WATER & WASTEWATER ENGINEERING(18CV62)

MODULE 1

Module-1

Water Demand & Conveyance

Introduction: Need for protected water supply. Demand of Water, Factors affecting per capita demand, Peak factor, Design period and factors governing design period. Different methods of population forecasting -with merits and demerits. Numerical Problems.

Sources: Surface and subsurface sources suitability with regard to quality and quantity, Collection and Conveyance of water: Intake structures - types of intakes –Factors to be considered in selection of intake structures. Pumps: Types of pumps. Numerical Problems. Pipes: Design of the economical diameter for the rising main; Numerical Problems. BIS' Drinking water quality standard.

Module-2

Water Treatment & Distribution:

Objectives, Treatment flow chart – significance of each unit, Aeration- types, Sedimentation- settling tanks types, design. Coagulation -types of coagulants, Filtration- theory, types of filters, slow sand, rapid sand and pressure filters including construction, operation, cleaning. Operational problems in filters. Design of slow and rapid sand filter without under drainage system.

Softening: Overview of Lime soda, Zeolite process, RO and Nano filtration Disinfection: Methods of disinfection with merits and demerits, Theory of disinfection, Method of Fluoridation and De-fluoridation treatment of water.

Distribution system: Methods- Gravity, Pumping, Combined gravity and pumping system

Module-3

Wastewater & Its Conveyance:

Introduction, need for sanitation, methods of sewage disposal, types of sewerage systems, dry weather flow, wet weather flow, factors effecting dry and wet weather flow on design of sewerage system, estimation of storm flow, time of concentration flow,

Sewer materials: Material of sewers, shape of sewers, laying and testing of sewers, ventilation of sewers. Sewer appurtenances, manholes, catch basins, basic principles of house drainage, typical layout plan showing house drainage connections, Low-cost waste treatment; oxidation pond, septic tank

Module-4

Sewer Design & Effluent Disposal:

Design of sewers, hydraulic formula for velocity, design of hydraulic elements for circular sewers for full flow and partial flow conditions.

Disposal of effluents by dilution, self-purification phenomenon, oxygen sag curve, zones of purification, sewage farming, sewage sickness, numerical problems on disposal of effluents, Streeter-Phelps equation

Module-5

Wastewater Treatment:

Wastewater sampling, significance and techniques, BIS Effluent characteristics of wastewater, flow diagram for municipal waste water treatment, unit operations; screens - types, design, grit chambers, skimming tanks, equalization tanks,

Secondary treatment: Suspended growth and fixed film bio process, design of trickling filters, activated sludge process, Introduction to sequential batch reactors, moving bed bio reactors, sludge digesters.

Course outcomes:

After studying this course, students will be able to:

1. Identify the various water demand, available sources and the conveyance of water in municipal water supply scheme
2. Design the water supply scheme treatment units by understanding the basic principle.
3. Quantify wastewater generation and to make suggestions about operation and maintenance of the sewage collection and conveyance systems.
4. Design the sewer system and review the effect of disposal of municipal wastewater to streams and the concept of self-purification capacity
5. Understand and apply the design principles and criteria in designing wastewater treatment units

Need for Protected Water Supply

- water is absolutely essential not only for survival of human being but also for animals, plants and all other living creatures
- must be safe in all respects and it should not contain unwanted impurities or harmful chemical compounds or bacteria"s in it
- imperative in modern society to plan and build suitable water supply schemes which will provide potable (safe for drinking) water
- WSS - supplying safe whole some water to the people for drinking cooking, bathing, washing, etc.., - keep the diseases away - promoting better health, - maintaining better sanitation & beautification of surroundings.

Need for Protected Water Supply

- it shall ensure a safety against fire by supplying sufficient quantity of water to extinguish it.
- attracting industries - industrialization and modernization of the society - reducing unemployment and ensuring better living standards - promoting health, wealth and welfare of entire community as a whole.
- water may be considered as a carrier for bacteria and not multiplier thus the control of pathogens is possible by simple disinfection principles (process)

DEMAND FOR WATER

- i) Domestic water demand
- ii) Industrial and commercial water demand
- iii) Demand for public uses
- iv) Fire demand
- v) Water required compensating losses in wastes and thefts.

DOMESTIC WATER DEMAND:

- water required in private buildings for drinking, cooking, bathing, lawn sprinkling, Gardening, sanitary purposes etc….
- varies according to the living conditions of the consumers
- on an average this in a Indian city - 135 litres /day/person
- total domestic consumption generally amounts to 50- 60% of the total water consumption

AVERAGE DOMESTIC WATER CONSUMPTION IN A INDIAN CITY

USE	**CONSUMPTION IN LPCD**
Drinking	5
cooking	5
Bathing	55
Washing of clothes	20
Washing of utensils	10
Washing and clearing of houses and residences	10
	30
TOTAL	135 lpcd

INDUSTRIAL AND COMMERCIAL WATER DEMAND

- quantity of water required to be supplied to offices, Factories, different industries, hospitals, hostels, etc….
- vary considerably with the nature of the city and with the number and types of industries and commercial establishment
- no direct relation of this consumption with the population
- 50 lpcd -450 lpcd
- 20-50% of overall demand

INDUSTRIAL AND COMMERCIAL WATER DEMAND

Type of building	Age consumption in lpcd
1.Factories a) where bathrooms are required to be provided b) where no bathrooms are required to be provided	45 30
2.Hospitals (including laundry) per bed a) Number of beds < 100 b) Number of beds > 100	 340 450
3.Nurse homes and medical quarters	135
4.Hostels	135
5.Hotels (Per bed)	180
6.Restauvants (Per seat)	70
7.offices	45
8.Cinemas, Auditoriums and theatres (per seat)	15
9.Schools a) Day schools b) Residential school	 45 135

DEMAND FOR PUBLIC USES (MUNICIPAL CONSUMPTION)

- quantity of water required for public parks, gardening, washing and sprinkling on roads, use in public fountains etc…..
- not exceeding 5% of the total consumption

PURPOSE	WATER CONSUMPTION
Public parks	1.4 litres/m^2/day
Road watering	1-1.5 litres/m^2/day
Sewer cleaning	4.5 litres/head/day

FIRE DEMAND

- fire hydrants with separate water mains at about 100-150m apart are provided
- Pressure in pipe should be 10 to 15 m of water
- Flow rate is 1100 l/min of 3 jets,. Left of property - On the property- Right of the property

EMPIRICAL FORMULAS FOR FIRE DEMAND

- **Buston's Formula** $Q = 5663\sqrt{p}$
- **Freeman Formula** $Q = 1136.5\left[\frac{p}{10} + 10\right]$
- **Kulchiling's Formula**

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

FIRE DEMAND

- **NATIONAL BOARD OF FIRE UNDERWRITER'S FORMULA**

When population $\geq$ 2,00,000

$$Q = 4637 \sqrt{p}\left[1 - 0.01\sqrt{p}\right]$$

- When population is > 2,0,000 a provision for 54600 litres/ min + 9100 to 30400 litres/min for a second fire.
- For a Residential City
- 1. Small low buildings = 2200 litres/min
- 2. Large and higher buildings = 4500 litres/min
- 3. High value Residences apartment and tenements = 7650-1350 litres/min
- 4. Three storeyed buildings = 27,000 litres/min

WATER REQUIRED TO COMPENSATE IN THEFTS, WASTES, etc..

- water lost in leakage due to bad plumbing or damaged meters, stolen water due to unauthorized water connections and other losses and wastes, etc.…
- losses should be taken into account while estimating the total requirement.
- Even in the best managed water works this amount is usually taken as 15% of the total consumption.

PER CAPITA DEMAND (RATE OF DEMAND) (Q)

$$= \text{Per capita Demand in litres/day/head} = \frac{\text{Total yearly water requirement of the citty in Litres}}{365 \times \text{Design population}}$$

$$q = \frac{V}{365P}$$

USE	CONSUMPTION (LPCD)
Domestic use	135
Industrial use	50
Commercial use	20
Civic or public use	10
Waste, theft. Etc…	55
Total	270 lpcd

FACTORS AFFECTING PER CAPITA DEMAND

- 1. Size and type of city
- 2. Climatic conditions
- 3. Class of consumers
- 4. Quality of water
- 5. Pressure in the

- 6. Sewerage Facilities
- 7. System of supply
- 8. Policy of metering system
- 9. Cost of water

ASSESSMENT OF NORMAL VARIATIONS

- **MAXIMUM DAILY CONSUMPTION**
  - Maximum daily demand(MDD) = 1.8 Average daily demand (ADD) = 1.8q
- **MAXIMUM HOURLY CONSUMPTION**
  - Maximum hourly consumption = 1.5 Average hourly consumption
- **PEAK DEMAND** -Maximum hourly consumption of Maximum day

Peak Demand $= 1.5\left[\frac{MDD}{24}\right]$

$$= 1.5\left[\frac{1.8\,q}{24}\right]$$

$$= \frac{2.7\,q}{24}$$

Maximum hourly consumption of the maximum day

= (2.7 Annual Average hourly demand)

COINCIDENT DRAFT = MAXM DAILY DEMAND (MDD) + FIRE DEMAND (FD)

ASSESSMENT OF NORMAL VARIATIONS

GOODRICH FORMULA $P=180\ t^{-0.10}$

- P = % of annual average draft for the time t in days
- T = Time in days from to 1/24 - 365

When t= 1 day (For daily variations)

$$P = 180 \times 1^{-0.10}$$

$$P = 180\%$$

$$\therefore \frac{MDD}{ADD} = 180\%$$

When t = 7 days (For weekly variations)

$$P = 180 \times (7)^{-0.10}$$

$$P = 148\%$$

$$\therefore \frac{MWD}{AWD} = 148\%$$

T = 30 days (For monthly variations)

$$P = 180 \times (30)^{-0.10}$$

$$P = 128\%$$

$$\therefore \frac{MMD}{AMD} = 128\%$$

1. The design population of a town is 15000 Determine the Average daily, Maximum hourly demand under suitable assumptions

Soln: Assuming Average percapita demand as 270 Lpcd

i. ADD = design population $\times$ Avg. per capita demand

$= 1500 \times 270$

$= 4050000$ Litres/day

ADD = 4050 m^3/day

ii. MDD = 1`.8 $\times$ Average daily demand

$= 1.8 \times 4050$

MDD = 7290 m^3/day

iii. Maximum hourly demand of maximum day

$= 2.7 \times$ Annual Avg. hourly demand

$$= 2.7 \times \frac{q}{24}$$

$$= 2.7 \times \frac{4050}{24}$$

$= 455.625$ m^3/hr or

$= 10935$ m^3/day

EFFECTS OF VARIATION

Design consideration

- 1. The sources of supplies(well) = MDD
- 2. The pipe mains from source to service reservoirs = MDD.
- 3. The filter and OTHER UNITS at water treatment plant = MDD. Additional provision for reserve = break down and repairs = 2ADD
- 4. The pumps = MDD plus some additional reserve (2ADD)
- 5. The distribution system = maximum hourly demand of the maximum day or coincident draft (whichever is more) .
- 6. Service reservoirs = 8 days consumption.

DESIGN PERIOD

- like dams, reservoirs, treatment works, penstocks etc…., which cannot be replaced or increased in their capacities, easily and conveniently for example
- The future period or the number of years for which a provision is made in designing the capacities of the various component of the w.s.s- DESIGN PERIOD.
- 20-30 years - distribution system
- 1.Length or Life of Structures. -Steel Pipes 25 – 50 Years , Cast Iron -100 yrs
- 2.Ease of Extension. - Treatment Plant : Easy to enlarge. Tubewell : Easy to drill
- 3.First Cost - High first cost & Rate of interest – Less D P
- 4.Economy of Scale - production cast/unit decreases with increase in scale (size) of facility
- 5.Lead Time – (Time for Design of Project + Construction Time) - lead time is more→ high DP

POPULATION FORECASTING

- Arithmetical increase method
- Biometrical increase method
- Incremental increase method
- Decreasing rate of increase method or decreasing rate method
- Simple graphical method

- Comparative graphical method
- Master plan method or Zoning method
- Ration method or Apportionment method
- Logistics curve method.

POPULATION FORECASTING

- **ARITHMETICAL INCREASE METHOD**
- assumption that the population is increasing at a constant rate, ie.
- The rate of change of population with time is constant.

$$Pn = P + nIa$$

- **GEOMETRICAL INCREASE METHOD**
- percentage increase in population from decade to decade is constant.
- for young cities expanding at faster rates and useful for old developed cities.

$$Pn = P\left[I + \frac{Ig}{100}\right]^n$$

POPULATION FORECASTING

INCREMENTAL INCREASE METHOD

- combination of the AIM, GIM methods
- advantages of both and hence gives satisfactory results average decreases in the increases

DECREASE GROWTH RATE METHOD

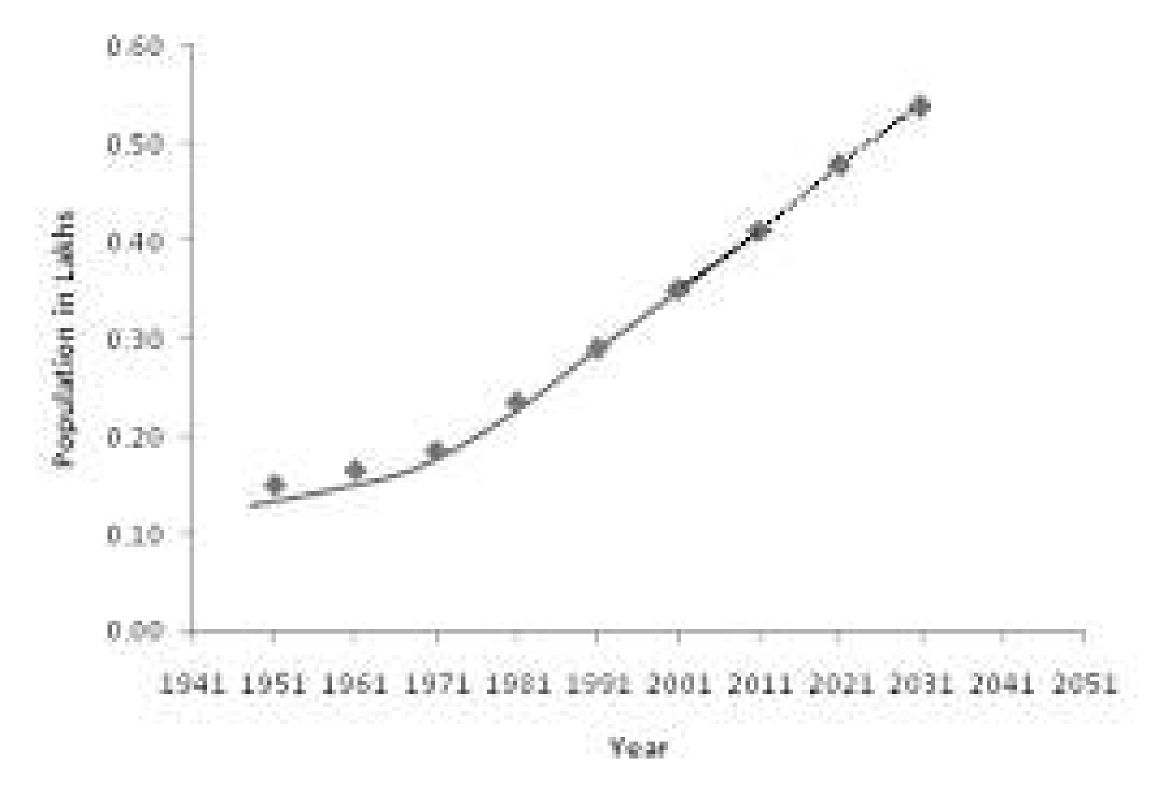
- Rate of increases in population – reduces - as a city reaches towards etc. saturation value this method which makes use of decreases in the % increases

$$p_n = p_o + nI_a + \frac{n(n+1)}{2} L_i$$

latest increases for each successive decade - average decreases in the increases

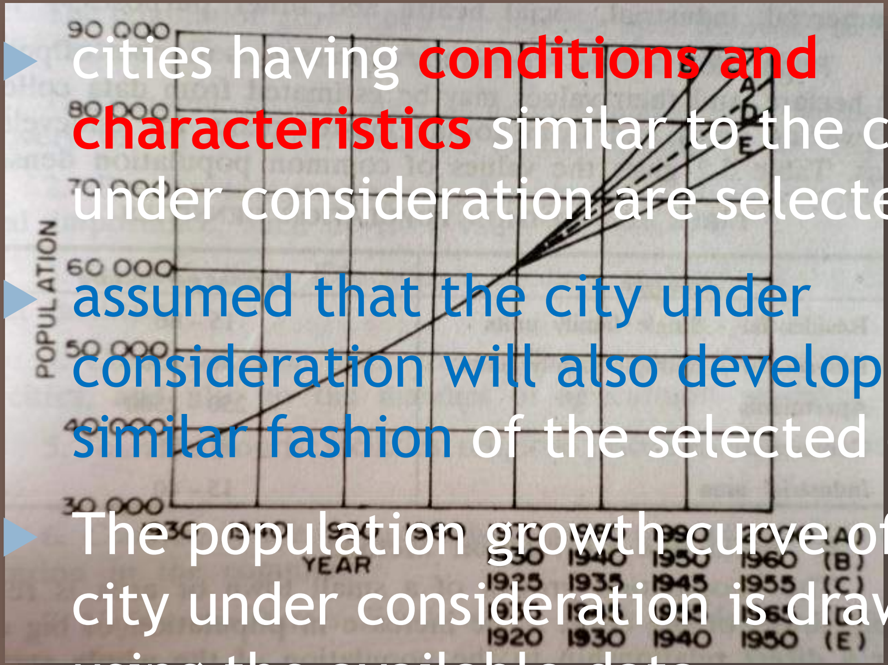
POPULATION FORECASTING

- **SIMPLE GRAPHICAL METHOD**
- graph is plotted between time and population the curve
- then smoothly extended up to the desired year.
- Gives very approximate results, extension of the curve is done by the intelligence of the designer.



POPULATION FORECASTING

- **COMPARATIVE GRAPHICAL METHOD**



- cities having **conditions and characteristics** similar to the city under consideration are selected.
- assumed that the city under consideration will also develop in similar fashion of the selected cities.
- The population growth curve of the city under consideration is drawn using the available data

POPULATION FORECASTING

MASTER PLAN OR ZONING METHOD

- By & metropolitan cities are generally not allowed to develop in a haphazard & natural way but are allowed to develop only in planned ways
- The master plan prepared - divides the city into zones & thus to separate each
- The population densities are fixed.
- Give us as to when & where the given no of houses, industries etc.. Would be developed. - Very easy to calculation design

RATIO METHOD & APPORTION METHOD

- In this method the cities population record as % of poplin of the whole country.
- The rations of local to national poplin are worked out for past 4-5 decades.
- A graph in plotted between ratios & year design period to calculation future population.

If the poplin of a town is plotted w.r.t time, the curve so obtained under normal condition are be as in figure & is called as ideal growth curve or logistics curve

- early growth JK at an increasing rate i.e. geometric growth or log growth,

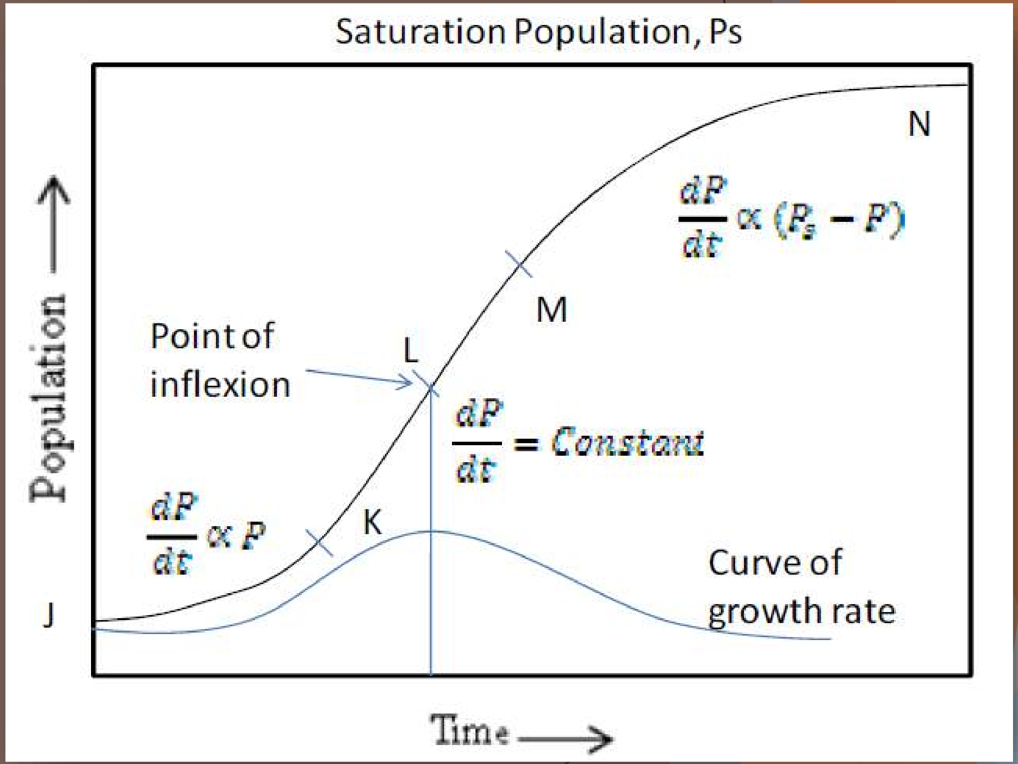
$$\frac{dP}{dt} \propto P,$$

- middle curve KM follows arithmetic increase i.e. constant.

$$\frac{dP}{dt} = \text{Constant}$$

- e rate of change of population is proportional to difference between saturation population and existing population, i.e.

$$\frac{dP}{dt} \propto (P_s - P).$$



POPULATION FORECASTING

- **Logistic curve method**
- Ps= Saturation population
- P_0 = Population of the city at the start point
- P_0,P_1,P_2 are population at t_1,t_2 and t_3 time
- m & n constants

$$P_s = \frac{2P_0P_1P_2 - P_1^2(P_0 + P_2)}{P_0P_2 - P_1^2}$$

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

$$n = \frac{2.3}{t_1} \log_{10}\left[\frac{P_0}{P_1}\frac{P_s - P_1}{P_s - P_0}\right]$$

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

Population forecasting

Athematic Increase Method

Year	Population
1940	8,000
1950	12,000
1960	17,000
1970	22,500

Calculate the probable population in the year 1980, 1990 and 2000.

