HAND WRITTEN NOTES:

OF

CIVIL ENGINEERING

SUBJECT:

ENVIRONMENTAL ENGINEERING
Environment: The environment consists of atmosphere, hydrosphere, lithosphere in which the light resources of the earth are contained.


Ecology is the relationship b/w living organisms with their surrounding environment. The ecological balance is required for sustainable development.

The role of environmental engineers is to design, construct and operate treatment facilities for water, air, solid waste.

Steps in design of water supply scheme for a town:

(i) Assessment of future population during design period and assessment of total water demand.
(ii) To search for the source of water.
(iii) To study water quality parameter.
(iv) Design of water treatment units.
(v) Design of water carriage system.
(vi) Design of distribution reservoir and distribution network.
Water Demand and Population Forecasting

The total water demand consists of following 6 components:

(i) Domestic water demand: According to manual of water supply of Govt. of Indiat should be b/w 135 lpd/dog to 225 lpd. Which is nearly 50 to 60% of total water demand. It is generally taken as 135 lpcd for low income group and 200 lpcd for high income group.

(ii) Industrial water demand: It is generally taken as 50 lpcd however it depends upon type of industry and number of industrial units in the city.

Type of Industry
Sugar
Fertilizer
Pulp and paper (maximum)
Automotive
Leather
Petroleum refining

(iii) Institutional and commercial water demand: On an average 20 lpcd is considered sufficient. But in highly commercial cities it is as high as 50 lpcd.

<table>
<thead>
<tr>
<th>Commercial unit</th>
<th>Water demand</th>
</tr>
</thead>
<tbody>
<tr>
<td>Office</td>
<td>45 lpcd</td>
</tr>
<tr>
<td>School</td>
<td>45 to 135 lpcd</td>
</tr>
<tr>
<td>Hotel</td>
<td>180 lpcd</td>
</tr>
<tr>
<td>Hospitals</td>
<td>350 to 450 lpcd</td>
</tr>
<tr>
<td>Cinematheaters</td>
<td>15 lpcd</td>
</tr>
</tbody>
</table>
iv. Demand for public uses: It includes maintenance of parks, gardens, roads this demand is 5% of total demand.

(v) Fire demand: This quantity is very small for big cities. For population, more than 50 lakh fire demands may be less than 1/lpcd but at the time of breakout of fire this water should be available suddenly.

Guidelines for the fire demand

(i) The minimum water pressure available at the fire hydrant should be 1 to 1.5 kg/cm² and must be maintained for 4 to 5 hrs.

(ii) Three jets should be thrown simultaneously: one on burning body and 2 on adjacent bodies.

(iii) Discharge at each stream should be about 1100 1/hr/ per minute.

(iv) No. of fire jets required depend on the size of population and is given by \( F = 2.8 \sqrt{P} \)

\( P \) - Population on thousands.

If population is 40,000 then \( P = 40 \)

(v) The rate of water demand for fire depends on population and can be computed using the following empirical formula.

Kuichling formula \( Q = 3182 \sqrt{P} \)

\( P \) - in thousands.

\( P = 5000 \) then \( Q = 5 \)

Freeman formula \( Q = 1136 \left( \frac{P}{10} + 10 \right) \)

\( Q \) = 1/hr/min \( P \) = thousand

National bowed formula

(American insurance Association formula)
(a) For high valued commercial cities population ≤ 2 lakhs,

\[ Q = 4637 \sqrt{P} \left(1 - 0.01P\right) \]

(b) For high valued commercial cities population > 2 lakhs,

\[ Q = 54600 \text{ a/min. and extra provision second and third fire.} \]

For residential cities

- Small buildings. \( Q = 2200 \text{ lhr/min} \)
- High buildings. \( Q = 4500 \text{ lhr/min} \)
- High valued apartments. \( Q = 7650 \text{ lhr/min} \)

**Boston Formula** \( Q = 5.663 \sqrt{P} \)

6. Water Demand For losses and theft

On an average, 15% of total demand may be provided for losses and theft, which is nearly 55 lpcd.

Total maxm water demand. It is the sum of above 6 demands, and IS code permits for India total maxm demand of 335 lpcd.

<table>
<thead>
<tr>
<th>Domestic Demand</th>
<th>200 lpcd</th>
</tr>
</thead>
<tbody>
<tr>
<td>Industrial</td>
<td>50 lpcd</td>
</tr>
<tr>
<td>Commercial</td>
<td>20 lpcd</td>
</tr>
<tr>
<td>Public</td>
<td>10 lpcd</td>
</tr>
<tr>
<td>Waste &amp; theft</td>
<td>55 lpcd</td>
</tr>
<tr>
<td>Fire</td>
<td>&lt; 1 lpcd</td>
</tr>
</tbody>
</table>

Total ≈ 335 lpcd.
The total water demand depends on size of population and for the design of water supply scheme for a given population size following guidelines may be adopted:

Size of Population

- < 50,000
- 50,000 to 2,000,000
- 2,000,000 to 10,000,000
- > 10,000,000

Total water demand

- 110 to 150 lpcd
- 150 to 240 lpcd
- 240 to 300 lpcd
- 300 to 335 lpcd

Factors affecting water demand:

(i) Size of population
(ii) Climatic conditions
(iii) For industrial and commercial activities
(iv) Habits of people
(v) Quality of water supply
(vi) Pressure available in distribution system
(vii) Development of sewage facilities
(viii) Cost of water

Variation in water demand:

Smaller towns have more variation

(i) Max \( m \) monthly consumption \( \rightarrow 1.28 \) times average monthly consumption

(ii) Max \( m \) weekly consumption \( \rightarrow 1.48 \) times average weekly

(iii) Max \( m \) daily consumption \( \rightarrow 1.8 \) time average daily consumption

(iv) Max \( m \) hourly consumption peak demand \( \rightarrow 2.7 \) time of average hourly consumption

It is also equal to 1.5 times max \( m \) daily hourly consumption.
Goodrich formula to compute maxm/peak demand

\[ P = 1.8 (t)^{-0.1} \]

\[ P = \frac{\text{maxm demand}}{\text{Aug demand}} \]

\[ t = \text{time in days} \]

\[ t = 1 \text{ for maxm daily} \]

\[ t = \frac{1}{24} \text{ for maxm hourly} \]

"GSI manual of water supply recommended peak factor btw 2.5 to 3.0 depending upon population size."

"Coincident Draft" It is the maxm of

1. Maxm daily demand = P e demand
2. Maxm hourly demand

Design periods of water supply unit:

- Water treatment unit
- Dams and reservoirs
- Wells and tube wells
- Main supply pipes (water main)
- Water treatment units
- Overhead tank, sewage reservoir

Design Discharge
Design period
Average annual demand
50yrs
Maxm daily demand
30-50yrs
Maxm daily demand
30yrs
Design period for fire demand, breakdown storage
15yrs
To care of hourly fluctuation
Population Forecasting methods

**Total population (P)**

![Graph showing population growth over time]

- **AB**: \( \frac{dp}{dt} \propto p \) → Increasing growth rate
- **BCD**: \( \frac{dp}{dt} = \text{const} \) → High growth rate
- **DE**: \( \frac{dp}{dt} \propto (P_s - p) \) → Decreasing growth rate

**Methods of population forecasting**

- (i) Arithmetic increase method
- (ii) Geometric increase method
- (iii) Incremental increase method
- (iv) Decreasing rate of growth method
- (v) Geographical extension method
- (vi) Geographical comparison method
- (vii) Zoning or master plan method
- (viii) Ratio or co-relation method
- (ix) Logistic curve method or Verhulst method
1. Arithmetic increase method

This method assumes that population increases at a constant rate:
\[ \frac{dp}{dt} = \text{constant} \]

If \( P_0 \) is population of last known decade then population after \( n \) decades will be

\[ P_n = P_0 + n\bar{x} \]

where \( \bar{x} \) = average arithmetic mean of population increase over the given decades.

Example:

Population of 5 decades is given below find the population in the year 2040, using arithmetic increase method.

<table>
<thead>
<tr>
<th>Year</th>
<th>Population</th>
<th>Increase in population</th>
</tr>
</thead>
<tbody>
<tr>
<td>1970</td>
<td>25,000</td>
<td></td>
</tr>
<tr>
<td>1980</td>
<td>28,000</td>
<td>3,000</td>
</tr>
<tr>
<td>1990</td>
<td>31,000</td>
<td>3,000</td>
</tr>
<tr>
<td>2000</td>
<td>42,000</td>
<td>8,000</td>
</tr>
<tr>
<td>2010</td>
<td>59,000</td>
<td>5,000</td>
</tr>
</tbody>
</table>

\[ \bar{x} = \frac{22,000}{4} = 5,500 \]

\[ P_{2040} = P_0 + n\bar{x} \]

\[ = 47,000 + 3 \times 5,500 \]

\[ P_{2040} = 63,500 \]

2. Geometric increase method / Uniform increase method

Compounding rate method
Geometric Increase method

But since geometric average is always less than arithmetic average hence for all practical approach.

conservative approach

Example: Population of a town is given in the table using geometric increase method find population in year 2040.

<table>
<thead>
<tr>
<th>Year</th>
<th>Popn</th>
<th>% Growth Rate</th>
</tr>
</thead>
<tbody>
<tr>
<td>1970</td>
<td>23,000</td>
<td>12%</td>
</tr>
<tr>
<td>1980</td>
<td>28,000</td>
<td>21.43%</td>
</tr>
<tr>
<td>1990</td>
<td>34,000</td>
<td>23.53%</td>
</tr>
<tr>
<td>2000</td>
<td>42,000</td>
<td>11.90%</td>
</tr>
<tr>
<td>2010</td>
<td>47,000</td>
<td></td>
</tr>
</tbody>
</table>

\[ n = \left( n_1, n_2, n_3, n_4 \right)^{1/4} \]

\[ = \left( 12 \times 21.43 \times 23.53 \times 11.90 \right)^{1/4} \]

16.38%
\[ P_n = P_0 \left(1 + \frac{r}{100}\right)^n \]

\[ P_{2040} = 47,009 \left(1 + \frac{18.38}{100}\right)^3 \]

\[ P_{2040} = 74,408.7 \]

**Incremental increase method**

It is also called method of varying increment. This method combines both arithmetic and geometric average method and this method is used when growth is not constant.

Population after \( n \) decades as given as:

\[ P_n = P_0 + n \bar{X} + \frac{n(n+1)}{2} \bar{Y} \]

where \( \bar{X} \) average increase in population of known decades

\( \bar{Y} \) average of incremental increase of the known decades.

Note: Geometric increase method gives high results which is suitable for cities growing with fast rate such as new cities whereas as arithmetic increase method gives low result which is suitable for cities growing with slow rate such as old cities. However, incremental increase method gives moderate results which can be used for new and old cities both.

*Find the population in the year 2040 using incremental increase method.*
Year | Pop. (m) | Increase | Incremental Increase
--- | --- | --- | ---
1970 | 25,000 | | |
1980 | 28,000 | 3000 | |
1990 | 34,000 | 6000 | |
2000 | 42,000 | 8000 | |
2010 | 47,000 | 5000 | |

\[ \overline{x} = \frac{22,000}{4} = 5,500 \]

\[ \gamma = \frac{2000}{3} = 666.67 \]

\[ P_0 = 47,000 + 3 \times 5,500 + \frac{3(3+1)}{2} \times \frac{2000}{3} \]

\[ P_{2010} = 67,500 \]

The results of this method are moderate to/so arithmetic average method and geometric average method.

4. Decreasing growth rate method:

If population is approaching saturation and growth rate is decreasing then this method is suitable. In this method, average decrease in the % increase is calculated and then subtracted from the last % increase. Computation are made for each successive years.

(i) Find % increase in population for each decade
(ii) and workout decrease in % increase in each decade and find average % decrease say \( x' \).
(iii) The population of next decade from the last known decade is given as

\[ P_1 = P_0 + \left( \frac{100 - x'}{100} \right) P_0 \]
where $P_0$ = Population of last known decade.
$R_0$ = Growth rate of last decade
or Average decrease in growth rate.

Population after 2 decades from the last known decade is given as

$$P_2 = P_1 + \left( \frac{(R_0 - 2R')}{100} \right) P_1$$

Example

The census record of a particular town is shown in table. Estimate the population for the year 2020 by decreasing growth rate method.

<table>
<thead>
<tr>
<th>Year</th>
<th>Population</th>
<th>% Increase</th>
<th>Decrease in %/6Dr</th>
</tr>
</thead>
<tbody>
<tr>
<td>1960</td>
<td>55,500</td>
<td>14.97%</td>
<td></td>
</tr>
<tr>
<td>1970</td>
<td>63,500</td>
<td>11.93%</td>
<td>-2.84%</td>
</tr>
<tr>
<td>1980</td>
<td>71,300</td>
<td>11.50%</td>
<td>-0.43%</td>
</tr>
<tr>
<td>1990</td>
<td>79,500</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

$R_0$ = Growth rate of last known census = 11.50%.

or Average decrease in $G.R.$ = \( \frac{2.84 + 0.43}{2} \) = 1.635%.

$$P_{2010} = P_{2010} + \left( \frac{R_0 - 3R'}{100} \right) P_{2010}$$

$$P_1 = P_{2010} + \left( \frac{11.5 - 1.635}{100} \right) \times 79,500$$

$P_{2010} = 87,343$

$$P_{2015} = 81,343 \times \left( \frac{11.5 - 2.84 - 1.635}{100} \right) \times 79,500$$

$P_{2010} = 94,581$
(a) Graphical extension method or simple graphical method

A graph is plotted b/w population and time for available data and the curve is smoothly extended for the desired year. For example let population is desired in the year 2030 and let population data are known for the last 50 years 1970 - 2010.

\[ P_{2000} = 94581 + \left( \frac{11.5 - 3 \times 1.635}{100} \right) \times 94581 \]

\[ P_{2020} = 1,000,765 \] Ans.

(b) Comparative Graphical method

Cities of similar condition and characteristics are selected which have grown in similar fashion in the past and their graphs are plotted. The mean graph from available data may be used for required city.
Here it required population of Lucknow for year 2040. Growth of Lucknow is from 2010 to 2040 will be similar to the which is avg. of growth of

1. Delhi - 1940 to 1970
2. Mumbai - 1930 to 1960
3. Varanasi - 1980 to 2010

Master plan or zoning method:

Generally in metropolitan cities growth is controlled by development authorities in a planned manner only those extension are allowed which are proposed in master plan. For example let no. number of flats will be constructed in next decade and 4 people allowed in one flat then population added in 2 decade will be 16m

Ratio or apportionment method:

In this method population of any town is expressed as a % of population of whole country and by taking the average growth rate of country population may be projected.

Logistic Curve method or Verhulst's method:

Let \( P_0 \) population at the beginning of census record,
\( P_1 \) population after time \( t_1 \) years,
\( P_2 \) population after time \( t_2 \) years,
\( P_s \) saturation population.
Then population after any time from the start is given as

\[ P = \frac{P_S}{1 + m \log e \left( \frac{m t}{t_0} \right)} \]

\[ P_S = \frac{2P_0 P_1 P_2}{P_0 P_2 - P_1^2 (P_0 + P_2)} \]

\[ m = \frac{P_S - P_0}{P_0} = \text{const} \]

\[ n = \frac{1}{t_1} \log e \left[ \frac{P_0 \left(P_S - P_1\right)}{P_1 \left(P_S - P_0\right)} \right] \]

**Example:** In two periods of each 20 years a city has grown from 30,000 to 1,70,000 and then to 3,00,000. Determine

iv. Saturation population

vii. Equation of Logistic growth

viii. Population after 60 years from the start
\[ P_0 = 20,000 \quad P_2 = 3,000,000 \]
\[ P_1 = 1,700,000 \quad t_1 = 30 \text{ years} \]
\[ t_2 = 40 \text{ years} \]
\[ t = 60 \text{ years} \]

1. \[ P_s = \frac{2P_0 P_1 P_2 - P_1^2 (P_0 + P_2)}{P_0 P_2 - P_1^2} \]
\[ P_s = 3,26,000 \]

2. \[ m = \frac{P_s - P_0}{P_0} \]
\[ \frac{3,26,000 - 30,000}{30,000} = 9.87 \]

\[ \eta = \frac{1}{30} \log_e \left[ \frac{30,000 (3,26,000 - 1,79,000)}{1,79,000 (3,26,000 - 30,000)} \right] \]
\[ \eta = 0.119 \]

\[ P = \frac{3,26,000}{1 + 9.87 \log_e^{-1} (-0.119 t)} \]

3. If \( t = 60 \text{ years} \)
\[ P = \frac{3,26,000}{1 + 9.87 \log_e^{-1} (-0.119 \times 60)} \]
\[ P = 3,23,000 \]
Water sources and their distribution in earth surface:
(i) Oceans - 97.3%
(ii) Ice and Glaciers - 2.17%
(iii) Ground water - 0.61%
(iv) Fresh water in lakes - 0.009%
(v) Salinewater in lakes - 0.008%
(vi) Fresh water in rivers - 0.0001%

Water Quality Parameters

Physical
- Suspended solids
- Turbidity
- Colour
- Taste and odour
- Temperature

Chemical parameters
- Dissolved solids
- pH
- Alkalinity
- Hardness
- Fluorides
- Chlorides
- Nitrates
- BOD, COD (organics)
- Metals

Biological parameter
- Pathogenic bacteria, viruses, protozoa, helminths
- Non pathogenic
- Coliforms

Physical parameters
Suspended solids: Total solids may be present in the form of
(i) Suspended solids: 10^-01 to 10^-03 mm size
(ii) Colloidal solids: 10^-03 to 10^-06 mm size
(iii) Dissolved solids: < 10^-06 mm size

Suspended & Colloidal
Suspended solids may consist of inorganic and organic content like silts, clays, immiscible liquids like oil and greases, plant fibers and algae etc.

These are undesirable because:

1. They make water aesthetically displeasing.
2. They are biologically active may cause growth of disease producing micro-organism.
3. These may attract or absorb chemical and biological agents.

Measurement is done by gravimetric test involving mass of residues measurement.

The total solids can be determined by evaporation and if dried mass is burned at 600°C, then organic matters will oxidised and only inorganic matter will be left.

$$\text{Organic matter} \xrightarrow{\text{Heat at 600°C}} \text{CO}_2 \text{f} + \text{H}_2\text{O} \text{f} + \text{other gases}$$

Note- (i) The permissible total solids (Dissolved + Suspended) < 500 ppm (mg/L) for drinking water.

and in no case it should be greater than 2000 ppm.

(ii) Permissible limit of suspended solid is 30 mg/L for drinking water according to environmental protection agency E.P.A.
2. Turbidity: Turbidity is a measure of the extent to which light is either absorbed or scattered by suspended material in water. It is not a direct measure of the quantity of suspended solids. Mostly turbidity is due to colloidal materials like clay, silts, stock fragments, metal oxides, vegetation fibres and microorganisms.

Due to presence of turbidity there is interference in light penetration which affects photosynthesis and disinfection of turbid water is difficult because suspended solids may physically seal sites where the organisms from disinfection.

Measurement of Turbidity:

It is measured photo-chromatically by determining the % of light of a given intensity i.e. either absorbed or scattered.

Measuring Devices:

(i) Turbidity Rod
(ii) Jackson Turbidity Meter
(iii) Baylis Turbidimeter (scattering principle)
(iv) Modern Nephelometer (scattering principle)

(ii) Turbidity Rod: Turbidity rod with platinum needle is inserted inside the water and the depth at which platinum needle just becomes invisible gives the turbidity in mg/litre or ppm.

x The Turbidity is expressed in a standard unit called JTU (Jackson Turbidity Unit) which is based absorption principal.

x One JTU is equivalent to turbidity produced by 1 mg of fine silica (SiO₂) dissolved in one litre of distilled water.
Jackson Turbidimeter - A burning candle is placed below a Jackson Turbidimeter and visibility is determined by measuring the depth of water when detecting candle is just distinguishable from the top. Longer the visibility lesser is the turbidity.

Turbidity of 25 FTU or more is detectable by the device.

10.8 cm path is equivalent to 200 FTU and 25 20.5 cm path is equivalent to 100 FTU.

Baylis Turbidimeter and Nephelometer are based on colour matching technique i.e. on scattering principle in this method small turbidity less than 1 unit can also be measured. Hence can be used for domestic water supply.

Since it is based on scattering principle hence turbidity caused by dark substances which absorb light rather than reflected should be measured by absorption technique.

In this method standard unit is FTU or NTU (Formazin turbidity unit)

1 FTU = Turbidity produced by 1mg of formazin polymer dissolved in one liter of distilled water.

Note: For drinking water permissible limit of turbidity is 5 to 10 FTU but it should be as less as 1 FTU.
Temperature 2- Temperature affects chemical and biological activities if temperature increases by 10°, biological activities are doubled. Dissolved temp for drinking water at 10°C and in no case should be more than 25°C.

By increasing, temp solubility of dissolved gases decreases. At 20°C DO (Dissolved oxygen) in water is nearly 9.2 mg/ltr. If DO falls below 4 mg/ltr then water species may die such as fish, crabs, lobsters.

Thermal shock - If hot water powerplants and automobile industry of discharged in natural streams then due to resist of heat, DO may fall below 4 ppm which may dead in death of water species. Such an stage is called thermal shock.

CHEMICAL WATER Quality Parameters:

Total Dissolved Solids

Dissolved Solids - Dissolved substances may be organic or inorganic. Inorganic minerals, inorganic gases whereas organic may be plant fibers, organic gases, organic chemicals.

TDS are often determined by measuring electrical conductivity of water using ionic water tester. The ability of water to conduct electricity...
**Colour:**

Colour is due to either suspended matter or dissolved matters. Colours caused by suspended matter is called apparent colour which can be removed by filtration. Where as colour caused by dissolved solid is called true colour which is due to dissolved mineral, metals and gases.

Measurement of Colour:

It is done by colour matching techniques. True colour is measured on Burgess scale by Nessler's tube.

The standard unit of colour is TCU (true colour unit). 1 TCU is equivalent to colour produced by 1 mg of platinum cobalt in the form of chloro platinate ions mixed in one 1th of distilled water. (It has yellowish brown colour).

Note:
The colour testing must be done within 12 hrs of collection of sample otherwise biological activity may alter the colour.

The permissible limit of colour for drinking water is 5 ppm and in no case > 20 ppm.

**Taste and odour:**

Taste and odour are expressed by threshold odour number at supress dilution ratio at which odour can not be detected.

\[ T-ON = \frac{A + B}{A} = \frac{\text{Vol. of diluted sample}}{\text{Vol. of undiluted sample}} \]

A: Volume of water sample undiluted
B: Volume of distilled water required to be added

The permissible limit for drinking water is 1 TON and in no case it should be more than 1.
Alkalinity: It is defined as the quantity of ions in water that will react to neutralise hydrogen ion. It means it is solubility of water to neutralise acids. Major compound causing alkalinity are CO₃⁻ bicarbonate Caustic alkalinity

Minor compounds are + HS₄O₃⁻, H₂BO₃⁻, HPO₄²⁻ & H₂S⁻

For purpose of computation alkalinity caused by minor compound is neglected. The alkalinity in water comes due to minerals or due to atmospheric CO₂ mixed in water or due to microbial decomposition of organic compound.

1. CO₂ + H₂O ⇌ H₂CO₃ [Dissolved CO₂ & Carbonic acid]
2. H₂CO₃ ⇌ H⁺ + HCO₃⁻ [Bicarbonate]?
3. HCO₃⁻ ⇌ H⁺ + CO₃²⁻ [Carbonate]?
4. CO₃²⁻ + H₂O ⇌ HCO₃⁻ + OH⁻ [Hydroxide]

The fourth reaction is a weak reaction but algae present in water then reaction may take place.
Due to consumption of carbonate, Algae may produce OH ion which may cause high pH, 9 to 10.

If pH is found less than 9 then usually OH may not present.

The reaction \( Ca(OH)_2 \) may combine alkalinity of water and may form solid precipitate of \( CaCO_3 \), which may get deposited on pipe surface and may cause incrustation in pipe. (Tuberculosis)

**Notes:** If water is acidic then there will be corrosion reaction.

**Measurement of Alkalinity**

Measurement is done by titration of water with an acid. The alkalinity is expressed in mg/l as \( CaCO_3 \). If 0.02 M \( H_2SO_4 \) is used as titrant then 1 ml of acid will neutralize 1 mg of alkalinity expressed as \( CaCO_3 \).

\[
\begin{align*}
OH^- + H^+ &\rightleftharpoons H_2O \\
CO_3^{2-} + H^+ &\rightleftharpoons HCO_3^- \\
HCO_3^- + H^+ &\rightleftharpoons H_2CO_3
\end{align*}
\]

If acid is added then pH of water falls and this recorded gradually during titration then following curve will be obtained.
The affinity of OH ions to react with H⁺ ion is maximum followed by affinity of CO₃²⁻ and the least HCO₃⁻.

If pH falls from original level to 8.3 then all the hydroxyl ion and \( \frac{1}{2} \) CO₃ ions are neutralised when pH falls from 8.3 to 4.5 then remaining half of carbonate ions and bicarbonate ions are neutralised.

Let \( P \) = Amount of acid reqd. to reach pH from original level to 8.3.
\( M \) = Amount of acid reqd. to reach pH from original level to 4.5.

Conclusion ⇒
\[ \text{If } P = M \Rightarrow \]
alkalinity is due to OH ions.

\[ \text{If } P = \frac{M}{2} \Rightarrow \text{It means there is carbonate alkalinity} \]

\[ \text{If } P = 0 \text{ and } M > 0 \Rightarrow \text{Hence there is bicarbonate alkalinity} \]

\[ \text{If } P = 0 \text{ and } M = 0 \Rightarrow \text{No alkalinity} \]

\[ \text{If } P < \frac{M}{2} \Rightarrow \text{Bicarbonate and carbonate ions are predominant} \]

\[ \text{If } P > \frac{M}{2} \Rightarrow \text{OH⁻ ion and } CO₃^{2⁻} \text{ are predominant} \]

Note: Mol wt of a compound = Sum of atomic wt (gms)
\[ \text{CaCO}_3 = 40 + 12 + 3 \times 16 = 100 \text{ gm} \]
\[ \text{H}_2\text{SO}_4 = 1 \times 2 + 32 + 16 \times 4 = 98 \text{ gm} \]
\[ \text{NaCl} = 23 + 35 \]
\[ \text{NaOH} = 23 + 16 + 1 = 40 \text{ gm}. \]

(ii) Equivalent weight \(= \frac{\text{Mol. wt}}{\text{Valency}} \)

\[ \text{Eqwt. of CaCO}_3 = \frac{100}{2} = 50 \text{ gm} \]
\[ \text{H}_2\text{SO}_4 = \frac{36}{2} = 36 \text{ gm} \]
\[ \text{NaOH} = \frac{40}{1} = 40 \text{ gm} \]

(iii) 1 Mole = Mol wt. of compound

1 Mole of CaCO\(_3\) = 100 gms CaCO\(_3\)

1 Mole of \(\text{H}_2\text{SO}_4\) = 98 gms \(\text{H}_2\text{SO}_4\)

1 Mole of \(\text{NaOH}\) = 40 gms \(\text{NaOH}\)

(iv) Concentration \(= \frac{M}{L}\) (mole/L).

If 1 mole of CaCO\(_3\) is present in 1 Ltr of water then Con\(^o\) is 1 M/L of caCO\(_3\).

It means 100 gms of CaCO\(_3\) is present in 1 Ltr of dist. water = 1 M/L

If 1 gm of CaCO\(_3\) is present in 1 Ltr then Con\(^o\) = \(\frac{1}{100}\) M/L

\[ = 0.01 \text{ M CaCO}_3 \]

No. of Moles = \(\frac{\text{Wt. in gms}}{\text{Mol. wt. in gms}}\).
If 1490 gm of H₂SO₄ is present in water then
conc. us: \( \frac{1490}{98} \) M/L
\[ \frac{1490}{98} = 5\text{ M/L} \]

If 490 mg of H₂SO₄ is present then conc. us
\[ \frac{490}{0.005} = 0.005\text{ M H₂SO₄} \]

1 gm equiv. = Eqv. unit of compound in gms
1 gm equiv. of CaCO₃ = 50 gm of CaCO₃
\[ \text{H₂SO₄} = \frac{49\text{ gm}}{98} \text{ H₂SO₄} \]

\[ \text{NaOH} = \frac{40\text{ gm}}{98} \text{ NaOH} \]

1N = No. of gm eq. present in 1 ltr of water

Ex. 1490 mg of H₂SO₄ = \( \frac{1490 \times 10^{-3}}{98} \) = 0.01 gm equiv.

If 490 mg of H₂SO₄ is present in 1 ltr of water

Then Conc. us. = 0.01 N H₂SO₄

If 980 mg of H₂SO₄ is present in 1 ltr of water then Conc. us = 0.02 N H₂SO₄.

1M = 1N if valancy is 1

1Mole/lt = 1 gm equiv/L of NaOH

1M = 1N of NaOH

If valancy 1M = 2N

2. \( 0.01\text{M} = 0.02\text{N} \) \( 1\text{M} = 3N \) if valancy > 1
1 gm equiv. of OH⁻ is equal to 1 gm equiv. of carbonate ions (CaCO₃)

Note: If x gm equiv. of OH⁻ ions and y gm equiv. of carbonate ions and z gm equiv. of bicarbonate ions are present than total alkalinity is equal to (x + y + z) gm equiv. of CaCO₃.

