applied per day to the aeration tank through influent sewage in gm.

V = The volume of the aeration tank in m³.

Q = sewage flows into the aeration tank in m³.

Y₀ = BOD₅ in mg/lit (or gm/m³) of the influent sewage.

(iii) $\frac{F}{M} = \frac{QY_0}{VX_t}$

Where,

$\frac{F}{M}$ = Food (F) to Microorganism (M) ratio

QY₀ = Daily BOD applied to the aeration system in gm.

Y₀ = five day BOD of the influent sewage in mg/lit.

Q = The flow of influent sewage in m³/day.

X_t = MLSS (Mixed liquor suspended solids) in mg/lit.

V = The volume of the Aeration Tank (lit).

M = V X_t Total microbial mass in the system in gm.

(iv) Sludge Age (θ_c)

(a)

$$\theta_c = \frac{\text{Mass of suspended solids (MLSS) in the system (M)}}{\text{Mass of solids leaving the system per day}}$$

(b)
$$\theta_c = \frac{VX_T}{Q_w X_R + (Q - Q_w) X_E}$$

Where,

X_T = The Concentration of solids in the influent of the Aeration Tank called the MLSS, i.e. mixed liquor suspended solids in mg/lit.

V = Volume of Aerator

Q_w = The volume of waste sludge per day

X_R = The Concentration of solids in the returned sludge or in the wasted sludge (both being equal) in mg/lit.

Q = Sewage inflow per day.

X_E = The Concentration of solids in the effluent in mg/lit.

(v) Sludge Volume Index (S.V.I)

$$S.V.I = \frac{V_{ob}}{X_{ob}} \times 1000$$

Where,

X_{ob} = Concentration of suspended solids in the mixed liquor in mg/lit.

V_{ob} = Settled sludge volume in ml/lit.

S.V.I = Sludge volume index in ml/gm.

(vi) Sludge Recycle and Rate of Return Sludge

$$Q_R \cdot X_R = (Q + Q_R) \times t$$

$$\frac{Q_R}{Q} = \frac{X_t}{X_R - X_t}$$

Where,

Q_R = Sludge recirculation rate in m³/day.

X_t = MLSS in the aeration tank in mg/lit.

X_R = MLSS in the returned or wasted sludge in mg/lit.

$$X_R = \frac{10^6}{S.V.I}$$

S.V.I = Sludge volume index in ml/gm.

• Specific substrate utilization rate

$$U = \frac{Q(Y_0 - Y_E)}{V \cdot X_t} \quad \left[\frac{1}{\theta_C} = \alpha_y U - k_e \right]$$

$\alpha_y = 1$ for MLSS and 0.6 for MLVSS, $k_e = 0.06$

Oxygen Requirement of the Aeration Tank

$$O_{2(\text{ required })} = \left[\frac{Q(Y_0 - Y_E)}{f} - 1.42Q_W \cdot X_R \right] \text{ gm / day}$$

Where,

$$f = \frac{BOD_5}{BOD_u} = \frac{5 \text{ day BOD}}{\text{Ultimate BOD}} = 0.68$$

Oxidation Ponds

➤ Depth → 1.0 to 1.8 m.

- Detention period → 2 to 6 weeks.
- Organic loading → 150 to 300 kg/ha/day.
- Under hot conditions → , 60 to 90 kg/ha/day.
- Under cold conditions.
- Length to width ratio = 2
- Sludge Accumulation = 2 to 5 cm/year
- Minimum depth to be kept = 0.3 m.

For Inlet Pipe Design

Assume V = 0.9 m/s

Assume flow for 8 hrs.

For Outlet Pipe Design

Dia of outlet = 1.5 dia of the inlet pipe

Septic Tank

- Detention time = 12 to 36 hr.
- Sludge accumulation rate = 30 lit/cap/year.
- Sewage flow = 90 to 150 lit/capita/day.
- Cleaning period = 6 to 12 months
- Length to width ratio = 2 to 3 m
- Depth = 1.2 to 1.8 m
- Width ≠ 0.9 m.
- Free board = 0.3 m

$$\text{Volume of septic tank} = \left[\frac{(\text{Sewage flow} \times \text{Detention time}) + (\text{Sludge accumulation rate}) \times \text{Cleaning rate}}{\text{Cleaning rate}} \right]$$