Year	Population	Increase in Population
1940	8,000	
1950	12,000	4000
1960	17,000	5000
1970	22,500	5500
	Total	14,500
	Average Inverse	4,833

Year	Population
1980	$22,500 + 1 \times 4833 = 27,333$
1990	$27333 + 1 \times 4833 = 32,166$
2000	$32166 + 1 \times 4833 = 36,999$

Year	*Population*	*Increase in Population*	*Percentage increase in Population*
1940	8,000	—	
1950	12,000	4,000	$\frac{4000}{8000} \times 100 = 50.0\ \%$
1960	17,000	5,000	$\frac{5000}{12000} \times 100 = 41.7\ \%$
1970	22,500	5,500	$\frac{5500}{17000} \times 100 = 32.4\ \%$
	Total	14,500	124.1
	Average per decade	4,833	41.37

Geometric Increase Method

Year	*Expected Population*
1980	$22{,}500 + \frac{41.37}{100} \times 22{,}500 = 31{,}808$
1990	$31{,}808 + \frac{41.37}{100} \times 31{,}808 = 44{,}967$
2000	$44{,}967 + \frac{41.37}{100} \times 44{,}967 = 63{,}570$

Incremental Increase Method

Year	*Population*	*Increase in Population*	*Incremental increase i.e. increment on the increase*
1940	8,000	—	—
1950	12,000	4,000	—
1960	17,000	5,000	+1000
1970	22,500	5,500	+0500
	Total	14,500	+1500
	Average	4,833	(+) 750

Decrease growth rate Method

Year	Population	Increase	Percentage increase in Population	Decrease in the percentage increase
1940	8000	—	—	—
1950	12000	4000	$\frac{4000}{8000} \times 100 = 50.0$	—
1960	17000	5000	$\frac{5000}{12000} \times 100 = 41.7$	+8.3
1970	22500	5500	$\frac{5500}{17000} \times 100 = 32.4$	+9.3
	Total	14500		17.6
	Average	4833		8.8

Year	Net percentage increase in Population	Population
1980	$32.4 - 8.8 = 23.6$	$22{,}500 + \frac{23.6}{100} \times 22{,}500 = 27{,}810$
1990	$23.6 - 8.8 = 14.8$	$27{,}810 + \frac{14.8}{100} \times 27{,}810 = 31{,}926$
2000	$14.8 - 8.8 = 6.0$	$31{,}926 + \frac{6}{100} + 31{,}926 = 33{,}842$

Year	Population	Increase in Population	% increase in Population	decrease in the % increase
1940	25000	—		
1950	28000	3000	$\frac{3000}{25000}\times 100 = 12\%$	
1960	32500	4500	$\frac{4500}{28000}\times 100 = 16.1\%$	(–) 4.1%
1970	40000	7500	$\frac{7500}{32500}\times 100 = 23.1\%$	(–) 7.0%
1980	45000	5000	$\frac{5000}{40000}\times 100 = 12.5\%$	(+) 10.6%
Total		20000		(–) 11.1 + 10.6 = (–) 0.5%
Average per decade		5000		$(-)\frac{0.5}{3} = (-)0.17\%$

Year	Net percent increase in Population	Population
1990	$12.5-(-0.17)=12.67$	$4500+\frac{12.67}{100}\times 4500 = 50702$
2000	$12.67-(-0.17)=12.84$	$50702+\frac{12.84}{100}\times 50702 = 57212$
2010	$12.84-(-0.17)=13.01$	$57212+\frac{13.01}{100}\times 57212 = 64654$

Sources and Characteristics

- Sources of Water
- Surface Sources
- Streams & Rivers
- Lakes & Ponds
- Oceans
- Subsurface Sources
- Open Wells & Tube wells
- Infiltration galleries
- Infiltration Wells

Sources and Characteristics

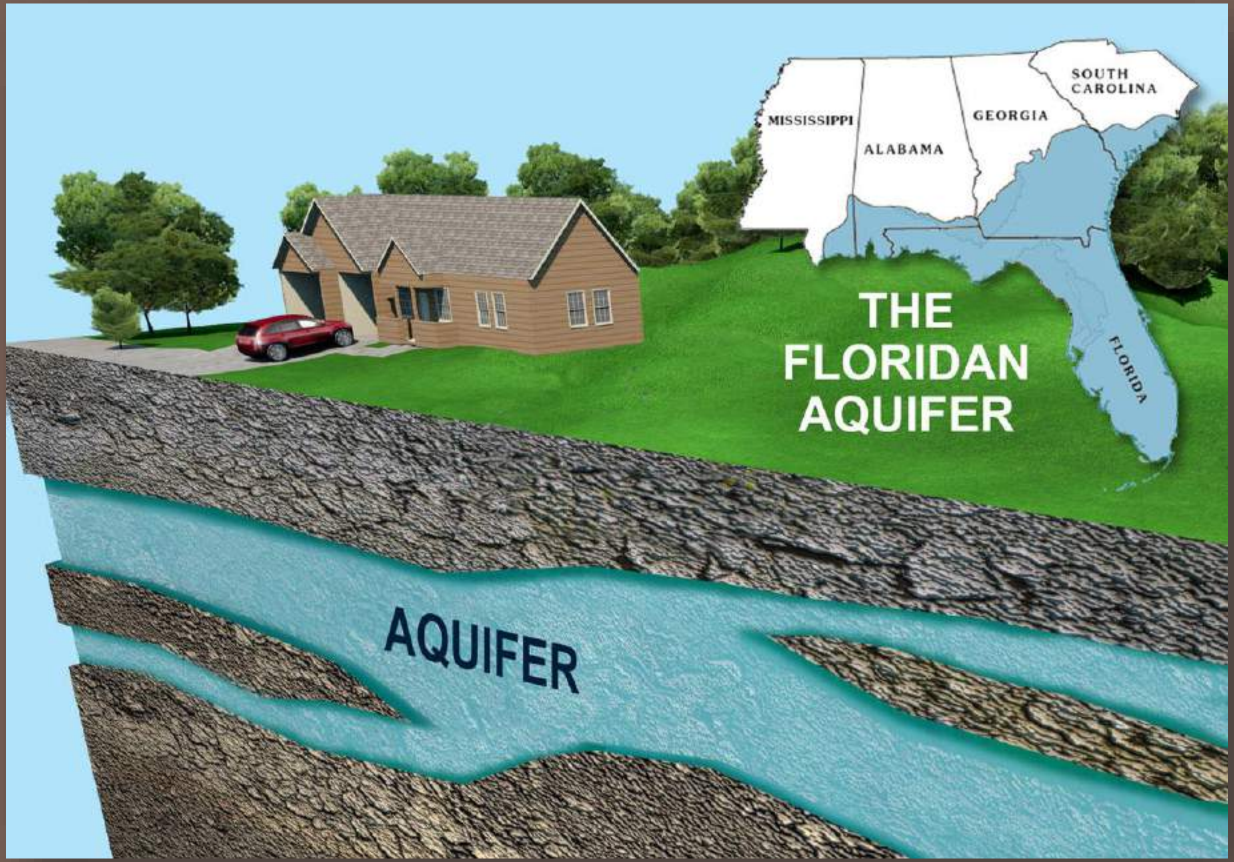
- **Seawater**: seawater in the world's oceans - salinity of about 3.5%., 35 grams/ every kilogram, or every liter. - cannot be used as potable source of water.
- **River:** A River is a natural watercourse, usually freshwater, flowing toward an ocean, a lake, a sea or another river. In a few cases, a river simply flows into the ground or dries up completely before reaching.
- Rivers have been used as a source of water, for obtaining food, for transport, as a defensive measure, as a source of hydropower to drive machinery, for bathing, and as a means of disposing of waste.

Surface Sources

- Pond & Lake: A natural large sized depression formed on the surface of the earth, when gets filled up with water is known as a pond or a lake. If the size of depression is small, it is termed as a pond and when the size is large it may be termed as lake.
- Stream: Stream is a flowing body of water with a current, confined within a bed and stream banks. important as conduits in the water cycle, instruments in groundwater recharge.
- Springs: The natural outflow of ground water at the earths surface is said to be spring. A pervious layer sandwiched between two impervious layer, give rise to natural spring . capable of supplying small amount of water, - therefore not considered as a source of supply

Subsurface Sources

- **Aquifer:** An aquifer is an underground layer of water-bearing permeable rock or unconsolidated materials (gravel, sand, silt, or clay) from which groundwater can be usefully extracted using a water well.
- **Infiltration Galleries:** Infiltration galleries are horizontal and nearly horizontal tunnel constructed at shallow depth along the bank of river through the water bearing strata.
- **Infiltration wells** are shallow wells constructed along the banks of the river in order to collect the river water seeping through their bottom. •
- constructed of brick masonry with open joints. generally covered at the top and kept open at the bottom


MISSISSIPPI
ALABAMA
GEORGIA
SOUTH CAROLINA
FLORIDA
THE FLORIDAN AQUIFER
AQUIFER

INFILTRATION GALLERIES OR HORIZONTAL WELLS

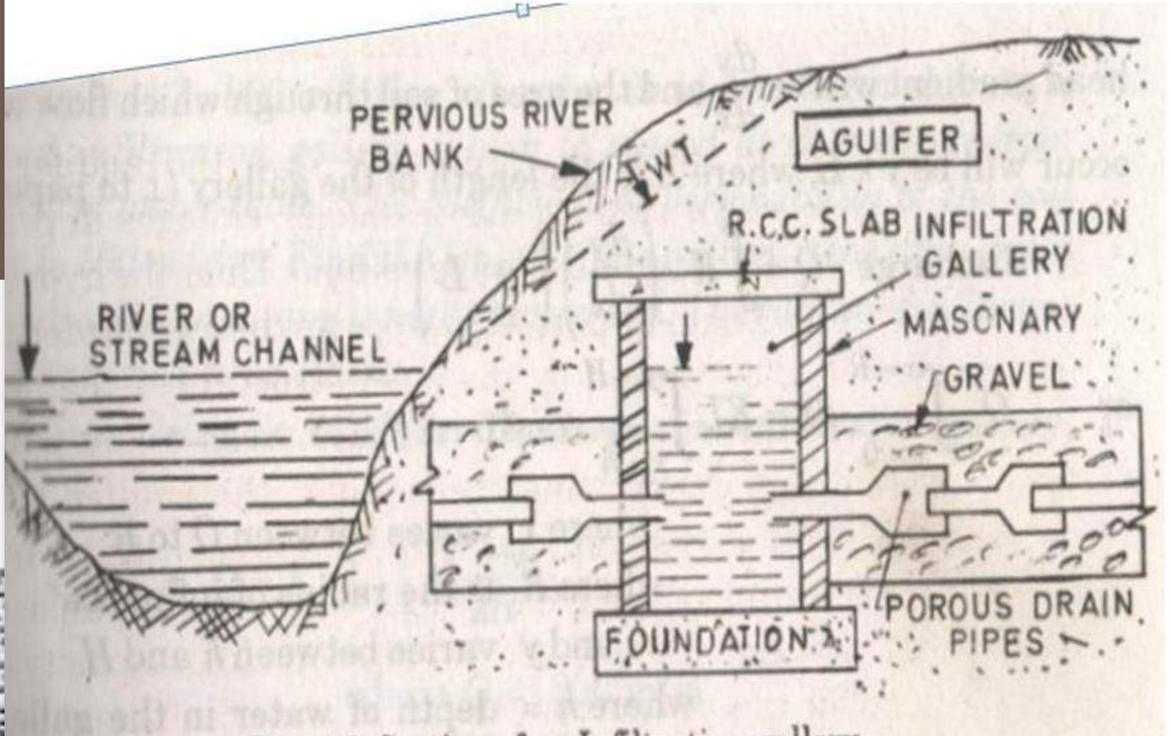


Fig. 4.8. Section of an Infiltration gallery.

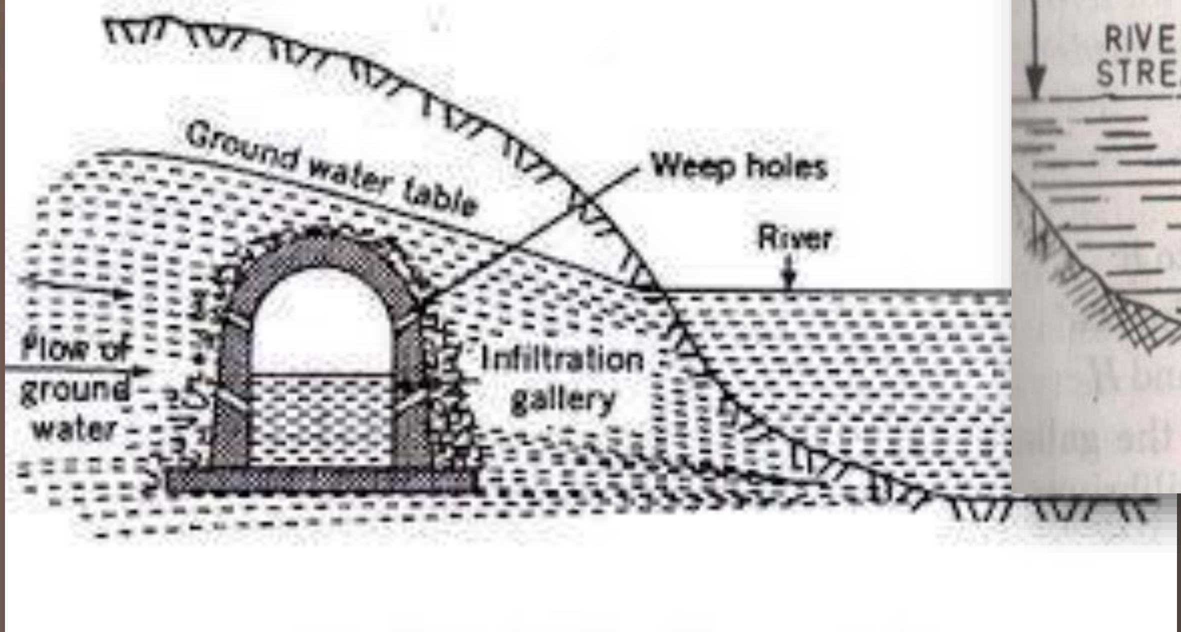


Fig. 4.6. Infiltration galleries.

Infiltration wells

- Vertical collection wells sunk on the river/stream banks - water infiltrates from both bottom and sides
- A hand pump, windmill or power pump is used to pump out water from the well
- Not affected by floods, silt/sand/gravel loads, and extremely low waters in rivers/streams
- Provides better quality water throughout the year (filtration)

The well can have radial porous pipes (jack wells)

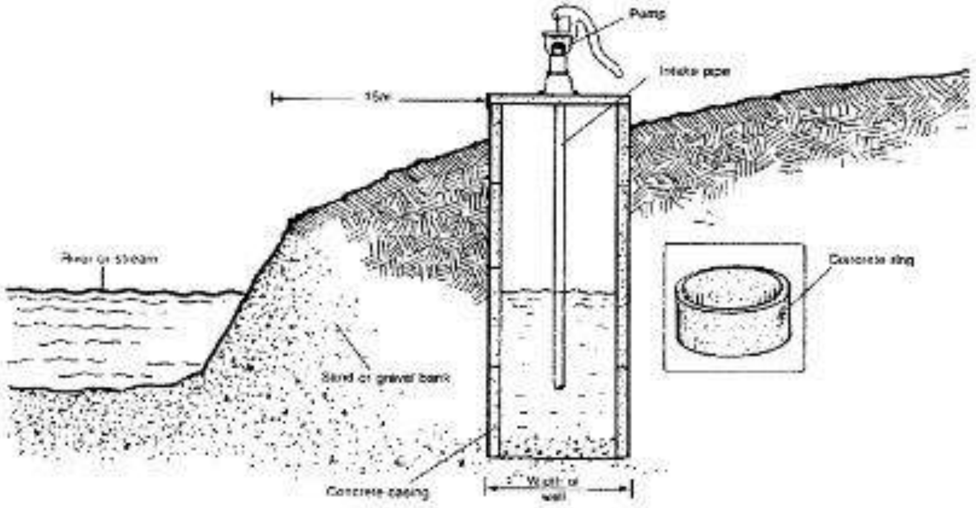
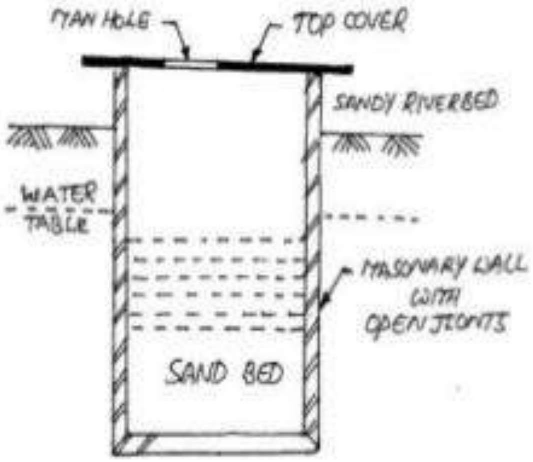
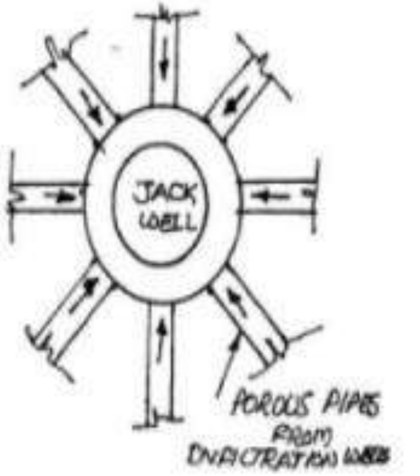


Figure 6. Riverside Well Intake

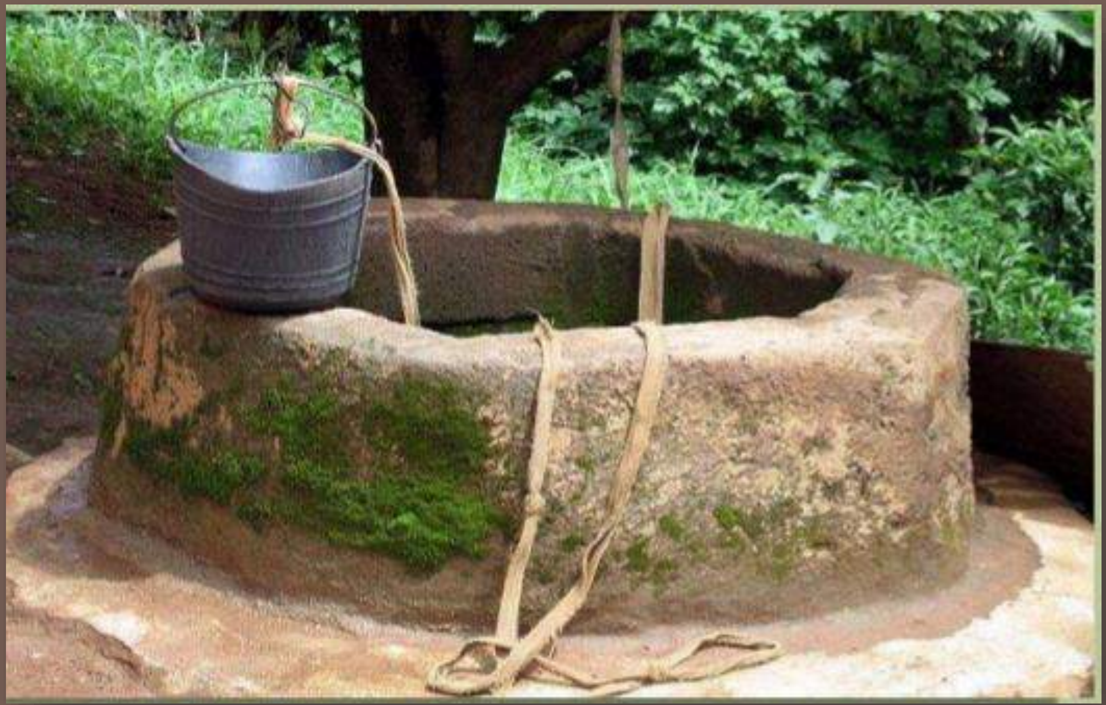


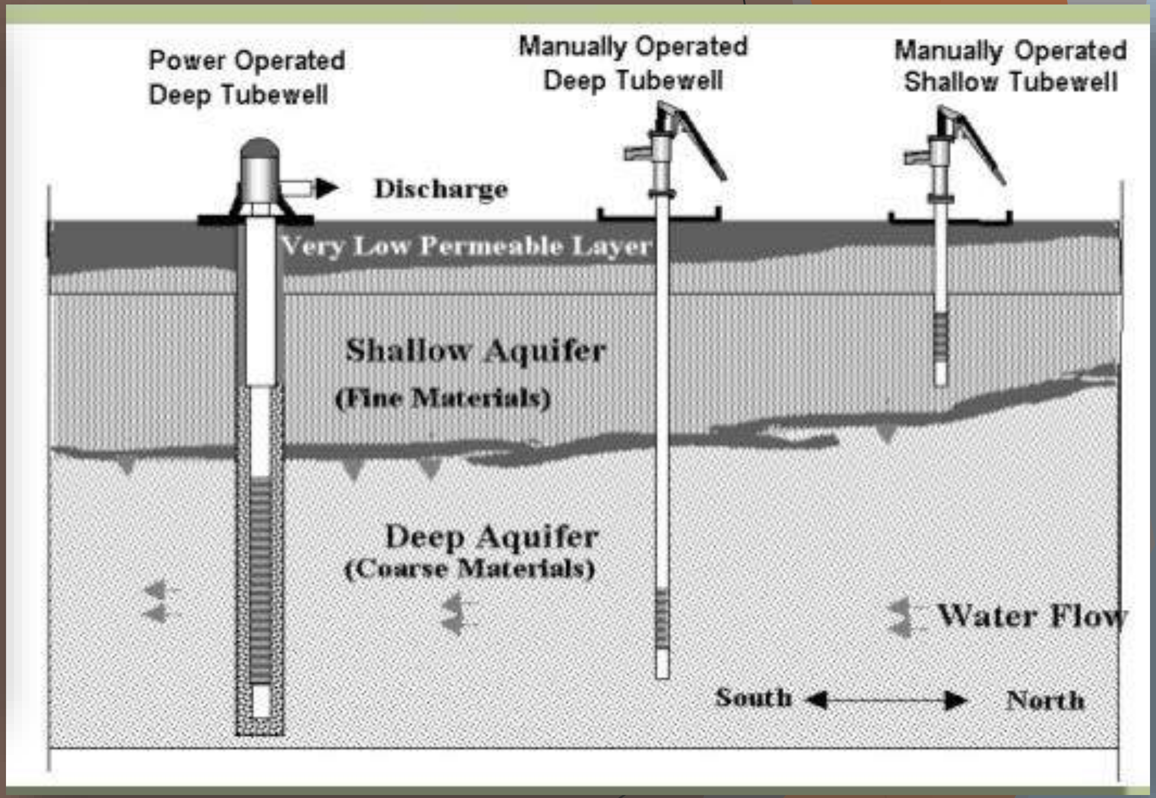


Subsurface Sources

- Open Wells: water has been utilized from ancient times
- Yield -1-5 l/s, diameter - 2-9 m , depth - 20 m .
- The yield is limited & can be excavated up to limited depth .
- source of water for a small community like a village.
- Tube wells
- To obtain large discharges tube wells a long pipe or a tube, is bored or drilled deep into the ground, intercepting one or more water bearing stratum
- Yield - 200 to 220 l/sec. Depth - 70 m to 300 m. Dia 0.5 to 0.6 m.