1 gm equiv. of OH⁻ ions = 1 gm of CaCO₃
1 gm equiv. of CO₃⁻ = 1 gm of CaCO₃
1 gm equiv. of HCO₃⁻ = 1 gm of CaCO₃

The total alkalinity in terms of weight of CaCO₃ = x + y + z

Usually alkalinity is represented in mg/l as CaCO₃.

Question: A sample of water contains 210 gm of CO₃²⁻, 122 gms of HCO₃⁻ and 68 gms of OH⁻. Find total alkalinity in the same water expressed as CaCO₃.

\[
gm \text{ equiv. of CaCO}_3 = \frac{\text{wt. of CO}_3^2}{\text{Equiv. wt. of CO}_3^2} = \frac{210}{30} = 7 \text{ gm equiv. (x)}.
\]

\[
gm \text{ of HCO}_3^- = \frac{\text{wt. of HCO}_3^- \text{ in gms}}{\text{Equiv. wt. of HCO}_3^-} = \frac{122}{61} = 2 \text{ gm equiv. (y)}.
\]

\[
gm \text{ equiv. of OH}^- = \frac{68}{17} = 4 \text{ gm equiv. (z)}.
\]

Total alkalinity = 7 + 2 + 4 = 13 gm equiv. of CaCO₃

Test of CaCO₃ = 13 x 50 = 650 gms
A 200 ml of sample of water has an initial pH of 4.5. 30 ml of 0.02 N H₂SO₄ is required to titrate the sample to fall pH to 8.3. Then find:

(i) Caustic Alkalinity (ii) Bicarbonate Alkalinity

Carbonate " Expressed as CaCO₃.

200 ml of sample of water
1 ml of 0.02 N H₂SO₄ neutralizes 1 mg of alkalinity as CaCO₃.

30 ml of 0.02 N H₂SO₄ will " 30 mg of alkalinity as CaCO₃.

It means 30 mg alkalinity as CaCO₃ is present in 200 ml of water.

Total alkalinity per litr of water = \( \frac{30 \times 1000}{200} \) = 150 mg/litr.

OH⁻ alkalinity as CaCO₃ = 5 mg/litr

\( \text{CO}_3^{2-} \) alkalinity as CaCO₃ + HCO₃⁻ alk as CaCO₃

\( \times 150 \times 5 \)

= 1125 mg/litr.
200 ml of sample of water.

\[ \text{OH}^- \text{ alkalinity present in 1 ltr} = 5 \text{mg/L as CaCO}_3 \]

\[ \because 200 \text{ ml of water} = \frac{5}{1000} \times 200 \]

\[ = 1 \text{ mg per 200 ml of water} \]

It means 1 ml of water 0.02 N\text{H}_2\text{SO}_4 will be used to neutralise 1 mg of \text{OH}^- as \text{CaCO}_3.

Hence 0.02 N\text{H}_2\text{SO}_4 used to neutralise \( \text{CO}_3^{2-} + \text{HCO}_3^- \) as \text{CaCO}_3 = 30 - 1 = 29 ml.

11 ml of 0.02 N\text{H}_2\text{SO}_4 is used for \( \text{OH}^- + \frac{1}{2} \text{CO}_3^{2-} = 11 \text{ ml} \)

\[ \Rightarrow \frac{1}{2} \text{CO}_3^{2-} = 11 - 1 = 10 \]

\[ \text{CO}_3^{2-} = 20 \text{ ml} \]

\( \text{CO}_3^{2-} \) ion conc. as \text{CaCO}_3 in 1 ltr of water

\[ = \frac{20}{200} \times 1000 \]

\[ = 100 \text{ mg/L as CaCO}_3 \]

\( \text{HCO}_3^- = 150 - 100 - 5 = 45 \text{ mg/L as CaCO}_3 \)
(3) pH of water
It represents the presence of $H^+$ ion concentration.
It is given as
\[ \text{pH} = -\log_{10}(H^+) \]
$H^+$ = Hydrogen ion concentration in mole/litre.
It is measured by potentiometer or colour matching indicators. Indicators used are:
(i) Methyl orange (Acidic indicator)
(ii) Phenolphthalein (Basic indicator).
For drinking water permissible pH is 7 to 8.5 but it should not be less than 6.5 and not greater than 9.2.

(4) Hardness
It is defined as concentration of multi valent cations present in solution. Hardness cation will react with anions in the water to form solid precipitate.
Note: The multi valent cation may be Ca$^{2+}$, Mg$^{2+}$, Al$^{3+}$, Fe$^{3+}$, Mn$^{2+}$, Sr$^{2+}$.
Mostly calcium and magnesium ions are predominant.
Hardness may be classified into 2 parts.
(i) Carbonate hardness (C-H)
\[ \text{Temporary hardness} \]
\[ \downarrow \]
\[ \text{Carbonate and } HCO_3^- \]
\[ \downarrow \]
\[ \text{Ca}^{2+} \text{ and } Mg^{2+} \text{ (other negligible)} \]
(ii) Noncarbonate hardness (N-C-H)
Permanent hardness
\[ \downarrow \]
\[ Cl^- \text{, } SO_4^{2-} \text{ and } NO_3^- \]
\[ \downarrow \]
\[ \text{Ca}^{2+} \text{ and } Mg^{2+} \text{ (other negligible)} \]
Hard water prevents formation of foam of or leather increase laundry expenses.

Hard water also cause incrustation in pipe and boiler. Hence boiler feed water should be free of hardness. Usually hardness is not harmful to digestive system but the taste is bad.

However excess presence of magnesium sulphate has laxative effect.

Calcium hardness does not cause any health hazard.

Temporary hardness can be removed by boiling or adding lime and permanent hardness can be removed by lime soda method and

Hardness is expressed as $CaCO_3$ equiv of $Ca^{2+}$, $Mg^{2+}$ and other multi valent

There are 3 methods to determine hardness

(a) Varsonate method (EDTA method)
(b) Clark’s Mtd
(c) Henner’s mtd

In Varsonate method hardness in water is determined by titration with varsonate stock.

EDTA is used with o-cresol black T as indicator. 0.01M EDTA is used for 1ml of titrant to neutralise (mg of hardness expressed as $CaCO_3$.

Total hardness is equal to

$$T-H = gm eqv of Ca^{2+} + gm eqv of Mg^{2+}$$
T.H = \left[ \text{gm equiv. of Ca}^{2+} + \text{gm equiv. of Mg}^{2+} + \text{gm equiv. of other multi valent ions} \right] \times \text{eqw. wt. of Ca}^{2+}

T.H \text{ as CaCO}_3 \text{ in gms}

\text{T.H. in mg/litre} = \left[ \frac{\text{conc. of Ca}^{2+}}{\text{in mg/l} \times \frac{20}{2}} + \frac{\text{conc. of Mg}^{2+}}{12} \right]

+ \left[ \frac{\text{conc. of other compound}}{\text{in mg/l}} \times \frac{\text{equiv. wt. of that compound}}{\text{mg/l}} \right]

\text{If other than Ca}^{2+}, \text{Mg}^{2+} \text{ ions are neglected then}

\text{T.H.} = \left[ \frac{\text{Ca}^{2+} \text{conc. in mg/l}}{50} + \frac{\text{Mg}^{2+} \text{conc. in mg/l}}{12} \right] \times 50

\text{mg/litre as CaCO}_3

Comparative study of Alkalinity and Hardness:

Alkalinity causing compds:

\text{CaCO}_3, \text{MgCO}_3, \text{Na}_2 \text{CO}_3

\text{Ca(HCO}_3)_2, \text{Mg(HCO}_3)_2, \text{NaHCO}_3

\text{(Ca(OH)}_2, \text{Mg(OH)}_2, \text{NaOH}

\text{Alkaline}}

\text{cause negligible hardness (effective only on very high pH).}
\[
\begin{align*}
Na_2CO_3 + CaCl_2 & \rightarrow CaCO_3 \downarrow + 2NaCl \\
2NaHCO_3 + CaCl_2 & \rightarrow 2NaCl + Ca(HCO_3)_2
\end{align*}
\]

Usually, raised alkalinity is negligible in water; and if permanent hardness is present, then salts such as chloride, sulphate, and nitrate of Ca and Mg react with sodium carbonate and bicarbonate, to convert them into \( \text{Ca}^{2+} \) and \( \text{Mg}^{2+} \) salt. Hence, sodium alkalinity will be present only when permanent hardness is absent. Under such conditions, carbonate hardness = \( T - H \).

**Case I** If permanent hardness is present then \( T - H \geq C.H. \) and \( C.H. = \text{Alkalinity} \).

**Case II** If permanent hardness is absent then \( T - H = C.H. \)

Alkalinity \( \geq C.H. \).

From case I: Permanent Hardness
\[ \begin{align*}
N:C.H. & = T - H = \text{Alkalinity} \\
\text{Hardness is expressed as mg/litr of CaCO}_3 \text{ of ppm or may be expressed in terms of degree of hardness.} \\
1 \text{ British degree of hardness} = 14.25 \text{ mg/litr and} \\
1 \text{ French} & = \quad \text{=} \quad 10 \text{ mg/litr of CaCO}_3
\end{align*} \]
Hardness in mg/ltr

0 to 55
55 to 100
100 to 200
200 to 500

Type of water

Soft
Slightly Hard
Moderate Hard
Very Hard

Note: For drinking water, desirable hardness is 75 to 115 ppm.
If it is less than 200 ppm, there will be no effect on health hazard.

Question: Results of water sample analysis are tabulated below. Find total hardness of water expressed as mg/ltr of CaCO₃.

(a) 44.8  (b) 89.5  (c) 111  (d) 358

Cations: Concentration (mg/ltr)  Equivalents

Na⁺  40  23
Mg²⁺  10  12.2
Ca²⁺  55  20
K⁺   2    39

\[ T\cdot H = \left[ \frac{\text{Ca}^{2+} \text{mg/ltr}}{20} + \frac{\text{Mg}^{2+} \text{mg/ltr}}{12.2} \right] \times 50 \]

\[ T\cdot H = \left[ \frac{55 + 10}{20} \right] \times 50 = 1.29 \text{ mg/ltr of CaCO}_3 \]
A Water contains following dissolved ions with their atomic ratio shown in figure.

<table>
<thead>
<tr>
<th>Cations</th>
<th>Conc. in mg/L</th>
<th>Atm. wt.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Na⁺</td>
<td>56</td>
<td>23</td>
</tr>
<tr>
<td>Ca²⁺</td>
<td>40</td>
<td>40</td>
</tr>
<tr>
<td>Mg²⁺</td>
<td>80</td>
<td>84</td>
</tr>
<tr>
<td>Al³⁺</td>
<td>3</td>
<td>27</td>
</tr>
<tr>
<td>HCO₃⁻</td>
<td>190</td>
<td>61</td>
</tr>
<tr>
<td>Cl⁻</td>
<td>165</td>
<td>35.5</td>
</tr>
</tbody>
</table>

pH of water is 7.

Qno. 1. The total hardness of sample in mg/L as CaCO₃ is
(a) 484 (b) 450 (c) 225 (d) 242

Qno. 2. M. C. H. of the sample is in mol/L as CaCO₃
(a) 225 (b) 156 (c) 86 (d) 0

Total hardness = \( \left( \frac{\text{Ca}^{2+} \text{ conc. in mg/L}}{80} + \frac{\text{Mg}^{2+} \text{ in mg/L}}{12} + \frac{\text{Al}^{3+} \text{ in mg/L}}{9} \right) \times 50 \)

= \( \left( \frac{40}{20} + \frac{80}{12} + \frac{20}{9} \right) \times 50 \)

= 242

Note: Not given

Qno. 3. Alkalinity as caused by CO₃²⁻, HCO₃⁻ & OH⁻

- OH⁻ 10⁻⁴ Negligible.
hence alkalinity is caused by bicarbonates.

Alkalinity as CaCO$_3$ = \[ \frac{[HCO_3^-]\text{in mg/l}}{\text{equiv. wt. of HCO}_3^-}\times \text{equiv. wt. of CaCO}_3}\]

\[ \frac{59}{61} \times 50 \]

\[ = 156 \text{ mg/l} \]

NCH = TH - Alk

\[ = 242 - 156 \]

NCH = 86

Grade 2007: Alk. and hardness of water sample are 250 mg/l and 350 mg/l as CaCO$_3$ respectively. The water has

(a) 350 mg/l as carbonate hardness & 0 N.C.H.
(b) 250 mg/l as C.H. & 0 N.C.H.
(c) 250 mg/l as C.H. & 350 mg/l as N.C.H.

Grade 2009: Common Data Question

Following chemical compounds were reported from a source.

<table>
<thead>
<tr>
<th>Compounds</th>
<th>Conc. in milli equiv/litr</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cl$^-$</td>
<td>15</td>
</tr>
<tr>
<td>Se$^{4-}$</td>
<td>15</td>
</tr>
<tr>
<td>CO$_3^{2-}$</td>
<td>05</td>
</tr>
<tr>
<td>HCO$_3^-$</td>
<td>30</td>
</tr>
<tr>
<td>Ca$^{2+}$</td>
<td>12 ( \times ) 1HCO$_3^-$</td>
</tr>
<tr>
<td>Mg$^{2+}$</td>
<td>18 ( \times ) 1HCO$_3^-$</td>
</tr>
</tbody>
</table>

pH: 8.5
**Qno.2**

Total ion activity of CaCO₃ is (a) 15,000 (b) 2000 (c) 3000 (d) 5,000

**Qno.3**

Alkalinity present in water is

- (a) 250
- (b) 1500
- (c) 1750
- (d) 5,000

Total alk. = \[ \text{CO}_3^{\text{2-}} + \text{Alk} + \text{HCO}_3^- + \text{OH}^- \]

- gm equiv. of \( \text{CO}_3^{\text{2-}} \) = gm equiv. of \( \text{HCO}_3^- \) = gm equiv. of \( \text{OH}^- \)

- gm equiv. of \( \text{CaCO}_3 \) = (meq/l + meq/l + meq/l)

\[ = \frac{\text{m eq}}{2000} \times 50 = 35 \times 50 = 1750 \text{ mg/L} \]

\( \text{OH}^- = 10^{-5.5} \text{ M/L} = 10^{-5.5} \text{ gm eq/L} \]

\( = 10^{-5.5} \times 10^3 \text{ mg eq/L} = 0.01 \text{ mg eq/L} \) (Negligible)

**Note:** Concentrations obtained for ground water sample have pH = 8.1 are as follows.

<table>
<thead>
<tr>
<th>Ion</th>
<th>Concentration (mg/L)</th>
<th>Actual</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ca²⁺</td>
<td>100</td>
<td>40</td>
</tr>
<tr>
<td>Mg²⁺</td>
<td>6</td>
<td>29</td>
</tr>
<tr>
<td>Na⁺</td>
<td>15</td>
<td>23</td>
</tr>
<tr>
<td>HCO₃⁻</td>
<td>250</td>
<td>61</td>
</tr>
</tbody>
</table>
Total hardness in mg/litre as CaCO₃ present in above water is
(a) 205 (b) 250 (c) 275 (d) 308

C.H. in mg/litre as CaCO₃ is
(a) 205 (b) 250 (c) 275 (d) 289.

Answer:

\[
\text{Alk} = \left( \frac{[\text{CO}_3^{2-}]}{\text{equiv.}} + \frac{[\text{HCO}_3^-]\text{mg/litr}}{\text{equiv. wt.}} \right) \times 50
\]
\[
= \frac{250}{61} \times 50 = 205 \text{ mg/litre}
\]

\[
\text{T-H.} = \left( \frac{\text{Ca}^{2+}}{20} + \frac{\text{Mg}^{2+}}{12} \right) \times 50
\]
\[
= 275
\]

\[
\text{N.C.H.} = \text{T-H.} - \text{Alk}
\]
\[
= 275 - 205
\]
\[
= 70
\]

\[
\text{C.H.} = \text{Alk} - 205
\]
Fluorides:

Fluorides are essential for healthy teeth. Permissible limit is (1-1.5) mg/ltr.

If fluorides is less than 1 mg/ltr, then it will cause more than 1.5 mg/ltr then it will cause fluorosis or discoloration of teeth.

When fluorides is more than 5 mg/ltr, then it may cause deformation of bones which is called bone fluorosis.

Phosphorous:
It is neither toxic nor harmful but it is indicator of future pollution of natural bodies. It facilitates rapid growth of aquatic plant like algae which may pollute water in natural streams. Phosphorous also interferes with chemical coagulation.

Iron and Manganese (Fe & Mn):
Excess iron causes hard-bad taste, discoloration of clothes and incrustation of pipe due to formation of limonothrix. It should not be more than 0.3 mg/ltr in drinking water.

Excess presence of Mn also causes discoloration of clothes and its permissible limit 0.05 mg/ltr.

Fe & Mn are generally found in water which is deficient in oxygen, such as underground water.

Copper (Cu):
Excess presence affects lungs and respiratory organ. If copper sulfate is more than 250 mg/ltr, then it may cause dermatitis effect.

Permissible limit 0.05 to 1.5 mg/ltr.
Reminiscible limit of sulphur - 200 mg/ltr.

Toxic metals (+) Arsenic, Berium, (+) Cadmium, Chromium, lead, Mercury and Silver are toxic and hazardous these enter in human body through food chain. Cyanide is poisonous and it should not be more than 0.2 mg/ltr.

Phenolic substances are also harmful and should not more than 0.001 mg/ltr.

Dissolved Gases :- (i) CH₄ - it is explosive in nature and causes green house effect

(ii) H₂S - it produces bad smell.

(iii) CO₂ - its presence indicate biological activities. It imparts bad taste and makes water more corrosive.

(iv) O₂ + O₃ is required for respiration of water species and it no case it should less 4 mg/l.


Presence of Organics (+) There are two types of organics

(i) Biodegradable (+) These are utilised at food by microorganisms ex. - starch, fats, protein, Alcohol, Acids, Amines, and amines. The utilisation of dissolved organic is either through oxidation process or through reduction process and oxidation may be aerobic or anaerobic.

Usually aerobic reaction gives stable product and the amount of O₂ consumed during microbial utilization called BOD. BOD for drinking water should be nil.
Non Biodegradable organics are lignic acid, cellulose, tannic acid, phenols, detergent compounds, and industrial waste.

COD is measure of biodegradable and non-biodegradable organics hence non biodegradable organics are represented by COD - BOD or COD - TOC.

**Type of aquatic plants**

(i) Sperrophyta → Waterweeds
(ii) Bryophyta → Mosses
(iii) Pteridophyta → Ferns & horsetails
(iv) Thallophyta → Algae → Photosynthesis plants

**Aquatic Animals**

(i) Vertebrata → Fish and Amphibian
(ii) Mollusca → Snails, Slugs, and limpets.
(iii) Arthropoda → Insects, spiders, slugs
(iv) Worms → Rotifers & thread worms.
(v) Metazoa → Polyzoa and hydra
(vi) Protzoa → E. Histolytica

**Biological Characteristics**

Those microorganisms which are harmful for human and animals are called pathogens these are capable of infecting and transmitting the diseases to human beings they are not native to aquatic system but
usually require an animal host for their growth and reproduction. They can be transported by water and air. The water born pathogen includes bacteria, viruses, protozoa and helminths.

16. Bacteria - Usually these may be rod shape but may be also spherical or spiral. These are single celled microorganisms which is simplest form of life they synthesize protoplasm from surrounding environment.

Rod shaped bacteria are called - bacilli
Spherical shaped are called - cocci
Spiral shaped - spirilla

Diseases caused by bacteria

Diseases | Bacteria
---|---
Typhoid Fever | Salmonella typhi
Cholera | Vibrio comma / Vibrio cholera
Bacilli Dysentery | Shiga bacillus
Lepto spiroisis | leptospirosis - Weil's Disease

10. Virus - Smallest biological structure which is known to contain all genetic information necessary for their own reproduction. These can be seen only by microscope.

Diseases | Virus
---|---
Polio | Polio
Jaundice / Hepatitis | Hepatitis
Diarriahoe | Cito - Pathogen
Protozoa: These are unicellular organisms which have complex functional activity as compared to bacteria and viruses. Protozoal infection is usually characterized by gastro intestinal disorders.

If drinking water is contaminated by sewage, then protozoal infection is common.

Disease: Protozoa

Entamoeba
Amoebic Dysentery
Giardia lamblia
Babesia
Isospora

(iv). Helminths: These are also called parasitic worms. They involve two or more animal host, one of which may be human or which will contain helminth.

Dracunculiasis → Dracunculus medesinesis

Measurement of Pathogens

Indirectly pathogens are determined by measurement of coliforms. Since coliform survive longer than survival of pathogen hence if in drinking water there is no presence of coliform, then there will be no presence of pathogens. Coliforms are harmless microorganisms which are found in intestine of all warm blooded animals. Common coliforms are B-coli & E-coli.

Methods to measure coliform

(i) Membrane filter technique
(ii) Test tube method (Most probable number method)
(iii) Coliform index
Membrane filter Technique is the most commonly used by environmental engineers to find coliform bacteria. In this test water sample is filtered through a membrane on the porous of which do not exceed 0.45 micrometers. Coliform bacteria are retained on the filter and then bacteria are placed on a selective media called McIlvaine Medium, which promotes growth of only coliform and not other species. Usually after 24 hours at suitable incubated temperature coliform grows into visible size which may be counted manually and results are represented as no. of coliform per 100 ml of water. For drinking water not more than 1 coliform colony should be present per 100 ml of water.

Test tube method or most probable number method

It is generally preferred by microbiologist. The water sample is mixed with lactose, and sample is incubated at 37°C for 48 hours. Sample is kept in 15 tubes (5 tubes of 10 ml, 5 tubes of 1 ml, 5 tubes of 0.1 ml).

After 48 hrs tubes are tested for presence of CO₂ and acids which indicates the presence of coliforms bacteria and results are referred with standardized statistical table given by McCord which can be used to find most probable no of BCol per 100 ml of water. Most probable no represents bacterial density which is most likely to be present.
<table>
<thead>
<tr>
<th>Sample size</th>
<th>No. of +ve test</th>
<th>No. of -ve test</th>
<th>Total no. of test tubes</th>
</tr>
</thead>
<tbody>
<tr>
<td>10 ml</td>
<td>4</td>
<td>1</td>
<td>5</td>
</tr>
<tr>
<td>1 ml</td>
<td>2</td>
<td></td>
<td>5</td>
</tr>
<tr>
<td>0.1 ml</td>
<td>1</td>
<td></td>
<td>5</td>
</tr>
</tbody>
</table>

+ve test combination = 4-2-1

Referring to mecardi table MPN is 26 per 100 ml vol. of water.
Water Purification System

Water Treatment units:

1. Purpose of water treatment
   - To remove colour, turbidity, dissolved solids, suspended solids.
2. To remove disease producing microorganisms.
3. To remove objectionable taste and odour.
4. Excess hardness and salinity.
5. Toxic metal and dissolved gases.

Method of treatment will depend upon the quality of raw water, presence of pollutant and desired quality of water.

Water Treatment units:

(a) Screening ++ Removal of floating objects are provided at intake points.
(b) Aeration — Removal of dissolved gases, iron and Mn up to some extent.
(c) Sedimentation or clarification — Removal of suspended impurities, silt, sands and other fine suspended particles and some of bacteria.
(d) Filtration — Most important operation of water purification. It removes suspended impurities, colloidal impurities, micro-organisms, etc.
(e) Disinfection — Removal of pathogenic microorganisms.
(f) Softening — Removal of hardness.
(g) Deferoxization — Removal of Fe and Mn chemically.
(h) Desalination — Removal of excess salts.
Fluidization & defluidization

Addition of flocculants in deficient condition & defluidization is removal of excess flocculants.

**Adsorption units** → Treatment with activated carbon. It is to remove suspended impurities.

**Types of water treatment**

**Case I**
When source of water provides clear water with low turbidity and free from taste and colour and no excess presence of metals and minerals. Under such condition plain chlorination is done i.e. Raw water → chlorination → supply

**Case II**
Treatment plant for treating hard water.

Raw water → Aeration → Softening → Flocculation & Sedimentation

Supply → Disinfection → Filtration →

**Case III**
Treatment of Turbid Surface water with organics

Raw water → Screening → Prechlorination → Aeration

Supply → Msc 4 → Disinfection →

Absorption → Filtration →

(Treatment with Activated carbon)
Screening

1. Coarse Screen
   2. Fine Screen

Coarse screens are in the form of bands of 25mm size spaced at 75 to 100 mm. C/C.

Screens are kept inclined at 45 to 60° with horizontal so that it helps in mashing reduces flow velocity by providing increased flow area.

Fine screens are in the form of wire mesh with opening less than 10 mm size. Since fine screens get clogged frequently hence head loss increases therefore those are not preferred instead for removal of fine floating particles sedimentation is preferred.

Aeration

Air is mixed in water to create self renewing interfaces b/w air and water. Aeration helps in removal of:

1. Bad taste and odour caused by dissolved gases and organic compounds.
2. pH increases DO content in water
3. Reduces corrosive property of water by removal of CO₂
4. Removes partly Fe & Mn.
5. Helps in increasing biological activities hence early completion of oxidation and some of bacteria may.
6. Removal of volatile liquids such as phenols, humic acids.

Types of aerators
1. **Gravity aerators + Trickling beds, Steps, Inclined apron with sponges.**
   
   Slat tray aerator (Most commonly used)
   
   Coral bed aerator

2. **Spray towers or nozzles** – Efficient in removing 
   
   (98) CO₂ and H₂S (97%)

3. **Air diffusers** + A compressed air is introduced in this method but this method is not efficient in removal of CO₂.

**Limitations of Aeration**

1. Not very efficient in removal of taste and odour caused by non-volatile substances like oil and greases.

2. Not efficient in removal of taste and odour caused by chemical such as due to waste.

3. Fe & Mn can be precipitated only when organic matter are not present.

4. Possibility of airborne contamination

**Sedimentation**

*Sedimentation is removal of suspended particles by gravitational settling.* Sedimentation tanks are designed to reduce velocity of flow so as to reduce turbulence. Sedimentation is based on Stokes law.

1. **Plain sedimentation (Type I Sedimentation)**
   
   It is called discrete settling. When no coagulants are added.
Coagulation sedimentation (Type - 2 settling).

Clarification

Chemicals are added to form flocs of suspended and colloidal substances.

Types of sedimentation tank

- Quiescent type (Still & Loros type)
- Continuous type

1. Quiescent type:

   Tank is filled with incoming water and is allowed to rest for detention period. Generally, 24 hrs. detention period is provided and 6 to 12 hrs are required for sludge removal (cleaning the tank) therefore total cyclic operation need 30 to 36 hrs. hence of minimum of 3 tanks are required to maintain constant supply.

   Tank are designed to treat maximum daily demand - 1.8 times average daily demand.

   These are obsolete now a days.

2. Continuous flow type:

   The aim of design is to achieve ideal condition of equal velocity at all points. Depending on type of flow there may be of following three types:

   (i) Horizontal flow tanks - These are rectangular
   (ii) Spiral or Radial flow - Circular
   (iii) Vertical flow - Hopper bottom tank

   Horizontal Flow rectangular sedimentation tank

Assumptions 1. Sifting of particle is same as in case
of quiescent type of equal depth

The flow is horizontal and steady and settling velocity is uniform.

The conc of suspended particles is same at all vertical c/s.

Particles are removed when they reach at the bottom of settling zone.

\[
\begin{align*}
\frac{V_f}{V_s} &= \frac{L}{V_s} \\
H &= \frac{V}{g} = \frac{(R \times L \times H)}{g}
\end{align*}
\]

Let \( t = \) Detention Time = \( \frac{L}{V_f} \)

\[
t = \frac{H}{V_s}
\]

\( V_s \) = Settling Velocity of a particle which enters at the top surface at the entry point and reaches to bottom surface of settling zone at cut point.

It means those particles which will settle with velocity \( V_f \)

will be 100\% removed just if settling velocity of particle is less than \( V_s \), will be fractionally removed because
Particles entering at lower depth may be removed. If particles have settling velocity smaller than \( V_s \) then \( \frac{1}{2} \) removal is given by

\[
\frac{1}{2} \text{ removal} = \frac{V_s}{V_s'} \times 100
\]

Let

\( h \) = Height at which particles enter (\( h < H \))

which are removed and have velocity \( V_s' \)

Then

\[
\frac{V_s'}{V_s} = \frac{h}{H}
\]

Note: \( V_s \) is also called overflow rate or surface overflow rate which is given by \( \frac{Q}{B \cdot L} = \frac{V_s}{V_s'} \)

Surface overflow rate is a design property of sedimentation tank.

\[
V_s = \frac{Q}{B \cdot H}
\]

The settling velocity of particle having diameter \( d' \) can be computed using Stoke's law

\[
V_s' = \frac{g \left( \rho_s - \rho_w \right) d^2}{18 \mu}
\]

Design Parameters

(c) Detention time

- 410.8 hrs for plain sedimentation
- 240.4 hrs for coagulation sedimentation
It is the theoretical time required by a particle to pass from entry to exit or time required for settling a particle from top to bottom with velocity $v_s$ or it is the time required to fill the tank.

$$t = \frac{H}{v_s}$$

$$t = \frac{B L H}{Q}$$

for rectangular

$$t = \frac{D^2 (0.011D + 0.785H)}{Q}$$

for circular

(ii) Overflow rate \( (v_s = \frac{Q}{B L}) \)

$$v_s = 500 \text{ to } 750 \text{ l/hr/m}^2 \text{ or }$$

$$12,000 \text{ to } 18,000 \text{ l/day/m}^2$$

Plain sedimentation

and

$$v_s = 1000 \text{ to } 1250 \text{ l/hr/m}^2 \text{ or }$$

$$24,000 \text{ to } 30,000 \text{ l/hr/m}^2$$

2. Horizontal flow velocity \( (v_f = \frac{Q}{B H}) \)

It is in the range of 0.15 to 0.9 m/minute and usually taken as 0.3 m/minute.