Disposal of treated effluent

Dilution and Dispersion

$$C = \frac{C_s Q_s + C_R Q_R}{Q_s + Q_R}$$

Where,

C = Final concentration of that material in the river

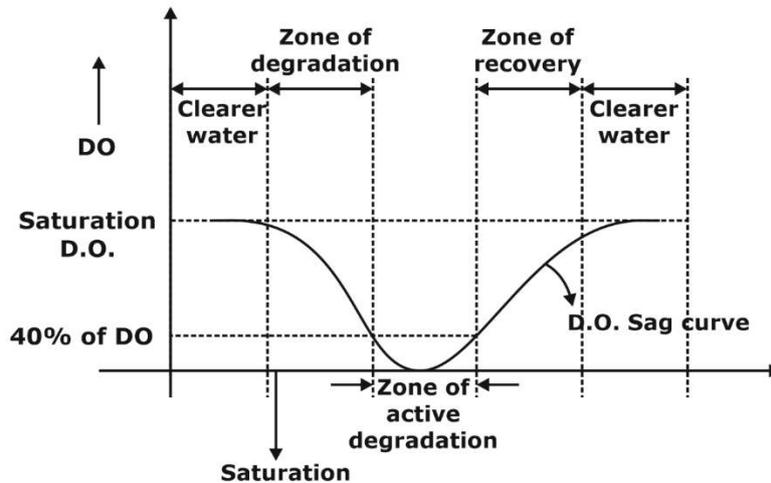
C_s = Concentration of material in sewage

C_R = Concentration of that material in the river

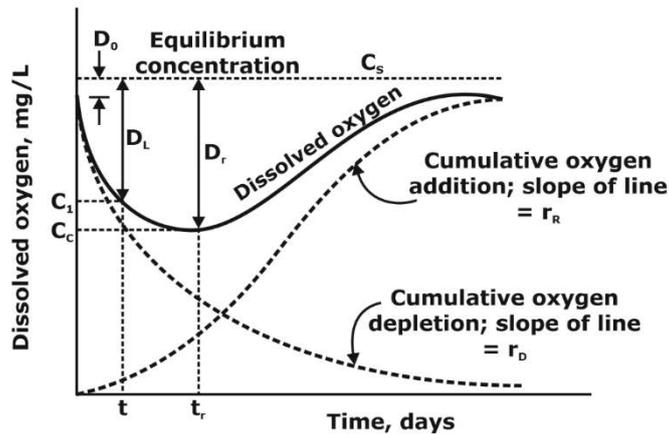
Q_s = Flow rate of sewage

Q_R = Flow rate of river

Zone of Pollution in River Stream



- Oxygen deficit = Saturation DO- Actual DO
- Saturation D.O at 20°C → 9.2 mg/lit.
- Saturation D.O at 30°C → 7.6 mg/lit.
- Saturation D.O at 0°C → 14.6 mg/lit.



Characteristics of the oxygen sag curve.

- The curve showing the rate of depletion with time is known as the **Deoxygenation curve**.
- . The curve showing the rate of accumulation of oxygen with time is termed as **Reoxygenation curve**.
- **Oxygen Sag curve:** In running a polluted stream, deoxygenation and reoxygenation occur simultaneously. The resultant oxygen deficit can be obtained by adding the deoxygenation and reoxygenation curve algebraically. The resultant curve is known as the oxygen sag curve.
- $TOD > COD > (BOD)_U > (BOD)_5$

Where,

TOD= Theoretical oxygen demand

BOD= Biological oxygen demand

COD= Chemical oxygen demand

(BOD)_u= Ultimate BOD (Y_u)

Stretcher-PHELPS EQUATION

$$D_t = \frac{k_D L}{k_R - k_D} \left[(10)^{-k_D t} - (10)^{-k_R t} \right] + \left[D_o \cdot (10)^{-k_R t} \right]$$

$$k_{D(T^{\circ}C)} = k_{D(20^{\circ}C)} [1.047]^{(T-20^{\circ}C)}$$

$$k_{R(T^{\circ}C)} = k_{R(20^{\circ}C)} [1.016]^{(T-20^{\circ}C)}$$

$$f = \frac{k_R}{k_D}$$

$$t_c = \frac{1}{k_D (f-1)} \log_{10} \left[\left\{ 1 - (f-1) \frac{D_o}{L} \right\} f \right]$$

$$D_C = \frac{L}{f} \cdot [10]^{-k_D \cdot t_c}$$

Where,

D_t = D.O deficit in mg/lit after t days.