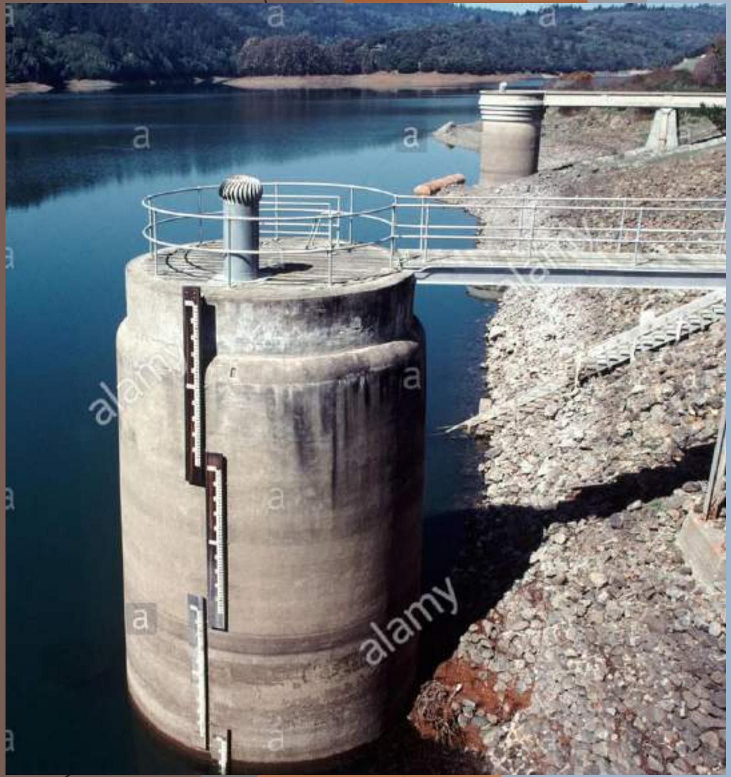
surface and subsurface sources





INTAKE STRUCTURES

- Intakes - masonry or concrete structures whose function is to provide calm and still water, free from floating matter for water supply.
- consists of the opening, strainers or gratings through which the water enters and the conduit for conveying the water, usually by gravity to a sump well. From the well the water is pumped to the treatment plant.



Factors Governing Location of Intake

- should provide good quality water so that its treatment may become less expensive
- Heavy water currents should not strike the structure directly
- Approach to the intakes should be easy
- As far as possible intakes should not be selected in the vicinity of sewage disposal
- Site should be nearer to the treatment plant so that it reduces the cost of conveyance of water
- They should not be located in navigation channels
- In meandering rivers, the intakes should not be located on curves or at least on sharp curves
- located at a place from where it can draw water even during the driest periods of the year.
- Site should be such as to permit greater withdrawal of water, if required of a future date.

Reservoir intake

- This feature makes it usually desirable to take water from about 1m below the surface.
- Due to fluctuations in water level, - ports at various heights with gate valves.
- Gate valves are used to regulate water supply. When water level goes down, gate valve of lower portion is opened.
- The access to the ports is made by means of an operating room

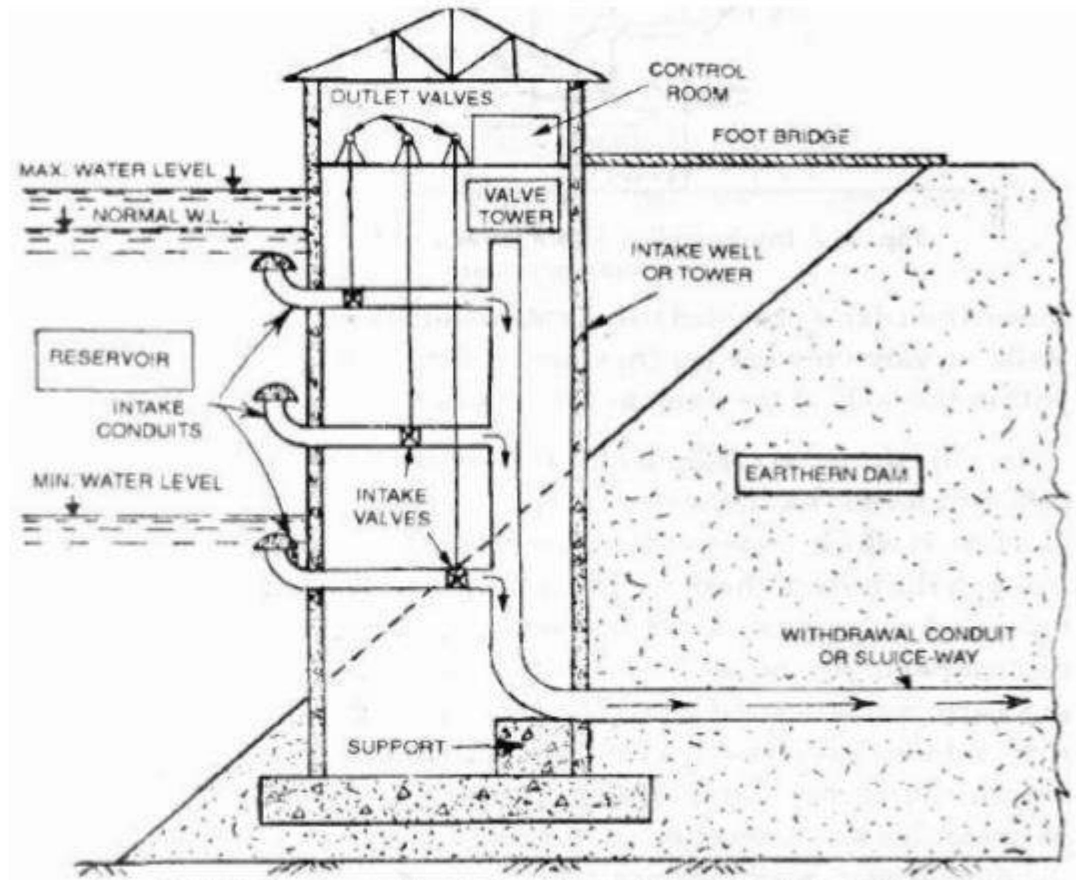
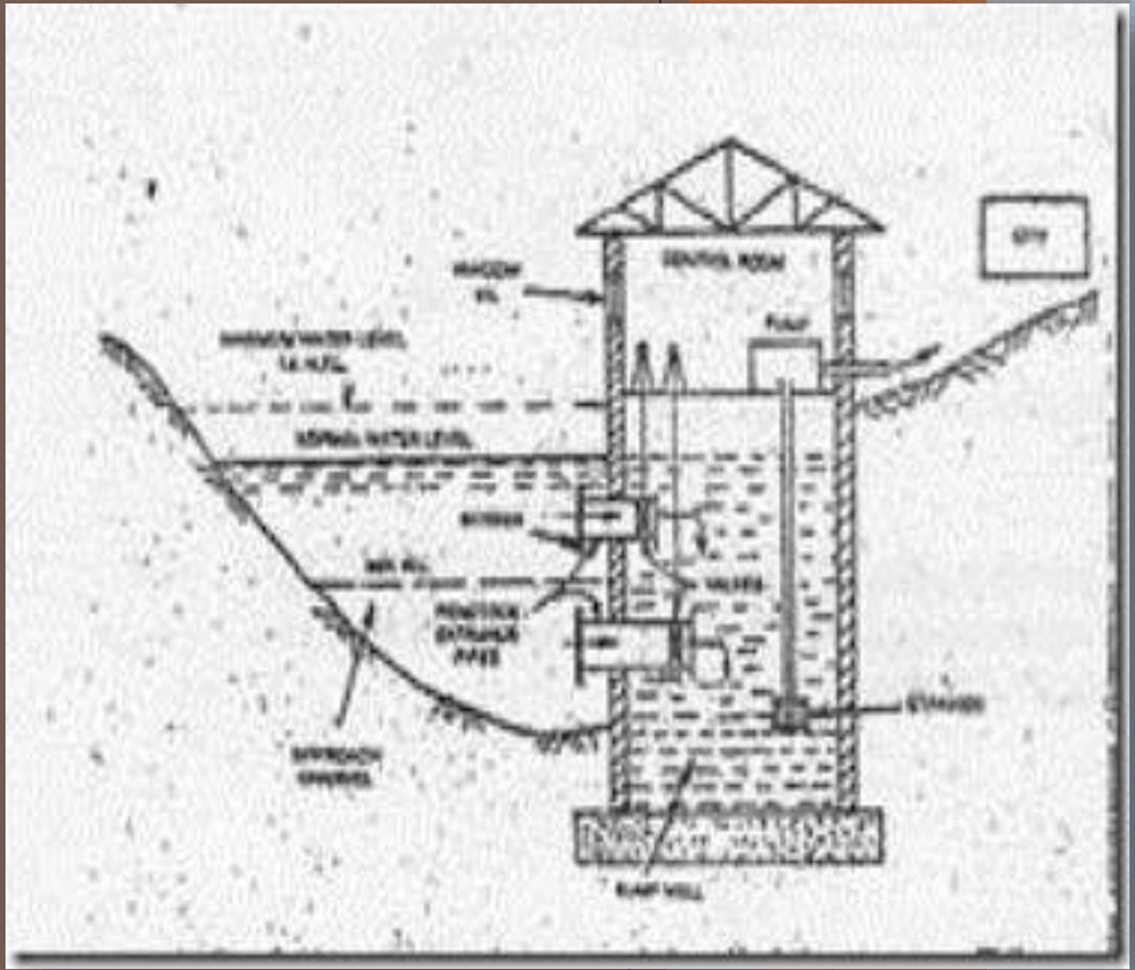


Fig. 5.11. Valve tower situated at the upstream toe of an earthen dam.

River Intakes

- A river intake consists of a port (conduit) provided with a grating and a sump or gravity well.
- The conduit must 1-2m above the bottom to prevent entry of silt. & kept 1m below the top surface to avoid entry of floating particles.
- Velocity < 0.15 m/s to prevent entry of small fish.
- constructed above the point of sewage disposal or industrial waste water disposal.
- River intakes are likely to need screens to exclude large floating matter. The bottom of the river intake must be sufficiently stable.



Lake intakes

- If the lake shore is inhabited, the intake should be constructed so that the danger of pollution is minimized.
- Intake opening above 2.5m the bottom so that the entry of silt with water is minimized.
- Entering velocity must be low to prevent entering of floating matter, sediment, fish or ice. Entering velocity of 0.15 m/s is usually used.
- Off shore winds tend to stir up sediments carries for long distances. must be located 600-900 meter.

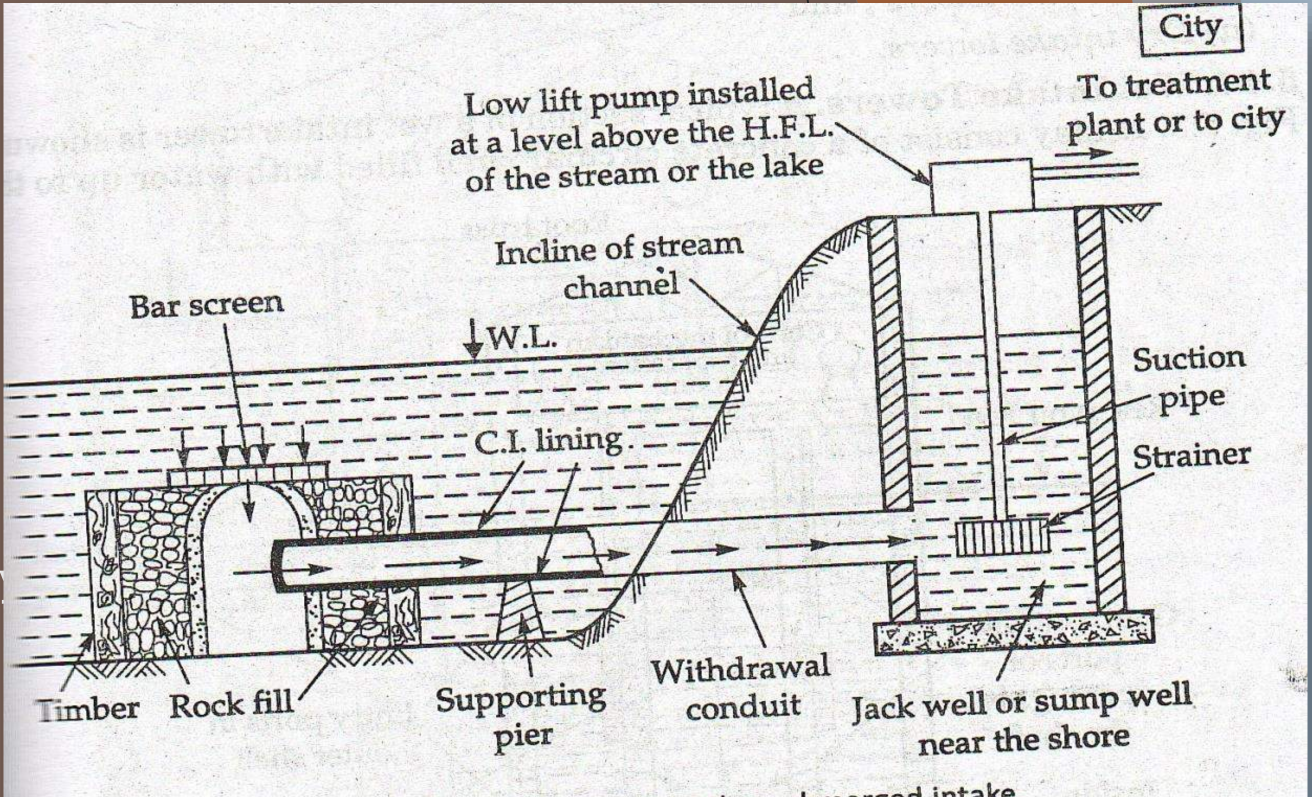
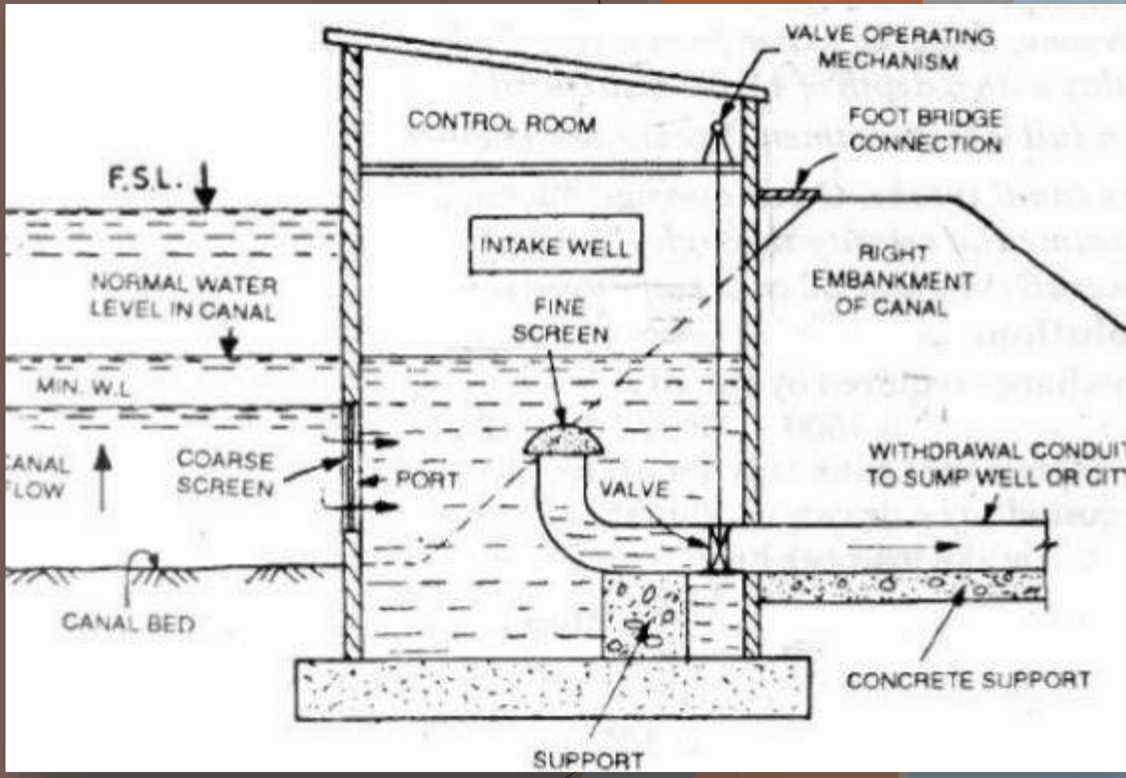


Fig. 5.2. Rock filled timber crib—submerged intake.

CANAL INTAKE

- a very simple structure constructed on the bank.
- It consists of a pipe placed in a brick masonry chamber constructed partly in the canal bank on one side of the chamber an opening is provided with coarse screen for the entrance of water.
- The end of the pipe inside chamber is provided with a bell mouth fitted with a hemispherical fine screen.
- The outlet pipe caries the water to - other side of bank / to treatment plants one sluice value is operated by a wheel from the top of the masonry chamber provided to control the flow of water in the pipe.



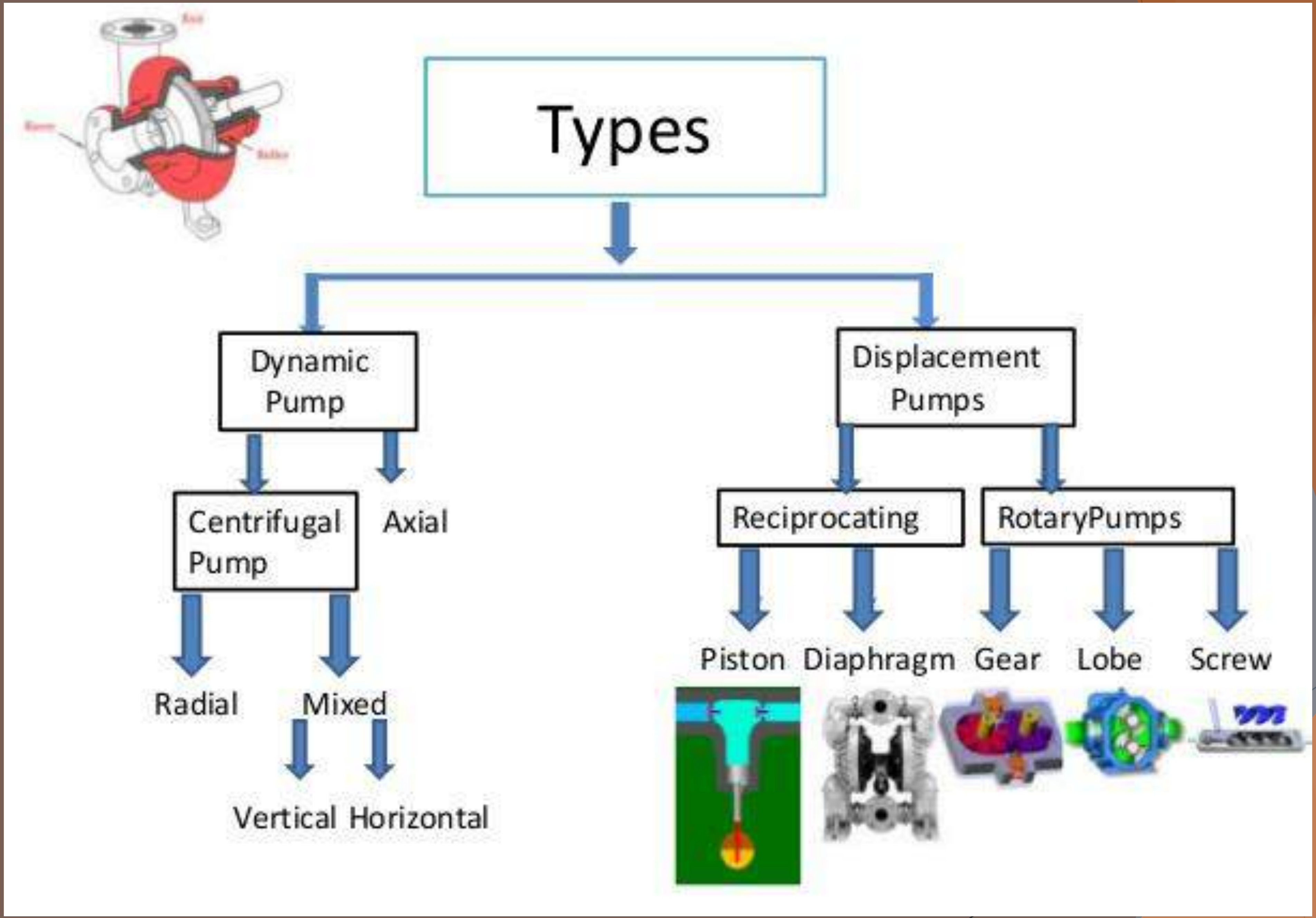
Pumps for Lifting Water

- The **function of the pump is to lift the water or any fluid at higher elevation** or higher pressure. In water works pumps are required under the following circumstances.
- **At the source of water to lift the water from rivers, streams, wells etc.. and to pump it to the treatment works.**
- **At the treatment plant to lift the water at various units so that it may flow in them due to the gravitational force** only during the treatment of the water.
- **For the back washing of filters and increasing their efficiency**.
- **For pumping chemical solution at treatment plants**.
- **For filling the elevated distribution reservoirs or overhead tanks**
- **To increase the pressure in the pipe lines by boosting up the pressure**.
- **For pumping the treated water directly in the water mains for its distribution**.