(iv) Slowing through period \( (T) \)

It is the average time required for a batch of water to pass through the settling tank. Since the central portion of water has short circulating effect hence

$T$ detention period (t)
Displacement efficieny: It is defined as ratio of flowing through period to detention period. Generally its value is 0.25 to 0.50 but it is desirable to keep displacement efficiency more than 0.3.

Tank dimensions:
- Width (B): 8 to 12 m. (+12)
- Length (L): 4 B
- Depth (H): 3 to 4.5 m. (+18 m, 6 m)

Sludge Zone: If sludge is mechanically removed continuously by scraper then no extra provision for Sludge zone is required but if sludge is removed manually, periodically then 0.8 to 1.2m Sludge storage is required.

Circular tank: These may have radial or spiral flow. Effluent comes at the centre and moves radially outward. At the circumference water is provided in a circular tank velocity of water continuously decrease from centre towards circumference. Overflow weir of V-notch is provided to reduce short circuiting and to ensure smooth drop of water.

Hoffer bottom tank with vertical flow: Water enters through centrally placed inlet pipe and is deflected downwards by the deflector box. Water travels vertically downward. The sludge settles at the bottom of hoffer from where it is removed with the help of sludge pipe connected to pump.
Determination of settling tank efficiency in horizontal flow tank (size, weight composition analysis). Efficiency of a sedimentation basin indicates the removal of suspended matter at a given overflow rate ($V_o$).

These particles which settle with velocity $V_s' > V_s$ will be fully removed and those particles which have velocity $V_s' \leq V_s$ will be fractionally removed. The efficiency can be determined by using cumulative frequency curve.

![Cumulative Fraction of Particles with Velocity < $V_s$](image)

**Setting Velocity ($V_s'$)**

The overall removal of particles in sedimentation tank is given as

$$R = (1 - x_s) + \sum \frac{V_s'}{V_s} \Delta x$$

$x_s$ = Fraction of particles which have velocity $< V_s$

It means $(1 - x_s)$ as fraction of those particles which have settling velocity $> V_s$.

$$\sum \frac{V_s'}{V_s} \Delta x$$ is fraction of particles removed having velocity $V_s'$ ($< V_s$)

$\Delta x$ is fraction of particles having velocity $V_s'$ ($< V_s$)
Settling efficiency may be reduced by

1. Eddy currents
2. Surface currents
3. Vertical convection currents
4. Density current (cold water moves below hot water)

Example 2. A water sample having particle size distribution of sediments is shown below:

<table>
<thead>
<tr>
<th>Size (mm)</th>
<th>% of particles</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1</td>
<td>10%</td>
</tr>
<tr>
<td>0.2</td>
<td>20%</td>
</tr>
<tr>
<td>0.3</td>
<td>15%</td>
</tr>
<tr>
<td>0.4</td>
<td>8%</td>
</tr>
<tr>
<td>0.5</td>
<td>30%</td>
</tr>
<tr>
<td>0.6</td>
<td>20%</td>
</tr>
</tbody>
</table>

% particles present

\[ V_s' (\text{mm/sec}) \]

Total solids in suspension from one liter of sample is 1000 gms and

overflow rate of the tank is 0.35 mm/second.

Then find:

1. Weight of suspended solids removed in gms
2. Settling tank efficiency in %

Solution:

Fraction of S.S. Removed

\[ R = (1 - x_s) + \frac{\sum V_s'}{V_s} \]

\[ x_s = \text{Fraction of particles with } V_s' < V_s \]

\[ = 0.1 + 0.20 + 0.15 \]

\[ = 0.45 \leq V_s \]

\[ (1 - x_s) = 0.55 \geq V_s \]

\[ \frac{\sum V_s'}{V_s} = \frac{0.20 \times 0.10 + 0.25 \times 0.20}{0.35} \]

\[ = \frac{0.35 \times 0.10}{0.35} = 0.0328 \]
\[ \% \text{ removal} = \left( 1 - X_b \right) + 1 \times \frac{V_0}{V_i} \times 100 \]

\[ = (0.55 + 0.328) \times 100 \]

\[ = 88.2\% \]

Weight of suspended solids removed = \[ 88.2\% \times 840 \]

\[ = 744.48 \text{ kg} \]

Ques: 2 ML/day of water passing through a sedimentation tank which is 6 m wide, 15 m long and having water depth of 1 m. Find (i) detention time (ii) average horizontal flow velocity (iii) Overflow rate and weight of dry suspended solids removed per day if concentration of suspended solids in influent water is 60 mg/l and tank efficiency is 70%.

Solution: detention time \[ t = \frac{Q}{B} \]

\[ t = \frac{2 \times 10^3 \times 3 \times 84 \times 10^{-6}}{2 \times 10^{-6} \times 10^3} \]

\[ = 1.1664 \text{ sec} \]

\[ = 3.24 \text{ hrs} \]

Horizontal flow velocity \[ v_h = \frac{Q}{B} \]

\[ = \frac{2 \times 10^3}{6} \]

\[ = 33.33 \text{ m/hr} \]

Surface velocity \[ v_s = \frac{Q}{B} \]

\[ = \frac{2 \times 10^6}{3 \times 60} \]

\[ = 6.67 \text{ cm/min} \]

Overfall rate \[ v_o = \frac{Q}{B} \]

\[ = \frac{2 \times 10^6}{3 \times 60 \times 15} \]

\[ = 1.11 \text{ m/hr/m}^2 \]
Total weight of S.S. entering tank per day

\[ 60 \text{ mg} \times 2 \times 10^{06} \times \frac{114}{10^6} = 120 \text{ kg/day} \]

% removal = 70%.

\[ \text{wt of suspended solids removed} = 120 \times 0.7 = 84 \text{ kg/day} \]

A circular sedimentation tank filled standard sludge removal equipment is to handle 3.5 MLD of raw water. If detention period of tank is 3hr 5hrs and depth of tank is 3m. Find the dia of the tank.

Solution

Volume of circular tank

\[ V = D^2 \left( 0.011D + 0.785x \right) \]

Quantity of water to be treated in detention time

\[ 3.5 \times 10^{06} \times 10^{-03} \times 3.5 \times 10^{06} \times 5 = 29.16 \text{ m}^3 \]

\[ 29.16 = D^2 (0.011D + 0.785x) \]

D = 16.84m

In a continuous flow settling tank having depth 2.5m and L = 60m. What velocity of water will be permitted for effective removal of 0.025mm particles having specific gravity 2.65 and 1m² of water 0.01cm²/sec at 25°C.

For effective removal

\[ V_s > V_c \]

\[ 0.5 = V_s \quad \text{and} \quad (s + 1) d^3 \]

\[ V_s = \frac{9.81L (2.65 - 1)(0.025 \times 10^{-03})^2}{18 \times 0.01 \times 10^{-09}} \]
\[ \nu_s' = 5.62 \times 10^{-04} \text{ m/sec} \]

\[ \frac{V_f}{V_s} = \frac{L}{H} \]

\[ V_f = \frac{60 \times 5.62 \times 10^{-04}}{0.5} = 0.0135 \text{ m/sec} \]

\[ V_f = 1.35 \text{ cm/sec} \]

**Type 2 Sedimentation**

It is also called classification or sedimentation with coagulation. Coagulants are added to neutralise the negative protective charge on colloidal particle and allows them to coagulate.

The coagulants should remove following impurities:

(i) Fragments of vegetation matter and plants.
(ii) Organic colouring matters.
(iii) Fine minerals present in colloidal form
(iv) Bacteria and other microorganisms.
(v) Dissolved organic compound added from sewage.

Factors affecting Coagulation:

1. Type of coagulant
2. Quantity or Dose of coagulant
3. Characteristics of water
4. Type and Quantity of suspended matter
5. Temperature
6. pH of water
7. Time and method of mixing
A thick gelatinous precipitate known as floe is formed. Aluminium and ferrous ions having the charge attract negative charge colloidal particles of clay and turbidity. Commonly used coagulants.

1. Alum
2. Ferrous sulphate (Copperas) \( \text{FeSO}_4 \)
3. Chelated copperas
4. Sodium aluminate \( \text{Na}_3 \text{AlF}_6 \)
5. Magnesium carbonate
6. Polyelectrolytes

Alum \( \rightarrow \) It is also called aluminium sulphate

\[
\text{Al}_2(\text{SO}_4)_3 \cdot 18\text{H}_2\text{O}
\]

Alum reacts with \( \text{Na}_2(\text{HCO}_3) \) to form \( \text{Al(OH)}_3 \), which is a precipitate

\[
\text{Al}_2(\text{SO}_4)_3 \cdot 18\text{H}_2\text{O} + 3\text{Na}_2(\text{HCO}_3) \rightarrow 2\text{Al(OH)}_3 \downarrow + 3\text{Na}_2\text{SO}_4 + 6\text{CO}_2\uparrow + 18\text{H}_2\text{O}
\]

Addition of Alum produces \( \text{CO}_2 \), which forms carbonic acid, which decreases pH and hence corrosiveness as increased. Addition of lime imparts permanent hardness because it converts bicarbonate into sulfate.

Alum is effective only in pH range of 6.5 to 8.5. Therefore, in order to increase the pH, Alkali may be required such as \( \text{Ca(OH)}_2 \) or SODA \( \text{Na}_2\text{CO}_3 \) to raise the pH.
The dose of alum 10 to 30 mg/ltr and on an avg.
17 mg/ltr is usually provided.
Alum is cheaper and flocs formed are
stable but disadvantage is that huge quantity
of sludge is produced.
If quick lime (CaO) is added to increase
the PH, then following reaction will take
place.

\[
\text{CaO} + \text{CO}_2 \rightarrow \text{CaCO}_3
\]
\[
\text{CaCO}_3 + \text{H}_2\text{O} + \text{CO}_2 \rightarrow \text{Ca(HCO}_3\text{)}_2
\]

\[
\text{Al}_2(\text{SO}_4)_3 \cdot 18\text{H}_2\text{O} + 3\text{Ca(CH}_3\text{CO}_3\text{)}_2 \rightarrow 2\text{Al(OH)}_3\downarrow + 3\text{CaSO}_4 + 6\text{CO}_2\uparrow + 18\text{H}_2\text{O}
\]

From above reaction it is clear that 1 mole
of alum requires 3 moles of CaO for alkaling.
If hydrated lime is added, then following
reaction will take place.

\[
\text{Al}_2(\text{SO}_4)_3 \cdot 18\text{H}_2\text{O} + 3\text{Ca(OH)}_2 \rightarrow
\]

\[
2\text{Al(OH)}_3\downarrow + 3\text{CaSO}_4 + 18\text{H}_2\text{O}
\]
It means 1 mole of alum produce 2 moles of

\[
\text{Al}_2(\text{OH})_3\text{ precipitate}
\]
If soda is added then following reaction will
take place.

\[
\text{Al}_2(\text{SO}_4)_3 \cdot 18\text{H}_2\text{O} + 3\text{Na}_2\text{CO}_3 \rightarrow \text{Al(OH)}_3\downarrow + 3\text{Na}_2\text{SO}_4
\]

\[
+ 3\text{CO}_2\uparrow + 18\text{H}_2\text{O}
\]
From above reaction it is clear that 1 mole of alum (636 gm) produces 2 moles of Al(OH)₃ (2×78 = 156 gm). Hence 1 gm of Alum will produce \( \frac{156}{636} \) gm of dry sludge, or 

\[ \frac{156}{666} \text{ gm of Al(OH)₃} = 0.234 \text{ gm of Al(OH)₃} \]

When flocs are formed then impurities are caught/trapped hence total sludge weight will include weight of impurities + weight of Al(OH)₃ precipitate.

Note: Alum is most widely used for dewatering and water treatment.

Copperas (Ferrous sulphates)⁷⁺ These are used with lime in order to increase the pH.

\[ \text{FeSO₄} \cdot 7H₂O + \text{CuSO₄} \cdot 5H₂O \]

When Copperas is added in water with lime, the basic precipitate of Fe(OH)₃ is formed.

\[ \text{FeSO₄} \cdot 7H₂O + \text{Ca(OH)}₂ \rightarrow \text{CaSO₄} + \text{Fe(OH)}₃ + 7H₂O \]

\[ 4\text{Fe(OH)}₂ + \text{O}_2 + 2\text{H}_2\text{O} \rightarrow 4\text{Fe(OH)}₃ \downarrow \]

It means 1 mole of ferrous sulphate produces 1 mole of Fe(OH)₃ precipitate.

Potom salts are active only in pH range of more than 8.5 Therefore addition of lime is essential. The use of Potom salts in water is
water and gives precipitate of calcium aluminate. Since it removes permanent hardness hence it is often used treating boiler feed water.

It works in pH range of 6 to 8.5.

\[
Na_2Al_2O_4 + Ca\left(CHCO_3\right)_2 \rightarrow CaAl_2O_4 + Na_2CO_3 + CO_2 + H_2O
\]

\[
Na_2Al_2O_4 + CaCl_2 \rightarrow CaAl_2O_4 + 2NaCl
\]

\[
Na_2Al_2O_4 + CaSO_4 \rightarrow CaAl_2O_4 + Na_2SO_4
\]

It is rather than Alum.

Magnesium Carbonate with lime water, magnesium carbonate is used with lime water.

\[
Mg\left(CO_3\right) + Ca\left(OH\right)_2 \rightarrow Mg\left(OH\right)_2 + Ca\left(CO_3\right)\downarrow
\]

Soluible in water, hence Sludge produced in the form of slurry which is difficult to remove because it is bekleen hence this is not commonly used.

Poly electrolytes: These are high molecular weights water soluble polymers. These work on wide pH range but are costly hence not preferred.
limited because it maintains high pH beyond the permissible limit of drinking water. Moreover, corrosion problem in pipe increases. Dose of ferrous sulphate is also 10 to 30 mg/l.

Chlorinated copperas is when chlorine is added with ferrous sulphate, then ferrous sulphate and ferrous chloride is formed and resulting combination of these 2 is called chlorinated copperas. The ferrous sulphate (Fe₂(SO₄)₃) is active in pH range 4 to 7 and at pH more than 9 whereas as ferrous chloride is active in the pH range 2.5 to 6.5 and at pH over 8.5. Hence resulting combination is effective in wide range of pH. 

\[
\text{Fe}(S\text{O}_4)\text{3} + 3\text{Ca(OH)}\text{2} \rightarrow \text{CaSO}_4 + 2\text{Fe(OH)}\text{3} + 2\text{FeCl}_3 + 3\text{Ca(OH)}\text{2} \rightarrow 3\text{CaCl}_2 + 2\text{Fe(OH)}\text{3} + 
\]

Note 1: Iron salts are more commonly used in treating sewage water.

2. Iron salts produce Fe(OH)_3 which causes turbidity in pipes hence produces bad taste and odour. Therefore not preferred for treating drinking water.

3. Iron salt produces heavy floc and also the time required for floc formation is less.

Sodium Aluminate (Na₂Al₂O₄)

It reacts with permanent hardness present in
\[ \text{MgCO}_3 + \text{Ca(OH)}_2 \rightarrow \text{Mg(OH)}_2 \downarrow + \text{CaCO}_3 \downarrow \]

Hence, it is not commonly used.

Poly electrolytes:

These are high molecular weight, water-soluble polymers. These work over a wider pH range, but are mostly used because they are not off-putting.

Methods of feeding coagulants:

1. Dry feeding: It is simple and requires less space for working, and it is cheaper, but its control is difficult and mixing is not very effective.

2. Wet feeding: The solution of coagulant of required strength is prepared and mixed with water to facilitate thorough mixing. In this method, there is better control of quantity.

Mixing devices:

- Centrifugal Pump: In most cases, centrifugal pumps are used to cause the water to the settling tanks. Coagulants may also be added in the suction line of the pump, which will get mixed with the water.
1. Compressed air agitation (O = \frac{1}{2})

2. Mixing basin with baffle walls

3. Mixing basin with mechanical devices - it is used in most of the modern water treatment plants. The mixing device is called flash mixer, which consists of paddles which are operated by power driven motors. The intensity of mixing is dependent upon temporal mean velocity gradient (\Gamma). The turbulence and mixing is based on the rate of power input.

Generally, Flash mixer has speed of 900 to 1400 rpm & detention time of 30 to 60 sec.

The temporal mean velocity gradient is defined as the rate of change of velocity per unit distance normal to the section.

The dimension of \( \Gamma \) = m/sec/m = sec^{-1}

\[
\Gamma = \sqrt{\frac{P}{\rho \cdot \Delta V}}
\]

where:
- \( P \) = Power input required in watt/m^3/hr of flow (1 = 3)
- \( \rho \) = Density in kg/m^3
- \( \Delta V \) = Voltage in watt/m^3/hr of
- \( V = \) V to which power is applied for mixing

\( \nu \) = Dynamic viscosity (N-s/m^2)

(\text{absolute viscosity})
Flocculation: It is the process of inducing formation of flocs. Flocculation is a slow mixing process in which colloids are brought together to form flocs. The rate of flocculation is dependent upon several factors such as concentration of turbidity, type of coagula, temporal mean velocity gradient, etc.

If temporal mean velocity gradient is combined with retention time or displacement time then non-dimensional parameter is formed called conjunction opportunity.

\[ C.O. = \frac{L}{V} x \frac{t_1}{t_2} \times \sec^{-1} x \sec = 1 \]

\[ C.O. = \frac{\sqrt{\frac{P}{N}} \times \frac{V}{A}}{\frac{P}{V}} = \frac{P}{V} = \frac{\sqrt{PV}}{\mu} \]

Flow

Power Induction flow = \( \sqrt{\frac{PV}{\mu}} \)

Note: If \( C.O. \) is large & \( t \) is short then small and dense flocs are formed which subsequently together to form large flocs which can be removed easily.
Note:
If $Q$ is small and $t$ is long, then lighter floccs are created.

Rule:
$Q$ is recommended $\geq 300 \text{ sec}^{-1}$ and $t$ is recommended $\leq 60 \text{ sec}$.  

A water treatment plant is required to process $98,800 \text{ m}^3/day$ of raw water. The rapid mixing tank impacts a velocity gradient of $900 \text{ sec}^{-1}$ to mix/blend $35 \text{ mg/L}$ of alum with the flow for an detention time of 2 min. If $f = 1000 \text{ kg/m}^3$, $\mu = 10^{-3} \text{ m}^2/\text{sec}$, then find power input in watts required for rapid mixing.

\begin{align*}
Q &= 900 \text{ sec}^{-1} \\
t &= 2 \text{ min} = 120 \text{ sec} \\
Q &= 98,800 \text{ m}^3/day \\
&= 2880 \frac{\text{m}^3}{\text{hr}} \\
&= 0.333 \\
V &= 28800 \text{ m}^3 \\
&= 0.333 \times 120 \\
&= 40 \text{ m}^3 \\
\mu &= 10^{-3} \times 1.5 \text{ m}^2 \\
f &= \frac{Q^2 v \mu}{L} \\
&= (900)^2 \times 40 \times 1 \times 10^{-3} \\
&= 324,000 \text{ watts}
\end{align*}
Filtration

In Filtration the turbidity, colloidal metals, non settleable metal, some dissolve metals, organic compd, micro organisms are removed.

Following are the 4 operations during filtration:

Mechanical straining
Sedimentation
Biological action
Electrolytic action

1. Mechanical straining: Impurities are trapped in sand surface. It is a kind of screening which occurs just at the surface of sand.

2. Sedimentation: The particles which are finer than the voids may also removed by filtration by such particles stick stick to already settled gelatinous mass.

![Diagram showing sand and gravel layers](image)
Biological Action: - A layer of sticky particulate organic matter, silica, clay, etc., An & Bacterial deposit is formed at the top surface of filter medium which is called slimy skin or SCHMUTZDECKE which may contain bacteria, protozoa and helminths also.

The organic matter become food & microbe consume hence biological activities takes place sometimes bacteria destroy each other to maintain bacterial balance.

Electrolytic Action: - The particulate matter may be removed by electrostatic exchange. The charge on filter medium neutralizes the charge on fibers. During the process of backwash the electrostatically neutral material is removed & change on filter medium is regenerated.

Classification/Types of Filters:

1. Slow sand filter
2. Rapid sand filter / rapid gravity filter
3. Pressure filters

1. Slow sand - filter: Generally these are designed rectangular basins. H/B ratio 1.5 to 3. The pretreatment needed for such water plain sedimentation without regeneration.
The filter medium consists of sand having size 0.2 to 0.4 mm & coefficient of uniformity b/w 5 to 5 & depth of slow sand filter 90-110mm.

Below the sand a gravel base is provided having particle size 3-6.5 mm of thickness of gravel base of 30-75 cm.

The top 10-15 cm of the sand is usually finer than the lower part which is retaining the impurities. The area of each filter unit ranges b/w 100 to 200 m² having its size of the order of 30m x 60m.

The filtration rate is small of the order of 100 to 200 L/hr/m² is the

The design period of filter is 10-15 yrs & design discharge is taken as much daily demand.

Freshly clean filter have low head loss of the order of 10-15 cm only but with the passage of time head loss increase hence to maintain worst discharge telescopic tube is adjacent.

The slow sand filters are very efficient in removal of bacteria & removal up to 98.9% bacterial content & suspended solid.
The water is prechlorinated then effluent may be up to 99.9%. These may remove turbidity up to 50 ppm.

The cleaning of filters is done once in 1 to 3 months in which 1.5 to 3 cm layer is scraped and washed manually. Amount of water is used for cleaning is 0.2 to 0.6 filtered water of a day.

Since these are very efficient, they are suitable for small (discharge being less) treatment plant where low colour, low turbidity and low bacterial turbidity is seen but hence these are large area hence uneconomical for large public supply. A large public supply rapid sand filter is used.

The number of filter bed seen depends on total filter area but usually one filter bed is provided at standby.

Guidelines to adopt number of filter beds:

| Area   | No of Filter Beds
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>≤ 20 m²</td>
<td>2 ((1 + 1))</td>
</tr>
<tr>
<td>20 - 950</td>
<td>3 ((2 + 1))</td>
</tr>
<tr>
<td>950 - 650</td>
<td>4 ((3 + 1))</td>
</tr>
<tr>
<td>650 - 1200</td>
<td>5 ((4 + 1))</td>
</tr>
<tr>
<td>≥ 1200</td>
<td>6 ((5 + 1))</td>
</tr>
</tbody>
</table>
Design a slow sand filter for following data:

- Population \( P \) = 50,000
- Per capita water demand = 150 lpcd
- Max daily demand = 1.8 avg daily demand
- Scale of filtration = 180 l/hr/m²

Design 6 beds of filter assuming 1 bed will be kept at standby.

Avg daily demand = \( 150 \times 50,000 \times 750,000 \)
= 75,000,000

Max daily demand = \( 1.8 \times 75,000,000 \)
= 135 \times 10^6 l/day

Design discharge in l/hr = \( \frac{135 \times 10^6}{180} \)
= 5625 \times 10^3 l/hr

Total filter area reqd = \( \frac{5625 \times 10^3}{180} \) m²
= 3125 m²

No of opn filtered bed = 5
Area of each opn filtered bed = \( \frac{3125}{5} \)
= 625 m²

\[ L: B = 2 \]
Area = \( 2B \times B = 2B^2 = 625 \)
\[ B = 17.67 \text{ m} \]
\[ L = 35 \text{ m} \]
Rapid sand filter:

The water to be treated must be coagulated & sedimented. The filter media is sand having size b/d 0.35 to 0.60 mm. Uniformity coefficient ranges b/d - 1.3 to 1.7. The depth of sand is about 75 cm. The sand is laid at decreasing increasing size from top to bottom.

Below the sand, gravel base is provided having size 3 m to 4.0 m. This depth is 60 - 90 cm.

The size of each filter unit is smaller by filtration rate is more, the area of a filter bed will be 10 to 80 m². A size of each filter unit may be 5 m to 8 m.

Rate of filtration - 3000 ltr/hr/m² - 6000 ltr/h (30 times of slow sand filter).

This are washed mechanically through backwashing either using water & compressed air. Backwashing is done at every 1 to 3 days. An amount of water used for back washing 10% of filtered water of a day's sand. Usually 20 min time is spent for back washing. Hence if backwashing is done the filter remains for 83 1/2 hrs.
These are less efficient in removal of bacteria (80-90%) but are highly efficient in removal of colour. They may remove 85-90 ppm turbidity.

**Design Steps**

1. Estimate total vol. of water to be filtered per day which is equal to max. publ. daily demand for popn.
2. Assume suitable filtration rate (if not given in range of 3000 – 6000 lt/hr/m²)
3. Find total area of filter, sq.m = Vol. of water to be filtered / filtration rate (lt/hr/m²)
4. Find the total no. of filters, sq.m for 10-80 m² having area of each filter 1/2

If no. of filters unit is 4-5 then provide a standby unit but no of filters unit may also be worked out by Manuel & Wallace formula:

\[
\text{no of filter units} = \frac{1.25A}{Q}
\]

\(Q \rightarrow \text{Ml/day}\)

Design each filters unit to be rectangular having L/B ratio b/w 1.5 to 3.
Design of under drainage system of rapid gravity filters (back washing system)

Gravel

75 cm

60 to 90 cm

Strainer

Main pipe/Manifold

Lateral pipes

Revetment/Gravity basin

\[ d = \left( \frac{D}{2} - \frac{d}{2} \right) \quad (D = \text{Dia of manifold}) \]

\[ s = 10 \text{ cm to } 30 \text{ cm} \quad (\text{known to us}) \]

No of laterals on one side = \( \frac{\text{length}}{\text{spacing}} \)

= \( \frac{1}{5} \)
Total no of lateral on both side = 21

Ltips:

The laterals are of two types:

Pipes having perforations of whole size (6-30 mm)
Pipes having strainers. Generally perforated type used high velocity for cleaning hence strainer type is preferred preferred.

2. The total c/s area of perforations/strainers should be abt 0.2% of total filter area and the c/s area of laterals should be 3-4 times c/s area of perforations. Area of lateral may be taken as time of the perforation area when ratio of perforation dia 13 mm & it should be taken 4 times when dia is 6m.

3. The c/s area of main pipe should be abt 3 times the c/s area of all laterals.

The ratio of length to dia of each lateral should not be more than 60

\[ \frac{l}{d} \leq 60 \]

The spacing of laterals should be kept 10-30 cm.

The max permissible velocity in manifolds to provide adequate backwashing but to...
prevent up flow of sand is 1.8 to 3.9 m

Note: The scale of application of wash water & settling velocity of smallest particles to retained in filter to avoid boiling of sand.

\[
\left( \frac{V_b}{V_t} \right)^{0.22} = n
\]

where \( n \) = porosity of sand/porous media/filter

\( V_t \) can be found by

\[
V_t^2 = \frac{4}{3} \left( \frac{g \cdot \rho_s - 1}{C_D} \right) \cdot d
\]

where \( C_D \) = drag coeff. 
\( \rho_s \) = sp. gr. of solids
\( g \) = gravity
\( d \) = dia. of settling particle or dia. of particle

Note: When backwashing is done to expand porous bed/sand hydrodynamically then head loss occurs

\[
h_1 = (\rho_s - 1)(1-n)^L
\]
\[ l = \text{length/thickness of embankment} \]

\[ \eta = \text{porosity} \]

In rapid gravity filters, the permissible head loss is 0.5 - 3.5 m, and permissible weir head is 0.8 - 1.2 m.

Design the dimensions of a set of rapid gravity filters (2 filters) for treating water used for 50,000 pop\( \text{th} \). The water demand is 180 lph, and max demand is 18 times of avg daily demand. Assume filtration rate, 5000 l/hr/m\(^2\). Assume 30 min time of avg daily demand. Assume filtration rate is 5000 l/hr/m\(^2\). Assume the ratio of filtrate to backwash water is 5000 l/hr. Assume the ratio of filtrate to backwash water is 5000 l/hr.

Total max daily demand = \(1.8 \times 50,000 \times 180 \text{ l/hr} \)

Total water eqd to be filtered per day = \(1.05 \times 1.8 \times 50,000 \int_{180} \)

account for backwash water

\[ = 17.01 \times 10^6 \text{ l/day} \]

Total water to be filtered per hr = \(17.01 \times 10^6 \text{ l/hr} \)

\[ = 723,829.7 \text{ l/hr} \]

Filtration area eqd = \( \frac{723,829.7}{5000} \text{ m}^2 \) = 144.76 m\(^2\)
There are two filter beds

\[ \text{area of each} = \frac{194.76}{2} = 72.38 \]

Assume \( L : B = 1.5 \)

\[ A = 1.5B^2 = 72.38 \]

\[ B = 6.94 \text{ m} \]

\[ L = 10.42 \text{ m} \]

Design R.G. filter for 4 MLD of water supply.