L= Ultimate first stage BOD of the mix at a point of waste discharge in mg/lit.

D_o = Initial oxygen deficit of the mix at the mixing point in mg/lit.

K_R = Reoxygenation constant

K_D = Deoxygenation constant

f= Self-purification constant

t_c = Critical time at which minimum dissolved oxygen occurs i.e.

D_C = Critical maximum oxygen deficit.

Air Pollution

COMPOSITION OF AIR

Air is a mixture of gases present in the atmosphere. Major gases

| Name of the gas | Percentage by volume (%) |
|-----------------|--------------------------|
| Nitrogen | 98.09% |
| Oxygen | 20.95% |
| Argon | 0.90% |
| Carbon dioxide | 0.032% |

| | |
|-----------------|--------|
| Remaining gases | Traces |
|-----------------|--------|

2. STRUCTURE OF ATMOSPHERE

| Name of the zone | Extended up to (km) |
|----------------------------|---------------------|
| Troposphere | 0 to 12 km |
| Stratosphere | 12 to 52 km |
| Mesosphere | 52 to 92 km |
| Ionosphere or thermosphere | 92 km |

3. SOURCES OF AIR POLLUTION

a. Natural sources of air pollution:

b. Man-made sources of air pollution:

- (i) Combustion of fuels: (CO_2), (SO_2), (NO_2).etc. accumulate in the atmosphere.
- (ii) Industries: Emit undesirable gases (SO_2 , CO_2 , NO_2 , NH_3 , CO).
- (iii) Thermal Power Plants: Mainly, they emit Sulphur dioxide.
- (iv) Automobiles: Exhaust contains carbon monoxide (CO), methane, un-burnt carbon. CO is the main source of air pollution in congested cities.
- (v) Agricultural activities: Crop spraying and field burning.
- (vi) Nuclear Power Plants: Emit various radioactive substances

Air pollutants are classified according to how they are formed, and this classification is primarily categorized into three distinct types

3.1. Based on Origin

1. Primary air pollutants
2. Secondary air pollutants

3.2. Based on Nature

1. Organic air pollutants
2. inorganic air pollutants

3.3. Base on the State of Matter

1. Particulates (Aerosols)
2. Gasses and vapours

a. Primary air pollutants

- Particulate matter such as dust and aerosols.
- Pollens
- Sulphur compounds (SO_2 , SO_3 , H_2S)
- Nitrogen compounds (NO , NO_2 , NH_3)
- Carbon monoxide (CO) and carbon dioxide (CO_2)
- Photochemical oxidants

b. Secondary Air Pollutants:

- Ozone
- PAN (Peroxy Acetyl Nitrate)

- SMOG (smoke+fog)
- Acid rain- Sulphuric acid (H_2SO_4) is formed by a simple chemical reaction between sulphur dioxide (SO_2) and water (H_2O) vapour. It causes acid rains.

4. SMOG

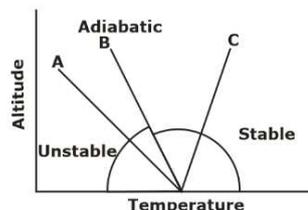
- Smog is a synchronism of two words — i.e., Smoke and Fog.
- **SMOG= SMOKE + FOG**
- Smog can be of two types, Photochemical or Coal induced.
- Smog is caused by the interaction of some hydrocarbons and oxidants under sunlight, giving rise to dangerous Peroxy acetyl nitrate (PAN).
- Smog reduces visibility, causes eye irritation and damage to vegetation.

5. VERTICAL DISPERSION OF POLLUTANTS

- The rate at which dry air cools as it rises is called the dry adiabatic lapse rate. It is independent of the ambient air temperature.
- The dry adiabatic lapse rate can be calculated from the first law of thermodynamics (1.0 per 100m).
- The saturated adiabatic lapse rate is not a constant. Since the amount of moisture that the air can hold before condensation begins is a function of temperature.
- A reasonable average value of the moist adiabatic lapse rate in the troposphere is about $6^\circ C/Km$.