Pumps for Lifting Water

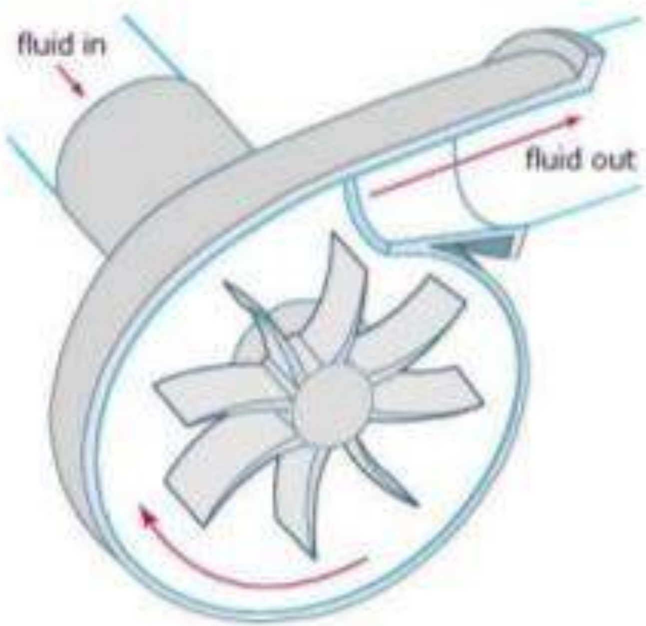
Classification of pumps

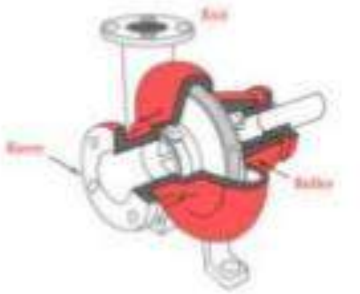
- **(i) Classification based on principles of operation**
- Displacement pump
- Centrifugal pumps
- Air –lift pumps
- Impulse pumps
- **(ii) Classification based on type of power required**
- Electrical driven pumps
- Gasoline engine pumps
- Steam engine pumps
- Diesel engine pumps
- **(iii) Classification based on the type of services**
- Low lift pumps
- High lift pumps
- Deep-well pumps
- Booster pumps
- Standby pumps


Types
Dynamic Pump
Displacement Pumps
Centrifugal Pump
Axial
Reciprocating
RotaryPumps
Radial
Mixed
Vertical
Horizontal
Piston
Diaphragm
Gear
Lobe
Screw

Principle of Operation

- <u>Dynamic Pump</u>: . *In dynamic pumps, energy is added to the fluid continuously through the rotary motion of the blades. This increase in energy is converted to a gain in Pressure Energy when the liquid is allowed to pass through an increased area.*





Centrifugal Pump

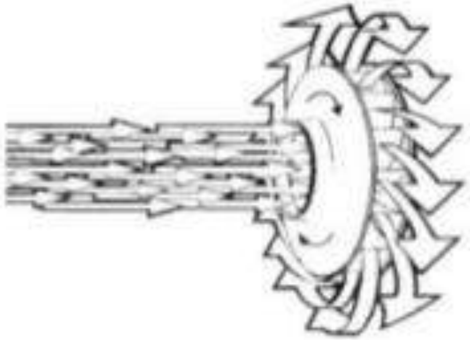
- <u>Working Principle</u>:

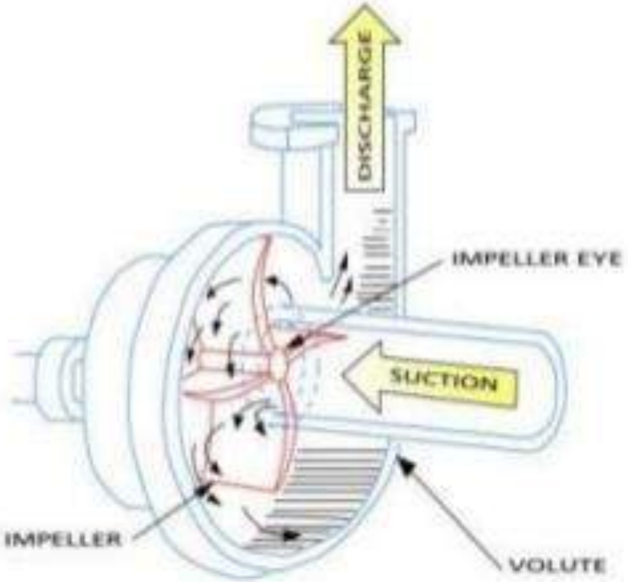
Works on the principle of centrifugal force. This is the force that pushes the liquid away from the centre(in tangential direction).

Converting Prime Mover energy into Mechanical energy through shaft .

Converting Mechanical energy into fluid energy impeller.

Converting kinetic Energy into pressure energy through the volute casing.





reciprocating pump

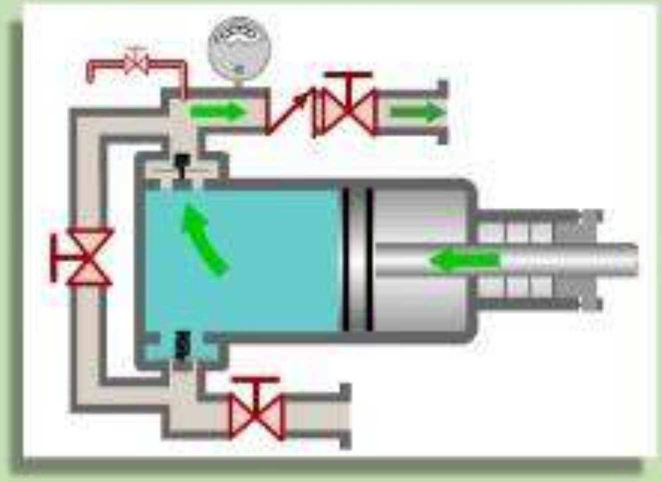
- Reciprocating pump is a positive displacement pump. In this pump the liquid is discharge due to the simple to and fro motion or reciprocating motion of the piston or plunger working in the cylinder of the pump. Hence ,it is called Reciprocating pump .

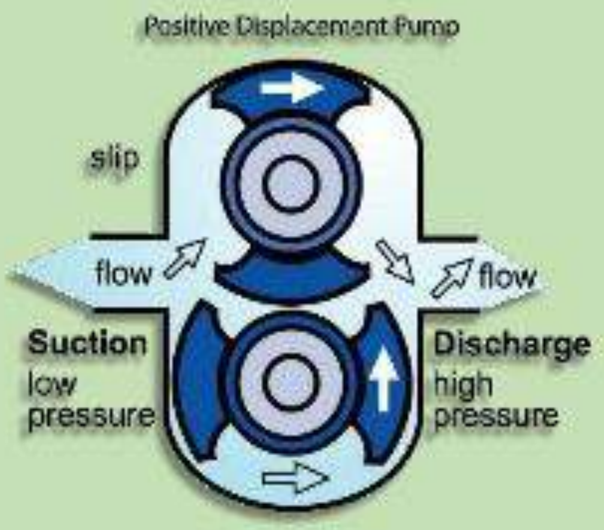
Displacement Pumps

- **In these types of pumps vacuum is created mechanically by the movable part of the pumps.** In the vacuum first the water is drawn inside the pumps, which on the return of mechanical part of the pump is displaced and forced out of the chamber trough the valve and pipe. The back flow of the water is prevented by means of suitable valves.

(i) Reciprocating Pumps

(ii) Rotary Pumps

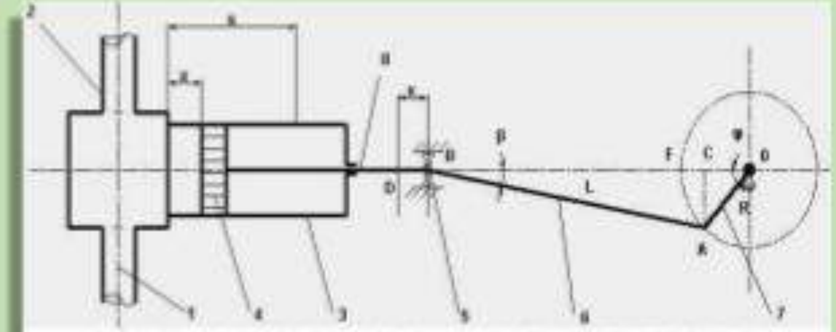




Reciprocating Pump

- Reciprocating pumps may be of the following types
- **Simple hand-operated reciprocating pump**
- **Power operated deep well reciprocating pump**
- **Single-acting reciprocating pump**
- **Double-acting reciprocating pump**

Reciprocating Pump



Schematic of Single Acting Reciprocating Pump Working Principle
1. suction valve 2. discharge valve 3. liquid cylinder 4. piston
5. crosshead 6. connecting rod 7. crank shaft 8. stuffing box

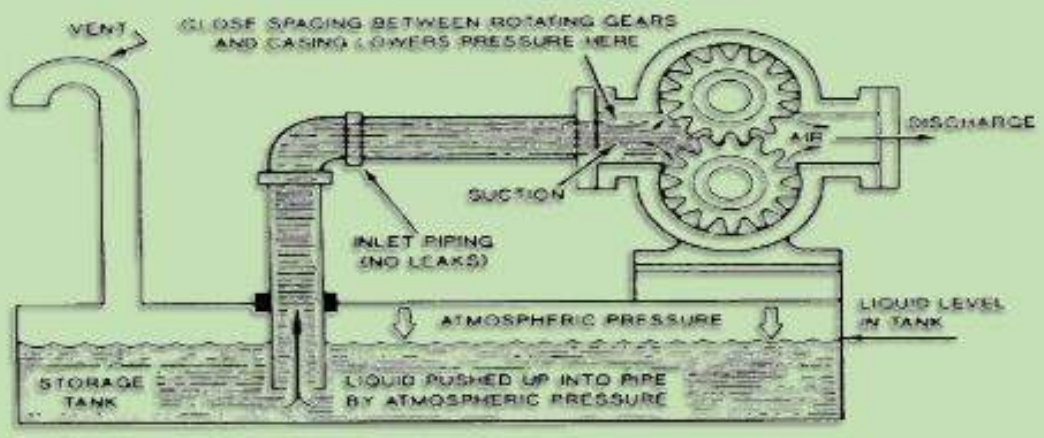
Rotary Pump

(a) Rotary pumps with gear

(b) Rotary pumps with cams

- **The revolving blades fit closely in the casing and push the water by their displacement**. The blades revolve in a downward direction at the Centre and the water is carried upward around the side of the casing. In this way the water is pushed through the discharge pipe and partial vacuum is created on the suction side. The intensity of vacuum mainly depends on the tightness of the parts.

Rotary Pump



Advantages of Rotary Pumps

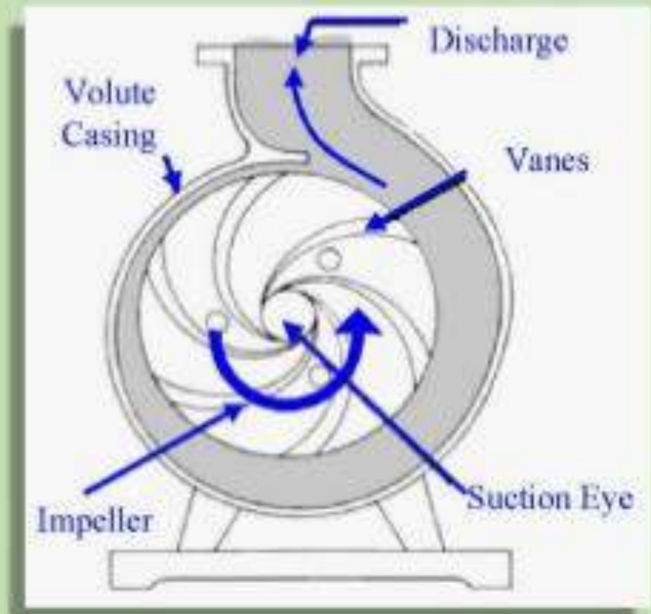
- They **do not require any priming as they are self-primed**.
- **The efficiency of these pumps is high** at low to moderate heads up-to discharge of 2000 l/m
- **These pumps have no valves, are easy in construction and maintenance as compared with reciprocating pumps**.
- **These pumps give steady and constant flow**
- **These pumps are deployed for the individual building water supply and for fire protection**.

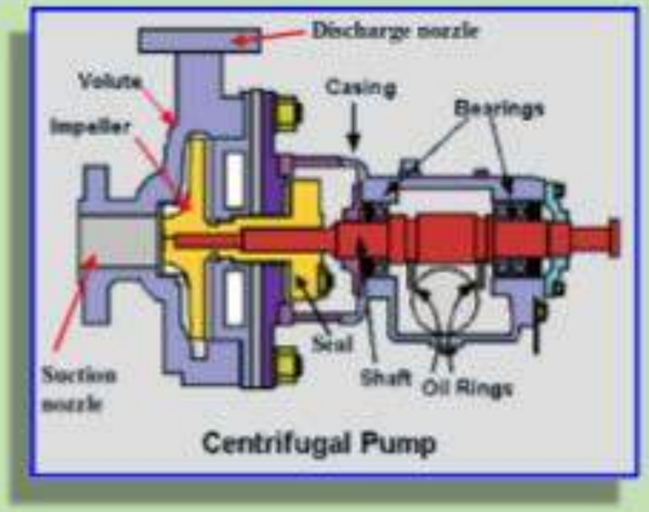
Disadvantages of Rotary Pumps

- The **initial cost of these pump is high**
- **Their maintenance cost is high due to abrasion of their cams and gears**.
- **They cannot pump water containing suspended impurities** as the wear and abrasion caused by the impurities will destroy the seal between the cans and the casing.

Centrifugal Pumps

- **These pumps work on the principle of centrifugal force, therefore, they are called centrifugal pumps.** The water which enters inside the pump is revolved at high speed by means of impeller and is thrown to the periphery by the centrifugal force.





Advantages and Disadvantages of Centrifugal Pumps

- The centrifugal pumps have the following advantages
- Due to **compact design, they require very small space**.
- They **can be fixed to high-speed driving mechanism**
- They **have rotary motion due to which there is low or no noise**
- They **are not damaged due to high pressure**

Disadvantages of centrifugal pumps

- **They require priming**
- The **rate of flow of water cannot be regulated**.
- **Any air leak on the suction side will affect the efficiency of the pump.**
- They have **high efficiency only for low head and discharge**.
- The **pump will run back, if it is stopped with the discharge valve open.**

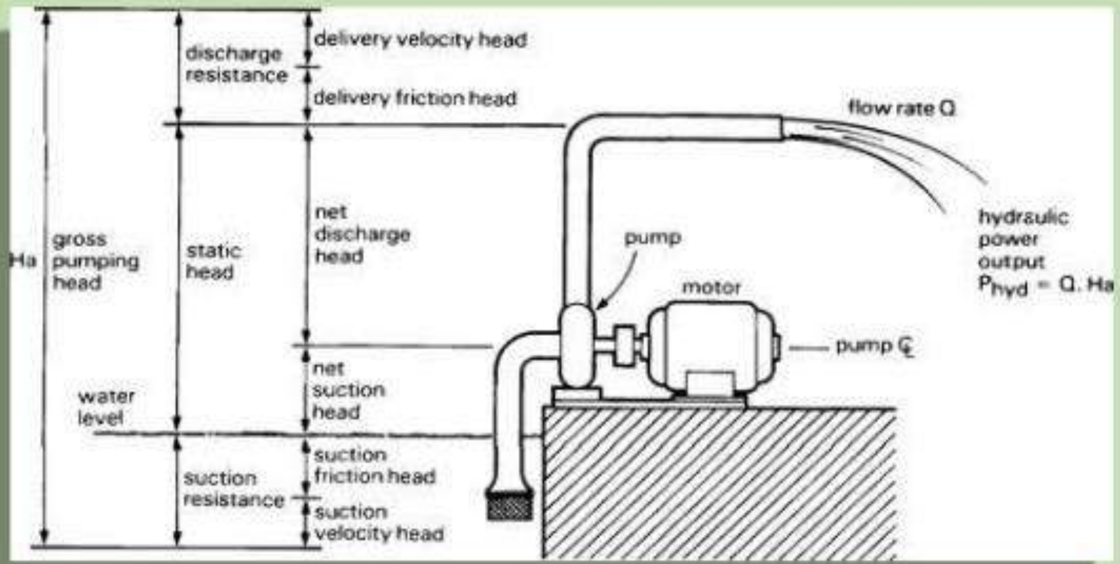
Design of Pumps

Total Head Or Lift Against Which The Pump Has To Work

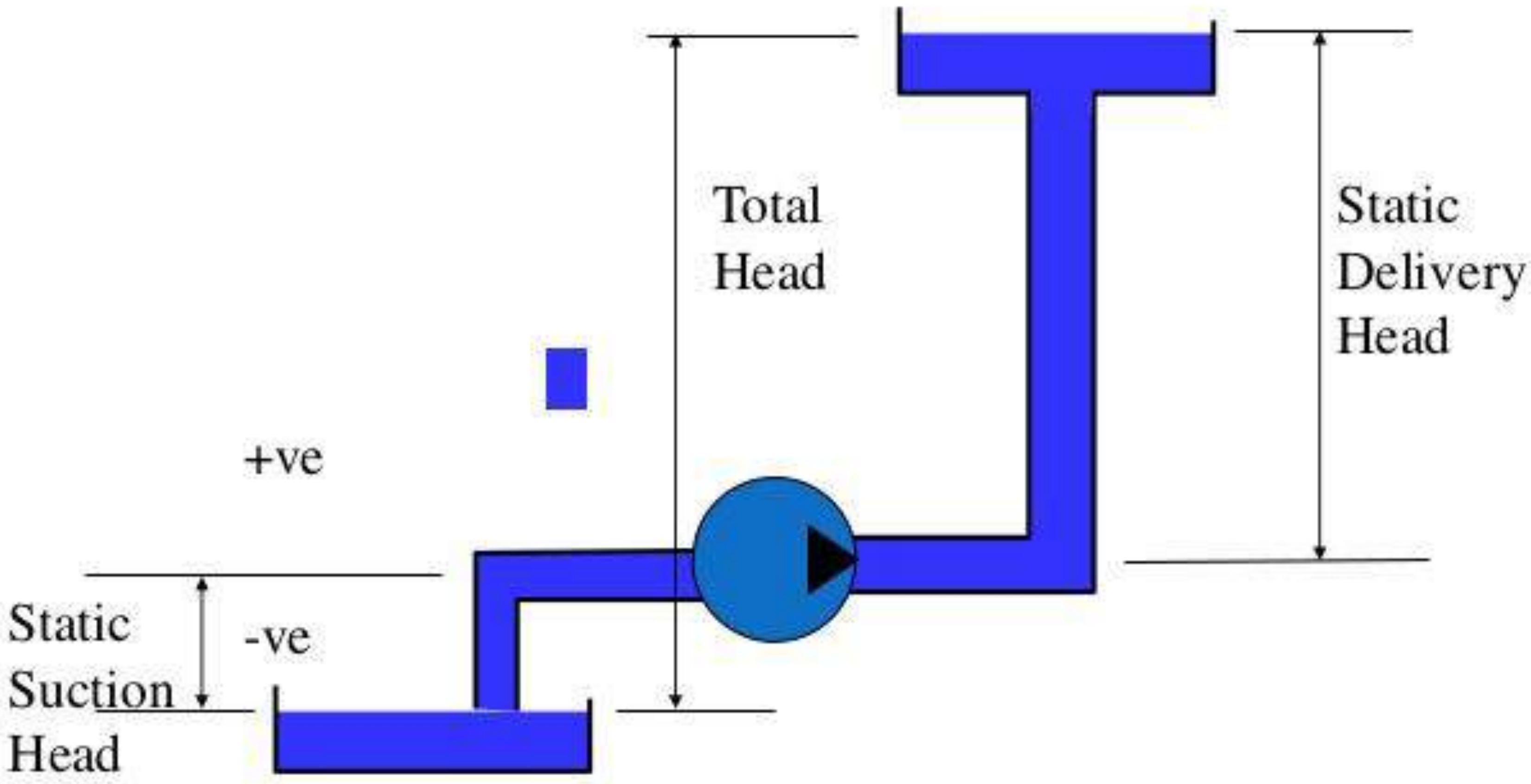
- **Suction lift .** It is the difference between the lowest water and the pump
- **Discharge lift or delivery Head.** It is the difference between the point of discharge or delivery and the pump.
- Generally only the friction losses is considered for the design as minor losses are very small if the length of the pipe is greater

- If **H_s- Suction lift or Head**
- **H_d- Delivery or discharge head**
- **H_l- Total loss of head then,**
- **The total head against which the pump has to work is given by:**
- **$H = H_s + H_d + H_1$**

Total Head Or Lift Against Which The Pump Has To Work



Pump Head



Total Head Or Lift Against Which The Pump Has To Work

- Friction loss can be found out by Darcy Weisbach equation
- Darcy Weisbach Eq^n - $H_f = \frac{4 f l v^2}{2 g d} = \frac{f' l v^2}{2 g d}$

Where ,

l- length of pipe

d- dia of pipe

v- velocity of flow

f- coefficient of friction

f'- friction factor Value of friction factor varies between (0.02 to 0.075)

Power required by the pump or capacity of pump

- The horse power (H.P.) of the pump can be determined by calculating the work done by the pump in raising the water up to the height H.
- Let the pump raise W kg of water to height H meter.
- Then the work done by the pump- W x H (m.kg)
- $- \gamma Q H$ (m. kg/sec)
- **Where, γ - Unit Weight of water**
- **Q- Discharge to be pumped in m^3 /sec**
- **H- Total head in meter**
- Water Horse Power (WHP) - $\dfrac{\gamma Q H}{75}$
- Brake Horse Power (B.H.P) = $\dfrac{W.H.P}{\eta}$

$$= \frac{\gamma Q H}{75 \eta}$$

Economic diameter of rising main

- For pumping a particular fixed discharge of water, there are two options
- **It can be pumped through bigger diameter pipe at low velocity**
- **Through lesser diameter pipe at high velocity**
- ***If the dia of the pipe is increased, it will lead to higher cost of the pipe line on the other hand if the pipe diameter is reduced the velocity would increase which will lead to higher frictional head loss and will require more Horse Power for pumping,*** thereby increasing the cost of pumping, also cost of fitting will increase.
- For obtaining the optimum efficiency, it is necessary to design the diameter of the pumping main which will be overall most economical in initial cost as well as maintenance cost for pumping the required quantity of water. **The diameter which provide such optimum condition is known as "economic diameter" of the pipe.**

Design of Rising Main

- An empirical formula given by lee is commonly used for determining the dia of the pumping or rising main
- **D- 0.97 to 1.22 $\sqrt{Q}$**
- Where,
- **D- Economic dia of pipe in meters**
- **Q- Discharge to be pumped in cumecs**

Hazen Williams Equation

- $V = 0.85\ C_H R^{0.63} S^{0.54}$
- Where,
- V- velocity of flow
- S- Slope of H.G.L
- $= \frac{Hl}{L} = \frac{\text{Head Loss}}{\text{Length of Pipe}}$

R- Hydraulic mean Radius of the pipe $= \frac{A}{P}$

If pipe is running full then

$R = \frac{\pi/4\ d^2}{\pi d} = d/4$

CH- Hazen William's coefficient which depends on age, quality and material of pipe.