Assume filter rate 5000 g/hr/m², amount of back washing 1% of filtered water. 1% of lost in back washing in 30 minutes. Find (i) no. of filter beds & size of each filter bed (ii) Manifold lateral drainage system.

Assume \( s = 15 \text{ cm} \)

Total water need to be filtered per day =

\[ \frac{1.04 \times 4 \times 10^6}{9.16 \times 10^6} \]

\[ = 23.5 \]

\[ = \sqrt{0.177 \times 10^6} \text{ g/hr} \]

No of filter units = \( 1.22 \sqrt{A} \)

\[ = 1.22 \sqrt{416} \]

\[ = 24.88 \] (2 no. of filter)

Total area of filter need = \( \frac{0.177 \times 10^6}{5000} \)

\[ = 35.4 \text{ m}^2 \]
Area of each filler = \( \frac{35.4}{2} \)

\[ = 17.7 \text{ m}^2 \]

\( \cos \theta = 1.5 \)

\[ 1.5 B^2 = 17.7 \]

\[ B = 3.43 \text{ m} \]

\[ L = 5.15 \text{ m} \]

Using chainage system

Area of perforations = 0.2\% of filler area

\[ = \frac{0.2}{100} \times 17.7 \]

\[ = 0.0354 \text{ m}^2 \]

Let us adopt 13 mm size of perforation

Area of latrals = \( \pi \times \text{Area of perforat} \)

\[ = \pi \times 0.0354 \]

\[ = 0.0708 \text{ m}^2 \]

Area of main pipe (mainifold) = \( \pi \times 0.0708 \)

\[ = 0.1416 \text{ m}^2 \]

\( \text{dia of main pipe} \rightarrow \)

\[ \frac{\pi D^2}{4} = 0.1416 \]

\[ D = 0.484 \text{ m} = 48.4 \text{ cm} \]

Length of latral = \( \frac{B - D}{2} \)

\[ l = 3.43 - 0.48 \]
\[ l = 1.505 \text{ m} \]

\[ n = \frac{L}{s} = \frac{5.11500 \times 100 \text{ cm}}{15 \text{ cm}} \]

\[ = 34.33 \approx 35 \]

Total no of laterals = \( 2 \times 35 = 70 \)

Area of all laterals = 0.0708 m²

Area of each lateral = 0.001 m²

\[ a = 2 \times 0.035 = 0.070 \]

Area of manifold = \( 2 \times \text{Area of lateral} \)

\[ = 2 \times 0.070 \]

0.14 m²

\[ \frac{\pi d^2}{4} = 0.001 \]

\[ d = 0.035 \text{ m} \]

\[ = 3.5 \text{ cm} \]

\[ \frac{L}{d} = \frac{1.505}{3.5} = 0.433 = 43 \text{ cm} \]

\[ \approx 60 \text{ cm} \]

It means no of filter units = \( 70 / 6 = 12 \) \( \frac{1}{6} \)

Size of each filter = \( 3.43 \times 5.15 \)

No of lateral pipe = \( 35 + 35 = 70 \)

Dia of lateral = 3.5 cm

\[ s = 15 \text{ cm} \]

Size of perforation = 13 mm

\[ d = 1.505 \text{ m} \]

\[ d = 150 \text{ cm} \]

\[ L = 15 \text{ cm} \]
Rapid Sand Filters are to be provided in a water treatment plant to process the water for a town with a population of 275000. The water demand is calculated. The rate of filter is 15 m³/hr. Allow 5% of filtered water for back washing by a 30 min period for back washing alumina. The no. of filter bed required including 1 filter unit stand-by.

Assume area of each filter 10 m x 4 m. Also compute the upflow velocity & head loss to expand the porous bed to length of 0.6 m.

Assume \( n = 0.5 \) (porosity)
\[ \phi = 0.5 \]
\[ d = 0.6 \text{ mm} \]
\[ C_s = 2.5 \times 10^3 \]
\[ \dot{v} = 0.1012 \times 10^{-5} \text{ m}^3/\text{sec} \]

Total water required to be filtered per day = \( 1.8 \times 275000 \times 0.95 \times 24 = 8.9 \times 10^6 \text{ ltr/hr} \)
\[ = 9.42 \times 10^3 \text{ m}^3/\text{hr} \]

Total area of filter required = \( 4.42 \times 10^3 \times 15 = 6.63 \times 10^4 \text{ m}^2 \)

No. of operational filters = \( \frac{294.89}{10 \times 4} = 7.36 \approx 8 \)

So total no. of filters = 8 + 1 (stand-by) = 9
\[ v_t^2 = \frac{4}{3} (C_5 - 1) \frac{g \cdot d}{C_D} \]

\[ 88 = \frac{4}{3} (0.5 - 1) 9.81 \times 0.6 \times 10^{-3} \]

\[ v_t^2 = 0.47 \times 10^{-3} \]

\[ v_t = 0.022 \text{ m/s} \]

\[ \left( \frac{v_b}{v_t} \right)^{0.22} = 0.5 \]

\[ \left( \frac{v_b}{0.022} \right)^{0.22} = 0.5 \]

\[ v_b = 9.4 \times 10^{-4} \text{ m/s} \]

\[ h_t = (C_5 - 1)(1 - n) \frac{L}{1 - 0.5} 0.066 \]

\[ h_t = 0.4915 \text{ m} \]

Filter Troubles: The problems in filter raise value to either a poor design or poor operation. Following are the troubles:

1. Cracking and breaking of filter bed
2. Formation of drum balls
3. Air binding
4. Sand migration
5. Jetting or sand boils
6. Sand leakage
Cracking & idlogging of filter bed :- It is caused by accumulation of solids on the top surface of the filter media. There is compression & shrinkage of soft gelatinous grains which may cracks. The cracks are more prominent near wall junction. Due to cracks dirty metals matter may pass with the water hence efficiency may reduced. To minimize cracking proper back washing is sued.

Formation of mud balls :- When turbid floccs combined with sands & other binders near the top of filter then mud balls are formed. They are formed due to insufficient washing sand grains. It reduces the filter rate. It can be avoided by adequate back washing & surface washing.

Air binding :- When water percolates then it loses its head which further increases with time when high the pressure in water becomes & when it falls below vapour pressure air gets separated from the water when produces bubbles which are trapped on the base hence airlocks are cracked sand filter mate is reduced.

The problem of air binding is more
when temp. of water is high, growth of algae have seen. Water is super saturated with excessive headloss.

Air binding can be minimising by avoid headloss or excessive –ve head by providing extra over filter bed control of algae growth. By adding C avoiding heating of water avoiding supersaturation with air

Sand incrustation — when sticky gelatino material combines with sand due to crystallisation of CaCO3 then eff size of sand is increased. This prob can be minimized by carbonating the influent water or by adding sodium hexa meta phosphate to keep CaCO3 in dissolved state.

\[
CaCO_3 + H_2O + CO_2 \rightarrow Ca\left(CHCO_3\right)_2
\]

dissolve form

setting or sand boils — during backwash high head loss may start coming up. Cond of quick sand may be revealed. It may be minimised by surface or air wash. It may be eliminated by using
Sand leakage: The upward movement of sand and gravel from the sand bed may occur due to improper grading or when the gravel is displaced by backwashing. It may be minimized by well-grading or by using coarse grained layer below the sand and gravel.

Pressure Filter: It is a kind of rapid sand filter which is enclosed in a container and water passes under pressure. The pressure may vary from 3-7 kg/cm² which may be developed by pumping. These may be horizontal or vertical type.

Design Specification:

- Dia of tank: 2.5 m
- Length/height: 9 m
- Filter rate: 6000 - 15000 ft³/hr/ft²
  which is 2.5 times that of rapid sand filter
- Filter material: Sand followed by gravel

It is often used for swimming pool water and also for softening industry.
Double Filtration or Roughening Filter

To increase the filtration rate through a slow sand filter without compromising the quality we use a rapid sand filter before the slow sand filter. This process is called double filtration and a rapid sand filter is used called roughening filter or scrubber.
Disinfection

Disinfection means killing of pathogenic bacteria whereas sterilisation means killing of pathogenic & non-pathogenic bacteria both.

Disinfectants either destroy or inactivate microorganisms by following four mechanisms:

1. Damage of cell wall of microorganisms
2. Alteration of cell permeability
3. Changing the nature of cell protoplasm
4. Inactivating the enzyme system which is responsible for metabolism

Methods of Disinfection:

Mild methods:

1. Boiling: It is the best method which removes 100% bacteria and other microorganisms. This is adopted by individuals but not suitable for mass public supply.

2. Treatment with excess lime: As lime increases pH and when pH is more than 11 then it removes 99-100% bacteria. Generally, it is used for boiler feed water. But it also reduces hardness. The necessary dose of lime (CaO) is to do mg/l. The excess lime can be removed by carbonation of the water.
5. **Treatment with silver ion / electrocatalytic process:**

Metallic silver ion have strong killing action. They require in contact killing period of 15 min to 3 hrs. A DC supply voltage may be required of the order of 15 volt to produce silver ion. This method is costly.

2. **Treatment with iodine & Bromine:**

It may be used for army troops & by private plants.

5. **Treatment with UV light:**

This method is efficient method of sterilisation. UV rays are generated using mercury vapour lamp. The blue green region of spectrum is more destructive (~2000 Å to 3000 Å).

This method is also costly and is often adopted for treating surgical water & swimming pool water.

6. **Treatment with KMnO₄ or (Pink Powder):**

It is commonly adopted for open wells in rural area. It can kill bacteria and can oxidise bed taste producing organic matter. If pink colour disappears quickly then it indicates oxidation is sufficient of organic matter. If 1-2 mg/litre may be sufficient
Ozone: It is a powerful oxidising agent if it can be produced in a high voltage electrical field from oxygen. Since it is highly unstable hence it should be produced at sight. Ozonised water is tasteless & no residue is left & also no colour is produced.

\[
\begin{align*}
\text{O}_2 & \rightarrow O + O \\
O + O_2 & \rightarrow O_3
\end{align*}
\]

Note: Since residual O₃ is not left hence for prevention of future contamination a small quantity of Cl is added.

Note: This method is adopted in Europe and was 1st in Chandigarh in India since it is costly hence it has limited use in India.

Chlorine dioxide (ClO₂): It's properties are similar to O₃. It doesn't form chloroforms & solubilizes in water. It is effective in oxidising phenolic compounds also. It is also unstable hence it should be produced at the sight.

Major Method:

1. Chlorination: When Cl₂ is added in H₂O, it forms hypochlorous acid (HClO₂)
2. hypochlorite ion (OCl⁻). If pH is less than 5 then HCl does not react and it remains as elemental Cl which is not effective in killing bacteria.

3. If pH = 5 to 7 then mostly HOCl is found which is most effective in killing bacteria. (80 times more active than OCl⁻ ions).

4. If pH > 8 then mostly OCl⁻ ions are formed.

\[
\text{HOCl} \quad \text{pH} > 8 \quad \rightarrow \quad H^+ + OCl^- \quad \text{pH} = 5 \text{ to } 7
\]

- It means for most effective killing action of all pH of water should be blue 5 to 7.

Factors Affecting disinfection:

1. Form of chlorine
2. pH
3. Concentration of HCl
4. Type of m.o.
5. Temp.
6. Contact time

Forms of chlorine:- 
- may be applied to the water in any of the following forms:
  a) bleaching powder (hypochlorite or perhypochlorite)
  b) chloramines (HCl + NH₃)
e) free chlorine (either liquid or gaseous)
d) chlorine dioxide (ClO₂)
e) chlorine tablets

w) Bleaching Powder: It contains 30-33% of available chlorine whereas perchlorite contains 60-70% of available chlorine.

\[
\begin{align*}
Ca(OCl)_2 & \rightarrow Ca^{2+} + 2OC\frac{-}{^-} \\
\text{mol wt} & = 142 \text{ g/mole} \\
2OC\frac{-}{^-} + H^{+} & \rightarrow HOCl \\
\text{pH > 8} & \\
\end{align*}
\]

This process is also called hypochloritisation.

1 gm of bleaching powder produces = \frac{102}{142} = 0.718 gm of Hypochlorite ions

Since bleaching powder increases pH hence its use is restricted to swimming pools only.

Chloramines: \( NH_3 \) is added just before of addition of Cl₂ usually 1 part of \( NH_3 \) & 45 parts of \( Cl_2 \) are added.

\[
\begin{align*}
H_2O + Cl_2 & \rightarrow HOCl + HCl \\
\text{(monochloramine)} \\
NH_3 + HOCl & \rightarrow H_2O + NH_2Cl \\
\text{(dichloroamine)} \\
NH_2Cl + HOCl & \rightarrow H_2O + NHCl_2 \\
\end{align*}
\]
NHCl₂ + HOCl \[\xrightarrow{\text{pH < 4.4}}\] H₂O + NCl₃ (Trichloramine)

Type of chloramine depends on pH of water. If pH < 4.4 then trichloramine is formed which has no disinfection property. Most commonly used disinfection compound is NHCl₂.

NH₃ + HOCl together is called combin.

Free chlorine is either it may be applied in gaseous or liquid form. It is 2.48 times more heavier than air. One vol. of liquid chlorine gives 4.62 vol. of gaseous chlorine.

When chlorine gas is subjected to press of 1 kg, then it gets converted into liquid.

The release of chlorine depends upon

- Organic matters
- pH value
- Amount of CO₂ in present in water
- temp
- contact period

The free available chlorine after 10 min of contact time should be of the order of 0.2 mgl/l.

(a) Chlorine chloride (ClO₂): It is unstable as produced from sodium dichloride (Na₂ClO₂)
\[\text{NaClO}_2 + \text{Cl}_2 \rightarrow 2\text{NaCl} + 2\text{ClO}_2\]

It is 2.5 times more active than free \(\text{Cl}_2\).

It may also be used when phenolic compounds are present.

If \(\text{pH} = 8\) to 10, it may also removes
organic matters. Since it is unstable
hence it is difficult to prevent future
contamination.

4) Chlorine Tablets: - A single ref. tablet of
0.5 gm is sufficient for
100 ltr of water. It is used by individuals.

Chlorine demand: - It is the chlorine b/w
applied \text{Cl}_2 and that
at the end of specified contact period

\[
\text{Chlorine demand} = \text{applied chlorine} - \text{residual chlorine}
\]

Water is satisfactorily disinfected if residual
\(\text{Cl}_2\) is of the order of 0.2 mg/l.

Disinfecting reaction of \(\text{Cl}_2\):

\[
\text{HOC}^+ + \text{Cl}_2 \rightleftharpoons \text{HOCl} + \text{Cl}^-
\]

\[
K_f = \frac{[\text{H}^+][\text{Cl}^-]}{[\text{HOCl}]}, \quad K_b = \frac{[\text{HOCl}]}{[\text{H}^+][\text{Cl}^-]}
\]

concentration is in mol/l.
Zone I: The consumption of Cl₂ in zone I changes oxidation of Fe³⁺ to Mn⁴⁺ to NO₂⁻ (Nitrite).

Zone II: A small consumption is due to formation of chloro-organic compounds & chloramines. It also indicates an initial point of killing.

Zone III & IV: Actual oxidation of organic comp. starts and bad taste appears whereas at C oxidation is completed & bad taste disappears.

Zone IV: There is no consumption of Cl₂ in Zone IV. If Cl₂ is added in Zone IV then it is called superchlorination stage & entire Cl₂ remains as free residual Cl₂.
Notes: If Cl₂ is added in distilled water then
Cl₂ curve will start from origin & entire
added Cl₂ will remain as residual Cl₂.

Types of Chlorination:

1. Plain chlorination
2. Pre chlorination
3. Post chlorination
4. Double chlorination / Multiple chlorination
5. Break point chlorination
6. Super chlorination
7. Dechlorination

1. Plain chlorination: Cl₂ is applied to the
    raw water & in other
    treatment as provided. It kills & checks
    the growth of weeds, algae, m.o. &
    organic matters. It is suitable when turbidity
    less than 50-30 mg/l.

9. Pre chlorination: Cl₂ is applied before sedimentation. It helps in reducing
    amount of coagulants & also oxidises
    organic matters. Therefore, reduces bacterial
    load of filter. Hence, increases life of
    filter & controls algae & plant growth.
    It also removes taste & odour.

If cysts of D. histolytica are present then
right close with higher concentration is
Post chlorination: It's a step process to apply it after sediment filtration.

Double chlorination: To do post chlorination both are applied. This is used when raw water contains large amount of bacterial life or organic matters.

Break pt. chlorination: It removes taste & odour, Mn & Fe & leaves desired residual Cl for future disinfection.

1st kill, bacteria then oxidises organic compound.

Break pt. is that pt. at which all the added Cl disappears as free residual Cl. It represents completion stage of disinfection. Pure residual Cl remains in water in the form of hypochlorous acid or hypochlorite ion or combination of both depending upon the pH of water. At pH < 5 it may remain as elemental Cl.

Super chlorination: If Cl is added beyond break pt. then it is called super chlorination. It is used for removal of high count of organic matters, cysts of E. histolytica as well as for prevention of future contamination.
Dechlorination: It is the process of removal of excess of dechlorination.

It is done either by saturation or by using chemicals such as Sodium Thiosulphate (Na₂S₂O₃), Sodium metabisulphate (Na₂S₂O₅), Sodium sulphate (Na₂SO₃), Potassium permanganate (KMnO₄), Sodium bisulphite (NaHSO₃), activated carbon, sulphur dioxide.

Testing of residual methods:

Ortho Toludene Test.

Sodium Bromide Test.

Chlorophenol Test.

O₂ D Test.

Cl gas used for disinfection combines with water to form HOCl. The HOCl dissociates to form OCl⁻ ions.

\[ \text{HOCl} \rightarrow \text{H}^+ + \text{OCl}^- \]

The equilibrium is governed by pH. The sum of HOCl & OCl⁻ is called free chlorine residual.

HOCl in 90%, & remaining is OCl⁻ in free & total pH of water should be

1. 4.8
2. 6.6
3. 7.5
4. 9.5

(90°, → HOCl = 10⁻¹, – OCl⁻)

pH is 5 to 7 only one option = (b)
\[ k = \frac{[H^+][OCC^-]}{[HOCC]} \]
\[ 2.7 \times 10^{-8} = \frac{[H^+]}{9} \]

\[ H^+ = 2.43 \times 10^{-7} \]

\[ pH = -\log_{10} H^+ = -\log_{10} [2.43 \times 10^{-7}] = 7 - \log_{10} 2.4 = 6.6 \]

Al gas: 8 mg/l as Cl₂ was added to drinking water, if free residual was measured as 2 mg/l as Cl₂ & pH was measured as 7.5. What is the weight of OCC ions in the water.

Assuming Cl₂ gas is completely converted into HOCC & OCC⁻.
At wt of Cl⁻ = 35.5
\[ OCl^- + H^+ \rightarrow HOCC \]
\[ [HOCC] + [OCC^-] = 8.16 \times 10^{-5} \text{ mole/lt} \]
\[ \text{pH} = 7.5 \]
\[ H^+ = 10^{-7.5} \]
\[ K = \frac{[HOC]}{[H^+][OCl^-]} \]
\[ 10^{-7.5} = \frac{[HOC]}{10^{-7.5} [OCl^-]} \]
\[ \frac{[HOC]}{[OCl^-]} = 10^{7.5} \times 10^{-7.5} \]
\[ [HOC] = [OCl^-] \]

\[ [OCl^-] + [HOC] = 50 \text{mL} \]

\[ [OCl^-] = \frac{2 \times 316 \times 10^{-5}}{2} = 1.408 \times 10^{-5} \text{ mol/L} \]
Water Softening:

Removal of Temporary hardness:

Methods:

1. Boiling:
   - Usually CaCO₃ gets dissolved in water if Cl₂ is present.
   
   \[ \text{CaCO}_3 + \text{CO}_2 + \text{H}_2\text{O} \rightarrow \text{Ca(HCO}_3\text{)}_2 \]
   
   When water is boiled above the heat
   
   \[ \text{Ca(HCO}_3\text{)}_2 \rightarrow \text{CaCO}_3 \downarrow + \text{H}_2\text{O} + \text{CO}_2 \uparrow \]
   
   CaCO₃ can be sedimented out in settling tanks.

2. Addition of lime:
   
   \[ \text{MgCO}_3 + \text{Ca(OH)}_2 \rightarrow \text{CaCO}_3 \downarrow + \text{Mg(OH)}_2 \downarrow \]
   
   \[ \text{Mg(HCO}_3\text{)}_2 + \text{Ca(OH)}_2 \rightarrow \text{Ca(HCO}_3\text{)}_2 + \text{Mg(OH)}_2 \downarrow \]
   
   \[ \text{Ca(HCO}_3\text{)}_2 + \text{Ca(OH)}_2 \rightarrow 2\text{CaCO}_3 \downarrow + 2\text{H}_2\text{O} \]

3. 1 mole of MgCO₃ requires 1 mole of lime but 1 mole of Mg(HCO₃)₂ requires 2 moles of lime.
Removal of permanent hardness:

Methods:

1. Lime Soda Method
2. Base exchange method / Wilke method
3. Demineralisation process

(1) Lime Soda Method:

$$Ca(OH)_2 + Na_2CO_3$$

$$Ca(HCO_3)_2 + Ca(OH)_2$$ $\rightarrow$ 3$CaCO_3 \cdot H_2O$

$$MgCO_3 + Ca(OH)_2$$ $\rightarrow$ Mg(OH)$_2 \cdot CaCO_3$

$$Mg(HCO_3)_2 + Ca(OH)_2$$ $\rightarrow$ Ca(HCO$_3$)$_2$ + Mg(OH)$_2$

Hence lime removes temp. hardness

$$MgCO_3 + Ca(OH)_2$$ $\rightarrow$ Mg(OH)$_2$ + Ca$CO_3$

$$MgSO_4 + Ca(OH)_2$$ $\rightarrow$ Mg(OH)$_2$ + CaSO$_4$

$$Mg(NO_3)_2 + Ca(OH)_2$$ $\rightarrow$ Mg(OH)$_2$ + Ca(NO$_3$)$_2$

It means lime converts Mg hardness into ion hardness hence soda is required to remove ion hardness (permanent)
Sodium is univalent

\[ \text{CaCl}_2 + \text{Na}_2\text{CO}_3 \rightarrow \text{CaCO}_3 \downarrow + 2\text{NaCl} \]

\[ \text{CaSO}_4 + \text{Na}_2\text{CO}_3 \rightarrow \text{CaCO}_3 \downarrow + \text{Na}_2\text{SO}_4 \]

\[ \text{Ca(NO}_3)_2 + \text{Na}_2\text{CO}_3 \rightarrow \text{CaCO}_3 \downarrow + 2\text{NaNO}_3 \]

Note 1: Lime converts non-carbonate hardness of Mg into non-carbonate hardness of Ca.

Note 2: Lime helps in removal of CO₂.

\[ \text{Ca(OH)}_2 + \text{CO}_2 \rightarrow \text{CaCO}_3 \downarrow + \text{H}_2\text{O} \]

Which prevents corrosive action (CO₂ eliminated)

Note - The optimum range of pH for ppt of CaCO₃ is 9 to 9.5 and for ppt of Mg(OH)₂ is 11. Therefore, excess lime may be needed to be added. Excess lime may be removed by carbonation:

\[ \text{Ca(OH)}_2 + \text{CO}_2 \rightarrow \text{CaCO}_3\downarrow + \text{H}_2\text{O} \]

Note - If lime & soda is added with alum i.e., softening & coagulation is combined then demand of coagulant is reduced.

Note - Lime soda process increases alkalinity and reduces corrosive action & it also helps in removal of Fe & Mn.
Note: In this process there is large quantity of sludge produced if lime soda is added with alum then sludge with contain following compound. The dry sludge will have wet of following compound:

- wt of Al(OH)_3 (Product of alum)
- wt of Ca(OH)_2 (Product of lime-soda process)
- wt of Mg(OH)_2
- wt of suspended solid removed (Product of sedimentation)
- wt of activated carbon which may be added for removal of excess lime.

2. Zeolite Method: Zeolites are natural materials which remains are synthetic material. Zeolites are made of salts, clay, silicates of sodium & aluminium having general formula as Na_2O·Al_2O_3·kSiO_2·yH_2O

Sodium Zeolite = Z = Na_2O·Al_2O_3·kSiO_2·yH_2O

Sodium Zeolite when reacts with Ca or Mg salts play produce Ca_2 or Mg_2 and Sodium Salts

Na_2Z + Ca/Mg
\[
\begin{align*}
\text{Na}_2\text{Z} & \rightarrow \text{CO}_3^{2-} + \text{HCO}_3^- + \text{SO}_4^{2-} + \text{Cl}^- + \text{NO}_3^- \\
\text{Ca}^2 & \text{ or } \text{Mg}^2 + \text{Na}_2\text{Salts}
\end{align*}
\]
It is regenerated by treating Ca²⁺ or Mg²⁺ with 5 to 10% Brønsted solution (as NaCl).

\[ \text{Ca}^2\text{+ or Mg}^2\text{+ + }2\text{NaCl} \rightarrow \text{Na}_2\text{Z} + \text{CaCl}_2/\text{MgCl}_2 \]

(5 to 10%)

**Note:** Zeolite or resin have excellent property of exchanging their cations. These resemble with sand filter in which filter medium is zeolite rather than sand.

Zeolite are green hence often called green sand whereas synthetic resins are white which is called perlite which is manufactured using clay, talc, kaolin, soda.

**Advantages of Zeolite Method:**

1. No sludge produced.
2. Zero hardness can be achieved.
3. Running & Maintenance cost is less.
4. No problem of incrustation in pipes.

**Disadvantages:**

1. If Fe²⁺ & Mn²⁺ are present then Fe²⁺ from zeolite & Magnese zeolite are formed which cannot be regenerated into sodium zeolite hence process becomes costlier.
3) It leaves NaHCO₃ in H₂O which develops foams in H₂O especially in boilers.

3) Not suitable for turbid water.

(3) Demineralization - This process removes minerals from H₂O & it is suitable when hardness is to be precisely controlled of desired quantity; the demineralized water is also called ionised water.

Treatment with Activated Carbon

Activated carbon is porous having adsorptive property which attracts impurities such as gases, liquids, fine solids, and it is used to remove taste and odour.

Activated carbon is produced by burning (charing) the wood at 500°C in a closed vessel.

The activated carbon is available in the market either in granular or in the powder form or in the name of DARCO & NUCAR.

It can also remove phenolic substances.

It is added either before or after the coagulation but before filtration. The best way is to add partly before sedimentation & partly after sedimentation.
Advantage of activated carbon \textit{conagulation}:
If it is added before, then it helps in conagulation.
It reduces \textit{cd} demand.
It removes taste, odour, colour caused by iron, Mn, & phenols.
It's over dose not harmful.

\textit{Treatment with Copper Sulphate (CuSO}_4 \cdot 5H_2O\textit{)}
It helps removal of colour, taste & odour and it is used to kill, send which growth of algae. It often applied in deep reservoirs and lakes.

\textit{Desalination (Removal of Suspended Solids)}

\begin{enumerate}
\item Reverse Osmosis
\item Distilisation
\item Electrolysis
\item Freeze \textit{ing process}
\item Solar distillation
\item \textit{Fluidisation} - If fluorine presence in H_2O is less than \textit{limit} then fluorine may be added by adding following compound.
\end{enumerate}
1. Sodium fluorides
2. Sodium Silicofluoride
3. Hydroxyfluoride

Defluoridation

If fluorine presence more than 1.5 mg/l then excess fluorine may be removed by following mtd:

1. Adding activated alumina
2. Natgonda technique
3. Reverse Osmosis
4. Adding flocces
5. Adding bone charcoal
6. Activated charcoal
7. Ion exchange mtd
8. Tricalciumphosphate
9. Adding lime
The treated water is to be supplied to individual houses. The layout of the distribution network depends upon the street pattern of the city and may be of the following 4 types:

1. Dead end system
2. Grid iron system
3. Ring system
4. Radial system

1. Dead end system:

![Diagram of Dead End System]

- Main
- Submain
- Branch pipe

(Tree System)

It follows network of streets and it is suitable for the localised town in which the development is irregularity (half-hazard).
Advantages:
- Easy computation
- Less no. of cut of valves
- Shorter pipe length
- Cheap & simple

Disadvantages:
1. There is only one route of supply hence if it is damaged then entire life supply is affected.
2. There are many dead ends when water will be stagnated which may get polluted.
3. Difficult to get additional water during fire brake down.
4. Pressure keeps on decreasing towards dead end.

2. Grid Iron System:
It is suitable for well planned city having grid iron road pattern.

Advantages:

1. There are more than 2 paths of flow hence friction losses and pipe size get reduced.
2. In case of repair only small area is affected.
3. Dead ends are eliminated hence due to continuous flow hence there is no stagnation & poll.
4. During fire more water can be charged from alternative sources.

Disadvantages:

1. More length of pipe
2. Larger no. of E.P. values

5. Ring System: It may be circular or rectangular in which main pipes runs along the periphery.

Grid area is divided into circular or radial blocks.

It is suitable for well planned cities.
If speeds are radial then centre is C²
Stand in the dish reservoir tanks from
 embodiment through radially laid pipes.

Radial System:
This mtd. maintains high pressure and efficient water distribution.