6. ATMOSPHERIC STABILITY

- When the environmental lapse rate and the dry adiabatic lapse rate are the same, Such atmosphere is neutral.
- The atmosphere is super adiabatic when the environmental lapse rate ($-dT/dz$) is greater than the dry adiabatic lapse rate.
- When the environmental lapse rate is less than the dry adiabatic lapse rate, Such an atmospheric condition is called stable, and the lapse rate is said to be sub adiabatic.
- When the ambient lapse rate and the dry adiabatic lapse rate are the same, the atmosphere has neutral stability.
- Super adiabatic condition prevails when the air temperature drops more than $1^\circ C/100m$
- Sub adiabatic conditions prevail when the air temperature drops at the rate of less than $1^\circ C/100m$.



- Temperature profiles to the left of the adiabatic lapse rate correspond to an unstable atmosphere (line) A; profiles to the right are stable (line C). The dry adiabatic lapse rate is line B. The speckled area is meant to suggest slopes that correspond to the wet adiabatic lapse rate.

Vertical temperature gradient: _____

Normal state

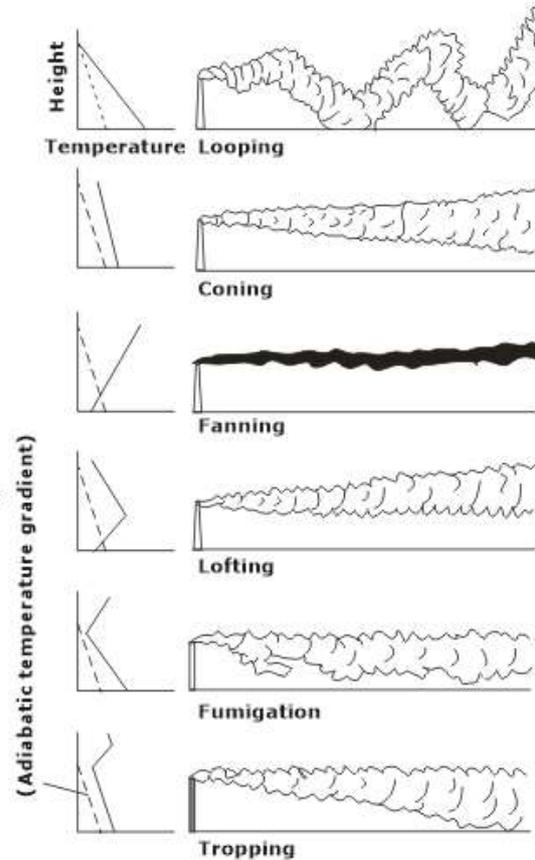
Almost isothermal state

Inverse state

A combination of an inverse state (above the ground) and a normal state (from a height slightly below the chimney orifice)

A combination of a normal state (above the ground) and an inverse state (above the chimney orifice)

A combination of a normal state (above the ground) and an inverse state (above the chimney orifice)



Particulate air pollutants, their sources, and effects

| Pollutant | Sources | Effects |
|--|---|--|
| Suspended particulate matter/dust from domestic, Depends on the specific composition | Smoke from domestic, Depends on the specific composition matter/dust industrial and vehicular soot R | Depends on the specific composition matter/dust industrial and vehicular soot Reduces sunlight and visibility, increases corrosion, Pneumoconiosis, Asthma, cancer, and other lung diseases. |
| Fly ash | Part of the Smoke released from Settles down on vegetation, houses. Adds chimneys of factories and to the suspended particulate matter (SPM) power plants | Settles down on vegetation, houses. Adds chimneys of factories and to the suspended particulate matter (SPM) Power plants in the air. Leachates contain harmful material |