- Find out the head loss due to friction in a rising main from the following data:
- Length of the rising main= 600 m
- Diameter of pipe= 0.2 m
- Discharge required to be pumped = 1200 l/min
- Friction factors= 0.025

- Velocity of flow = $\frac{Q}{A}$

$Q = 1200\ l/min$

$= \frac{1200 \times 10^{-3}}{60}\ m^3/sec$

$= 0.02\ m^3/sec$

$V = \frac{0.02}{\frac{\prod (0.2)^2}{4}} = 0.637\ m/sec$

- $Hf = \frac{f' l v^2}{2gd}$

$= \frac{0.025 \times 600 \times (0.637)^2}{2 \times 9.81 \times 0,2}$

$= 1.551\ m$

- A city with 1.5 lakh population I to be supplied water at 100 lpcd from a river 1 km away. The difference in water level of sump and reservoir is 30 m. if the demand has to be supplied in 8 hr., determine the size of the main and B.H.P of the pumps required.
- Take f- 0.0075, velocity in the pipe as 2.0 m/sec and efficiency of pump as 75 %

- Population of a city= 1,50,000
- Rate of water supply= 100 lpcd
- Therefore the average demand of the town= 1,50,000 x 100
- = 15×10^6 l/day
- Maximum daily demand= 1.5 x avg demand
- = $1.5 \times 15 \times 10^6$
- = 22.5×10^6 l/day
- = 22.5 MLD

- As the full demand is to be supplied through pumps in 8 hrs
- Discharge required= $\dfrac{22.5 \times 10^6}{8}$ l/ hours
- = $\dfrac{22.5 \times 10^6 \times 10^{-3}}{8 \times 3600}$ = 0.781 m^3/sec

- The maximum velocity in the pipe is given as 2.0 m/sec
- Therefore Cross sectional area of the pipe required
- $A = \frac{Q}{v} = 0.781 = 0.39\ m^2$
- If d is the dia of the pipe then
- $\frac{\prod d^2}{4} = 0.39$

D= 0.704m

= 0.75 m

Total lift is given as 30 m

- Friction loss hf can be found by using Darcy Weisbash eq n
- $Hf = \frac{4\ f\ l\ v^2}{2\ g\ d}$

$$= \frac{4 \times 0.0075 \times 1000 \times (2.0)^2}{2 \times 9.81 \times 0.75}$$

= 8.155 m

- Thus the total lift against which the pump has to work or lift water
- - 30 + 1.155
- - 38.155m
- BHP of the pump- $\dfrac{\Upsilon Q H}{75\,\eta}$

$$- \frac{1000 \times 0.781 \times 38.155}{75 \times 0.75}$$

- 526.75 - 530 HP

1 hp(I) - 745.699872 W - 0.745699872 kW

1 hp(I) - 745.699872 W - 0.745699872 kW

- From a clear water reservoir 3 m deep and maximum water level at RL 35 m water is to be pumped to an elevated reservoir at RL 80 m at the constant rate of 9 lakh litres per hour. The distance is 2000 m. Find the economic diameter of the rising main and the water horse power of the pump. Neglect minor losses and take f-0.01

- The Discharge Q= 9 lakh per hour
- $= \dfrac{9,00,000}{1000 \times 60 \times 60} = 0.25\ m^3/sec$

Economic diameter of rising main can be found by

$D = 1.22 \sqrt{Q}$

$= 1.22 \sqrt{0.25}$

$= 0.61$ m

- Maximum Suction Head = 3 m (depth of reservoir)
- Maximum delivery head = (80- 35)= 45 m
- (Difference between maximum water level and height of elevated reservoir)
- Suction + Delivery= 3 + 45 = 48 m

- Friction Head loss can be found by
- $Hf = \dfrac{4\,f l v^2}{2\,g\,D}$

$$V = \frac{Q}{A} = \frac{0.25}{\frac{\prod (0.61)^2}{4}} = 0.855 \ \text{m/ sec}$$

$$Hf = \frac{4 \times 0.01 \times 2000 \times (0.855)^2}{2 \times 9.81 \times 0.61}$$

$Hf = 4.886$ m

The total Head = 48 + 4.886 m

H = 52.886 m

- Water horse power of Pump $= \dfrac{\Upsilon\, Q\, H}{75}$

$$= \frac{1000 \times 0.25 \times 52.866}{75}$$

= 176.22 HP

IS 10500-1991	Desirable : 5 Hz. , Permissible : 25 Hz.
Risks or effects	Visible tint, acceptance decreases
Sources	Tannins, Iron, Copper, Manganese Natural deposits
Treatment	Filtration, Distillation, Reverse osmosis, Ozonisation

Nitrate

	Desirable : 45 mg/l, Permissible : 100 mg/lit
	Methemoglobinemia or blue baby disease in infants
	Livestock facilities, septic systems, manure lagoons, Household waste water, Fertilizers, Natural Deposits,
	Ion Exchange, Distillation, Reverse Osmosis

Arsenic

IS:10500-1991	Desirable: 0.05 mg/l Permissible: No relaxation
Risks or effects	Weight loss; Depression; Lack of energy; Skin and nervous system toxicity
Sources	Previously used in pesticides (orchards) Improper waste disposal or product storage of glass or electronics, Mining Rocks
Treatment	Activated Alumina Filtration, Reverse Osmosis, Distillation, Chemical Precipitation, Ion exchange, lime softening

pH	
IS 10500-1991	Desirable :6.5 – 8.5, Permissible :No relaxation
Risks or effects	Low pH - corrosion, metallic taste High pH – bitter/soda taste, deposits
Sources	Natural
Treatment	Increase pH by soda ash Decrease pH with white vinegar / citric acid

Total Dissolved Solids (TDS)	
IS 10500-1991	Desirable : 500 mg/l , Permissible : 2000 mg/l
Risks or effects	Hardness, scaly deposits, sediment, cloudy colored water, staining, salty or bitter taste, corrosion of pipes and fittings
Sources	Livestock waste, septic system Landfills, nature of soil Hazardous waste landfills Dissolved minerals, iron and manganese
Treatment	Reverse Osmosis, Distillation, deionization by ion exchange

Hardness	
IS 10500-1991	Desirable :300 mg/l , Permissible : 600 mg/l
Risks or effects	Scale in utensils and hot water system, soap scums
Sources	Dissolved calcium and magnesium from soil and aquifer minerals containing limestone or dolomite
Treatment	Water Softener Ion Exchanger , Reverse Osmosis

Alkalinity

IS 10500-1991	Desirable : 200 mg/l , Permissible : 600 mg/lit
Risks or effects	Low Alkalinity (i.e. high acidity) causes deterioration of plumbing and increases the chance for many heavy metals in water are present in pipes, solder or plumbing fixtures.
Sources	Pipes, landfills Hazardous waste landfills
Treatment	Neutralizing agent

Iron

	Desirable : 0.3 mg/l , Permissible : 1.0 mg/l
	Brackish color, rusty sediment, bitter or metallic taste, brown-green stains, iron bacteria, discolored beverages
	Leaching of cast iron pipes in water distribution systems Natural
	Oxidizing Filter , Green-sand Mechanical Filter

Manganese

IS 10500-1991	Desirable : 0.1 mg/l , Permissible : 0.3 mg/l
Risks or effects	Brownish color, black stains on laundry and fixtures at .2 mg/l, bitter taste, altered taste of water-mixed beverages
Sources	Landfills Deposits in rock and soil
Treatment	Ion Exchange , Chlorination, Oxidizing Filter , Green-sand Mechanical Filter

Sulphate

IS 10500-1991	Desirable : 200 mg/l, Permissible : 400 mg/l
Risks or effects	Bitter, medicinal taste, scaly deposits, corrosion, laxativ effects, "rotten-egg" odour from hydrogen sulphide gas formation
Sources	Animal sewage, septic system, sewage By-product of coal mining, industrial waste Natural deposits or salt
Sulphate Treatment	Ion Exchange , Distillation , Reverse Osmosis

Chl or i de

IS 10500-1991	Desirable : 250 mg/l , Permissible : 1000 mg/l
Risks or effects	High blood pressure, salty taste, corroded pipes, fixtures and appliances, blackening and pitting of stainless steel
Sources	Fertilizers Industrial wastes Minerals, seawater
Treatment	Reverse Osmosis , Distillation, Activated Carbon

e

91	Desirable : 1.0 mg/l, Permissible : 1.5 mg/l
cts	Brownish discoloration of teeth, bone damage
	Industrial waste Geological
	Activated Alumina, Distillation, Reverse Osmosis, Ion Exchange

Ar s eni c

IS:10500-1991	Desirable: 0.05 mg/l Permissible: No relaxation
Risks or effects	Weight loss; Depression; Lack of energy; Skin and nervous system toxicity
Sources	Previously used in pesticides (orchards) Improper waste disposal or product storage of glass or electronics, Mining Rocks
Treatment	Activated Alumina Filtration, Reverse Osmosis, Distillation, Chemical Precipitation, Ion exchange, lime softening

Chromium

IS 10500-1991	Desirable : 0.05 mg/l, Permissible : No relaxation
Risks or effects	Skin irritation, skin and nasal ulcers, lung tumors, gastrointestinal effects, damage to the nervous system circulatory system, accumulates in the spleen, bones, k and liver
Sources	Septic systems Industrial discharge, mining sites Geological
Treatment	Ion Exchange, Reverse Osmosis, Distillation

Copper

IS 10500-1991	Desirable : 0.05 mg/l, Permissible : 1.5 mg/l
Risks or effects	Anemia, digestive disturbances, liver and kidney damage, gastrointestinal irritations, bitter or metallic taste; Blue-green stains on plumbing fixtures
Sources	Leaching from copper water pipes and tubing, algae treatment Industrial and mining waste, wood preservatives Natural deposits
Treatment	Ion Exchange, Reverse Osmosis, Distillation

Cyanide

IS 10500-1991	Desirable : 0.05 mg/l, Permissible : No relaxation
Risks or effects	Thyroid, nervous system damage
Sources	Fertilizer Electronics, steel, plastics mining
Treatment	Ion Exchange, Reverse Osmosis, Chlorination

lead
82
Pb
207.2

Lead

IS 10500-1991	Desirable : 0.05 mg/l, Permissible : No relaxation
Risks or effects	Reduces mental capacity (mental retardation), interference with kidney and neurological functions, hearing loss, blood disorders, hypertension, death at high levels
Sources	Paint, diesel fuel combustion Pipes and solder, discarded batteries, paint, leaded gasoline Natural deposits
Treatment	Ion Exchange, Activated Carbon , Reverse Osmosis, Distillation

zinc
30
Zn
65.39

Zinc	
IS 10500-1991	Desirable :5 mg/l, Permissible : 15 mg/l
Risks or effects	Metallic taste
Sources	Leaching of galvanized pipes and fittings, paints, dyes Natural deposits
Treatment	Ion Exchange Water Softeners, Reverse Osmosis, Distillation

Mercury	
IS 10500-1991	Desirable : 0.001 mg/l, Permissible : No relaxation
Risks or effects	Loss of vision and hearing, intellectual deterioration, kidney and nervous system disorders, death at high levels
Sources	Fungicides Batteries, fungicides Mining, electrical equipment, plant, paper and vinyl chloride Natural deposits
Treatment	Reverse Osmosis, Distillation

Total Coliform Bacteria	
IS 10500-1991	95% of samples should not contain coliform in 100 ml 10 coliform / 100ml
Risks or effects	Gastrointestinal illness
Sources	Livestock facilities, septic systems, manure lagoons Household waste water Naturally occurring
Treatment	Chlorination , Ultraviolet, Distillation, Iodination

E. coliform Bacteria	
IS 10500-1991	Nil / 100ml
Risks or effects	Gastrointestinal illness
Sources	Livestock facilities, septic systems, manure lagoons Household waste water Naturally occurring
Treatment	Chlorination , Ultraviolet, Distillation, Iodination

HEALTH EFFECTS OF CHEM CAL PARAMETERS

Parameter	BIS Guideline value (maximum allowable)	General & Health effect
Total dissolved solids	2000 mg/L	Undesirable taste; gastro intestinal irritations; corrosion or incrustation
PH	6.5-8.5	Affects mucous membrane; bitter taste; corrosion; affects aquatic life
Alkalinity	600 mg/L	Boiled rice turns yellowish
Hardness	600 mg/L	Poor lathering with soap; deterioration of the quality of clothes; scale forming; skin irritation; boiled meat and food become poor in quality
Calcium	200	Poor lathering and deterioration of the quality of clothes; incrustation in pipes; scale formation
Magnesium	100	Poor lathering and deterioration of clothes; with sulfate laxative
Iron	1.0	Poor or sometimes bitter taste, color and turbidity; staining of clothes materials; iron bacteria causing slime
Manganese	0.3	Poor taste, color and turbidity; staining; black slime

HEALTH EFFECTS OF CHEM CAL PARAMETERS

Parameter	BIS Guideline value (maximum allowable)	General & Health effect
Aluminum	0.2	Neurological disorders; Alzheimer's disease
Copper	1.5	Liver damage; mucosal irritation, renal damage and depression; restricts growth of aquatic plants
Zinc	15	Astringent taste; opalescence in water; gastro intestinal irritation; vomiting, dehydration, abdominal pain, nausea and dizziness
Ammonia	-	Indicates pollution; growth of algae
Nitrite	-	Forms nitrosoamines which are carcinogenic
Nitrate	100	Blue baby disease (methemoglobineamia); algal growth
Sulfate	400	Taste affected; laxative effect; gastro intestinal irritation
Chloride	1000	Taste affected; corrosive
Fluoride	1.5	Dental and skeletal fluorosis; non-skeletal

HEALTH EFFECTS OF CHEM CAL PARAMETERS

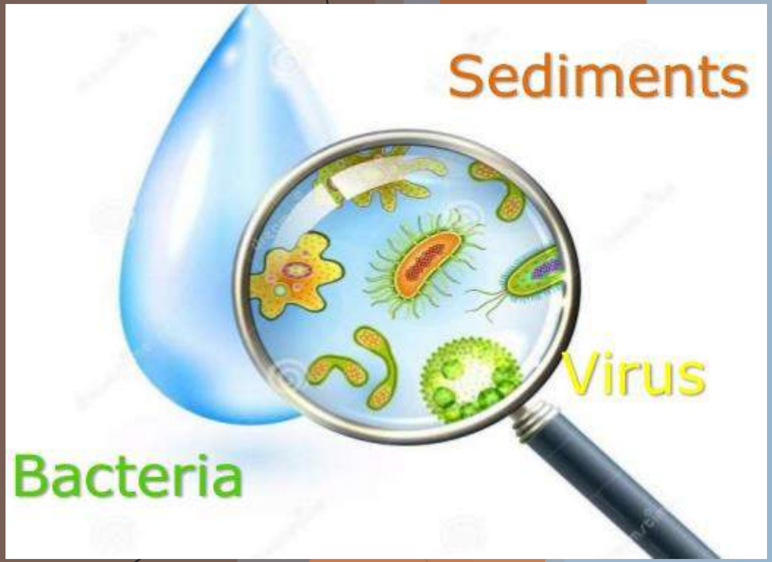
Parameter	BIS Guideline value (maximum allowable)	General & Health effect
Phosphate	-	Algal growth
Arsenic	0.05	Toxic; bio-accumulation; central nervous system affected; carcinogenic
Mercury	0.001	Highly toxic; causes 'minamata' disease-neurological impairment and renal disturbances; mutagenic
Cadmium	0.01	Highly toxic; causes 'itai-itai' disease-painful rheumatic condition; cardio vascular system affected; gastro intestinal upsets and hyper tension
Lead	0.05	Causes plumbism-tiredness, lassitudes, abdominal discomfort, irritability, anaemia; bio-accumulation; impaired neurological and motor development, and damage to kidneys
Chromium	0.05	Carcinogenic; ulcerations, respiratory problems and skin complaints
Pesticide	0.001	Affects central nervous system
Detergent	-	Undesirable foaming

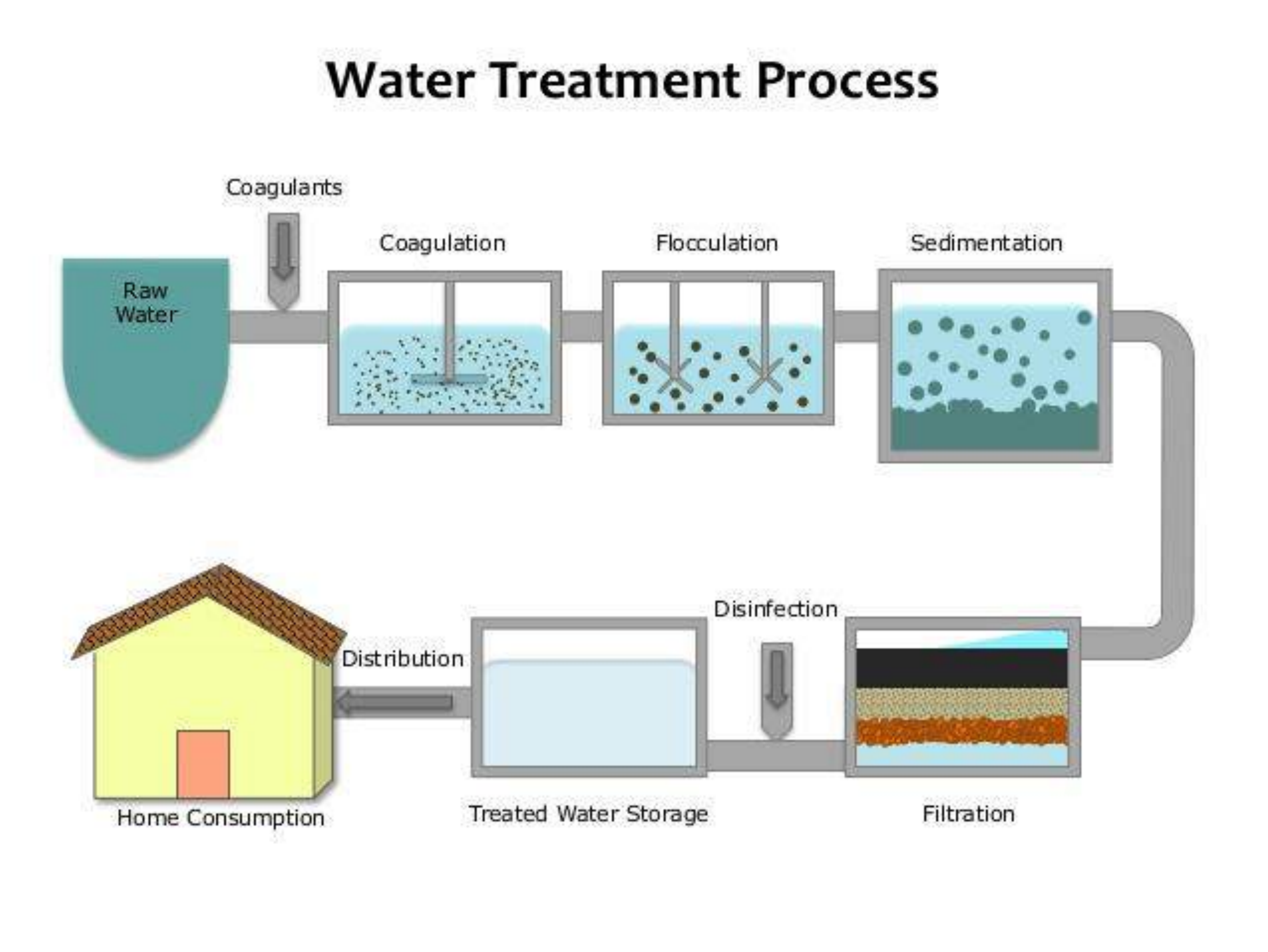
Water & wastewater treatment engineering

Module 2

Water Treatment processes

- Following are the purpose of Water treatment
- To remove color, dissolved gases and turbidity.
- To remove taste and odour
- To remove disease causing microorganisms so that water is safe for drinking purposes.
- To remove hardness of water.
- To make it suitable for a wide variety of industrial purpose such as steam generation, dyeing etc.