Analysis of pn in distribution system:

Methods:

1. Hardy Cross Method
2. Equivalent Pipe Mtd
3. Circle mtd.
4. Graphical mtd.
5. Pitometric Mtd
6. Electric Network analyser mtd

1. Hardy Cross Method

It involves trial and error approach and is based on following 2 rules:

(a) In each separate pipe or element there will be a relation between head loss and discharge given as:

$$h_y = k \cdot Q^n$$

where $n = 1.85$

(b) At each junction, the algebraic sum of quantity of water entering and leaving the junction is zero:

$$\sum Q = 0$$
In any closed loop or path, the algebraic sum of head loss in the individual element is zero.

\[ \sum h = 0 \]

Type of water supply system:

1. Continuous system → water supplied 24 hrs.
2. Intermittent system → water is supplied during fixed hrs. (e.g., 6 am to 8 am, 4 pm to 8 pm)

The intermittent system has drawbacks as:

- Lack of water during fire breakdown
- During non-supply times there is stagnation hence chance of contamination
- Greater size of pipes needed
- Need of domestic storage

 Determination of storage capacity of distribution reservoir:

The total capacity consists of:

1. Balancing storage
2. Breakdown storage: 25% emergency storage
3. Fire storage
V = Bal cap + emergency cap
   = Bal cap + 0.25V

Determination of balancing storage

Methods

1. Mass curve Ltd
2. Analytical Ltd

Mass curve Ltd: Mass curve is a plot of accumulated inflow vs. time or accumulated outflow vs. time

Case 1: Continuous pumping

Slope of dead zone of consumption vs. time
Balancing storage = A + B

Case - 2: Pumping has one morning 6 AM to 10 AM

A → excess demand (max)
B → max. excess supply
Bal. cap = A + B

pumping is done in two shift but tank is not full in single shift
Analytical Mtd 8: In this mtd cumulative hourly demand & cumulated supply are tabulated for all 24 hrs. And hourly excess demand & hourly excess supply is computed. The sum of maxm excess demand & maxm excess supply will give bal cap.

**House Connection:**

1. Female → Goose neck → Service pipe → stopcock → stop cock stop cock
2. Goose neck: It is curved & made of lead having abt length 75 cm. It connects service pipe with the female. It is provided for
to absorb shocks & disturbances.

Service pipe: made of Galvanized iron having size less than 50 mm dia.

Stop cock: Provided before water meter at entry level of house used to stop flows during repair work & for other purposes.

Water meter: To record & measure the quantity of water.

Bibcock: To control the flow.
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Basic term in Sanitary Engg.

10. **Refuse**: All the waste that is generated in the form of Solid + Semi-Solid + Liquid form.

11. **Garbage**: Dry form of waste of refuse. Organic in nature i.e., vegetable matter, fruit peeling, biodegradable, putrefied.

12. **Rubbish**: Dry form of waste i.e., waste paper, pieces of card board, office waste, restaurants (Inorganic in nature). (Combustible).

13. **Sewage**: All the waste generated from the domestic and industrial process in the form of liquified state. (~ 99.9% liquid form. Dark in colour, foul smell, remaining in solids)

14. **Surfage**: It is the waste in the liquid form generated from kitchens, bathrooms, household utilities. Less foul in nature. Light greasy.

15. **Storm water**: It is the runoff that is coming from all the roads, building sites and other catchment areas.

16. **Sewer**: It is the pipe line or conduit which conveys “surfage”.

17. **Dry weather flow**: It is the normal flow that is occurring in any season.

18. **Sewage system**: Collection and conveyance of sewage & Disposal of it through sewers.

**Sewage system**
- Collects, Transports, disposes sewage
(i) Separate system
(ii) Combined
(iii) Partially Separate system

1. Separate system is designed in the areas of uneven rainfall distribution.
   - Separate system is adopted in hilly areas.
   - Above system is suitable where deeper excavation is not possible.
   - Sewage and storm water is carried out in separate pipe line.

2. Combined system: In combined system sewage and storm water is collected and carried out in the same pipe line.
   - This system is adopted where there is even rainfall distribution.
   - The above system is suitable in the plain areas where deeper excavation is possible.
   - In combined system, as the sewage and storm water is collected in one conduit, it is easy to provide a larger net of pipe line in the congested areas, i.e., metropolitan cities.

3. In this system the pipe lines are designed for the entire sewage and a part of the storm water

4. Generally, there are two sewage carriage systems
   (i) Conventional system
   (ii) Water carriage system
In conventional systems all the sewage is collected from all the sources and it is dumped in the outskirts of the city and disposed off carefully.

In water carriage systems water is used as a medium for the safe transportation of sewage. Pipelines are laid down below the ground level. Sewage is treated properly and safely it is disposed of.

**Type of Sewers**

- House sewer
- Lateral sewer (Submain)
- Main sewer, trunk sewer
- Outfall sewer
- Treatment plant
- Disposal area

- House Sewer - The waste in the form of sewage is collected from the house hold and immediately disposed off into lateral sewer. The pipe line which carries sewage from the house to lateral sewer is known as house sewer.
Lateral or Submain Sewer - The conduit or pipeline which carries all the sewage from its source and leading it to the main sewer.

The main sewer carries all the waste up to the treatment plant by collecting the sewage from all the lateral sewers.

The section of the pipe line which carries the sewage from the treatment plant to the disposal area is known as outfall sewer.

* Sewer pipe designed for 1/3 or 1/2 full.
* Sewage Treatment plant should be located in low lying area and outskirts of the city.

Design Discharge of Sewage

Population of the city = P
Per capita consumption of water = x l/pd
Total drinking water supply = px

Drinking water supply per day

Sewage generated from a city need not be equal to the water supply.

The sewage discharge is equal to 75-80% of drinking water supply.

\[ q = (75\% \text{ to } 80\% \times px) \]

Design sewage discharge can be calculated based upon flow condition.
Maximum daily sewage flow rate = 2q

q = Average sewage flow.
Maximum hourly flow is 1.5 times that of daily flow rate.

= 1.5(2q) = 3q

The sewer pipeline should be checked for min flow rate so that silting should not occur.

The minimum daily sewage flow rate is 2/3 of q.

Minimum hourly sewage flow rate = 1/2 min daily sewage flow rate.

= \frac{1}{2} \times \frac{2}{3} q = \frac{1}{3} q (One third of average flow rate).

Always sewage system is designed max hourly sewage flow rate (3q) and checked for min hourly sewage flow rate.

Design period:

1. Treatment plant = 30 years
2. Trunk sewer = 15-20 year.
3. Pumping system = 5-10 year.
average value of taken as

\[ K = \frac{1}{A_1 + A_2 + \ldots + A_n} \]

(131)
Storm water calculations:

Runoff from the building areas, road sides and other catchment area. (32)

Runoff = Rainfall - losses.

Storm discharge is calculated by Rational formula:

\[ Q_p = \frac{KP_c A}{36} \, \text{m}^3/\text{sec.} \]

where \( K \) = Runoff Coefficient:
0.9 to 1.0 (for the impervious areas pavement).

For Grass = 0.15

\( P_c = \) Critical rainfall intensity, \( \text{(cm/hr)} \).
\( A = \) Catchment area in "Hectare" \( (1 \text{Ha} = 10000 \text{m}^2) \).

Critical rainfall intensity

\[ Q_p = \text{Storm water discharge, in } \text{m}^3/\text{sec.} \]

is defined as the maximum runoff obtained in the catchment area during time of concentration \( (t_c) \).

Time of concentration is defined as the period during which maximum runoff is contributed to catchment area, which is equal to

\[ t_c = t_e + t_f \]
\[ t_i = \left( \frac{0.885}{1 - 0.885} \right)^{0.885} \]
where $t_e =$ time of entry (critical rainfall duration)

$tf =$ time of flow.

Time of entry is defined as the time taken by the farthest drop of runoff to enter into the inlet section of the sewer.

$te =$ Catchment area

$tf =$ Sewer

Time of flow is defined as the time taken by the farthest drop of runoff to enter into the inlet section of the sewer.

Note: The units of $te$, $tf$, etc. in minutes.

If time of concentration $t_c$ is 5 to 20 minutes

$$P_c = \frac{75}{t_c + 10} \text{ cm/hr}$$

If $t_c > 20$ to 100 minutes

$$P_c = \frac{100}{t_c + 20} \text{ cm/hr}$$

**Empirical Formula**

$> 1000 \text{ ha} \quad \text{Southern India}$

**Dickan's Formula** (North India)

$$Q_p = C_d A^{3/4}$$
Engli formula - Old Bombay state of India

\[ Q_p = 12.3 \sqrt{A} \]

Kawab Jung Bahadur formula - derived from Hyderabad deccan catchment

Hedge or Burge's formula -

\[ Q_p = 19.6 \frac{A}{L^{1.3}} \]
Catchment A = Area = in km²

Cd: Coefficient depends upon geological terrain and other factors like runoff coefficient.

Cd = 11.5

(6) Pyve's Formula (South India)

\[ Q_P = C_d A^{2/3} \]

\[ Cd = 6.8 \]

\[ Q_P = 136 \]

Note: Sewage system is designed for the sewage discharge and storm water put together in a combined system.

Total design discharge = Sewage discharge + Storm water

Discharge

Question: The city has a population of 10 lakh with water supply rate of 200 lpcd. Assuming 75% of water supply reaches the sewer, the DWF in m³/sec is

\[ P = \text{Population} = 1,00,000 \]

\[ x = 200 \text{ lpcd} \]

Total drinking water supply = \( 10^5 \times 200 \text{ l/day} \)

\[ Q = 0.75 \times 200 \times 10^5 \]

\[ Q = 75 \times 200 \times 10^5 \times 24 \times 3600 \]

\[ 12 \times 72 \]

\[ 72 \]

\[ Q = 125,000 \times 10^{-3} \]

\[ Q = 0.125 \text{ m}^3/\text{sec} \]

\[ Q = 0.17 \text{ m}^3/\text{sec} \]

A residential area has 50 hectares of land. Assume critical rainfall intensity as 1 cm/hr,
Runoff coefficient as 0.8. Calculate storm discharge in cumecs

\[ Q_p = \frac{K \cdot P \cdot C}{36} \]

\[ Q_p = \frac{0.8 \times 1 \times 50}{36} = 1.11 \text{ m}^3/\text{sec} \]

**Question**

A city with a population of 5 lakhs has an area of 100 ha. Rate of water supply is 200 lpcd and average runoff coefficient for the entire area is 0.5. Time of concentration is 50 min. Assume 80% of water supply wets the sewer. DWF and storm water flows for the sewer line for the above data and also calculate the discharge for which a combined sewer system is to be designed.

\[ P = \text{Population} = 500,000 \]

\[ P_{pc} = \text{per capita consumption} = 200 \text{ lpcd} \]

\[ \text{Total drinking water supply} = P \cdot P_{pc} = 5,000,000 \times 200 \text{ lpcd/day} \]

\[ 80\% \text{ of water of sewage} = \frac{80}{100} \times 5 \times 10^5 \text{ lpcd/day} \]

\[ \text{DWF} = q = 0.926 \text{ m}^3/\text{sec} \]

\[ \text{Average daily flow} \]

\[ Q_p = \frac{K \cdot P \cdot C}{36} \]

**Storm water** \( Q_p \)
Design the section of a combined circular sewer for full flow from the data given below.

Area to be served = 150 ha.

Population of locality = 50,000

Max. permissible velocity = 3.2 m/sec

Time of entry = 5 min.

T of flow = 20 min.

Rate of rain supply = 200 lpcd.

Runoff coefficient = 0.45

Assuming 75% of water supply converts into sewage.

Ans

Total drinking water supply = P.P.E = 50,000 x 27

75% of drinking water as sewage = \( \frac{75}{100} \times 50,000 \times \frac{x^2}{1000 \times 0.4} \)

\[ q = 0.117 \text{ m}^3/\text{sec} \]
Storm water \( Q_p = \frac{kpA}{36} \)

\[
= 0.45 \times P_e \times 150 \\
= \frac{0.45 \times 2.22 \times 150}{36} \\
= 4.16 \text{ m}^3/\text{sec}
\]

Total design discharge \( = 3q + Q_p \)

\[
= 3 \times 0.117 + 4.16 \\
= 4.51 \text{ m}^3/\text{sec}
\]

Assuming "d" = dia of circular sec.

\[
Q = A v \\
4.51 = \frac{\pi}{4} \times d^2 \times 3.2 \\
\Rightarrow d = 1.34 \text{ m}
\]

IES

Due Length of lines in a drainage area and inlet tank are marked in the figure given below. Find the time of concentration for catchment and discharge by taking \( k = 0.3 \) and velocity of flow in the sewer as \( 0.75 \text{ m/sec} \). Assume frequency of design rainfall as 5 years. From the graph:

Given below:

Most remote point

\[
\begin{array}{c|c|c}
\text{Area} & \text{Area} & t_e \\
0.016 \text{ km}^2 & 0.032 \text{ km}^2 & 5 \text{ min} \\
\text{te} & 5 \text{ min} & 8 \text{ min} \\
\text{i} & 18 \text{ m} & A = 4 \text{ km} \\
\end{array}
\]
Time of concentration of area $A_1 = t_c + t_f$,

$$t_{c_1} = 5 + \left( \frac{120 + 180}{0.75} \right)$$

$$t_{c_1} = 1167 \text{ min}$$

$$t_{c_2} = 5 + \frac{180}{0.75} = 9 \text{ min}$$

$$t_{c_3} = 8 \text{ min}$$

$$V_0 = 100 \text{ mm/hr} \quad \text{(from above graph).}$$

$$Q_p = \frac{K P_c A}{36}$$

$$= \frac{0.3 \times 100 \times 7.2}{36 \times 10}$$

$$Q_p = 0.6 \text{ m}^3/\text{sec}.$$

$$A_1 = 0.016 \times (1000)^{0.5} \frac{10,000}{10,000}$$

$$A = 1.6 \text{ ha}$$

$$\text{Total area } A = A_1 + A + A$$

$$= 0.01600.032$$

$$+ 0.024$$

$$= 0.072 \text{ ha}$$

$$A = 7.2 \text{ ha}$$
Sewage Characteristic

Sewage contains 99.9% water remaining solids.

Aerobic process
Grasses + (In the process of oxygen)
\( \text{CO}_2, \text{NH}_3 \) (Nitrification)

Anaerobic process
Grasses + (In the absence of oxygen)
\( \text{CH}_4 \)
\( \text{CO}_2, \text{methan} \)
\( \text{H}_2\text{S}, \text{NH}_3 \)

1. Physical parameters
2. Chemical parameters
3. Biological parameters

4. Physical Parameters

11. Turbidity - Because of presence of sand, silt, grit type of particles and solids.

21. Colour - Fresh sewage greyish/light - foul smell less

    Septic - black in colour/dark - foul smell

3. Temperature - If temperature increases in sewage, DO gets reduced, hence effect following.
Biological activity  

- Dissolved gases

Total Solids
- Suspended solids, $\theta > 1 \mu$
- Colloidal solids, $\theta = 1 \times 10^{-3} \mu$
- Dissolved solids, $\theta < 1 \times 10^{-6} \mu$

- Organic 45%, Inorganic 55%

- Undergo decomposition process

Total solids can be checked by evaporation test:
- Sewage taken in a container is subjected to process of evaporation, residue left after evaporation is amount of total solids ($10^3 \times 10^5$) mg/l

Suspended solids can be found by subjecting the sewage sample to the process of filtration of 1 μ size filter. The particles which are residing on filter paper = quantity of suspended solids.

Total solids = Suspended solids + Dissolved solids

Dissolved solids can be found difference b/w total solids and suspended solids

Suspended solid can be classified as volatile solid and fixed solid.
Volatile solid can be found by keeping suspended sample of sewage sample in a muffle furnace at 550°C for 20 minutes. Organic matter which was burnt in muffle furnace = Volatile solids and remaining solids are known as fixed solids.

Fixed Solids = Suspended Solids - Volatile solids

Settleable solids can be found in the laboratory by the Imhoff Method cone test. By keeping sewage sample in the Imhoff cone for a period of 2 hrs, from the graduated markings present on the cone we can get directly settleable solid.

The pH of the sewage sample for fresh sewage appears to be light grey colour and possess pH of 7.2 - 7.5 (Alkaline in nature).

As the time passes sewage becomes septic in nature and it becomes acidic in nature. To prevent the acidity of the sewage lime should be added.
Dissolved Oxygen \( O_2 \) - For the freshness of the sewage, DO content should be more. The permissible value of DO after removing toxic elements from the sewage sample is above 4.2 mg/l or ppm. As temperature increases, DO quantity reduces. DO can be found by the Winkler's method.

Do levels variation causes the biological activity and dissolved gases.

**Biological Oxygen Demand (Biochemical)**

(BOD) It is the amount of \( O_2 \) or demand of oxygen that is required for biological decomposition of biological degradable matter at a specified temperature for a specific duration of each time. BOD is an indication for the presence of organic matter.

Eutrophication As we dump the sewage containing high BOD into the fresh water body, as time passes the accumulation of organic matter will be and thus will be high BOD after some time increase in level of organic matter the first water becomes absolute; this phenomena is known as eutrophication.
BOD is reported at 20°C for a period of 5 days (BOD$_{5,20°C}$).

BOD takes care of only biologically active organic matter.

Organic Matter $\rightarrow$ Biologically Active

Organic Matter $\rightarrow$ Biologically Inactive

$\therefore$ COD $>$ BOD

COD (Chemical Oxygen Demand)
The amount of Oxygen or demand of Oxygen i.e., required for the chemical decomposition of organic matter is called COD.

COD can be tested by chemical reagent like Potassium Dichromate ($K_2Cr_2O_7$) and Potassium permanganate ($KMnO_4$). COD takes care of both biologically active organic matter. For the COD, 2 to 3 hour duration of time is enough but BOD takes several days.

Theoretically Oxygen Demand (TOD)
It is the sum of BOD and COD. The total consumption of organic matter calculated from the chemical in theoretically.

$$C_6H_{12}O_6 + 6O_2 \rightarrow 6CO_2 + 6H_2O$$

TOD $>$ COD $>$ BOD

BOD Calculation:

$$\text{time} \uparrow \frac{\text{Em}}{\text{Organic}} \downarrow \rightarrow \text{Organic Matter} \downarrow \rightarrow \text{BOD} \downarrow$$

Consumed Oxygen: 1.4L
The rate of decomposition of organic matter w.r.t time is directly proportional to amount of organic matter at any time:

\[
\frac{dl}{dt} \propto -lt
\]

\[
\frac{dl}{dt} = -Klt
\]

\[
\frac{dl}{dt} + Kl_t = 0 \quad \text{--- (i)}
\]

Solution of above eqn:\n
\[
\log_e (l_t) = -Kt + c \quad \text{--- (ii)}
\]

at \( t = 0 \) \( \Rightarrow l_t = L \)

\[
\log_e L = c
\]

Putting the value of \( c \) in eqn (ii) we get:

\[
\log_e (l_t) = -Kt + \log_e L
\]

\[
\log_e \left( \frac{l_t}{L} \right) = -Kt
\]

\[
l_t = L e^{-Kt}
\]

BOD at any time \( t = 1 - L_t = BOD_t\)

\[
\Rightarrow 1 - L e^{-Kt}
\]

\[
\Rightarrow L (1 - e^{-Kt})
\]

The rate of decomposition of organic matter at any time is directly proportional to amount of organic matter present at that time (-ve sign indicates as time passes the decrease in the organic matter).

\[
BOD_{t0} = L (1 - e^{K_{t0}t}) = L (1 - 10^{-K_{t0+t}})
\]

Where \( K_t \) = Rate constant at a particular temp \( T^\circ C \)

\[
K_{25^\circ C} = 0.23 \ \text{per day}
\]

\( K_o \) = Deoxygenation rate constant
\[ K_D = \frac{K_t}{2.3} \text{ per day} \]

\[ K_{26^\circ C} = \frac{0.23}{2.3} = 0.1 \text{ per day} \]

\[ K_{T} = K_{26^\circ C} \left( 1.047 \right)^{T-26^\circ C} \]

*Temp at C rate constant is required*

Similarly \[ K_{D} = K_{26^\circ C} \left( 1.047 \right)^{T-26^\circ C} \]

\[ \text{base } 10 = K_D \]

\[ \text{base } e = K \]

\[ I \rightarrow \text{represents initial or ultimate BOD (BOD_u)} \]

\[ \text{BOD}_{26^\circ C} = 68.1 \% \text{ of organic matter decompose} \]

\[ \text{at } 20 \text{ days} = 95-99 \% \text{ decomposition occurs} \]

\[ \text{BOD}_{35 \text{ days}} = 3 \text{ BOD_u} \]

Procedure to find out BOD in the laboratory:

1. Undiluted Sample
2. Blank Bottle
3. Diluted Sample

\[ \text{DO}_{0} \text{ or } \text{DO}_{11} \]

\[ \text{on incubation} \]

\[ \text{In incubator for 5 days} \]

\[ \text{DO}_{0} \text{ or } \text{DO}_{11} \]

\[ \text{DO}_{11} \]
1. Let a raw sewage sample is tested for the BOD. The volume of raw sewage is $V_s$ & its DO is $DO_{sw}$

2. Let the water used for the dilution process having a volume of $V_w$ & its DO is $DO_{w}$ or $DO_{uw}$

3. After dilution the dissolved oxygen of diluted sample is $DO_{dsi}$ (Before Incubation)

4. Keep the diluted sample in the incubator for a period of 5 days & its DO is calculated after incubation i.e $DO_{dsf}$

$$BOD_{s26^\circ C} = \frac{DO_{dsi} - DO_{dsf}}{DF}$$

Where

- Dilution factor = 100 \[ \frac{V_s + V_w}{V_s} \] (If x% of soil is added then)

$$DF = \frac{V_s + V_w}{V_s}$$

Where

- $V_s$ = Vol. of sewage
- $V_w$ = Vol. of diluted water

**Note:** If the water other than distilled water is used then the above formula reduces to

$$BOD = (DO_{b} - DO_{f}) DF = (DO_{b} - DO_{s})$$

Where

- $DO_{b}$ = DO of blank bottle after water remained in the bottle after dilution (after incubation)
- $DO_{f}$ = DO of diluted sample after incubation
- $DO_{s}$ = DO of undiluted sample after before incubation

If dissolved oxygen of diluted sample i.e initial DO is not given then it will be worked out from the known volume of $V_s$ & $V_w$ & from $DO_{s}$ & $DO_{w}$

$$DO_{t} = \frac{DO_{s} V_{s} + DO_{w} V_{w}}{V_s + V_w}$$
A waste water sample having 5 days 30°C BOD is 110 mg/litre.

Calculate the 5 day BOD at 20°C. Assuming \( k_{20°C} = 0.3 \text{ per day} \)

\[
BOD_{5, 30°C} = 110 \text{ mg/litre}
\]

\[
1(1-e^{-kt}) = 110
\]

\[
k_{20°C} = k_{30°C} \left( \frac{104.4}{30-20} \right)
\]

\[
= 0.15 \text{ per day}
\]

\[
L \left( 1-e^{-0.15x} \right) = 131.29 \text{ mg/l}
\]

\[
BOD_{20°C} = L \left( 1-e^{-0.15x} \right)
\]

\[
= 131.29 \left( 1-e^{-0.15x} \right)
\]

\[
= 89.71 \text{ mg/litre}
\]

\[
S\text{soln}\]

For determination of BOD of a waste water sample 5, 10 & 15 ml of waste water were diluted to 300 ml and incubated at 20°C in the BOD bottle for 5 days. The results were as follows:

Based on the data the avg. BOD for the 5 days of the waste water

<table>
<thead>
<tr>
<th>S.No</th>
<th>Vol of waste water</th>
<th>Initial DO</th>
<th>5 days factor</th>
<th>5 days BOD</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>5 ml</td>
<td>9.2</td>
<td>6.9</td>
<td>60</td>
</tr>
<tr>
<td>2</td>
<td>10 ml</td>
<td>9.1</td>
<td>4.4</td>
<td>30</td>
</tr>
<tr>
<td>3</td>
<td>50 ml</td>
<td>8.9</td>
<td>0</td>
<td>66</td>
</tr>
</tbody>
</table>

\[
V_5 + V_{10} = 300 \text{ ml}
\]

Avg BOD at 5 days = 109.8 mg/l

Determine the ultimate BOD of a waste water sample was subjected to the BOD determination as follows. 6 ml of waste water containing no dissolved oxygen was mixed with 2.294 ml water containing 8.6 mg/l of DO. After incubation at
20°C for 5 days. The DO of Nitr was 5.4 mg/litre. The BOD rate constant at the base e \( K_{20} = 0.25 \) per day.

\[
V_s = 6\text{ ml} \quad DO = 0
\]
\[
V_w = 294\text{ ml} \quad DO = 8.6 \text{ mg/l}
\]

**Dilution factor** \( \frac{6+294}{50} \)

After incubation, \( DO = 5.4 \text{ mg/l} \)

\[
DO_i = \frac{6 \times 0 + 294 \times 8.6}{6 + 294} = 8.428 \text{ mg/l}
\]

\[
BOD = (8.428 - 5.4) \times 50 = 151.4 \text{ mg/l}
\]

\[
151.4 = 1 \left(1 - e^{-K_{20}t}\right)
\]

Initial BOD \( = 212.19e \text{ mg/l} \)

**Question:** The 5 BOD test using 5% of dilution of the sample (15 ml of sample & 285 ml of dilution water) DO values for the sample & dilution water blank bottle after 5 days incubation at 20°C were given as 3.8 mg/l & 8.8 mg/l. DO originally present in the undiluted sample was 8.80 mg/l. The 5 days 20°C BOD of the sample is.

**Dilution factor** \( = \frac{100}{5} = 20 \)

\[
BOD = (DO_b - DO_i) \frac{100}{5} - (DO_b - DO_i)
\]

\[
(8.8 - 3.8) \times 20 = (8.8 - 8.8)
\]

\[
= 92 \text{ mg/litre}
\]

**Question:** The 5 day BOD of a waste water sample is obtained as 190 mg/l. Find \( k \). The ultimate BOD of the sample in mg/l will be.

\[
DO_i = \text{BOD} = 1 \left(1 - e^{-K_{20}t}\right)
\]

\[
190 = 1 \left(1 - e^{-0.01\cdot5}\right)
\]
1. $L = 271.89 \text{ mg/l}$

Qns: If the BOD of a waste water sample after 3 days is 75 mg/l and the first order constant $k$ (base e) is 0.345/day. The amount of BOD remaining in the given sample, after 10 days is

$$BOD_{10 \text{ days}} = L \left(1 - e^{-k \cdot t}\right)$$

75 = $L \times 0.644$

$L = 116.32 \text{ mg/l}$

$BOD_{10 \text{ days}} = L \left(1 - e^{-k \cdot t}\right)$

= 112.63 mg/l

Remaining BOD = 3.7 mg/litre

Qns: A portion of waste water sample was subjected to standard BOD test (5 days $20^\circ\text{C}$). The BOD value after 5 days was found 180 mg/litre. The first order constant (base e) at $20^\circ\text{C}$ is 0.18 per day. The first order constant at other temp may be estimated by.

$$K_{T^\circ\text{C}} = K_{20^\circ\text{C}} \left(1.047\right)^{T\text{ to } 20}$$

The temp at which portion of the sample should be tested to exert the same BOD in 2.5 days is

$$BOD_{5 \text{ days}} = 180 \text{ mg/l}$$

$k = 0.18 \text{ per day}$

$$L = 303.32 \text{ mg/l}$$

$$BOD_{2.5 \text{ days}} = L \left(1 - e^{-K_{T^\circ\text{C}} \cdot x \cdot 2.5}\right)$$

180 = $L \left(1 - e^{-K_{T^\circ\text{C}} \cdot x \cdot 2.5}\right)$

$1 - e^{-K_{T^\circ\text{C}} \cdot x \cdot 2.5} = 0.593 \Rightarrow K_{T^\circ\text{C}} = 0.36$

$K_{T^\circ\text{C}} = K_{20^\circ\text{C}} \left(1.047\right)^{T\text{ to } 20}$

0.36 = 0.18 (1.047)$^{T\text{ to } 20}$

$T = 35.09^\circ\text{C}$
Questions:

1. If BOD for the 2 days of a waste water is 75 mg/l and \( k = 0.015 \) per day, what is the ultimate BOD?

\[
175 = L \left(1 - e^{-kt}\right)
\]

\[
L = \frac{175}{1 - e^{-0.015 \times 2}} = 170.445 \text{ mg/litre}
\]

2. The following observations were made on a 4l dilution of wastewater. DO of diluted water used for dilution is 3 mg/l. DO of diluted sample after 5 days incubation = 0.8 mg/l. DO of original sample = 0.6 mg/l. Calculate the BOD of the sample after 5 days. Ultimate BOD of the two samples, \( k_D = 0.1 \text{ day}^{-1} \).

\[
DF = \frac{100 - 25}{4} = \frac{96}{4} = \frac{V_s + V_w}{V_w} \quad V_s = 4
\]

\[
BOD = (3 - 0.8) \times 25 - (3 - 0.6) = 52.6 \text{ mg/l}
\]

\[
52.6 = L \left(1 - 10^{-0.15 \times 5}\right)
\]

\[
L = \frac{52.6}{0.92} = 57.2 \text{ mg/l}
\]

\[
DO = \frac{DO_i \times V_s + DO_{in} \times V_w}{V_s + V_w} = 0.6 \times 4 + 3 \times 96 = 2.9
\]

\[
BOD = \frac{(DO_i - DO_f) \times DF}{V_s + V_w}
\]

\[
= \frac{(2.9 - 0.8) \times 25}{100} = 52.6 \text{ mg/l}
\]

3. Data from an unscheduled domestic waste water BOD test are: 500 ml of waste in 300 ml of water. Initial DO is 78 mg/l and final DO after 43 days, \( k = 0.1 \) per day. Calculate BOD and initial BOD.

\[
BOD = (78 - 4.3) \times 60 = 3000 \text{ mg/l}
\]

\[
BOD = 210 \text{ mg/litre}
\]

\[
\frac{210}{L \left(1 - e^{-0.1 \times 43}\right)} = L = 533.571 \text{ mg/l}
\]
The 5 days BOD at 20°C of a given sample is 450 mg/litre. Calculate the ultimate BOD at 35°C, given that it is 0.1 mg/litre per day.

\[ K_{d_{35^\circ C}} = K_{d_{20^\circ C}} (1.097)^{35-20} \]

\[ 0.199 \]

\[ BOD_{20^\circ C} = 1 \times (1 - 10^{-0.199 \times 5}) \]

\[ 450 \times 0.684 \]

\[ L = 658.11 \text{ mg/litre} \]

\[ BOD_{5,35^\circ C} = L \times (1 - 10^{-0.199 \times 5}) \]

\[ = 658.11 \times (1 - 10^{-0.199 \times 5}) \]

\[ BOD_{5,35^\circ C} = 591.54 \text{ mg/litre (initial BOD)} \]

**BOD - Types**

- BOD is of two types
- (i) 1st Stage BOD
- (ii) 2nd Stage BOD

- 98.1% of BOD is of Carbonaceous matter only

**BOD Ratio**

\[ \frac{BOD}{COD} = 0.92 \text{ to } 1.0 \] (Sewage in biodegradable type)

\[ BOD_{5} = 0.63 \text{ to } 0.68 \] (Sewage is said to have containing biodegradable waste)

**Population Equivalent**

Total BOD produced by the Industry per day of a city

\[ \frac{BOD \text{ produced by an individual}}{per \text{ day}} \]
Standard BOD produced by an individual per day = 0.08 kg

Relative Stability

It is defined as the ratio of oxygen present in the effluent to the amount of oxygen required for the decomposition of carbonaceous matter i.e. 1st stage BOD.