Gaseous air pollutants: their sources and effects

| Pollutant | Source | Harmful effect |
|--|--|---|
| Carbon compound (CO and CO ₂) | Automobile exhaust burning of wood and coal | <ul style="list-style-type: none"> • Respiratory problems • Green house effect |
| Sulphur compounds (SO ₂ and H ₂ S) | Power plants and refineries volcanic eruptions | <ul style="list-style-type: none"> • Respiratory problems in humans • Loss of chlorophyll in plants (chlorosis) • Acid rain |
| Nitrogen Compound (NO and N ₂ O) | Motor vehicle exhaust atmospheric reaction | <ul style="list-style-type: none"> • Irritation in eyes and lungs • Low productivity in plants • Acid rain damages material (metals and stone) |
| Hydrocarbons (benzene, ethylene) | Automobiles and petroleum industries | <ul style="list-style-type: none"> • Respiratory problem • Cancer causing properties |
| SPM (Suspended Particulate Matter) (Any soild and liquid particles suspended in the air, (flush, dust, lead) | Thermal power plants, Construction activities, metalurgical processes and automobiles | <ul style="list-style-type: none"> • Poor visibility, breathing problems • Lead interferes with the development of red blood diseases and cancer. • Smoge (skoke & fog) formation leads to poor visibility and aggravates asthma in patients |
| Fibres (Cotton, wool) | Textiles and carpet weaving industries | <ul style="list-style-type: none"> • Lung disorders |

Air pollution control & its principles

1. Source Control Technology

- Air quality management sets the tools to control air pollutant emissions.
- Control measurements describe the equipment, processes, or actions used to reduce air pollution.
- The extent of pollution reduction varies among technologies and measures.
- The selection of control technologies depends on environmental, engineering, economic factors, and pollutant type.

2. Settling Chambers

- Settling chambers use the force of gravity to remove solid particles.

3. Cyclones

Cyclones are efficient in removing large particles but are not as efficient with smaller particles. For this reason, they are used with other particulate control devices.

4. Venturi Scrubbers

- Venturi scrubbers use a liquid stream to remove solid particles.
- In the venturi scrubber, gas laden with particulate matter passes through a short tube with flared ends and a constricted middle.
- This constriction causes the gas stream to speed up when the pressure is increased.
- The difference in velocity and pressure resulting from the constriction causes the particles and water to mix and combine.

5. Electrostatic Precipitators (ESPs)

- An ESP is a particle control device that uses electrical forces to move the particles out of the flowing gas stream and onto collector plates.
- The ESP places electrical charges on the particles, causing them to be attracted to oppositely charged metal plates located in the precipitator.

Solid Waste Management

- Solid waste refers here to all non-liquid wastes. In general, this does not include excreta.
- Common ordinary household and commercial waste are called refuse or municipal solid waste (MSW).

1. Different categories of solid waste include:

- **Organic waste:** wastes from the preparation of food, market place etc.
- **Combustibles:** paper, wood, dried leaves, packaging for relief items, etc. (high organic and low moisture content)
- **Non-combustibles:** Metal, tin cans, bottles, stones, etc.
- **Ashes/dust:** Residue from fires used for cooking.
- **Bulky waste:** Tree branches, tyres, etc.
- **Dead animals:** Carcasses of domestic animals and livestock.
- **Hazardous waste:** Oil, battery acid, medical waste.
- **Construction waste:** Roofing, rubble, broken concrete, etc.

2. ON-SITE DISPOSAL OPTIONS

2.1. Communal pit disposal

- Consumers dispose of waste directly into a communal pit.
- The size of this pit will depend on the number of people it serves.
- The long-term recommended objective is six cubic meters per fifty people.

2.2. Family pit disposal

- Family pits may provide a better long-term option where there is adequate space.
- These should be fairly shallow (up to 1 m deep).

2.3. Communal bins

- A single 100-litre bin should be provided for every fifty people in domestic areas, one hundred people at feeding centres, and ten market stalls.
- In general, bins should be emptied daily.

3. OFF-SITE DISPOSAL OPTIONS

3.1. Landfilling

- Once the solid waste is transported off-site, it is normally taken to a landfill site.
- Here the waste is placed in a large excavation (pit or trench) in the ground, back-filled with excavated soil each day waste is tipped.
- Ideally, about 0.5m of soil should cover the deposited refuse at the end of each day to prevent animals from digging up the waste and flies from breeding.

3.2. incinerations

- Burning or incineration is often used for the disposal of combustible waste.

3.3. Composting

- Composting is environmentally friendly and beneficial for crops.

3.4. Recycling

It is used to make the waste reuseable.