SCREENING

AERATION

FLOCULATION

SEDIMENTATION

FILTERATION

DISINFECTION

SOFTENING

WHAT IS SCREENING ?

- Screening is done to carry out the remove of heavy suspended solid from the water.
  like:- plants, stones, animals,trees, etc.
- Screening is generally adopted for the treatment of surface water.
- Screening is done with the help of -------
  - **1. Coarse Screen**
  - **2. Fine Screen**

COARSE SCREEN :-

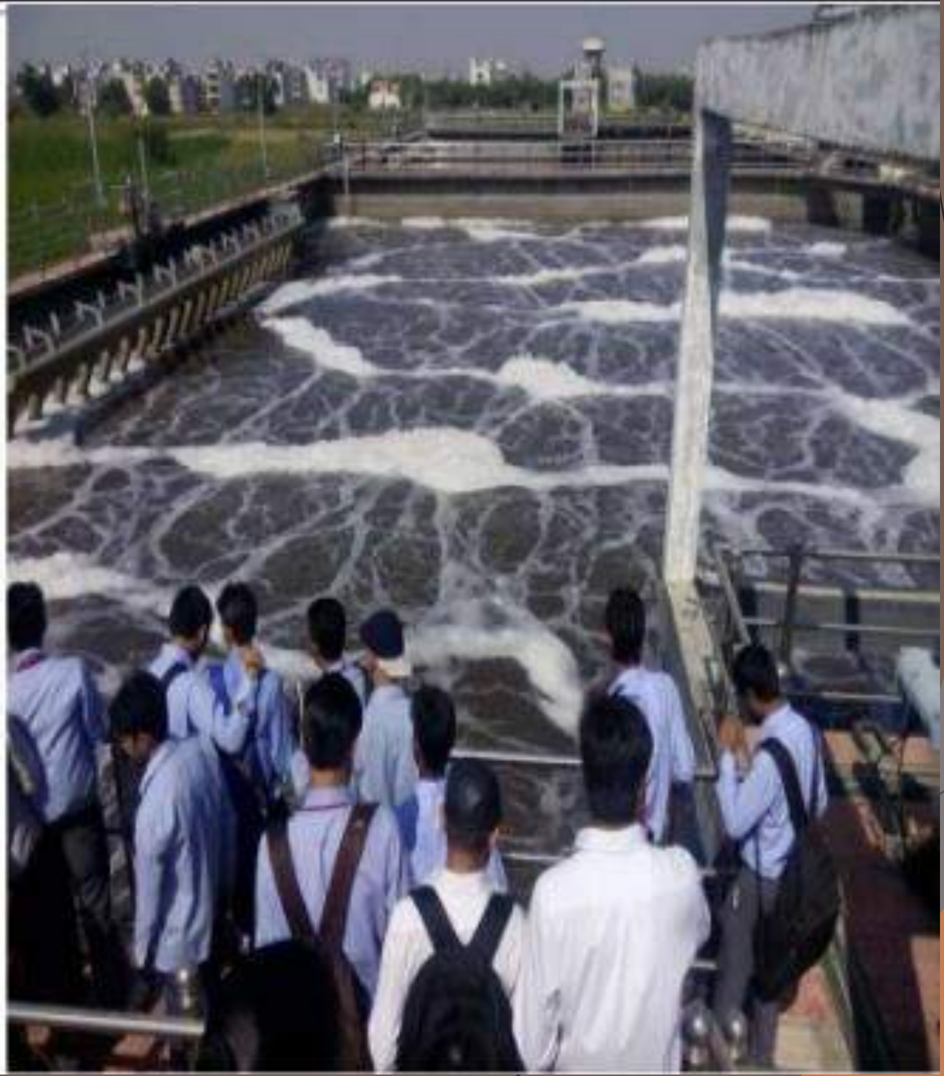
- **Coarse Screen:-** Coarse Screen in the form of bar of size **10mm** to **25mm** having sepcing of 2200mm center to center.

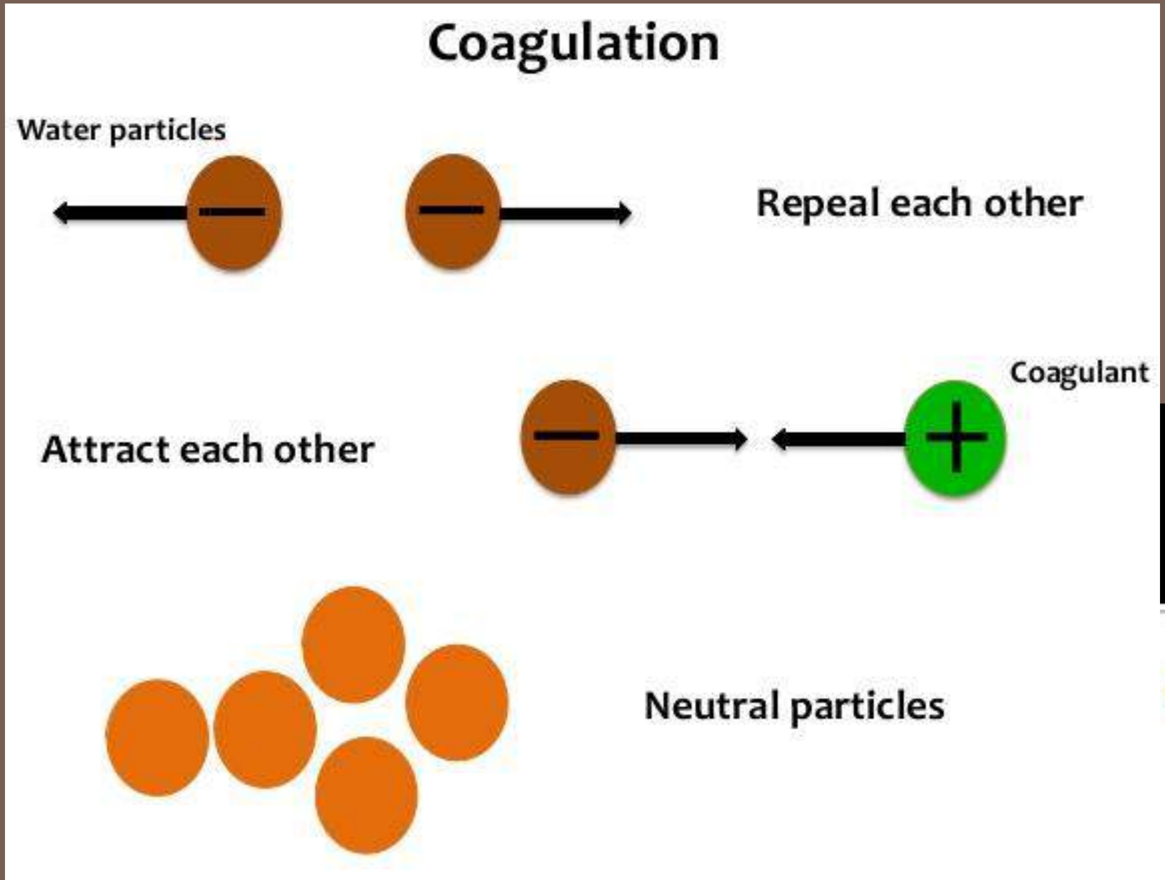
FINE SCREEN:-

- **Fine Screen:-** Fine screen iin the form of wire cruss of size 10mm

WHAT IS AERATION ?

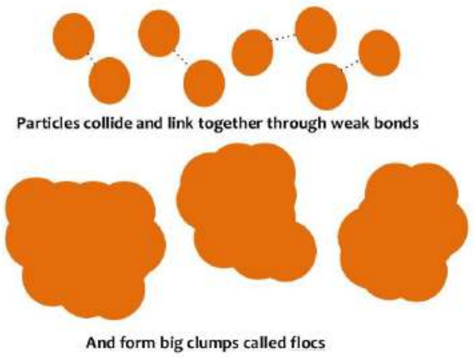
- **Aeration**:- It is the process in which water of brought intimate contact of air .
- It removes undesirable gases. CO_2 , H_2S.
- It removes undesirable organic mater.





FLOCULATION:-

- **Floculation:-** It is the process in which naturalize particle are in contact with each other , so as to promote their resulting in increased size.

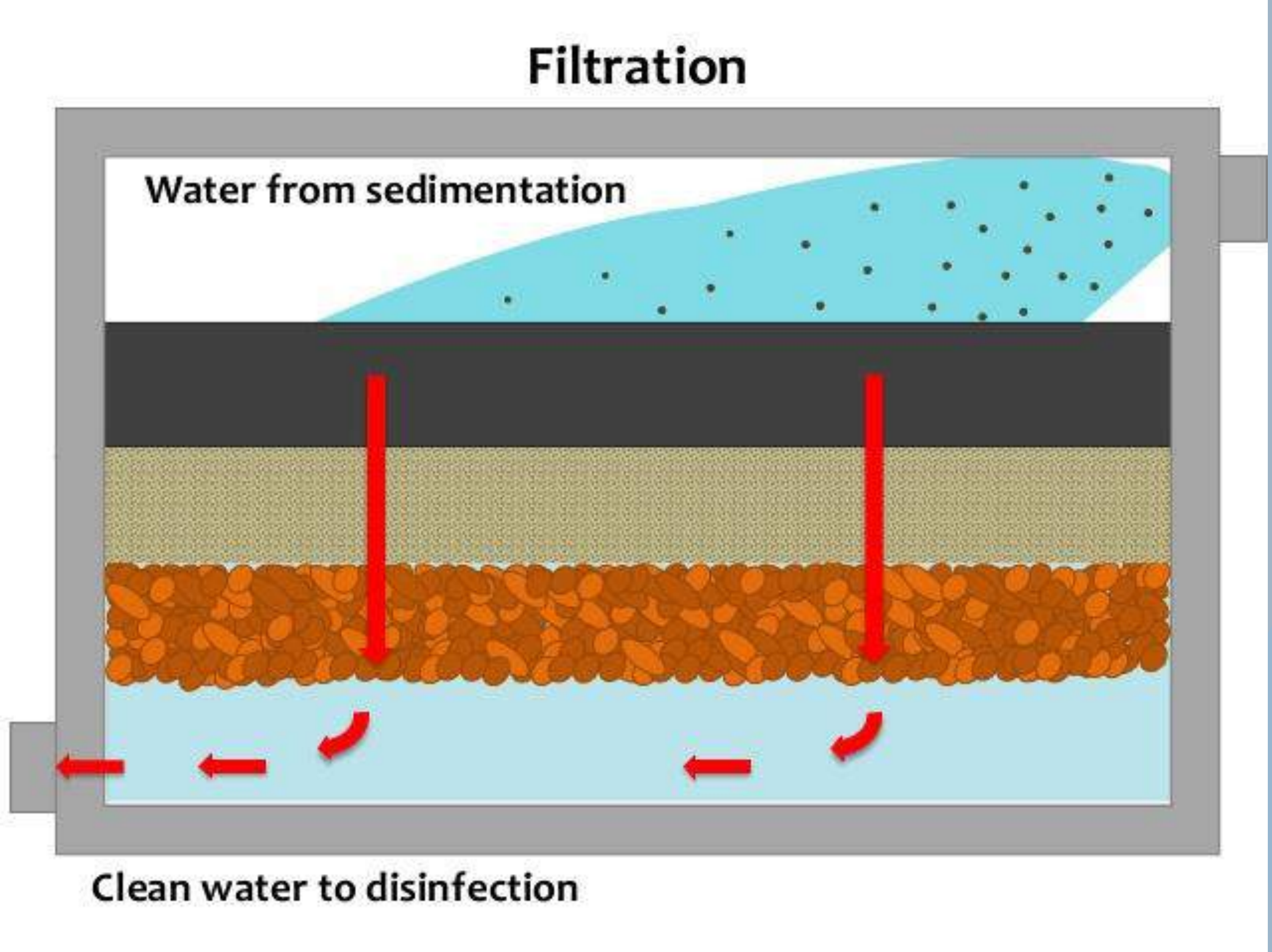


Sedimentation :-

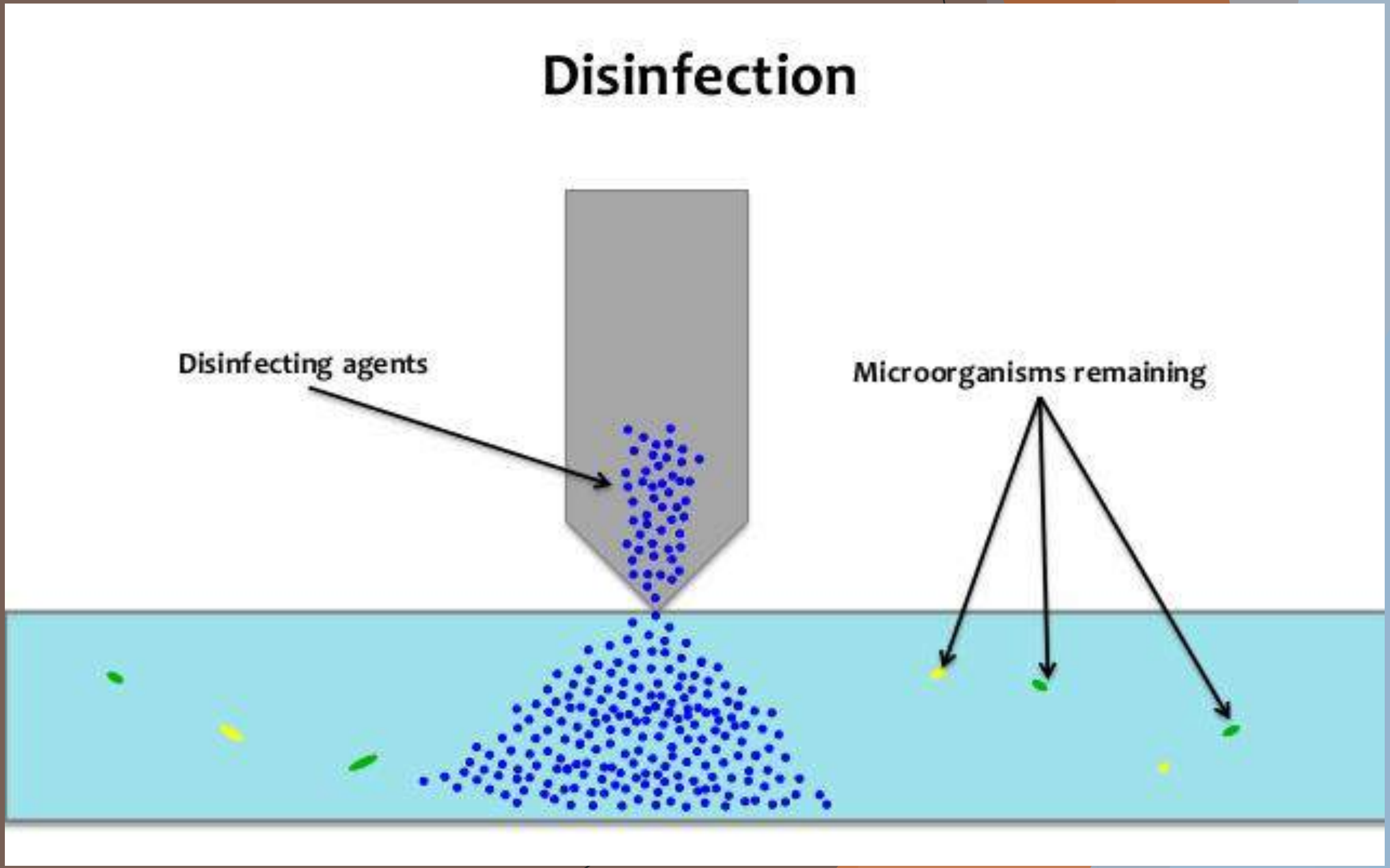
- **Sedimentation:-** Sedimentation is the process removes suspended particle form the water which could not be removes in the screening process.

FILTERATION :-

- **Filteration:-** It is carry out for the removes of fine suspended particles and flow from the water.
- Filteration also remove organic matter. Micro organism , minerals form the water,



Disinfection is the destruction or removal of all pathogenic organisms or organism capable of giving rise to infection, but not necessarily spores.



AERATION:

- Process of bringing the water in contact with excess of air or oxygen. It is one of important unit operation of gas transfer

Objectives

- It removes taste and odour caused by gases due to organic decomposition
- It increase the DO content of the water
- It remove H_2S and hence odour due to this is also removed
- It decreases CO_2 and thereby reduces corrosiveness and raises its pH
- It converts iron and manganese from their soluble states to their insoluble states, so that these can be precipitated and removed
- Due to agitation of the water during aeration, bacteria may be killed to some extent

Types of aerators

- Free fall aerators or gravity aerators
  - Cascade aerators
  - Inclined apron aerators
- Stat tray aerators
- Spray aerators
- Air diffuser basins.

Free fall aerators

Cascade aerators

- Simplest of free fall aerator, weir and waterfalls of any kind are cascade aerators.
- A simple cascade consists of a series of 3 to 4 steps of concrete or metal.
- Water is allowed to fall through a height of 1 to 3 m thereby it comes to close contact with air
- Cascade can be either in open air, or may be in room which has plenty of louvered air inlet
- CO_2 reduction usually 50 to 60%

Inclined apron aerator with riffle plates

- In this type water is allowed to fall along an inclined plain or apron which is usually studded with riffle plates in herring bone fashion
- The breaking up of sheet of water will cause agitation & consequent aeration

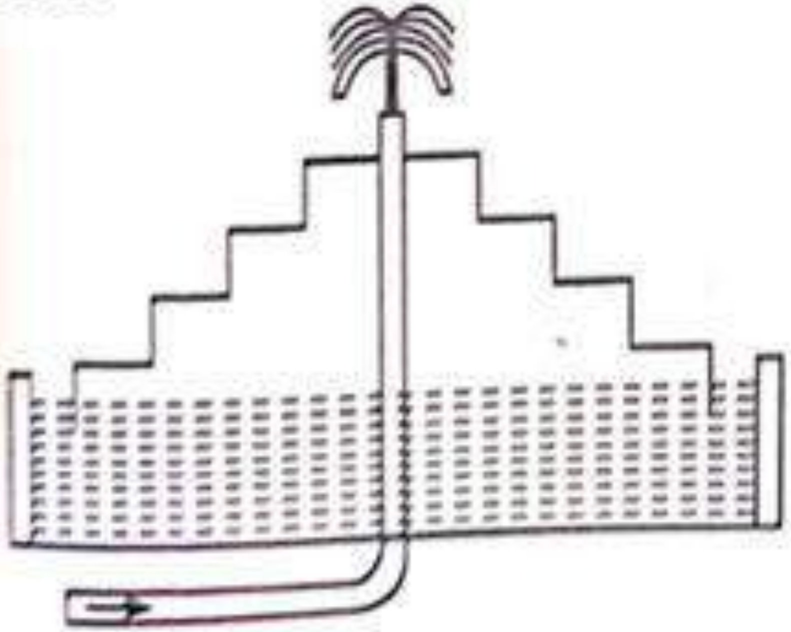


Fig. 6.5 : Circular type

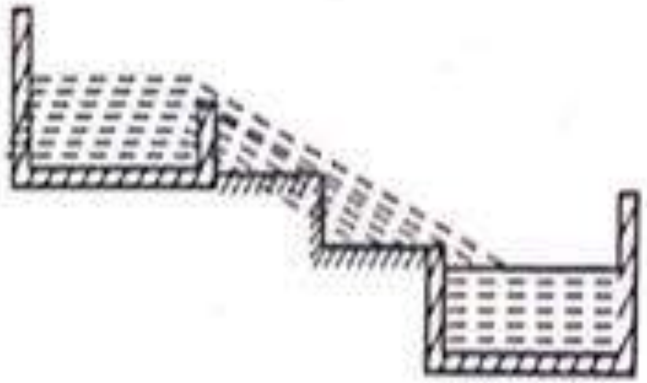


Fig. 6.6 : Straight steps

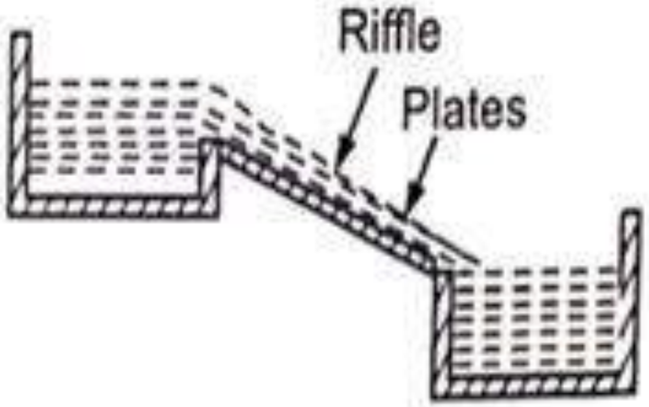
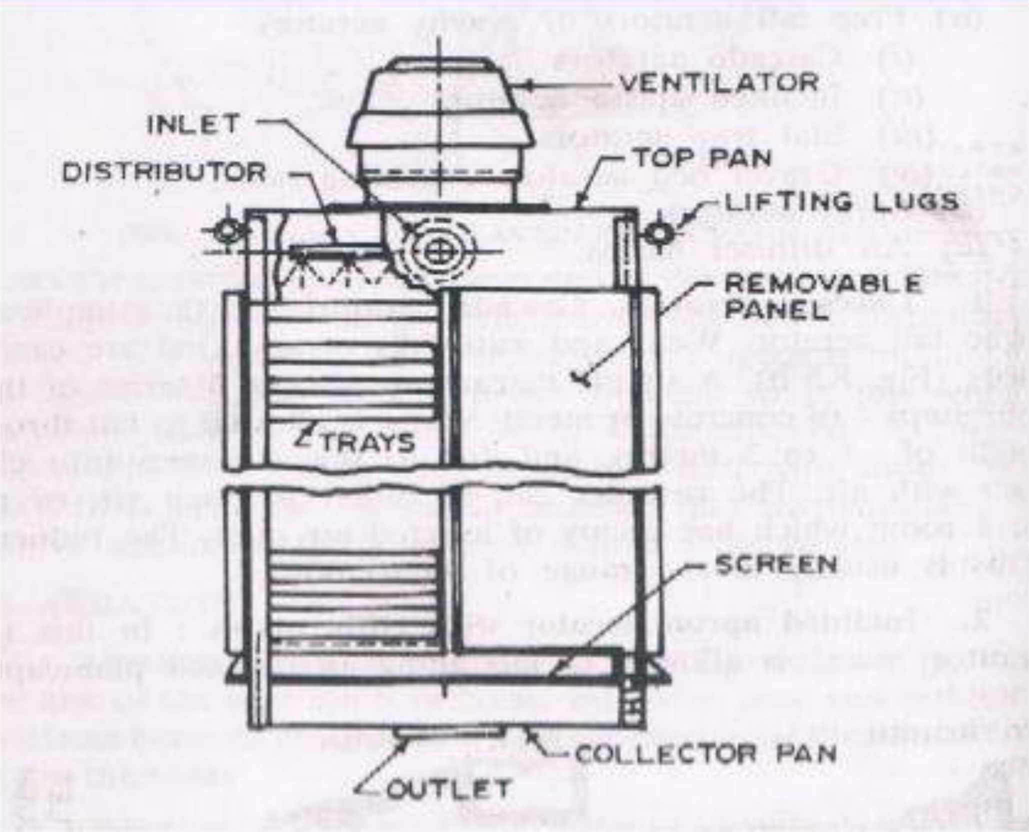