Relative Stability can be given by following eqn depending on the temp level

(i) \( \text{100} \times (1 - 0.794^{t_{20^\circ C}}) \)
(ii) \( \text{100} \times (1 - 0.604^{t_{37^\circ C}}) \)

Where \( t_{20^\circ C} \) = No. of days reqd. for the decomposition of carbonaceous matter at 20°C

\( t_{37^\circ C} \) = No. of days reqd. for the decomposition of carbonaceous matter at 37°C

Ques. Calculate the population eqn. of a city given that the avg. sewage from the city 95 x 10^6 lit/day & the avg. 5-day BOD is 300 mg/litre.

Population eqn. = \( \frac{\text{Total BOD contributed to city per day}}{\text{Individual BOD per day}} \)

\[ \text{Individual BOD per day} = \frac{95 \times 10^6 \text{ lit} \times 300 \times 10^{-3} \times 10^{-3} \text{ kg}}{\text{litre}} = 0.08 \text{ kg} \]

\[ \text{Population eqn.} = 356,250 \]

The period of Incubation is 10 days at 20°C, then the % of Relative Stability of the sewage sample is

\[ \text{Let} \quad 100 \times (1 - 0.794^{t_{20^\circ C}}) \]
\[ \quad 100 \times (1 - 0.794^{10}) \]
\[ \quad 90.04\% \]
Ques. Find out the BOD of a waste water containing 300 mg/l of ketone of chemical formula \( \text{C}_3\text{H}_6\text{O}_2\text{C}_2\text{H}_5 \). The oxidation eqn is given by
\[
\text{CH}_3\text{CO}_2\text{H}_5 + \frac{11}{2}\text{O}_2 \rightarrow 4\text{CO}_2 + 4\text{H}_2\text{O}
\]

\[
12 + 3\times1 + 12 + 16 + 2\times4 + 5 + \frac{11}{2}\times2\times16
\]

\[
92 + 176
\]

92 mg \rightarrow 176 mg of oxygen

300 mg \rightarrow ?

\[
\frac{176 \times 300}{92} = 733.3 \text{ mg/l}
\]

Design of sewer

- Designed for the \( \frac{1}{4} \) or \( \frac{3}{4} \) full

- Should have the self-cleansing velocity in order to prevent settling/ sedimentation

- Sewage pipes should be lay down at a particular gradient so that the flow occurs under gravity.

- Velocity in the sewer can be calculated by the following formula

\[ V = C \sqrt{RS} \]

- \( c \) = Chezy’s constant
- \( R \) = Hydraulic mean depth = \( \frac{A}{P} \) \( \frac{\text{Wetted area}}{\text{Wetted perimeter}} \)
- \( S \) = Gradient or slope at c sewer is place
- \( V \) = Velocity of flow for sewage
Manning's equation
\[ V = \frac{1}{n} \frac{R^{5/3}S^{1/2}}{N} \]

\[ n = \text{Manning's Coefficient} \]

There are 3 conditions in sewage pipeline to be maintained:

(i) Full flow condition
\[ R = \frac{\sqrt[3]{N} D^2}{\lambda D} = \frac{n}{\lambda D} \]

(ii) Half flow condition
\[ R = \frac{A}{P} = \frac{1}{\frac{\lambda D}{2}} = \frac{D}{4} \]

(iii) Partial flow condition
\[ MN = ON = OH \]
\[ MN = D \left( 1 - \cos \theta \right) \]
\[ A \Rightarrow 360^\circ \rightarrow A \left( \frac{\lambda D^2}{4} \right) \]
\[ \theta \rightarrow \frac{D}{A} \]

Wetted area
\[ A = \frac{A \lambda D^2 x}{360^\circ} - \frac{1}{2} \frac{D \cos \alpha}{2} \left( \frac{D \sin \alpha + D \sin \alpha}{2} \right) \]
\[ = \frac{A \lambda D^2 x}{360^\circ} - \frac{D^2}{2} \frac{\cos \alpha \sin \alpha}{2} \]
NOTE: These are known as hydraulic characteristics of a beam.

Ques: A 20 cm dia of beam is laid at a slope of 0.004 and is designed to carry a discharge at a depth of 10 cm. Manning's n = 0.014. Then find out the design discharge.

Som: It is condition of half flow.

\[ R = \frac{D}{4} = \frac{20}{4} = 5 \text{ cm} \]

\[ V = \frac{\eta R^{2/3} S^{1/2}}{D/4} = \frac{0.014 \times (5)^{2/3} (0.20)^{1/2}}{4} \]

\[ V = 13.209 \text{ cm/s} \]

\[ A = AV = \frac{\pi D^2}{4} \times 13.209 \]

\[ Q = 4149.84 \text{ cm}^3/\text{sec} \]

Ques: The slope of a 1 m diameter concrete sewer laid at 1 m/1000. It develops a velocity of 1 m/sec when flowing full. The velocity of flow of sewer when it is flowing half full will be.

\[ V_1 = \frac{R^{2/3} S^{1/2}}{R^{2/3} S^{1/2}} \]

\[ V_2 = \left( \frac{1}{\eta} \right)^{1/2} \left( \frac{1}{1000} \right)^{1/2} \]

\[ V_2 = 1 \text{ m/sec} \]
A 200m dia sewer with a slope of 1 in 500 is running full. Calculate the rate of flow in sewer given \( n = 0.058 \).

\[
V = \frac{1}{n} R^{0.5} S^{0.2}\]

\[
= \frac{1}{0.058} (20)^{0.5} (4)^{0.2} (1500)^{0.2}
\]

\[
V = 0.50 \text{ m/sec}
\]

\[Q = AV\]

\[Q = 0.015 \text{ m}^3/\text{sec}.
\]

- **DO** - BOD of municipal water - 100 to 500 mg/l
- **COD** > **BOD**
- **COD - BOD** = Non biodegradable organics
- **BOD** / **COD** considered to be an imp factor in water treatment

Water treatment = 2.6s
Sewage treatment

1. Preliminary → Primary treatment
   - Screening
     - Granit Chambers
     - In order to remove:
       - Sand
       - Floating matter
       - Delirks
         - (Inorganic)
         - Detention time: 2-5 mm

   - Skimming
     - (Remove:
       - OCT
       - Grease
       - Chc)

   - Detention
     - Particle size > 2 mm
     - Detention time: 1 hr
     - Flowing velocity: 0.3 m/sec

2. Primary treatment
   - Primary sedimentation tank
     - (Removal of:
       - Suspended particles)

3. Secondary treatment
   - Aerobic biological tank
     - Filters
     - Aeration tank
     - Oxidation
     - Pond & aerated lagoon

4. Final treatment
   - Anaerobic biological tank
   - Septic tank
   - Imhoff tank
8. The slope of a 1m dia. concrete sewer laid 1 in 1000, it develops a velocity of 1 m/s when flow in full, the velocity of flow of the sewer when it is flowing half full will be:

\[ V_1 = \frac{1}{n} \left( \frac{D}{14} \right)^{2/3} \left( S_n \right)^{1/2} \]

\[ V_2 = \left( \frac{D}{14} \right)^{2/3} \left( S_n \right)^{1/2} \]

\[ V_1 = V_2 \]
\[ = 1 \text{ m/s} \]

(1)

A 2m dia. sewer with a slope of 1 in 500 is running full. Calculate the rate of flow in sewer given that \( n = 0.42 \)

\[ V = \frac{1}{n} \left( \frac{D}{14} \right)^{2/3} S^{1/2} \]

\[ V = \frac{1}{0.42} \left( \frac{2.0}{4} \right)^{2/3} \left( \frac{1}{500} \right)^{1/2} \]

\[ V = 2089.09 \text{ m}^3/\text{s} \]

\[ Q = \frac{\pi (0.20)^2}{4} \times 0.50 \times 0.99 \]

\[ Q = 0.015 \text{ m}^3/\text{s} \]
Sewage Treatment:

1) Preliminary treatment
   - Screening: floating matter & debris
   - Grit chamber: inorganic (sand, silt, grit particles)
   - Skimming tank: oil, greases, etc.

   Detention time: 2-5 min

Primary Treatment → Primary sedimentation tank

   Removal of suspended particles
   Detention time: 2-3 hours
   Flowing velocity: 0.3 m/min

Secondary Treatment → Sewage treatment effluent

   Coming from the primary sedimentation tank, it is treated aeration or anaerobically to get the clearer effluents.
The removal of suspended solids in the primary sedimentation tank, this process of mining is called *sedimentation*.

**Secondary treatment**

- Aerobic treatment
  - Presence of O₂
    - Trickling filter (TP)
    - Activated sludge process (ASP)
    - Oxidation ponds
    - Aeration tanks
    - Activated lagoon

- Anaerobic treatment
  - Absence of O₂
    - Sludge digestion tank
    - Septic tanks
    - Imhoff tanks
    - Anaerobic lagoon

**Final Treatment** - Effluent coming out from the secondary treatment is treated with Elop (chlorination is done).

**Disposal**

[Flowchart for treatment]

- **Flowchart for treatment**
  - Raw sewage or peat sewage
  - Screening
  - Grit chamber
  - Skimming tank
  - Primary sedimentation tank
  - Suspended solid removed
  - Final treatment
  - Recirculation, sludge
  - Sludge digestion
  - Sludge disposal
  - Suspended solids removed
  - Secondary treatment
  - Sludge running over screen
  - Anaerobic treatment
  - Gas released (CH₄, CO₂)
  - Disposal

**Flowchart Details**

- **Flowchart Details**
  - Effluent from primary treatment
  - O₂ addition
  - Aeration
  - Final treatment
  - Disposal

[Flowchart Image]
Addition of active sludge to the sewage is known as seeding.

The rate of decomposition depends on:
- pH
- Temperature
- Stirring
- Seeding
- Growth of bacteria

**Septic Tank:**

\[ \frac{\text{influent}}{\text{Treatment}} \rightarrow \frac{\text{effluent}}{\text{BOD}} \]

\[ \frac{\text{influent}}{\text{BOD}} \rightarrow \frac{\text{treatment}}{\text{effluent}} \rightarrow \frac{\text{BOD}}{\text{removal}} = \frac{Y_i - Y_e}{Y_i} \times 100 \]

Efficiency of the tank:

\[ \text{efficiency} \text{ of the tank} = \frac{\text{BOD removal}}{\text{BOD applied}} \times 100 \]

Outlet from tank:

- Inlet
- Outlet
- Manure
- Sewage liquid
- Sludge
- Anaerobic action
Detention time = 12 to 36 hrs.

- Purposes involved → sedimentation
- Sludge digestion through anaerobic action

- Cleaning time period $t_c = 6$ month - 3 years.

- Sludge accumulation rate $q_a = 30$ lt/capital year

- Sewage flow rate $Q = \frac{L}{B} = 4$ to 5

- Removal of BOD = 100 - 200 ppm

- Freeboard = 0.3 to 0.45 m

Total Vol. of the tank: $V = V_{se} + V_{sludge}$

- $V_{se} = \text{Vol. of sewage water depth}$
- $V_{sludge} = \text{vol. of sludge}$

Total Vol. of tank $V = L \times B \times H$

where $L = \text{length of tank}, B = \text{width of tank}, H = \text{depth of tank}$
Design a septic tank for a colony of 200 people, the colony is supplied water at a rate of 135 lps. Assume retention period of 34 hrs and 75% of water becomes the waste water. The tank is cleaned once in a year. The rate of volatilization of sludge is 40 lts/person/year. The depth of the tank is to be kept at 2 m. Provide a freeboard of 0.3 m. Assume T ratio as 3.

Population of colony = 200
supplied water = 135 lps

Total drinking water = 135 \times 200 = 27,000 lps

\[ \frac{1000 \text{ lts}}{1 \text{ m}^3} \times 27,000\text{ lps} = 31.25 \times 10^{-4} \text{ m}^3/\text{sec} \]

sewage rate = \[ \frac{75}{100} \times 31.25 \times 10^{-4} = 2.34 \times 10^{-5} \text{ m}^3/\text{sec} \]

\[ \frac{2.34 \times 10^{-5}}{240 \text{ hrs}} = 9.75 \times 10^{-8} \text{ m}^3/\text{sec} \]

Vol of sewage water \[ V_s = 9.75 \times 10^{-8} \times 240 \times 60 \times 60 = 0.22 \text{ m}^3 \]

Vol of sludge \[ V_{sl} = \frac{9.75 \times 10^{-8} \times 240 \times 60 \times 60}{10^3} = 0.0003 \text{ m}^3 \]
Total Volume \( V = V_{50} + V_s \)

\[ \begin{align*}
V_{50} &= 8 + 20.22 \, m^3 = 98.42 \, m^3 \\
V_s &= 5.1 \, B^2 \\
5.1 \, B^2 &= 28.22 \\
B &= 2.35 \, m \\
L &= 7.05 \, m \\
H &= 2.0 \, m
\end{align*} \]

A. Estimate the design a circular sewage tank for a town population of 40,000. The avg. water demand is 140 lpcd. Assume that 70% of water reaches at the treatment unit & the max. design discharge of sewage is taken as 3.7 times the avg. sewage discharge demand.

- Population = 40,000
- Per capita consumption = 140 lpcd
- Total = 40000 x 140 l/d
- = 5600000
- = 1000 x 24 x 60 x 60
- = 0.0648

- Sewage rate \( = \frac{70 \times 0.0648}{100} = 0.0454 \, m^3/s\)
Max design discharge:
\[ Q = 2.7 \times 0.0454 = 0.1235 \text{ m}^3/\text{sec} \]

Assume: \( D_t = 2\text{hrs} \)

Vol of sedimentation tank = \( Q \times D_t = 0.1235 \times 2 \times 60 \times 60 = 88.2 \text{ m}^3 \)

Assume effective depth of the tank = 3 m (without freeboard)

Surface area of tank = \( \frac{\text{Vol}}{\text{Depth}} = \frac{88.2}{3} = 29.4 \text{ m}^2 \)

Since the tank is circular
\[ \frac{a^2 \pi}{4} = 29.4 \]
\[ a = 19.34 \text{ m} \]

By calculation of the surface loading for the sedimentation tank per day it becomes:

Surface loading of tank, \( S = \frac{\text{Capacity of tank or Volume of tank}}{\text{Surface area}} \)

\[ S = \frac{88.2}{29.4} = 3.0 \text{ m}^3/\text{m}^2/\text{day} \]

\[ S = \frac{0.1235 \times 24 \times 60 \times 60}{29.4} = 36 \text{ m}^3/\text{m}^2/\text{day} \]
The permissible surface loading for sedimentation tank is 50 m³/m²/day.

\[
\text{surface loading} = 30 \text{ m}^3/\text{m}^2/\text{day} < 50 \text{ m}^3/\text{m}^2/\text{day}
\]
so design is safe.

Estimate the size of septic tank. (\(\frac{H}{L} = 2.25\))
Liquid depth 2m with 300mm freeboard. Desludging (removal of sludge) intervals in years and the total trench area of percolation field for a small colony of 300 people. Assume a water supply of 100 lpcd/capital/day. The waste water flow if 80% of water consumption, the sludge product of 0.04 m³/capita/yr and the retention time of 3 days. Desludging is done when the tank is 1/3 full of the sludge. A percolation test indicated an allowable hydraulic loading of 100 ltr/m²/day.

Population = 300

Per capita consumption = 100 lpcd

Total = 300 x 100 l/day

\[
\text{sewage rate} = \frac{80 \times 300 \times 100}{100 \times 1000 \times 24 \times 3600 \times 60} = 2.77 \times 10^{-4} \text{ m}^3/\text{sec}
\]

\(V_3\) Vol of tank = \(\frac{Q \cdot t}{A}\)

\[
= 2.77 \times 10^{-4} \times 3 \times 24 \times 3600 \times 60
\]

= 71.7984 m³
total depth = 23 m
free board = 0.3
liquid depth = 2.0 m

\[ V_{se} = \frac{V}{3} \]
\[ V = V_s + V_{se} \]
\[ V = \frac{V_s + V}{3} \]
\[ V_s = V - \frac{V}{3} = \frac{2V}{3} \quad (3) \]

\[ W_s = LBD \quad V = LBH \]
\[ = 2.25B^2 \cdot 2 \]
\[ V = 4.5B^2 \quad (3) \]

\[ V = V_s + V_{se} \]
\[ = 4.5B^2 = \frac{V - \frac{V}{3}}{3} \]
\[ = \frac{V}{3} \cdot \left( \frac{4.5B^2}{3} \right) \]

\[ V = \frac{V}{3} \cdot \left( \frac{4.5B^2}{3} \right) \]

\[ B = \frac{V}{4.5B^2} \]

\[ \frac{2V}{3} = 71.984 \]
\[ V = 107.69 \text{ m}^3 \quad (4) \]

\[ 107.69 = 4.9B^2 \]
\[ B = 4.6 \text{ m}, \quad L = 10.5 \text{ m} \]
\[ H = 240.3 = 9.3 \text{ m} \]
Hydraulic loading is defined as the ratio of flow to surface area.

\[
\text{loading} = \frac{q}{A}
\]

\[
100 = \frac{2.77 \times 10^{-4}}{100 \times 3.24 \times 60 \times 60}
\]

\[
A = 339.33 \text{ m}^2
\]

\[
V = \frac{q}{3} = \frac{q}{a}
\]

\[
358.9 = 0.04 \times 360 \times 3.99
\]

\[
t_e = 2.99
\]

\[	t_e = 3 \text{ years}
\]

Sludge Digestion Tank (SDT):

It is an anaerobic process and involves the absence of O_2.

Paraeobtic treatment releases gases: CH_4 - 65-70%, CO_2 - 30%.

Influent \rightarrow SDT \rightarrow Sludge Thickener \rightarrow Sludge Flaking Body \rightarrow Final Treatment \rightarrow Disposal

Effluent: (Digested sludge)
40-60% of organic solids are not digested.

CH₄ gas released is around 0.9 m³ per kg of volatile solids of the digested sludge. volatile: (H₂ = 0.6 m³) for raw sewage.

Three stages of 'SOT' are:
1. Acid fermentation
2. Acid regression
3. Alkaline fermentation

The parameters which influence sludge digestion are:

1. temp.
2. pH
3. seeding phenomenon
4. mineral content of sludge
5. nature of bacteria: Mesophilic Bacteria = 39°C
6. Thermophilic Bacteria = 35-50°C

Acid Fermentation: Sludge digestion process contains the following 3 forms:

1. Digested sludge: Stable solid matter, light brown or black in colour free of pathogenic bacteria containing cysts of bacteria.

The digested sludge is dried up and can be used as a fertilizer.

2. Supernatant liquid: Finely divided solid matter and liquid.

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containing 3000 ppm quantity of BOD. It is recycled in the treatment plant and sent back to the raw sewage.

5. Gases of Decomposition: CH₄ around 65% - 70%, CO₂ - 30%. Other gases like N₂, H₂S are evolved. The various stages involved in the process of digestion of sludge are:

1. Acid Fermentation: The sewage will be acidic in nature and it indicates the beginning of the digestion process.

2. Acid Regression: At this stage of sludge digestion, BOD remains high and sludge becomes foamy. Scum is formed at the top due to which gases are trapped.

3. Alkaline Fermentation: During this stage, the liquid and the digested solids and gases get separated. The sludge becomes alkaline in nature. At this stage, BOD falls rapidly and large volume of the CH₄ gas with small amount of other gases is evolved.

The parameters which influence sludge digestion are:

1. Temperature: At temp. increases the rate of the digestion will be more.
In the case of sludge digestion at 39°C, it takes 80 days for the decomposition process. The mesophilic bacteria is acting in this process. As the temp increases there will be the action of thermophilic bacteria.

1. **pH**: The sewage in the form of sludge in the sludge digestion tank should have a pH of around 7.2 to 7.4.

   For the proper growth of bacteria to encourage bacterial action lime should be added in the form of calcium hydroxide (Ca(OH)₂).

2. **Seeding Phenomenon**: Proper seeding can be done by the addition of activated sludge (which is rich in concentration of active microorganism) to the raw sewage in order to reduce the time period of decomposition.

   Seeding a seeding sludge can be done by means of stirring rods & bacterial enzymes present in digested sludge should be mixed thoroughly for the better decomposition.

   Bacteria has two types:

   - **Mesophilic bacteria** works out at lower temp. It helps in the anaerobic reaction whereas the thermophilic bacteria acts at higher temp. and it decomposes the sludge at a faster rate.
a) The sludge from the sedimentation tank contains 95% moisture content. After treatment in the sludge digestion tank if the moisture content of the sludge is going to become more, then it can be reduced by seeding into sending it into sludge thicker tank. The m.c. of sludge can be calculated by:

\[ m.c. = \frac{w_0}{w_0 + w_s} = 95 \%
\]

where:

- \( w_s \) = wt. of solids
- \( w_0 \) = wt. of water

If sludge having a m.c. of \( P_1 \) & volume \( V_1 \) in initial t is dried up, \( V_1 (100 - P_1) = V_2 (100 - P_2) \) after drying the m.c becomes \( P_2 \) & volume \( V_2 \) then

\[ V_1 (100 - P_1) = V_2 (100 - P_2) \]

In this case, this eqn holds good.

For the solid content if the quantity of sludge with a m.c. of 98% is \( x \), then the quantity of sludge with a m.c. of 96% will be:

\[ V_1 (100 - P_1) = V_2 (100 - P_2) \]

\[ x (100 - 98) = V_2 (100 - 96) \]

\[ x \times 2 = V_2 \times 4 \]

\[ V_2 = \frac{x}{2} \]
A sedimentation tank is treating 4.5 MLD of sewage containing 275 ppm of suspended solids. The tank removes 55% of suspended solids. Calculate:

(a) wt of sludge produced per day assuming the moisture of sludge as 96%.

(b) If the sp gr of sludge is 1.02, calculate the sludge quantity in bulk (Volume).

\[ Q = 4.5 \text{ MLD} \]

Sewage contains 275 mg/l of suspended solids

Total discharge = \( 4.5 \times 10^6 \) ltr/day.

Total suspended solids in sewage = \( 4.5 \times 10^6 \times 275 \times 10^{-3} \)

= 1237.5 kg/day.

Total removal suspended solids = \( \frac{55}{100} \times 1237.5 \)

= 680.62 kg/day.

Moisture content of sludge = 96% = \( \frac{96}{100} \times 96 + 4 \)

\[ W = \frac{W_0 + 4}{100} \]

4 kg of solids producing 100 kg of sludge

\[ \frac{100}{4} \times 680.62 \]

= 17015.5 kg

= 17.015 tonnes

Density = \( \frac{\text{wt of sludge}}{\text{Vol of sludge}} \)
Vol. of sludge = \frac{\text{wt of sludge}}{\text{sp. gravity density}} = 17.05 \text{ tones}

\text{sp. gravity density} = 1.02 \text{ t/m}^3

= 16.68 \text{ m}^3.

\text{Total sp. gravity} = \frac{\gamma_{\text{sludg}}}{\gamma_0}

1.02 = \frac{\gamma_{\text{sludg}}}{1 \text{ t/m}^3}

\gamma_{\text{sludg}} = 1.02 \text{ t/m}^3

\text{Sludge Digestion Tank:}

V_1 (100 - P_1) = V_2 (100 - P_2)

\text{Vol. of sludge digested} = \frac{1}{3} \times \text{(volume intake)}

\text{The vol. of sludge} = V_1 - \frac{2}{3} (V_1 - V_2)

\text{Imhoff Tank:} -- \text{Where confined area is not possible to have sedimentation tank, an anaerobic tank (septic tank) separate then we use in the better lesser space by combining the section together in imhoff tank.}

\text{Physical forces are acting i.e. the sedimentation of the sludge is taking place by gravitational force.}

\text{The unit operation of imhoff cone, basically}
settling chamber or sedimentation tank

Converting the form of the sludge from one form into the another form i.e. the impurities are converted chemically into the other form by the anaerobic action.

Detention time can be kept up to 30 days.

Disposal Methods of sludge:

1. Absorption Trenches
2. soak pits
3. cess pools.

In an absorption trenches the digested sludge can be carried out through the open jointed pipes into the trenches dug below the GS. After 2-3 months of periods, the entire sludge discharge into the trench will become earthy material and it acts as a good manure in the absorption trenches.
we can grow the vegetation plants itself. This phenomenon known as organic farming.

2. Soak Pits: - Soak pit is a mechanism in which a pit is excavated up to a certain depth to the digested sludge effluent is allowed into the pit, the sludge m.e. is get reduced by the surrounding brick bats filled up in the pit. After sometime the sludge is converted into the manure in very form and it can be utilise as a manure.

3. Cess Pools: -

Cess pools, top portion acts as a absorption trench and bottom portion acts as septic tank. When the soil is porous there is danger of the percolation of sludge to the nearby wells. In the areas of cess pools, care should be taken that no open wells should exist in the surrounding area.
Aerobic Treatment - 1) Trickling filter
2) ASP
3) Oxidation Pond

Trickling Filter:

![Diagram of Trickling Filter Process]

1. Trickling filter is an aerobic Treatment.
2. Growth aerobic bacteria is dominant.
3. Biological decomposition takes place through the aerobic action.
4. The presence presence of attached growth bacteria in the filter beds makes easy for the aerobic decomposition.

Efficiency \( \eta \) = \( \frac{\text{BOD removed}}{\text{applied BOD}} \) \( \frac{y_i - y_e}{y_i} \times 100 \)

Types of T.F.:
1) Intermittent T.F.
2) Contact bed filters
3) Low rate (standard rate) filter
4) High rate filter
1. Intermittent TF → Filter beds contain coarse sand, BOD removal is 90-95%

2. Contact bed filters → Filter bed contains gravel 60-70% - BOD removed.

3.20 Low Rate (Standard Rate) Filters → Stone aggregate gated 70-80% BOD removed. Recirculation ratio = 0.

3. High Rate Filters → Stone aggregate, but sludge is recirculated - BOD removed 80-90%.

Problems of filters:

1. Flies (spreading of flies) → Psychoda.
2. Odour → Prevent by chlorination.
3. Ponds - Clogging of pores of filter beds of algae & other plants.

Aeration

Rotating arms nozzles

Outlet: filtered growth of bacteria

Aerobic decomposition

Ultraviolet biological film formation with enriched inert biomass

Rakes or shovels
Parameters useful for design of TF:

Organic loading \( u = \frac{Q h_i}{V} \)

where \( Q \) = discharge rate of sewage
\( V \) = Volume should be kept in ha-m

Recirculation Ratio \( R = \frac{R_i}{I} \)

\( R_i \) = rate of recirculation
\( I \) = rate of influent

In case of high rate filters, the recirculation of sludge is done whereas in case of low rate filters there is no need of recirculation.

Organic loading: It is defined as the amount of BOD i.e., supply to the trickling filter at the given flow rate per unit volume.