Fig. 6.7 : Inclined apron aerator

Slat tray aerators

- Most commonly used, consists of a closed round or square structure containing a series of closely stacked superimposed wood-slat trays.
- Water enters the top of the aerator & is evenly distributed over top most trays. The slats in the trays are staggered so that the films of water raining over the edges of tray falls on the centers of the slats in the tray just below.
- Air is supplied to the bottom of the aerator with the help of a blower, which blows it upward.
- Ventilator which provide at the top discharges air & gases to the atmosphere.
- Water is collected in the collector pan at the bottom, then

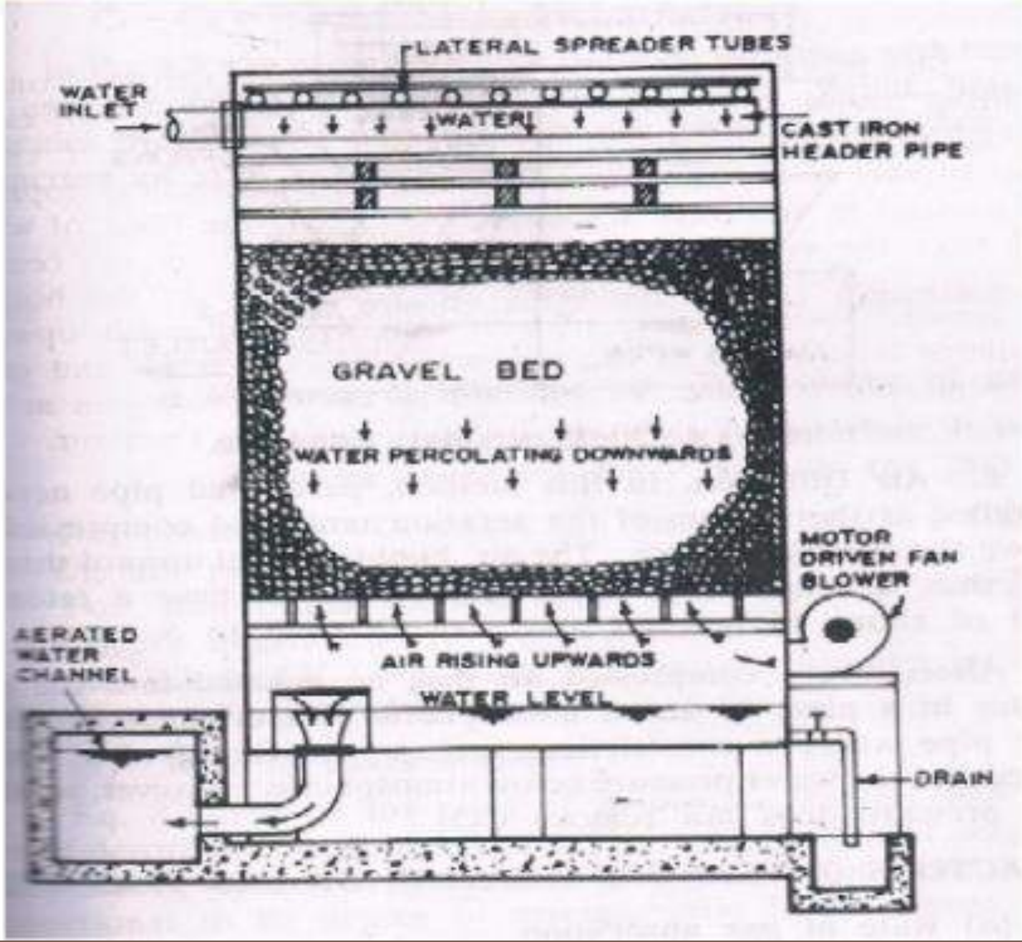
Slat Tray Aerators



Gravel bed aerator (Trickling beds)

- Cascading through beds of coke, lime stone or anthracite is believed to remove CO_2 more efficiently than other methods
- Water is applied at the top and trickles down while air is blow upwards.
- Thickness of gravel bed 1 to 1.5 m.
- Other form i.e., trickling beds contains 3 to 4 trays filled with coke, slag or stone are used
- Bed thickness 0.5 to 0.6 m, vertical distance b/n bed is 0.5 m.
- Water is applied from top through a perforated distribution pipe, during tricking process aeration takes place.

Gravel Bed Aerators



Spray aerators

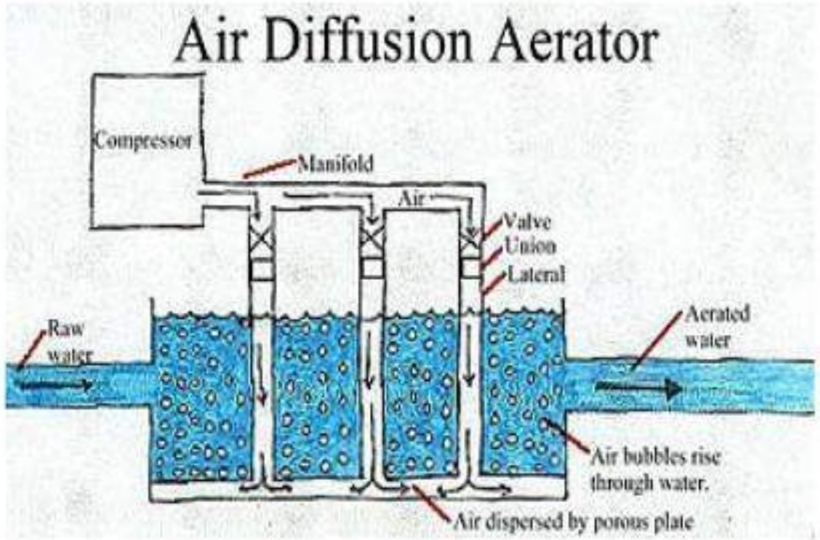
- Divides water flow into fine streams and small droplets which come into intimate contact with air in their trajectory.
- Water is sprinkled in fine jets through nozzles.
- Requires considerable head (0.75 to 1.5 kg/ cm^2), it reduces CO_2 by 70 - 90%

Air diffusion

- In this system perforated pipe network is installed at the bottom of the aeration tank & compressed air is blown through these pipes. Air bubbles travels upward through water causing aeration
- Air diffuser basin retention period is about 15 min, depth 3 to 5 m.
- Alternatively, compressed air may be injected into the flow of water in the pipe

Spray Aerators

Air diffusion



Limitations: of Aeration

- Not an efficient method of removal or reduction of taste and odour caused by relatively non-volatile substances such as oil of algae
- Odour removal is only 50% when symura was causative organism
- Taste & odours caused by chemicals due to industrial wastes discharges into receiving stream are not satisfactorily removed
- Fe & Mn can be precipitated by aeration only when organic matter is not present
- Possibility of air born contamination in water is there
- Additional lime may be required to neutralize CO_2 that would be removed by aeration
- Economical only in warmer month

SEDIMENTATION:

- **Sedimentation** is the removal of suspended particles by gravitational settling. Sedimentation tanks are designed to reduce the velocity of flow of water so as to permit suspended solids to settle out of the water by gravity.
- **Plain sedimentation:** when the impurities are separated from suspended fluid by the action of natural force alone, i.e., by the gravitational force and natural aggregation of the settling particles, the operation is called plain sedimentation
- **Sedimentation with coagulation (clarification):** when chemicals or other substances are added to induce or haste aggregation and settling of finely divided suspended matter, colloidal substances and large molecules, the operation is called sedimentation with coagulation or simply clarification

TYPES OF SEDIMENTATION TANK:

method of operation

- Quiescent or Fill and draw type
- The continuous flow type

Shape

- Rectangular tank with horizontal flow
- Circular tank with radial or spiral flow
- Hopper bottom tank with vertical flow

Fill and draw type:

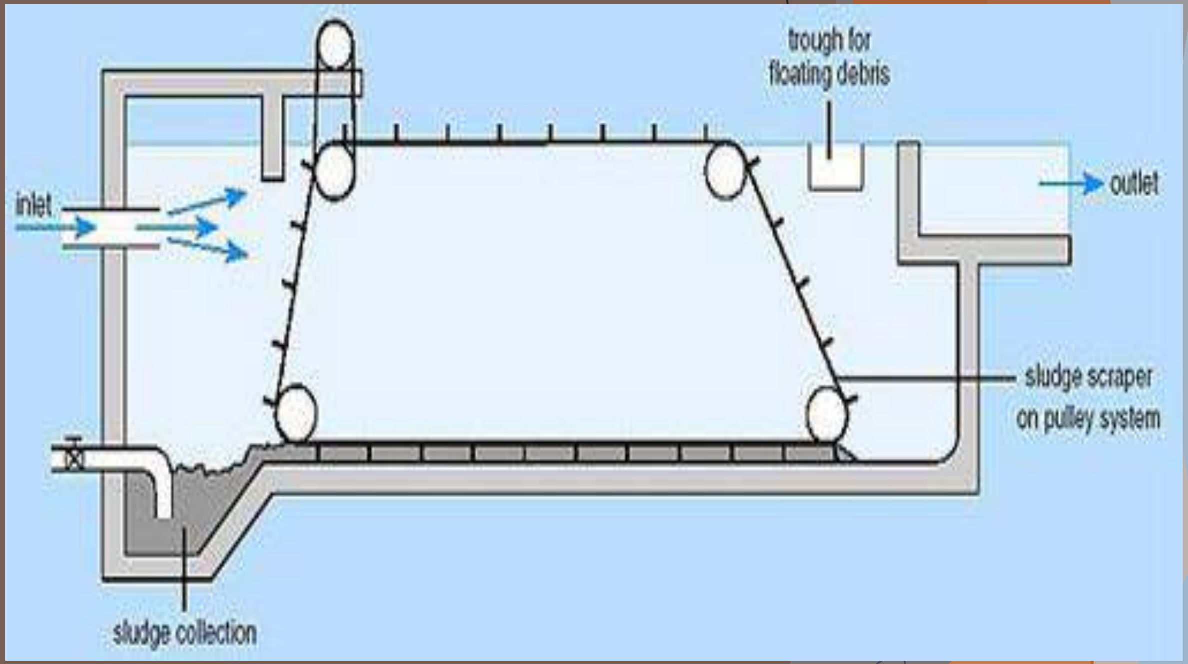
- tank is first filled with incoming water, and is allowed to rest for a certain time.
- During the rest period suspended particles settles down at the bottom of the tank, at the end of the period, the clear water is drawn off through the outlet valve.
- The tank is then cleaned of settled particles and filled again
- DT: 24 hrs., cleaning time 6 -12 hrs., hence cycles of operation takes 30 – 36 hr.
- Minimum 3 units are required to maintain constant supply

Continuous flow type tank

- Water is continuously keeps on moving in tank, through with a very with a very small velocity during which time the suspended particles settle at the bottom before they reach the outlet.
- Two types are
  - Horizontal flow tank
    - Generally rectangular in plan having L= 2W.
    - Water flows in horizontal direction, with a maximum permissible velocity 0.3 m/s
  - Vertical flow tank

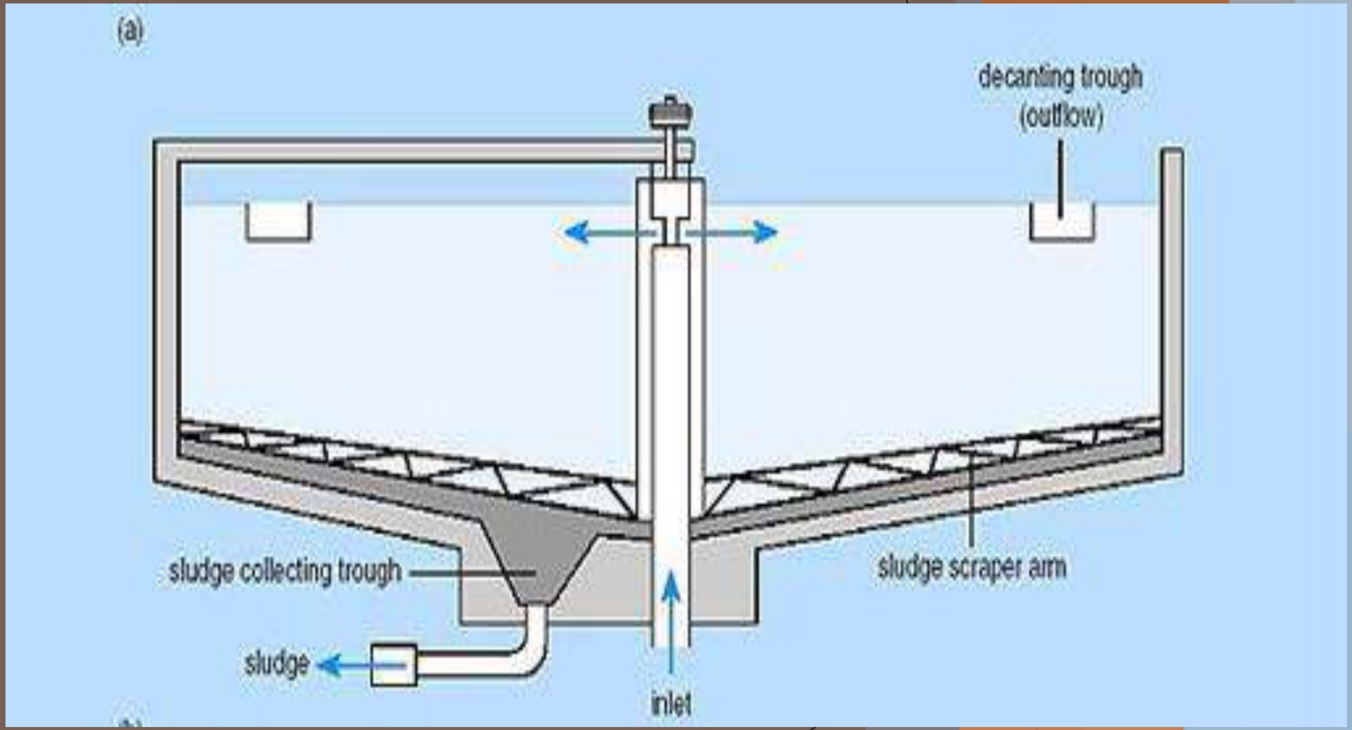
Rectangular tank with horizontal flow

- Baffles are provided to prevent short circuiting
- Figure sows rectangular tank, without baffles, but with a sludge hopper, & a sloping floor.
- Has high settling efficiency & provided with sludge removal equipment
- Sludge, scrapped by sludge scrapers & collected in hopper & removed through sludge withdrawal pipe



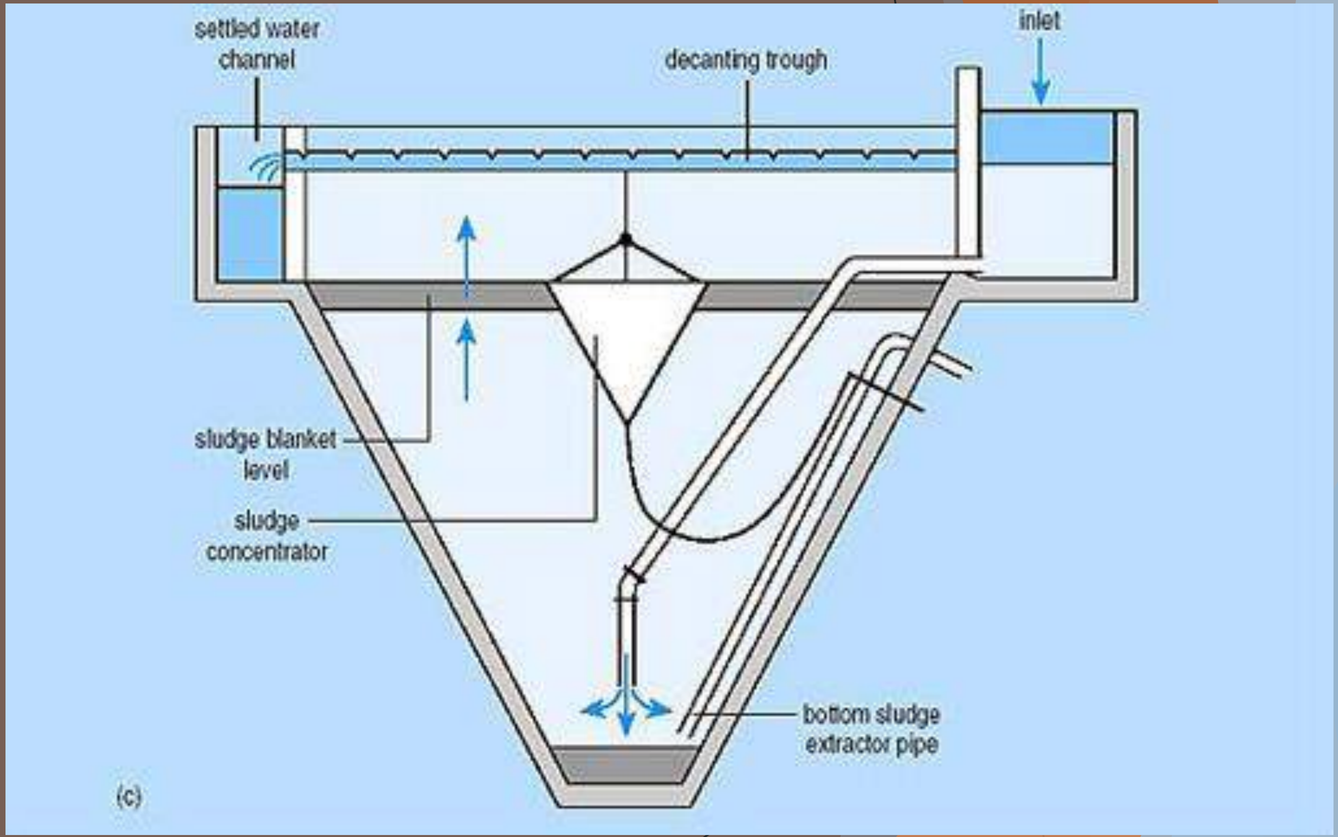
Circular tank with radial or spiral flow

- May have circular or spiral flow
- Influent enters through a central pipe & rises up to baffle ox or influent well & flows radialy (horizontally) towards the outlet provided circumferences
- Racking arms move slowly to scrap the sludge which is removed through sludge pipe connected to sludge pump



Hopper bottom tank with vertical flow

- Water enters through centrally placed inlet pipe & deflected downwards by deflector box
- Water travels vertically downwards & sludge settles at bottom of hopper removed by sludge pipe connected to sludge pump



Design Details

- **Detention period:** for plain sedimentation: 3 to 4 h, and for coagulated sedimentation: 2 to 2.5 h.
- **Velocity of flow:** Not greater than 30 cm/min (horizontal flow).
- **Tank dimensions:** L:B = 3 to 5:1. Generally L= 30 m (common) maximum 100 m. Breadth= 6 m to 10 m. Circular: Diameter not greater than 60 m. generally 20 to 40 m.
- **Depth:** 2.5 to 5.0 m (3 m).

Surface Overflow Rate: For plain sedimentation 12000 to 18000 L/d/r area; for thoroughly flocculated water 24000 to 30000 L/d/m2 tank area.

Slopes: Rectangular 1% towards inlet and circular 8%.