Recirculation factor: \( P = \frac{1 + R/I}{(1 + 0.1R/I)^2} \)

Efficiency of TF:

\[ \eta = \frac{100}{1 + 0.009 + \frac{1}{u}} \]

\( u \) = organic loading
For high scale filters

\[ \eta = \frac{100}{140.004 \times f} \]

where \( u \) = \( \frac{A}{V} \), \( f = \) recirculation factor depends on recirculation ratio

In case of design of TIP, the required vol. of the filter can be formed by making use of organic loading so vol of filter

\[ V = \frac{A}{u} \]

The depth of the tank is 2-3 m

Q: Estimate the efficiency of a 30 m dia & 1 m deep single stage high scale trickling filter for the following data:

- Sewage flow = 4.5 MLD
- Recirculation ratio = 1.4
- BOD of raw sewage = 250 ppm
- BOD removed in the primary clarifier (PST) = 25%

\[ \text{sewage flow} = 4.5 \times 10^6 \text{ m}^3/\text{day} \]
\[ A = 4.5 \times 10^6 \text{ m}^2/\text{day} \]
\[ \text{BOD of raw sewage} = 250 \times 10^{-6} \text{ g/l} \]

Total BOD of sewage \[ = 250 \times 10^{-6} \times 4.5 \times 10^6 \]
\[ = 11.25 \text{ kg/day} \]
BOD supply as influent to T.P. = $75\,\%$ of $11.25$ 

$= \frac{75}{100} \times 11.25$ 

$Q_{yi} = 8.4375$ 

Recirculation factor = 1.4 

$P = \frac{1 + \frac{R}{i}}{(1 + 0.1 \times \frac{R}{i})^2}$ 

$P = \frac{1 + 1.4}{(1 + 0.1 \times 2.4)^2}$ 

$P = 2.846$ 

Diameter of T.P. = $D = 30$ m 

Depth = 1 m 

Volume of T.P. = $\frac{\pi}{4} \times d^2 \times h = \frac{\pi}{4} \times (30)^2 \times 1$ 

$= 706.85 \, m^3 = 706.85 \%$ 

$q_{i} = \frac{100}{1 + 0.0044 \sqrt{Q_{yi}}} = \frac{100}{1 + 0.0044 \sqrt{8.4375}}$
\[ q = 100 \]

\[ 1 + 0.0044 \sqrt{\frac{843.75}{70.6 \times 10^3 \times 1.846}} \]

\[ q = 73.86\% \]

A trickling filter plant treats 1500 m³/day of sewage with a BOD of 220 ppm and a suspended solid concentration of 250 ppm. Estimate the total solids production assuming that the primary clarification removes 30% of BOD and 60% of influent solids. Take the solid production in trickling filter as 0.5 kg/s/kg of the applied BOD.

Trickling filter treats 500 m³/day of sewage.

BOD of sewage = 220 ppm.

Suspended solids BOD = 250 mg/l.

Primary clarifier removed = 30% of BOD & 60% of influent solids.

Solids production rate = 0.5 kg/s/kg.

\[ \text{influent BOD} \times \text{TP} = 70\% \text{ of BOD} \]

BOD removed = BOD of sewage given as = 230 mg/l.

\[ \frac{70 \times 230 \times 10^{-6}}{100} \]

\[ = 1.54 \times 10^{-4} \text{ kg/l} \]

The scale of sewage flow given in the problem.
Total rate of BOD sent to the TP = \(1.54 \times 10^{-4} \times 1500\) = 231 kg/day

1 kg → 0.5 kg

931 kg = 0.5 \times 231 = 115.5 kg

The suspended solids removal is 60% of influent, consisting of 250 ppm BOD in the PST.

Total sewage flow rate = 1500 m\(^3\)/day

Total wt. of solids in influent = \(1500 \times 10^3 \times 0.6 \times 10^{-6}\) = 0.375 kg/day

60% of influent is classified by PST

\(0.6 \times 1500 \times 10^3\) = 225 kg/day

\[\text{PST} \rightarrow \text{TF} \rightarrow \text{SST}\]

Suspended solid = 60%

Total wt. of suspended solids = 225 kg/day

Total wt. of solids producing the TF = 115 kg/day

Total solids produced = 225 + 115

= 340.5 kg/day
Q: Determine the dimensions of a high rate T.F. given that

a) sewage flow = 3 MLD
b) Recirculation ratio = 1.5
c) BOD of raw sewage = 250 mg/l

d) BOD removed in Primary tank = 25%
e) Final effluent BOD desired = 30 mg/l

By what % of the dia. of the filter will have to be modified if it is to be designed as standard rate T.F. for the above requirements.

\[ Q = 3 \times 10^6 \text{ m}^3/\text{day} \]

BOD of raw sewage = 250 mg/l

\[ \text{Total BOD of sewage} = 3 \times 10^6 \times 250 \text{ mg} \]

150 kg/day

Effluent BOD desired = 30 mg/l

Influent of T.F. = 75% = \[ \frac{75}{100} \times 750 = 562.5 \text{ kg/day} \]

After treatment in TF

Effluent BOD = 30 mg/l = \[ 30 \times 10^{-6} \times 3 \times 10^{-6} \]

= 90 kg/day

\[ M = \frac{y_i - y_f \times 10^6}{y_i} = \frac{562.5 - 90}{562.5} \times 100 \]

= 84%
Recirculation factor

\[ \eta = \frac{1 + R}{\left(1 + 0.1 R \right)^2} \]

\[ \eta = \frac{1 + 1.5}{\left(1 + 0.1 \times 1.5 \right)^2} \]

\[ \eta = 1.89 \]

\[ \eta = \frac{100}{1 + 0.0044 \sqrt{\frac{84.5}{\nu \cdot \eta}}} \]

\[ \eta = \frac{100}{1 + 0.0044 \sqrt{562.5}} \sqrt{\nu \cdot 1.89} \]

By assuming the depth = 3 m

\[ V = \frac{\pi d^2 h}{4} = \frac{\pi d^3 \times 0.84}{4} \]

\[ \frac{100}{1 + 0.0044 \sqrt{562.5}} \sqrt{\frac{\pi d^3 \times 1.89}{2}} = 0.84 \]

\[ d = 31.7 \text{ m} \]
(ii) For standard scale filter:

\[ R = 0 \]
\[ I = 1 \]
\[ f = 1 \]

\[ q = \frac{100}{1 + 0.0044 \sqrt{\nu}} = \frac{100}{1 + 0.0044 \sqrt{562.5}} \]

\[ \phi = 84 = \frac{100}{1 + 0.0044 \sqrt{562.5}} \]
\[ N = 3001.5 \text{ m}^3 \]

Assuming the depth of T.P. = 2 m.

Area \[ \frac{\pi d^2}{4} \cdot \frac{3001.5}{2} = 1500.75 \]

\[ \frac{\pi d^2}{4} = 1500.75 \]
\[ d = 43.71 \text{ m} \]

\[ \% \text{ of modification} = \frac{43.71 - 31.7}{31.7} \times 100 \]
\[ = 31.71 \]
\[ = 87.5 \% \]
Design a sludge digestion tank for the primary sludge i.e. raw sludge from the primary sedimentation tank with following data:

a) Avg. flow of sewage = 20 MLD.
b) Total suspended solids of raw sewage = 300 mg/l.
c) 60% of suspended solids is removed in the primary sedimentation tanks.
d) M.C. of the digested sludge is around 85%.
e) Sp. gr. of sludge = 1.02.
f) Digestion period in the sludge digestion tank = 30 days.

Avg. flow of sewage = 20 MLD
Total flow rate = 20 x 10^6 cft/day.

Total wt. of suspended solids = 300 mg/l.
Total wt. of suspended solids removed = 60%.
\[ = \frac{60}{100} \times 20 \times 10^6 \times 300 \times 10^{-3} \]
\[ = 36,000 \text{ kg/day} \]

Digestion volume of sludge = \( \frac{1}{3} \) of initial volume on assuming \( V_2 = \frac{1}{3} V_1 \).

Given that m.c. of digested sludge = 85%.

Let \( p_1 \) be the m.c. of undigested sludge.
Let \( p_2 \) be the m.c. of digested sludge.
\[ V_2 = V_1 / 3 \]

Using \( V_1 (100 - p_1) = V_2 (100 - p_2) \).
\[ N_1 (100 - P_1) = \frac{V_1 (100 - 85)}{3} \]

\[ P_1 = 95^\circ F \]

5 kg of solid in sludge — 95 kg of water in sludge

\[ \frac{8600}{5} \times 3600 \]

= 68400 kg of water

Total wt. of sludge = \( \omega_s + \omega_w \)

= 3600 + 68400

= 72000 kg

Given that sp. gr. of sludge = 1.02

Volume = \frac{weight \ of \ sludge}{sp. \ gr.}

\[ V_1 = 70.58 \ m^3 \]

\[ V_2 = \frac{V_1}{3} \]

\[ V_2 = 70.58 - 23.5 \ m^2 \]

Total Volume: \[ V = \left[ \frac{V_1}{3} - \frac{2}{3} (V_1 - V_2) \right] \times 36 \]

\[ V = 1175.8 \ m^3 \]

Depth = 6 m (Assume)

\[ \frac{11.758}{4} \]

\[ \phi \]

\[ d^2 = 19.34 \ m \]
An anaerobic reactor receives waste water at a flow rate of 500 m³/day with COD of 2000 mg/l.
The effluent COD is 400 mg/l. Assuming the waste water contains 20% of biodegradable solids, the daily vol of biogas produced by the reactor is 192 cubic m.

Q: A T-F is designed with an organic loading of 0.175 kg/m³/day. If the effluent influent of sewage is 150 mg/l then effluent BOD is?

The BOD of sewage entering into the T-F is 200 ppm. If the effluent of sewage is 50 ppm then the I of T-F = ?

\[ \text{Sludge age} \ 
\text{Qe} = \frac{V \cdot X_i}{Q_w \cdot X_r + (Q_i - Q_w) \cdot X_0} \]

\[ Q_i = \text{Inflow} \]

\[ V = \text{vol. of water} \]

\[ Q_R = \frac{R \cdot Q}{100} \]
Activated Sludge Process (ASP):

It is an aerobic process. Decomposition of sludge takes place under aerobic bacteria.

Making use of activated sludge

Activated sludge: sludge having high concentration of active microorganisms (from secondary sedimentation plant SST).

The activated sludge is mixed with new sewage in aeration tank (ASP) with high supply of variation or oxygen & kept for 9-8 hours. The active sludge mixed in aeration tank as mixed liquor.

The solids that are formed in the aeration tank are known as 'MLSS' (Mixed Liquor Suspended Solids).

\[ Q_i \rightarrow \text{Aeration Tank (MLSS)} \rightarrow \text{SST} \rightarrow Q_e \]

\[ Q_r = \text{flow rate of waste}, \quad X_w = \text{concentration of solids in the waste} \]

\[ Q_i = \text{flow rate of influent}, \quad X_i = \text{concentration of influent solids} \]

\[ Q_e = \text{flow rate of effluent}, \quad X_e = \text{concentration of effluent} \]

\[ Q_r = \text{Recirculation flow rate}, \quad X_r = \text{concentration of recycled solids} \]
Design consideration

Hydraulic Retention Time: It is defined as the volume of tank to the rate of flow of the sewage into the tank.

\[ \text{HRT} = \frac{V}{Q} \]

\( V \) = Vol. of loading

\( Q \) = Loading of organic

Volumetric Organic Loading: The volumetric organic loading is defined as BOD load applied per unit volume of the aeration tank.

\[ u = \frac{Q_{\text{org}}}{V} \]

Efficiency of Aeration Tank: It is defined as

\[ \eta = \frac{Y_t - Y_e}{Y_t} \times 100 \]

\[ \eta = \frac{\text{BOD removed}}{\text{BOD applied}} \times 100 \]

Sludge Thickening:

Sludge Bulking

Sludge Volume Index (SVI)

F:M ratio → Food to microorganisms
Sludge Thickening: Sludge from secondary settling tank contains too much of moisture that is 98-99%. The moisture is first reduced by sending into the sludge thickener where it will be reduced from 98-99%.

It helps in reducing the capacity of digestion tanks.

It is similar to the circular settling tank with a retention time of 12-24 hrs.

Bulking of Sludge: Under sick condition, the settled sludge may contain more moisture and their resulting swelling of the sludge volume. Due to the sludge bulking, it remains in the suspension in the efficient of secondary clarifiers.

To avoid the bulking of sludge, following are the remedial measures:

a) Eliminate the industrial waste.
b) Chlorinate the sewage.
c) Supply excess amount of oxygen and increased aeration.
d) Raising the pH of the sewage to 8 by the addition of lime.

Sludge Volume Index: It is defined as it is the volume occupied in ml by 1 gm of solids in the mixed liquor after settling for a period of 30 min.
SVI = Vol. of sludge settled in ml
MLSS in gm

SVI = 50 - 150 should be adopted

F/M Ratio = Food to Microorganisms Ratio
It is called as organic loading.
It is defined as the ratio of BOD applied per kg day to the MLSS in the aeration tank.

\[
\frac{F}{M} = \frac{O_{l}}{V \cdot C_{l}}
\]

where
V = Vol. of the sludge
\(C_{l}\) = concentration of mixed liquor solids.

High F/M ratio means is the removal of the BOD and vice versa.

For an activated sludge process F/M ratio should be 0.3 to 0.4.
The efficiency of aeration tank depends upon the quantity of BOD removal.

Oxidation Pond: Water Stabilization pond

Cheapest treatment → area requirement is more

Nuisance is more

Oxidation ponds of the eastern basins open channel constructed below the p.l. surrounded by the
In case of oxidation ponds, the principal involved is establishment of organic matter by the combined action of algae and other microorganisms by the symbiotic relationship.

In the case of oxidation pond the symbiotic relationship exist the algae & microorganism in the sense algae produced O₂ while growing in the presence of sunlight & the O₂ is utilized in the end products in the oxidation pond of CO₂, NH₄ & phosphates. The discharge in the form of waste i.e. coming from the oxidation pond after decomposition can be used in land irrigation system.

In the case of oxidation pond, the area required is around ½ hectare to 1 hectare and retention time is 20-30 days. BOD removal = 90%

Cleaning period of settled sludge = 6 years. To stimulate the process of decomposition, to reduce the time of decomposition sodium nitrate is added.
Sewage Effluent Disposal

Sewage is disposed by two methods:

1. Dilution method
2. Land treatment

Dilution Method:
Effluent is discharged into an outfall running river/stream source.

\[ Q_s \] sewage waste
\[ D.O \] permissible limits

River:

\[ Q_r \] river

Mixture:

\[ E_m = Q_r + Q_s \]

\[ D.O_{min} = \frac{Q_r D.O_r + Q_s D.O_s}{Q_r + Q_s} \]

BOD of mixture:

\[ Y_{min} = \frac{Q_r Y_r + Q_s Y_s}{Q_r + Q_s} \]
Stages of River in the Self Purification

1. Zone of clean water
   - Border of chemical aerobic

2. Zone of clear water
   a) Zone of degradation
   b) Zone of active decomposition
   c) Zone of recovery
   d) Zone of clean water

\[ k_p = \text{rate of constant which reoxygenation occurs} \]

\[ k_D = \text{Deoxygenation} \]
DO Deficit = Saturation DO - Present DO

Initial DO Deficit = Saturation DO - 50 min

Dilution coefficient

Self-purification constant of a river: \[ f = \frac{K_r}{K_d} \]

\[
D_f = \frac{K_d \cdot L}{K_r - K_d} \left[ 10 - K_d t - 10^{-K_r t} \right] \left[ 10^{-K_r t} \right]
\]

\[
D_{el} = \frac{K_d \cdot L}{K_r} \left[ 10^{-K_d t_c} \right] + \quad (1)
\]

\[
t_c = \frac{1}{K_d (f - 1)} \log \left[ 1 \cdot (f - 1) \frac{DO}{L} \right] f^{-1}
\]

where \( D_e \) = critical DO

\( t_e \) = time period at which critical DO deficit occurs

\( D_f \) = DO at any time \( t \)

For the dilution process of rivers, it is very important to check the qualities of running rivers or streams and the geological area.
Land Treatment: - Directly dumping the effluent sewage effluent onto the land. After some time, the pores of the soil will be clogged and filtration does not take place properly. Presence of effluent (sewage sickness) land pollution & leading of sewage or leachate forms GWI get polluted.

coloured water

Factors Affecting the self purification of system of mine:

Dilution: - If the sewage is mixed with large vol. of water or it is much diluted, sewage always remains in the aerobic condition & anaerobic condition never comes. The always DO remains present in the water.

Current: - The self purification of stream is directly depends upon the current. When there is no current, sewage matter deposit near the outfall causing formation of the sludge banks and causes foul odours. And in the slow current, sedimentation takes place causing the growth of algae resulting in the product of oxygen.
Temperature: As the activities of organisms depends upon the temp, the self purification will also depend upon the temp.

At lower temp the organisms activities are slow due to the slow rate of decomposition. As in the submerging sea summer season the temp increases, the stream will get self purification is less. In the winter season.

Sedimentation: With the slow current the heavier solids settle in the stream bed & start anaerobic decomposition. The products of decomposition are again mixed with the water of current of dilution is sufficient, anaerobic condition will not develop.

Sunlight: The pathogens are killed if they are expose to the sunlight; therefore sunlight helps in the self purification process. Algae also grow in sun light causing the production of oxygen.

Oxidation: The organic matter immediately mixed with the stream starts getting oxidised due to the development of oxidising & organisms present in water. The process prevails till the...
Complete oxidation of organic matter
The oxygen demand in satisfied
The stream becomes purified due to
this phenomenon

Reduction - It occurs in streams due to
hydrolysis of organic matter
biologically & chemically, anaerobic
organisms start the splitting of
complex organic substances present in
the sewage.
This action produces odours & gases
and thus, stabilization comes into
the picture.

A site adjacent to a river is
considered for the location of a
municipal waste water treatment plant
having an avg. inflow of 2500 m³/d.
The selected treatment sequence is as
follows:
1. grit channel
2. primary sedimentation process
3. activated sludge process
4. secondary sedimentation process

Design the grit channel assuming
max. hydraulic loading as the twice the
avg. inflow rate. Assume appropriate desk
parameters if required.
Hydraulic loading = 20 kg sewage flow

\[ \text{Q} = 2 \times 2500 = 5000 \text{ m}^3/\text{day} \]

By assuming the flow velocity as 0.3 m/s and detention time period \( t_d \) = 2 minutes

then the length of the grit channel \( L \) = velocity \( \times t_d \) \[ = 0.3 \times 2 \times 60 \]

\[ = 36 \text{ m} \]

The surface area of channel can be calculated as

\[ \text{area} = \frac{\text{design flow rate}}{\text{velocity}} \]

\[ = \frac{5000}{0.3} = \frac{5000}{24 \times 60} \]

\[ = 0.192 \text{ m}^2 \]

by assuming the depth of channel as 1 m

\[ B = \text{area} \times \frac{1}{\text{depth}} = \frac{0.192}{1} = 19 \text{ cm} \]

Hence we provide the dimension of grit channel as

\[ L \times B \times H = 36 \text{ m} \times 0.19 \text{ m} \times (1 + 0.3) \]

\[ L \times B \times H = 36 \times 0.19 \times 1.3 \]
A settling but on a sample drawn from the equalization tank, liquor of ASP (MLSS = 2800 mg/l) was carried out with 1 lt sample. The sludge yield, the settled vol. of 200 ml. The vol. of sludge vol. index shall be

\[
SVI = \frac{Vol.\ settled}{MLSS} \times 10^3
\]

\[
SVI = \frac{200 \times 10^3}{2800} = 71.4 \text{ mg/l}
\]

Query: 1 lt of sewage when allowed to settle for 20 min gives a sludge vol. of 27 cm³. If the dry wt of the sludge is 3 gm, the SVI = ?

\[
SVI = \frac{27}{3} = 9
\]

Thus, in certain situation, the waste water discharging into a river mixes with the river water instantaneously & completely. Following is the data available:

- Waste water DO = 2.00 mg/l
- Discharge rate = 110 m³/see
- River water DO = 8.3 mg/l

Initial amount of DO in the mixture of river & waste shall be
\( Q_0 = 2.00 \text{ m}^3/\text{min} \)
\( Q_s = 1.10 \text{ m}^3/\text{min} \)
\( DO_R = 0.3 \text{ mg/l} \)
\( Q_R = 8.7 \text{ m}^3/\text{min} \)
\( \text{temp} = 20^\circ \text{C} \)

\[ D.O_{\text{min}} = \frac{Q_R(DO)_R + Q_s(DO)_s}{Q_R + Q_s} \]

\[ = \frac{8.7 \times 8.3 + 1.10 \times 2}{8.7 + 1.10} \]

\[ = 7.59 \text{ mg/l} \]

**Ques.** If one of sludge is reduced from 98 to 96%, then the vol of sludge decreases by

\[ V_1(100 - P_1) = V_2(100 - P_2) \]

\[ V_1(100 - 98) = V_2(100 - 96) \]

\[ V_1 \times 2 = V_2 \times 4 \]

\[ \frac{V_1}{V_2} = 2 \]

\[ V_1 = 2V_2 \quad (50^\circ \text{C}, 0 = V_1) \]

**Ques.** Effluent from waste water treatment plant (flow rate 864.0 m\(^3\)/d, temp = 25\(^\circ\)C) is discharged to surface stream (flow rate 12 m\(^3\)/s, temp 15\(^\circ\)C). What is the temp of stream after mixing?
A 125 m$^3$/s sewage of a city is discharged in a perennial river (continuous) which is fully saturated with O$_2$ and flows at a rate of 1600 m$^3$/s with a velocity of 0.12 m/s. If BOD of sewage is growing, find out when critical DO will occur in the river. Assume $k_D = 0.44$ /day, $k_F = 0.11$ /day. Ultimate BOD is 125% of BOD mixture of waste & river water. Take saturation DO level of river is 9.2 mg/L.

\[
1 = \frac{k_F}{k_D} = \frac{0.44}{0.11} = 4
\]

\[
\frac{1}{k_D(f-1)} \log \left[ 1 - (f-1) \cdot \text{DO} \right] = f
\]

\[
t_c = \frac{0.11(4-1)}{0.11(4-1)} \log \left[ 1 - 3 \times \text{DO} \right]
\]
\[ D_{\text{min}} = Q_R(00)R + Q_s(00)s \]
\[ s_R + q_s \]
\[ = 1600 \cdot 9.2 + 125 \cdot 0 \]
\[ 1600 + 125 \]
\[ = 8.333 \]
\[ \text{(initial DO deficit)} \]
\[ D_0 = \text{saturated DO} - \text{DO\textsubscript{min}} \]
\[ = 9.2 - 0.53 \]
\[ = 0.66 \]
\[ \text{from eqn (1)} \]
\[ t_e = \frac{1}{0.11 \times 3} \log \left[ 1 - 0.8 \times 0.66 \right] \times 27.1739 \]
\[ t_e = 1.725 \text{ days} \]
\[ D_e = k_R \left[ 0.11 \right] - k_0 t_e \]
\[ k_R \]
\[ = 0.11 \times 27.1739 \left[ 10 \right] - 0.11 \times 1.725 \]
\[ = 0.44 \]
\[ D_e = 438.876 \]
\[ \text{Lengths of flow into river} = \text{velocity} \times \text{flow} \times t_e \]
\[ = 0.12 \times 1.72 \times 24 \times 60 \times 61 \]
\[ = 17832.96 \text{ m} \]
\[ = 17.83 \text{ km} \]
A River before its entry into the town had a discharge of 100 ft/sec & 20 mg/l. It was the concentration of a conservative parameter (BOD). The town's waste water outfall, for having 200 mg/l, concentration of the same conservative parameter raised the concentration of river to 50 mg/l after a complete mix with the river water. Determine the dilution ratio resulting from the discharge of waste water outfall.
Ecology

Atmosphere

- Troposphere
- Stratosphere
- Mesosphere
- Thermosphere

Ozone Layer

11 km

Air mass

Cloud formation

Sea-gas

Lakes, rivers, ponds

Subsurface water bodies

Aquifer

Aquitard

Hydrosphere

Subsurface water bodies

Surface water bodies

Eco System

- Dissolved oxygen
- pH, N, sunlight, non-living things
- Temperature, salts, moisture content

Biotic factors

(living things)

Abiotic factors

(living things)

Produce plants

Consumers, Animals

Decomposer: Bacteria, viruses, fungi,

Protozoa
Food chain in an ecosystem transfer the energy from one level to another level.

- Grazing food chain → Normal food chain
  - Detritus food chain
    - Stals from the decomposition process
    - Bacteria
Food chain: Eaters eaten by the another one

producer → consumer → Decomposer

Transfer of energy from one level to another level is known as Trophic level. Trophic energy along the food chain, least in the energy transfer to the last consumers.

Rules of thermodynamics; law of conservation of energy.

Eco System

- Natural ecosystem
  - Forest ecosystem
  - Ocean/Sea ecosystem
  - Desert ecosystem

- Artificial ecosystem
  - Limnic ecosystem
    - Lentic ecosystem (stagnated water, e.g., lake)
    - Lotic ecosystem (running water, e.g., river)

- Industrial (Terrestrial)
- Marine Ecosystem
- Aquarium Ecosystem
- Agricultural Ecosystem
- Anthropogenic Ecosystem
Ecology is the study of the ecosystem in relation to its surrounding environment.

The ecological system can be classified into two types:

1. Natural ecosystem
   - E.g.: forests, deserts, oceans

2. Artificial ecosystem
   - E.g.: industrial, agricultural, limnic, aquarium

Every ecosystem consists of abiotic factors (Non-living things) and biotic factors (living things) and a mutual inter-relationship between them. Each ecosystem consists of its own set by maintaining systematic balance.
Air Pollution: It is defined as release of excessive foreign matter into the air beyond its permissible level, which becomes toxic in nature and causes damage to the human beings, animals and property.

Pollution
↓
Pollutants
↓
toxic element beyond its permissible level

In the ecosystem
↓
Sources for the pollutants - which cause pollution

Natural pollution

1. pollen grains - Anther flowers
2. forest fires
3. volcanic eruption

Man made sources

1. Thermal power plants
2. Nuclear Reactors
3. Agriculture source
4. Automobiles

Sources:

Primary Air Pollutants - (i) SO₂, dust, NOx gases, carbon-related (CO₂, CO), hydrocarbons, hydrogen sulphide, hydrocarbon "smoke"
Secondary Pollutants:

Because of the reaction b/t the two primary pollutants.

Air Pollution is defined as the excessive concentration of foreign matter in the air which adversely affects the well-being of an individual or causes damage to property and affects plants, animals, buildings.

Sources of Air Pollution:

Air pollution sources can be classified as:
1. Natural Sources
2. Manmade Sources

- Natural Sources:
  - Products from the atm: reactions (chemical, heat, air oxidation, photochemical)
  - Polymers

- Aerosols:
  - Aerosols are finely divided solids and liquid particles of microscopic soils held suspended or dispersed in atm.
  - Eg: Dust, smoke, fog, mist, fume (generated from steel plant)

- Micro-organisms: which infect plants & animals
4) Pollen grains: These are the small grains from the anther flowers which cause allergic reactions.

5) Radioactive minerals: The radiation emanated from the reaction power plants causes damaging effects and gases.

6) Volcanic ash, gases, and odours from the marshy lands.

7) Man-made sources:

1) Combustion of fuel: \( \text{CO}_2, \text{SO}_2, \text{NO}_2 \),

2) Industrial emission such as \( \text{SO}_2, \text{CO}_2, \text{NO}_2 \) and \( \text{CO}, \text{NH}_3 \)

3) Thermal power plants: These mainly emit \( \text{SO}_2 \) gases and exhaust.

4) Automobiles: The exhaust from the automobiles contains CO, \( \text{CH}_4 \), and burnt carbon.

5) Agricultural activities: Activity related to agriculture using field burning, crops spraying.

6) The nuclear power plants emit toxic radioactive substances.
Primary Pollutants

1) Pollens
2) Sulphur Compounds \((\text{SO}_2, \text{CO}, \text{H}_2\text{S})\)
3) Nitrogen Compounds \((\text{NO}, \text{NO}_2, \text{NO}_3, \text{NH}_3)\)
4) Carbon Compounds \((\text{CO}, \text{CO}_2)\)
5) Lead
6) Radio-active Substances
7) Halogen Compound
8) Hydrocarbons

Secondary Pollutants

These are the pollutants which are formed in the air as a result of interaction of two or more primary air pollutants or by the reaction between primary pollutant and certain constituents by photochemical phenomenon.

1) \(\text{H}_2\text{SO}_4\)
2) \(\text{O}_3\)
3) PAN
4) Formaldehydes
5) Smog
6) Photochemical smog

\(\text{H}_2\text{SO}_4\) is formed by the simple chemical reaction of \(\text{SO}_2\) and water vapour. It causes acid rain.
Ozone, PAN, formaldehyde are formed by the photochemical reactions caused by the sunlight by the two primary air pollutants.

1. Industries related to the gases:
   a) SO$_2$ from the thermal power plant.
   b) CO$_2$ from automobiles → Carbon monoxide hemoglobin
   c) Lead is formed and acts as aphyxiant
   d) Leads to the cardiovascular diseases.
   e) HCl from the automobiles exhausts
   f) Lead → It is released into the rain by the automobiles running on petrol.
   g) NO$_x$ gases → This are released from the vehicular oxidation, electrical discharges, solar radiation, welding operations in the industries.

Effects of Air Pollution:

1. Air pollution causes Bronchiolitis (Asthma)
2. The high concentration of SO$_2$, NO$_2$, photochemical smog, pollengrains cause asthma.
3. Lead discharge from the automobiles cause lungs infection.
4. Hydrogen chloride released from the fluorine industries cause the motting of teeth (discoloration of color) on teeth.
5) To cause death due to asphyxiation.
6) Radioactive components cause cancer and leukemia.
   The carcinogenic agents lessen the age of genetic effects.
7) The finer gases cause eye irritation, nose infection, and respiratory symptoms (breathing problems).
8) Silicosis & Asbestosis are the diseases caused to the person who work in cement industry, coal mines & thermal power plants.

Effects on Plants:

The air pollutants can cause:

1. Chlorosis (discolourisation of plants i.e. yellow discoloration)
2. Necrosis (air pollutant excessives levels can lead to the tissue killing of plants)
3. Abscission (unnecessary falling of leaves due to the excessive air pollutant concentration is called as Abscission)
4. Epinasty: Die wrinkling of leaf due to the air pollution is known as Epinasty.

Global effects of Air pollution:

1) Acid rain & Global Warming
2) Ozone layer depletion
3) Greenhouse effect
Acid Rain - The rain water is slightly acidic, the acidity level increases with the SO₂ & NO₂. Mining of the secondary pollutants in air can lead to H₂SO₄, which causes acid rain.

Major part of the acid rain is due to the SO₂, which is produced mainly by the burning of coal & oil in the industry.

The pH of the acid rain is below 5 to 5.5. Acid rain causes damaged to the forest, crops, buildings & monuments (by the leaching action). Increased levels of acid rain cause dissolved oxygen to the flora & fauna and skin cancer & change in ecological system.

Global Warming - It is the outcome of the air pollution caused by the man-made activities. Global warming may lead to the forest fires and burning of the crops.

The increased level of temp in the atm. zone due to the release of various air pollutants & vehicular emissions trap the sunlight in the lower levels & prevent the reflected sunlight to go up into the higher zone. The trapping of sunlight leads to the increased temp at the global
Due to this phenomenon, it is known as Global Warming. Global Warming is due to the increased industrialization & emission of chlorofluorocarbons (CFCs), CCL4, HC, chloroform (CHCl3), isopropene, refrigerants, CO2.

Global Warming is affecting changes in sea levels.

2. Increased levels of seas are causing imbalance in the ecological system.

3. Global warming affecting the growth rate of age.

Ozone Layer Depletion

Ozone is present in stratosphere & it is called as stratospheric ozone. The unit of the ozone measurements are ‘DAASON’, due to the rate of global warming, ozone layer depletion is.

Ozone taking place is harmful ultraviolet rays are reaching the earth's surface and causing ill damage to the ecosystem.

When chemical pollutants such as oils & surfactants, often industrial emission in the excessive concentration levels, some of them break the ozone layer a
The concentration of ozone layer is gets reduced in the extra effects of ozone layer depletion:

1. Damage to the immune system
2. Disturbance in ecosystem
3. Increased skin cancer
4. Shortening of life of plants & lower yield of crop

Control Methods:

1. Electrostatic Precipitators (ESP)
2. Wet scrubber
3. Bag houses filters

ESP: Thermal Power plants (Fly ash)

The treatment of air pollution depends upon the concentration of air pollution.
Conversion of Suspended particulate matter (SPM)

From ppm to deg/m³

\[
\text{deg/m}^3 \left( \frac{mg}{m} \right) = \left(\frac{\text{PPM of SPM}}{\text{ppm}}\right) \times \text{gram} \times \text{mole mass of gas} \times 10^3 
\text{vol. of gas (lit/mole)}
\]

Any gas, at 1 atm pressure or 760mm

In 0°C i.e. 273°K, occupies 22.4lit/mole of vol.

Avagadro’s law

\[
P_1V_1 = P_2V_2 \quad \frac{T_1}{T_2}
\]

The 1 mole of any 1 gas occupied the same volume as 1 mole of any other gas at the same temp and pressure

At 273°K and 1 atm press. the vol. of gases is 22.4lit/mole.

To convert the vol. gas at other temp. & pressure the above avagadro’s law is used

Question: A sample of air analysed at 0°C & 1 atm press. is reported to contain 9 ppm of CO. Determine the equivalent CO concentration in micrograms/m³.
\[ CO = 12 + 16 = 28 \]

\[ \frac{mg}{l} \text{ of } CO = \frac{9 \times 28 \times 10^3}{22.4} \approx 112.5 \text{ o} \]

Thus, the mean indoor airborne chloroform concentration in a room was determined to be 0.24 ppb/m³ using the following data:

- \( T = 293^\circ \text{K} \), \( P = 1 \text{ atm} \), Gas Const. \( R = 8.314 \times 10^5 \text{J/m}^3 \text{K} \)
- Atomic wt. of C = 12, H = 1, Cl = 35.5

This concentration is expressed in parts per billion (ppb basis) or equal to

\[ \text{ppb} \times \frac{119.5 \times 10^3}{V} \]

- 0.4 = \( \frac{\text{ppb} \times 119.5 \times 10^3}{V} \)
- 0.4 = \( \frac{\text{ppb} \times 119.5 \times 10^3}{24.04} \)

\[ p = 0.08 \text{ ppb} \]

\[ T_1 = 293 \]
\[ T_2 = 273 \]
\[ V_1 = 24.04 \text{ l} \]
\[ V_2 = 24.04 \text{ l} \]

**Ans.** Particulate matter (flyash) removed by flue gases from the furnaces burning fossil fuels are better removed by:

a) Bag house filter
b) Wet scrubber
c) ESP

d) Minimum depth of air is zero
Two primary air pollutants are:

a) \( \text{SO}_2 + \text{O}_3 \)

b) \( \text{NO}_2 + \text{PAN} \) (P triggering acetyl hydride Nipate)

c) \( \text{SO}_2 + \text{HC} \)

d) \( \text{O}_3 + \text{PAN} \)

Meteorological conditions:

Air pollution changes:

1. Wind velocity (Anemometer)
2. Wind direction (Wind rose diagram)
3. Lapse Rate
4. Plume behaviour
5. Stack height

Lapse Rate:

a) Decrease of temp. with increase of altitude (ELR)

b) Adiabatic lapse rate (ALR): The temp. of a particular air parcel or air pocket decreases with the increase of altitude.

\[ \text{ELR} < \text{ALR} \quad \Rightarrow \quad \text{sub adiabatic lapse rate} \]

\[ \text{ELR} > \text{ALR} \quad \Rightarrow \quad \text{super adiabatic} \]

\[ \text{ELR} = \text{ALR} \quad \Rightarrow \quad \text{neutral state} \]

Inversion: Increase of temp. with increase of altitude i.e. negative lapse rate.

Dry air cools at rate of \( 1.0^\circ \text{C}/\text{km} \) → dry ALR

In saturated (wet air) state, it is calculated to be \( 6^\circ \text{C}/\text{km} \) → wet ALR.
Solid Waste Management

Solid waste is defined as the dry form of refuse.

Density of solid waste = 300 - 600 kg/m³

Effect of Meteorological condition on air pollution:

Wind velocity & wind direction: The higher the wind speed, the more rapidly the pollutants would be carried away from the source, and concentration of pollutants will get decrease.

Lapse Rate: Rate of change of temp of air with altitude as known as lapse rate.

Environmental lapse rate (ELR): In the lower atmosphere (known as troposphere) up to a height of about 11 km above the Earth surface, the temp decreases linearly with increase in the height.

In the upper region of atmosphere which extends from about 11 km to 30 km, const temp prevails.

Adiabatic Lapse Rate (ALR): The internal decrease of temp. with height which occurs in the rising parcel of air mass without exchange of heat from the surrounding is known as ALR.
Sub radiabatic lapse rate:

When $EIR < AIR$; the rising air parcel will be cooling more quickly than its surroundings and it will not be able to rise up to the greater altitude. Such an atm condition is said to be stable and however not favourable for effective dispersion of pollutants. It results more pollution.

Super radiabatic lapse rate $-$ ($EIR > AIR$);

The rising air parcel will always remain warmer and lighter than the surrounding environment. The air parcel will continue to accelerate & go up. In such a case the atm condition is said to be unstable its dispersion of pollutants is rapid and effective which leads to the less pollution.

Neutral State $-$ $EIR = AIR$; the atm condition is said to be neutral.

Negative lapse rate $-$ when the temp of air increase with increase in the altitude, then the lapse rate is known as negative lapse rate or inversion.
Solid Waste Management

Solid waste in dry form of refuse density - 300 - 600 kg/m³

Municipal solid waste - [used cans, cardboard pieces, papers, biodegradable waste, Rubbish, Garbage]

Solid waste

Causes: 1) Domestic 2) Industrial sources

Disposal of solid waste -

1) Sanitary land filling
2) Incineration or pyrolysis
3) Pulverization
4) Barging into the sea
5) Composting

Indoor Process → Anaerobic treatment
Anerobic process → Anaerobic treatment

1) Energy produce kJ/kg (dry basis) = \( \frac{\text{Energy in kJ/kg (of described)}}{100 - \% \text{mc}} \) 

2) Energy produced in kJ/kg (dry basis) with ashfree = 

\[ \text{energy in kJ/kg (as described)} \times \frac{100}{100 - \% \text{ash in mc}} \]
of the chemical formula of solid waste is given.

\[
= \text{modified Dulong formula} \\
= 33.7(C) + 14.28\left(\frac{H - 0}{8}\right) + 95(S)
\]

when 
- \(C = \%\) of carbon
- \(O = \%\) of oxygen
- \(H = \%\) of hydrogen
- \(S = \%\) of sulphur

Solid waste can be disposed by the following method:

1. By Sanitary Landfilling Method - Refuse is conveyed and dumped into the low-lying area discovered by earth material, so that refuse is not directly exposed.

   The waste is stabilized by aerobic & anaerobic process by the bacteria. The refuse is kept for a period of 2-4 months. Lechati - During the rainy season when excess water seeping through the area may contain the solid waste as a coloured liquid called as lechatu. The lechatu is highly toxic in nature & it pollutes the GWT.

Production of Gases - In the most cases, the decomposition of solid waste releases \(\text{CH}_4\) & \(\text{CO}_2\).
The gases can be collected by the gas collector and they can be used as fuel.

2. Incineration: Burning of refuse at high temp in furnace (incinerator) is known as incineration. Upon heating within the O2-free atmosphere, most of the organic substances can be split into the gases and solid fractions and this process of burning is known as pyrolysis.

3. Grading:

4. Pulverization: Refuse is pulverized in the grinding machine so that reduce the vol & change its physical character. After this, it is disposed in the trenches anaerobically or aerobically.

5. Burning into the sea: The solid waste is directly dumped into the sea & disposed of, but care should be taken to prevent the formation of sludge banks.

6. Composting: This decomposition is effected either under aerobic condition or under anaerobic condition or both.
The final end product is known as manure (compost) and it can be used as fertilizer in agricultural fields. In Indian wording, there are three methods available for the disposal of refuse & night soil.

1. Indore Process: It uses the mixture of refuse & night soil for the decomposition under anaerobic condition.

Bangalore Process: It is primarily anaerobic in nature & does not involve directly handling of mass) as it is in Indore method. This method is widely accepted by the municipal authority & in this method refuse & night soil are placed in the layer in an underground earthen trench.

This mass is covered by 15 cm of layer of earth & it is kept for the decomposition. After 4-5 months, the refuse gets stabilized, and changes into the brown colour, odourless manure called as Humus.

Thus, the following composition of solid waste is given below and the data given as:

<table>
<thead>
<tr>
<th>Component</th>
<th>% by mass</th>
<th>Energy kJ/kg</th>
</tr>
</thead>
<tbody>
<tr>
<td>Food waste</td>
<td>15</td>
<td>4600</td>
</tr>
<tr>
<td>Manure</td>
<td>85</td>
<td>16750</td>
</tr>
</tbody>
</table>
a) Compute the energy content of solid waste given in the above table form.

b) Determine the energy content on dry basis if the moisture of waste is 20%.

c) Determine the energy content on an ash-free basis assuming the ash content was 5%.

a) Total energy in KJ/kg:

\[
\begin{align*}
4600 \times 0.15 + 16750 \times 0.45 + & \\
16300 \times 0.10 + 32600 \times 0.10 + & \\
6500 \times 0.10 + 18600 \times 0.05 + & \\
700 \times 0.05 + & \\
0.15 + 0.45 + 0.10 + 0.10 + & \\
0.10 + 0.05 + 0.05 & \\
\end{align*}
\]

\[= 14732.5 \text{ KJ/kg}.\]

b) Energy content on dry basis:

\[14732.5 \times \frac{100}{100 - 20} = 18648.73 \text{ KJ/kg}.\]

c) Energy content on an ash-free basis:

\[14732.5 \times \frac{100}{100 - 21.3} = 19908.71 \text{ KJ/kg}.\]
A composition of a certain municipal solid waste sample and sport of its various components are given below. Calculate the sp. wt. of the municipal solid waste in kg/m³.

<table>
<thead>
<tr>
<th>Component</th>
<th>% by weight</th>
<th>Vol (m³)</th>
<th>sp. weight (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0: Food waste</td>
<td>23%</td>
<td>0.166</td>
<td>300</td>
</tr>
<tr>
<td>1: Dirt &amp; Ash</td>
<td>30%</td>
<td>0.06</td>
<td>500</td>
</tr>
<tr>
<td>2: Plastics</td>
<td>10%</td>
<td>0.1538</td>
<td>65</td>
</tr>
<tr>
<td>3: Wood &amp; Yard waste</td>
<td>10%</td>
<td>0.08</td>
<td>125</td>
</tr>
</tbody>
</table>

\[ \text{Total sp. wt} = \frac{\text{total wt}}{\text{total vol}} = \frac{100}{0.46} = 217.39 \text{ kg/m³} \]

Thus: Two biodegradable components of municipal solid waste are:

- Plastic & wood
- Cardboard & Glass
- Leather & tin tanks
- Food waste & Garden trimming.

50 gms of CO₂ & 85 gms of CH₄ are produced from the decomposition of municipal solid waste with a formula wt (molecular wt) of 120 gms.

What is the ratio per capita per year for the gas produced in a city of 1 million people with a municipal solid waste production rate of 500 tonnes/day?
City population = 1 million

Solid substance (MSW) = 120 gm

120 gm burnt \(\rightarrow\) 80 gm of \(\text{CO}_2\) + 25 gm of \(\text{CH}_4\)

120 gm MSW \(\rightarrow\) 50 gm of \(\text{CO}_2\)

500 tonnes \(\times\) \(\frac{50 \times 500 \times 10^3}{120} = \frac{208.3 \times 10^3}{10}\) kg

\(\frac{208.3 \times 10^3}{10}\) per million

30.3 \(\times\) \(\frac{10^3}{10^6}\)

120 gm MSW \(\rightarrow\) 45 gm of \(\text{CH}_4\)

500 tonnes \(\times\) \(\frac{25 \times 500 \times 10^3}{120} = \frac{104.167 \times 10^3}{10}\) kg

\(\frac{104.167 \times 10^3}{10}\) = 104.167 of \(\text{CH}_4\)

Total greenhouse gases = 208.3 + 104.167

= 312.5 gm per capita day

A. Estimate the theoretical vol. of \(\text{CH}_4\) gas that would be expected from anaerobic digestion of a tonne of waste having composition C_50 H_100 O_10 N. The chemical expression...
involved as follows:

\[ \text{C}_a \text{H}_b \text{O}_c \text{Nd} + \left( 4a - 2c - 3d \right) \text{H}_2\text{O} \rightarrow \frac{(4a + b - 2c - 3d)}{4} \text{C}_a \text{H}_b \text{O}_c \text{Nd} + \left( 4a - b + 2c + 3d \right) \text{CO}_2 + \text{dH}_2\text{O} \]

\[ \text{Vol} = \frac{380.5}{0.7167} = 417.23 \text{ m}^3 \]
The ultimate analysis of municipal solid waste is given below:

<table>
<thead>
<tr>
<th>Component</th>
<th>Mass (kg)</th>
<th>% by mass</th>
</tr>
</thead>
<tbody>
<tr>
<td>Carbon</td>
<td>34.57</td>
<td>36.3</td>
</tr>
<tr>
<td>Hydrogen</td>
<td>6.90</td>
<td>7.3</td>
</tr>
<tr>
<td>Oxygen</td>
<td>48.58</td>
<td>51.10</td>
</tr>
<tr>
<td>Nitrogen</td>
<td>0.43</td>
<td>0.50</td>
</tr>
<tr>
<td>Sulphur</td>
<td>0.13</td>
<td>0.10</td>
</tr>
<tr>
<td>Ash</td>
<td>4.47</td>
<td>4.7</td>
</tr>
<tr>
<td>Total</td>
<td>95.02</td>
<td>100</td>
</tr>
</tbody>
</table>

Estimate the energy content of waste by using modified DuLong formula:

DuLong formula: \[ 337(C) + 1428(H - \frac{C}{8}) + 95(S) \]

\[ \begin{align*}
&= 337 \times 36.3 + 1428 \left(7.3 - \frac{36.3}{8}\right) \\
&\quad + 95 \times 0.10 \\
&= 1354 - 5.65 \text{ kJ/kg}
\end{align*} \]

Estimate the mc of solid waste sample with the following composition:

<table>
<thead>
<tr>
<th>Component</th>
<th>% by mass</th>
<th>m.e.g.</th>
<th>wt of component</th>
<th>mc</th>
</tr>
</thead>
<tbody>
<tr>
<td>Foodwaste</td>
<td>60</td>
<td>70</td>
<td>0.30 kg</td>
<td>0.20x70/100 = 0.1</td>
</tr>
<tr>
<td>Paper</td>
<td>40</td>
<td>6</td>
<td>0.40 kg</td>
<td>0.0240</td>
</tr>
<tr>
<td>Cardboard</td>
<td>10</td>
<td>5</td>
<td>0.10 kg</td>
<td>0.0050</td>
</tr>
<tr>
<td>Plastics</td>
<td>5</td>
<td>2</td>
<td>0.05 kg</td>
<td>0.0010</td>
</tr>
<tr>
<td>Garden trimmings</td>
<td>5</td>
<td>60</td>
<td>0.05 kg</td>
<td>0.0050</td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td></td>
<td></td>
<td>0.0850</td>
</tr>
</tbody>
</table>
Wood | 5 | 20 | 0.05 W | 0.01 W
Tin can | 5 | 3 | 0.05 W | 0.0015 W

90%

Let total wet of MShW = W kg.

Moisture content = total wt of moisture

moisture content = total weight

0.9 kg of 0.1 lb by mass

0.215 W + 0.11 W

238

= 31.15%
Noise Pollution

Noise is defined as unwanted sound. The sound changes due to the change in the pressure levels due to the environmental changes.

Sound is periodic, regular and long duration of time, pleasant sound.

Sound is aperiodic, irregular and short duration noise.

Any sound that just audible is having a pressure level called at "Threshold level" = 20 mPa = 0.00002 N/m².

Sound in the form of noise is measured in Decibel.

The range of sound = 0.00002 N/m² to 200 N/m².

The range of sound = 80 db

The range of sound = 120 db

\{ Turbine 140 db \} & Jet
Measurement of sound w.r.t to S.P.L

\[ SPL = 20 \log_{10} \left( \frac{p}{p_{ref}} \right) \]

where \( p \) = measured sound w.r.t to the threshold level (20 \text{微皮安})

If two sources are there:

\[ \text{Leq} = 10 \log_{10} \left( \frac{\sum_{i=1}^{n} \left( L_i / 10 \right) \times t_i}{\sum_{i=1}^{n} t_i} \right) \]

where \( \text{Leq} \) = the equivalent sound pressure level in decibel

\( n \) = total no of sound pressure level recorded

\( L_i \) = values of the sound pressure level recorded in db with \( i = 1, 2, 3, \ldots \)

\( t_i \) = time duration of the diff. sound pressure levels recorded, expressed as the fraction of total recording time

\[ \begin{align*}
80 \text{ db} & : 5 \text{ min} \quad L_1 = 80 \text{ db}, \quad t_1 = \frac{5}{100} \\
90 \text{ db} & : 15 \text{ min} \quad L_2 = 90 \text{ db}, \quad t_2 = \frac{15}{100}
\end{align*} \]

\[ \text{Leq} = 10 \log_{10} \left( \frac{10^{80/10} \times 5/100 + 10^{90/10} \times 15/100}{100} \right) \]

\[ \text{Leq} = \text{81.90 db} \]
Causes of Sound Pollution

1. Domestic Sources
2. Vehicular Sources
3. Construction Activities
4. Machineries
5. Election campaign & loudspeakers | Public fun

Effects of Noise Pollution:

- Increased levels of blood pressure
- Decreased efficiency due to fatigue
- Nervous weakness leading to cardiac attacks
- Acoustic trauma
- Temporary deafness
- Irritation & headache

Remedial Measurements:

1. By Regularly acts & laws
2. ear plugs
3. using methods of Isolation
4. using of Insulating materials
5. minimising the traffic

Sound can be classified as three types

1. Continuous noise: It is an undisturbed sound level that varies less than 5 dB. 
   e.g. Running fan
2. **Intermittent Noise** - It is a noise which is continuous for more than 1sec, & it is interrupted intermittently. 

Eg: a drilling machine.

2. **Impact Noise** - It is characterised by change of sound pr. of atleast 40db within the duration of less than 1sec.

Eg: Turbines & hammers.

**Ans.**

The reference pr. used in the determinatio of sound pr. is:

a) 20 lpa  b) 20 db  c) 10 lpa  d) 10 db

2. According to the Noise pollution regulation by central rules of 2000 of the Ministry of environmental roadside forest of India, the day time & night time limits in ambient air for residential area expressed in db.

- Night - 45 db  
- Day - 55 db

Q: A 60 db of noise is measured with the 20 lpa noise, and another 60 db is measured with the 10 lpa reference level, then the total noise level in the db will be equal to 

**Ans.**
Note: When two sources are given for a sound, the equivalent pr of the noise can be calculated by the sine wave curve by means of root mean square pressure.

\[ P_{\text{rms}} = \sqrt{P_1^2 + P_2^2} \]

\[ SPL = 20 \log_{10} \left( \frac{P_1}{P_{\text{ref}}} \right) \]

\[ 3 \log P = 20 \log_{10} \left( \frac{P_1}{P_0} \right) \]

\[ P_1 = 80 \times 10^3 \text{ lpa}, \quad P_2 = \text{?} \]

\[ P_{\text{rms}} = \sqrt{P_1^2 + P_2^2} \]

\[ = \sqrt{2 \times (80 \times 10^3)^2} \]

\[ = 8.8284 \text{ lpa} \]

\[ SPL = 20 \log_{10} \frac{P_{\text{rms}}}{P_{\text{ref}}} \]

\[ = 20 \log_{10} \left( \frac{2.82 \times 10^4}{20 \text{ lpa}} \right) \]

\[ SPL = 63.01 \text{ dB} \]
A noise level of 80 dB is lasting for 10 min and followed by 60 dB for 80 min & 100 dB for 5 min one after the other. What is the equivalent sound pressure level if the recording time is 95 min?

\[ SPL = 10 \log \left( \sum_{i=1}^{n} \frac{10^{\frac{L_i}{10}} \times t_i}{\sum_{i=1}^{n} t_i} \right) \]

\[ = 10 \log \left[ \frac{10^{80/10} \times 10}{95} + \frac{10^{60/10} \times 80}{95} + \frac{10^{100/10} \times 5}{95} \right] \]

\[ = 27.3 \text{ dB} \]

**Thumb Rule:** If the two pr. intensities \( x \) & \( y \) decibals, and if the \( \Delta \) difference in pressure level is more than one equal to 16 dB, then the total noise level will be numerically equal to the magnitude of higher value.

If the two noise levels are \( x \) & \( y \) dB then the total noise level will be equal to \( (x + 3) \) dB.
While recording the weighted sound level readings have been taken at a sight site at different types of idle noise, these readings are 20, 56, 66, 42 dB. With reference to the 20 libra, then the any sound level expressed in the dB each.

\[ SPL = 20 \log_{10} \left( \frac{P}{P_{ref}} \right) \]

\[ 20 = 20 \log_{10} \left( \frac{P_1}{20} \right) \]

\[ P_1 = 2 \times 10^3 \text{ kPa} \]

\[ 56 = 20 \log_{10} \left( \frac{P_2}{20} \right) \]

\[ P_2 = 12619.14 \text{ kPa} \]

\[ 66 = 20 \log_{10} \left( \frac{P_3}{20} \right) \]

\[ P_3 = 39905.24 \]

\[ 42 = 20 \log_{10} \left( \frac{P_4}{20} \right) \]

\[ P_4 = 2517.8 \]

\[ P_{avg} = \frac{P_1 + P_2 + P_3 + P_4}{4} = 13310.45 \]

\[ SPL = 20 \log_{10} \left( \frac{13310.45}{20} \right) \]

\[ SPL = 56.78 \text{ dB} \]
Find out the vol. of anaerobic digesters for 5 MLD of domestic waste water treatment plant having 60% of suspended solids removed by the primary clarifier (PST) and 250 mg/l suspended solids were present in the waste water. The size of the influent sewage sludge (from PST) is 96%. Initial volatile solid content is 70%, volatile solids destroyed is 65%. Digested sludge solid content is 8%, sp. gr of the primary sludge is 1.05, sp. gr. of the digested sludge 1.04. Density of water is 1000 kg/m^3. The digestion period is 15 days.

\[ \text{Volatile solid content} = 70\% \]

\[ \text{Vol. of sewage} = 5 \text{ MLD} \]

\[ \text{Density of sewage} = 5 \times 10^6 \text{ kg/day} \]

\[ \text{Suspended solids in sewage} = 250 \text{ kg/d} \times 5 \times 10^6 = 1250 \text{ kg/d} \]

\[ 60\% \text{ of suspended solids removed by PST} \]

\[ = 0.6 \times 1250 \]

\[ = 750 \text{ kg/d} \]

\[ \text{Vol. of influent sewage sludge} = 96\% \]

\[ \text{1 kg of solid matter} = 100 \text{ kg of sludge} \]

\[ 750 \]

\[ 150 \times 750 \]

\[ = 18750 \text{ kg/day} \]
sp. gr. of sludge = \( P_{\text{sludge}} \)

\[ = 2.47 \]

density of sludge \( P_{\text{sludge}} = 1.03 \times 10^3 \text{kg/m}^3 \)

\[ = 1.03 \times 10^3 \text{kg/m}^3 \]

Vol. of sludge = \( \frac{\text{wt. of sludge}}{\text{density of sludge}} \)

\[ V_1 = \frac{18750}{1030} = 18.20 \text{ m}^3 \]

for calculation of \( V_2 \)

Total solids = volatile solids + non-volatile solid

\[ = \frac{70 \times 7.50 \times 0.35 + 30 \times 750}{100} \]

\[ = 408.75 \text{ kg/day} \]

Digested sludge having 8\% of solids

8\% of solids makes 100\% of sludge

\[ \frac{408.75 \text{ kg}}{8} = \frac{100 \text{ kg}}{408.75} \]

\[ = 5109.375 \text{ kg/day} \]

Density of the digested sludge = sp. gr. \times \rho_0

\[ = 1.04 \times 10^3 \]

\[ = 1.040 \text{ kg/m}^3 \]

Vol. of digested sludge = \( \frac{5109.375}{1040} \)

\[ = 4.9 \text{ m}^3 \]
\[ V_2 = 4.91 \text{ m}^3 \]

\[ V = \left( V_1 - \frac{3(V - V_2)}{3} \right) t_1 \]

\[ = \left( 18.26 - \frac{3(18.26 - 4.91)}{3} \right) \times 15 \]

\[ V = 1.28 \text{ m}^3 \text{ m}^3 = 140.1 \text{ m}^3 \text{ at } 15^\circ \]

A sewage containing 200 mg/l suspended solids is passed through primary sedimentation tanks. The solids from the PST were digested to recover the gases. Find the vol of the CH\(_4\) & CO\(_2\) produced in the digestion of the sludge from 10000 m\(^3\) of sewage. State clearly the assumptions made.

Total vol of the sewage = 10000 m\(^3\)

Total suspended solids = 200 mg/l.

Total suspended solids = \(10000 \times 10^3 \times 200 \times 10^{-6} = 2000 \text{ kg}\)

a) By assuming 60\% of the suspended solids are removed in the PST = \(\frac{60 \times 2000}{100} = 1200 \text{ kg}\)

b) By assuming 50\% of the removed suspended solids (1200 kg) = volatile solids as 65\%.

\[ \frac{65 \times 1200}{100} = 780 \text{ kg (volatile)} \]

0.9 m\(^3\)/kg of CH\(_4\) released.

1 kg = 0.9

\[ 780 \times 0.9 = 702 \text{ m}^3 \text{ remaining in tank} \]
A Population of town is 30,000, domestic sewage produced is 120lt/capita/day having BOD of 800 mg/ltr. Industrial sewage produced is 32105 lt/day having BOD of 800 mg/ltr. Design a high rate single stage trickling filter with the following data:

1) PSF removes 35% of BOD
2) Organic loading is 10000 kg per hectare/m per day
3) Circulation ratio = 1
4) Hydraulic loading is 170 ML/day
5) Find the efficiency of T.F. & the BOD of effluent

79.5, BOD = 40.8