SETTLING VELOCITY (Streamline settling)

$$V_s = \frac{g}{18}(G-1)\frac{d^2}{\upsilon}\left(for\ d\ <\ 0.1\ mm\right)$$

Where

V_s = *Settling velocity in m/* sec

d = *diameter of the particle in m*

G = *Specific gravity of the particle*

$$= \frac{\rho_s}{\rho_w} = \frac{Density\ of\ particle}{Density\ of\ water}$$

υ = *Kinematic viscosity of water in* $m^2/\sec$

Alternatively

$$V_s = 418\,(G\text{-}1)\; d^2 \left(\frac{3T+70}{100}\right)(G-1)(for\ d < 0.1\ mm)$$

Where

V_s = *Settling velocity in m/sec*

d = *diameter of the particle in m*

G = *Specific gravity of the particle*

$$= \frac{\rho_s}{\rho_w} = \frac{Density\ of\ particle}{Density\ of\ water}$$

υ = *Kinematic viscosity of water in* $m^2/\sec$

T = *Temperature of water in* ^0C

$$V_s = 1.8\sqrt{gd\left(G-1\right)} \quad \left(for\ d \quad > \quad 1.0\ mm\ \right)$$

$$V_s = 418\ (G\text{-}1)\ \ d\left(\frac{3T+70}{100}\right)(G-1)\left(for\ d\ lies\ between\ 0.1\ mm\ and\ 1mm\right)$$

Design – Problem

- Design a coagulation-cum-sedimentation tank with continuous flow for a population of 60,000 persons with a daily percapita water allowance of 120 litres. Make suitable arrangements wherever needed.

1.*Design of Settling Tank*

Average daily consumption

$$= \textit{Population X Per capita demand}$$

$$= 60000\ X\ 120$$

$$= 7.2\ X\ 10^6\ \textit{litres}$$

Assuming that the Max. daily demand as 1.8 times the average daily demand

The maximum daily demand $= 1.8\ X\ (7.2\ X\ 10^6)$litres

$= 12.96\ X\ 10^6$ litres

Quantity of water to be treated during an assumed detention period of 4 hours

The capacity of tank required $= \dfrac{12.96\ X\ 10^6}{24} X4$

The capacity of tank required $= \dfrac{12.96 \ X\ 10^{6}}{24} X4$

$$= 2.16\,X\,10^{6}\ litres$$

$$= 2.16\,X\,10^{3}\ \mathrm{m}^{3}$$

Assuming the flow rate as 1000 litres/hr/m^2 of plan area

(ie. between 1000 to 1250 litres/hr/m^2)

$$\frac{Q}{B.L} = 1000$$

$$where\ Q = \frac{2.16X\,10^{6}}{4} = 540X10^{3}\ litre\,/\,hour$$

$$B.L =$$

Plan area is 540 m^2

Assu min *g* the width as (B) 12 m

The length will be =

Hence use a tank of 45 m X 12 m X 4 m .

Provide extra depth of 0.5 m for sludge removal at the starting end and ($\frac{45}{50} = 1.4$m) at the downstreem side.

Use a free board of 0.5 m above the water level

Example 7·3. *A settling tank is designed for an overflow rate of 40000 litres per m² per hour. What percentage of particles of diameter (a) 0·05 mm (b) 0·02 mm, will be removed in this tank at 10°C?*

Solution :

Settling velocity $v_s = \frac{Q}{A}$

Here $Q = 4000$ litres/hour

$$= \frac{4000 \times 10^{-3}}{60 \times 60} \text{ m}^3\text{/sec.}$$

$A = 1 \text{ m}^2$

$$\therefore \quad v_s = \frac{4000 \times 10^{-3}}{1 \times 3600} \text{ m/sec}$$

$$= \frac{4000 \times 10^{-3}}{1 \times 3600} \times 100 \text{ cm/sec.}$$

$$= 0{\cdot}111 \text{ cm/sec.}$$

For 0·05 mm particles :

Assume $S_s = 2{\cdot}65$

$\therefore$ $v_s' = 418\ (2{\cdot}65 - 1)\ (0{\cdot}05)^2 = 1{\cdot}72$ mm/sec

$= 0{\cdot}172$ cm/sec

$\therefore$ $\%\ \text{settled} = \dfrac{0{\cdot}172}{0{\cdot}111} \times 100 > 100\ \%$

Hence all the particles of 0·05 mm dia. will settle.

For 0·02 mm particles :

$v_s' = 418(2{\cdot}65 - 1)\ (0{\cdot}02)^2 = 0{\cdot}276$ mm/sec

$= 0{\cdot}0276$ cm/sec

$\therefore$ $\%\ \text{settled} = \dfrac{0{\cdot}0276}{0{\cdot}111} \times 100$

$= 24{\cdot}8\ \%$

Example 7·7 *Design a rectangular sedimentation basin for the following data :*

(i) *Volume of water to be treated = 3 million litres per day.*

(ii) *Detention period = 4 hours.*

(iii) *Velocity of flow = 10 cm/min.*

Solution :

$$\text{Detention time} = 4 \text{ hours}$$
$$= 240 \text{ min.}$$

$$\text{Velocity of flow} = 10 \text{ cm/min.}$$

$\therefore$ $\quad$ Length of tank $= 0{\cdot}10 \times 240 = 24$ m.

Volume of water in 4 hours

$$= \frac{3 \times 10^6}{10^3} \times \frac{4}{24} = 500 \text{ m}^3$$

$\therefore$ $\quad$ Cross-section area

$$A = \frac{500}{24} = 20{\cdot}8 \text{ m}^2$$

Assume a working depth of 3 m.

$\therefore$ Width of tank $= \dfrac{20{\cdot}8}{3} \simeq 7$ m.

Provide an extra depth of 1 m for sludge storage and 0·5 m for free board making a total depth $= 3 + 1{\cdot}5 = 4{\cdot}5$.

Hence provide a settling tank of size 24 m $\times$ 7 m $\times$ 4·5 m.

Check :

Volume of water per hour

$$= \frac{3 \times 10^6}{24}.$$

$\therefore$ Surface loading rate

$$= \left(\frac{3 \times 10^6}{24}\right) \times \frac{1}{24 \times 7}$$

$= 744$ litres/hour/m^2 which is satisfactory.

Circular sedimentation tank

$$V = D^2 (0{\cdot}011\, D + 0{\cdot}785\, H).$$

- V= volume of tank
- D= Diameter of tank
- H= depth of tank

Rectangular sedimentation tank

$$V = \frac{Q}{D.T}$$

$L = V_H \times D.T$

$W = V_S \times D.T$

$V = L \times W \times H$

$$SA = \frac{Q}{SLR}$$

$SA = L \times W$

$C/S\ A = W \times H$

Example 7·8 *Design a circular sedimentation basin for the above data.*

Solution :

A circular sedimentation tank is generally provided with its bottom cone-shaped, with a slope of 1 vertical to 12 horizontal. Under this condition, its volume V, in terms of its diameter D and height H is given by

$$V = D^2\,(0{\cdot}011\ D + 0{\cdot}785\ H).$$

Now volume of water during detention period

$$= \frac{3 \times 10^6}{10^3} \times \frac{4}{24} = 500\ \text{m}^3$$

Let the effective depth $= 3$ m.

Substituting the values in the above expression, we get

$$500 = D^2\,(0{\cdot}011\ D + 0{\cdot}785 \times 3)$$

Solving this by trial, we get

$$D \simeq 14{\cdot}2\ \text{m}.$$

COAGULANTS

- The various chemical used for secondary sedimentation in the water treatment units are called coagulants.
- These coagulants are most effective when the water is slightly alkaline.

1. **Use of ALUMN as Coagulant**

ALUMN is the name given for Aluminium sulphate with (Al2SO4)3.18H2O

$$Al_2(SO_4)_3.18H_2O + 3Ca(HCO_3)_2 \rightarrow 3CaSO_4 + 2Al(OH)_3 \downarrow + 6CO_2 \uparrow$$

2. Use of COPPERAS as Coagulants

Copperas is the name given to ferrous Sulphate with its chemical formula FeSO4.7H2O

$$i)\ FeSO_4.7H_2O + Ca(OH)_2 \rightarrow CaSO_4 + Fe(OH)_2 + 7H_2O$$

$$ii)\ Fe(OH)_2 + O_2 + 2H_2O \rightarrow 4Fe(OH)_3 \downarrow$$

3. Use of Chlorinated COPPERAS as Coagulants

When chlorine is added to solution of copperas the two react chemically so s to form ferric sulphate and ferric chloride.

$$i)\ 6FeSO_4.7H_2O + 3Cl_2 \rightarrow 2Fe_2(SO_4)_3 + 2FeCl_3 + 42H_2O$$

$$ii)\ Fe_2(SO_4)_3 + 3Ca(OH)_2 \rightarrow 3CaSO_4 + 2Fe(OH)_3 \downarrow$$

FILTRATION

- It is a solid-liquid separation process in which the liquid passes through a porous medium to remove as much fine suspended solids as possible.

THEORY OF FILTRATION

- Mechanical straining
- Flocculation and sedimentation
- Biological metabolism
- Electrolytic changes

MECHANICAL STRAINING

- The suspended particles present in water, and which are of bigger size than the size of voids in the sand layers of the filter, cannot pass through these voids and get arrested in them. Therefore, the resultant water will be free from these impurities.
- Most of the particles are removed in the upper sand layers. The arrested particles including the coagulated flocs form a mat on the top of the sand bed, which further helps in straining out the impurities.

FLOCCULATION AND SEDIMENTATION

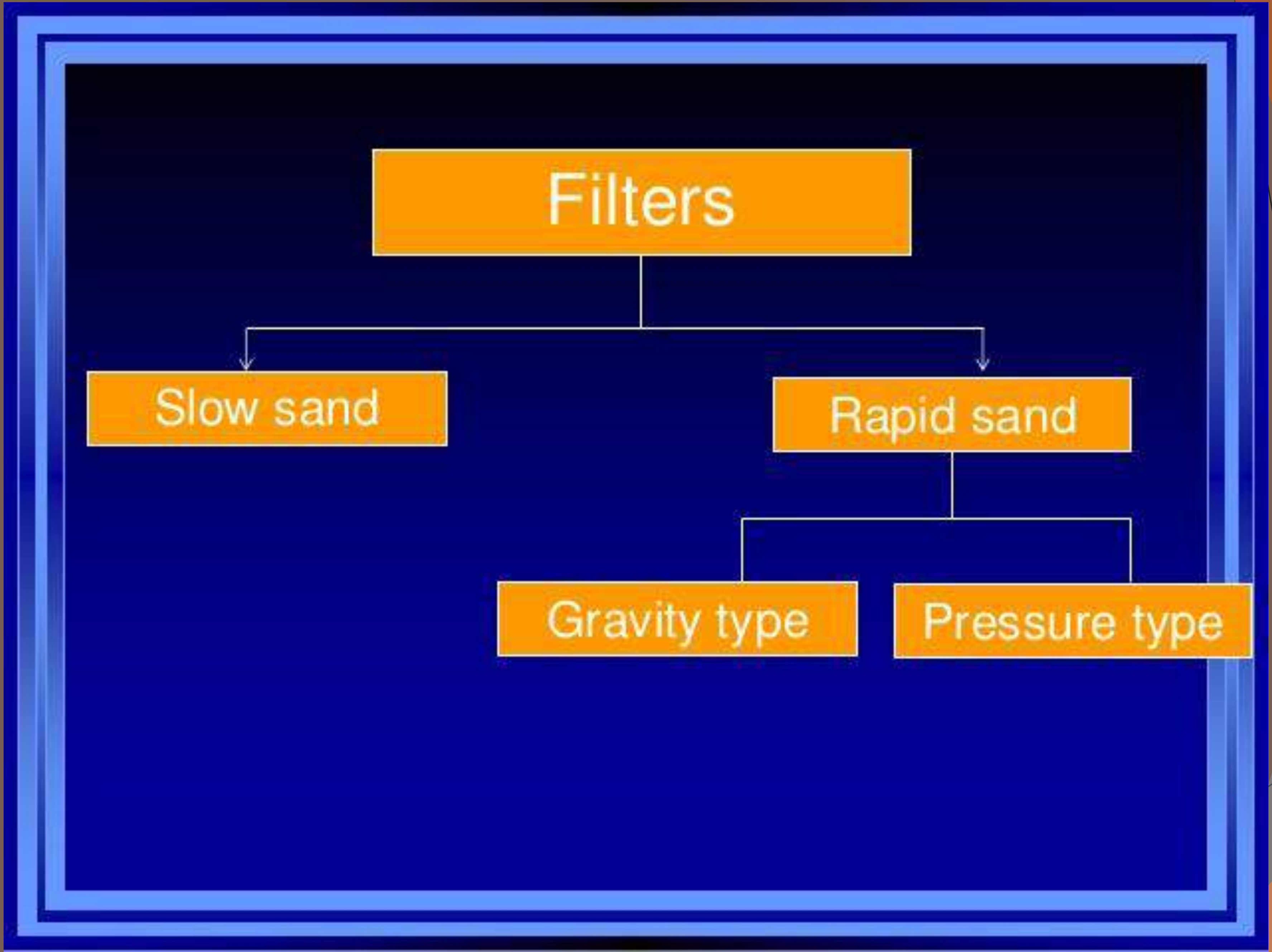
- The filters whose voids size is more than the size of particles, also able to remove such particles.
- This fact is possible by assuming the voids spaces acting as tiny coagulation-sedimentation tanks.
- The colloidal matter arrested in these voids is a gelatinous mass and , therefore, attract other finer particles. These finer particles settle down in the voids and get removed.

BIOLOGICAL METABOLISM

- Generally micro-organisms and bacteria are reside in voids as coatings over sand grains during the initial process of filtration. And, these organisms use organic impurities as their food and convert them into harmless compounds.
- Such harmless compounds form a layer on the top, which is called schutzdecke or dirty skin.
- This layer further, helps in absorbing and straining out the impurities.

ELECTROLYTIC CHANGES

- The purifying action of filter can also be explained by the theory of ionisation.
- This may be explained by that, the sand grains of filter media and the impurities in water, carry electrical charges of opposite nature.
- When these oppositely charged particles and the impurities come in contact with each other, they neutralise each other, there by changing the character of water and making it purer.
- After a certain interval, the electrical charges of sand grains get exhausted, and have to be restored by cleaning the filter.


Filters
Slow sand
Rapid sand
Gravity type
Pressure type

CONSTRUCTION OF SLOW SAND FILTERS

- The various parts of the slow sand filter are:
  - Enclosure tank
  - Filter media
  - Base material
  - Under drainage system
  - Inlet

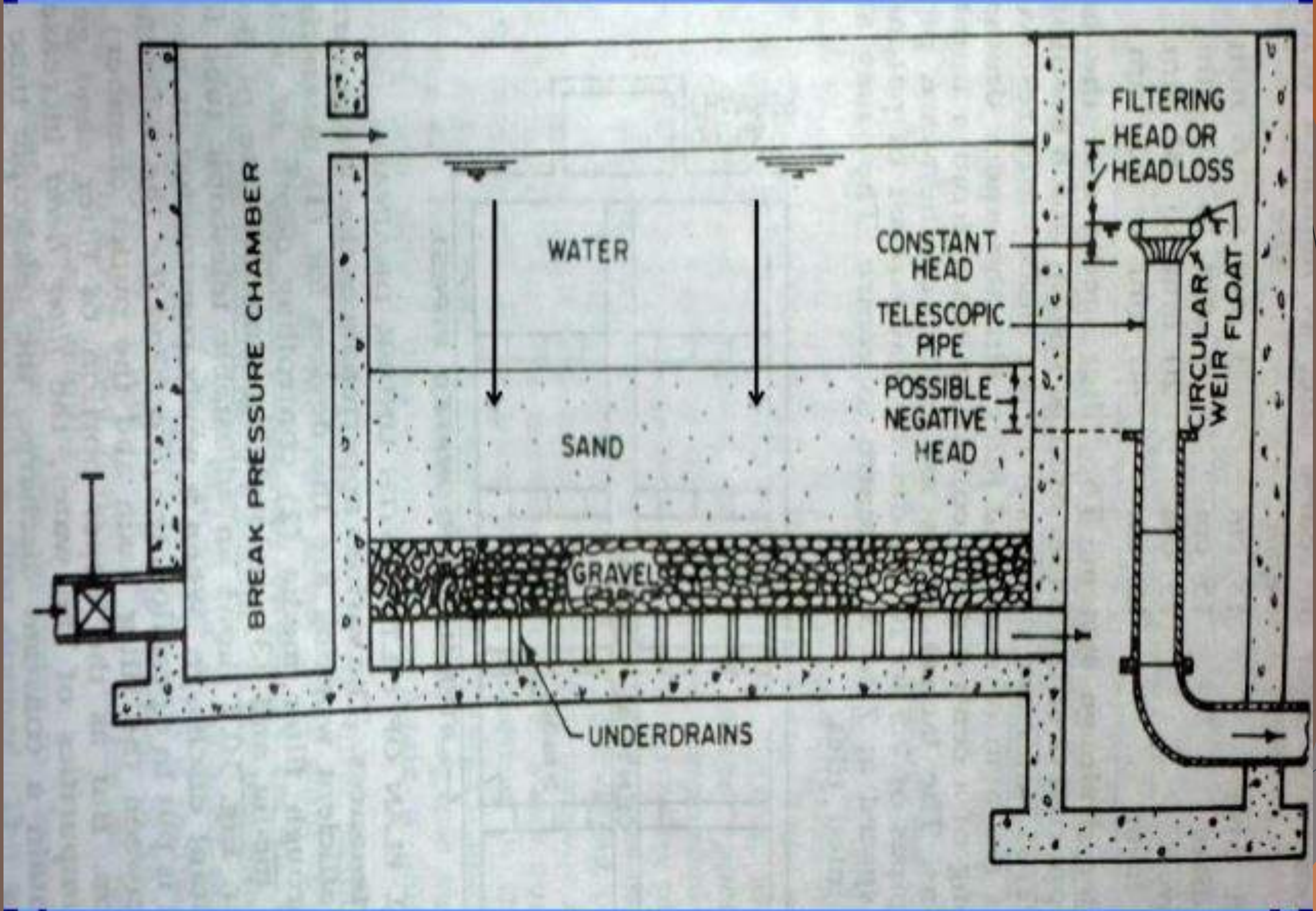
SLOW SAND FILTER

Enclosure tanks:

- An open water tight rectangular tank, made of masonry or concrete
- The bed slope is kept at about 1 in 100 towards the central drain
- The depth of tank is 2.5m to 3.5m
- The plan area of tank – 100 t0 2000 sq.km.

Filter media:

- The filter media consists of sand layer about 90 to 110 cm in depth and placed over a gravel support.
- The effective size of sand varies from 0.2 to 0.4mm and uniformity coe. Varies from 1.8 to 2.5


BREAK PRESSURE CHAMBER
WATER
SAND
GRAVEL
UNDERDRAINS
FILTERING HEAD OR HEAD LOSS
CONSTANT HEAD
TELESCOPIC PIPE
POSSIBLE NEGATIVE HEAD
CIRCULAR WEIR
FLOAT

Filter media:

- The filter media consists of sand layer about 90 to 110 cm in depth and placed over a gravel support.
- The effective size of sand varies from 0.2 to 0.4mm and uniformity coefficient Varies from 1.8 to 2.5
- The coarser layer should be at bottom and finer layer should be at top. Top layer should contain uniform in grain size

Base material:

- The base material is gravel, and it supports the sand. It consists of 30 to 75mm thick, with gravels of different sizes, which placed in 3 to 4 layers.
- Thickness of each layer around 15 to 20cm are used.
- The coarsest gravel is used in the bottom most layer and the finest layer is used in the top most layer.
- The size of gravel in each layer should be as :

  bottom most layer – 40-65 mm

  Intermediate layers – 20-40mm & 6-20mm

  Top most layer – 3-6mm

Under drainage system:

- The gravel support is laid on the top of an under drainage system. The under drainage system consists of a central drain and lateral drains.
- The lateral are open jointed pipe drains or some other kind of porous drains placed 3 to 5m apart on the bottom floor and sloping towards a main central covered drain.
- The laterals collect the filtered water and discharge it into the main drain, which leads the water to the filtered water well.

OPERATION OF SLOW SAND FILTER

- The effluent from the sedimentation tank is allowed to enter the inlet chamber of the filter unit and get distributed uniformly over the filter bed.
- The water percolates through the filter media and gets purified.
- Through gravels, the water comes out as filtered water. It gets collected in the layers through the open joints, which into the main drain.
- The main drain finally discharges into the filtered water well.

LIMITATIONS TO OPERATION

- The water entering the slow sand filter should not be treated by coagulants.
- The depth of water on the filter should also be decided and should not be allowed to under go large variations.
- The filter head is generally limited to 0.7 to 1.2m.

CLEANING OF SLOW SAND FILTER

- For cleaning slow sand filters, lot of manual labour is required and also small quantities of wash waters are needed.
- The cleaning is done by
  a) scrapping and removing the 1.5 to 3cm of top sand layer and
  b) The amount of wash water required is very small, for filtering
- Cleaning is repeated until the sand depth is reduced to about 40cm or so.
- The interval between two successive cleanings, depends upon
  i. Nature of impurities and
  ii. Size of filter media

  This interval normal ranges between one to three months.

SLOW SAND FILTERS

Rate of Filtration:

The rate of filtration for slow sand filters ranges between 100 to 200 litres/ hour/ sq. m. of filter area.

Efficiency and Performance:

- Highly efficient in removing bacteria and suspended matter, and also removes colours and turbidity up to 50 mg/l.
- The extent of bacteria removal is up to 98 to 99%

Disadvantages:

- Because of their smaller rate of filtration, they require huge surface areas, and large volumes of filtering materials.
- This makes them costly and uneconomical.

Enclosure tank

- Smaller in size, therefore can be placed under roof.
- Rectangular in shape and constructed of concrete or masonry.
- Depth – 2.5 to 3.5
- **Surface area – 20 to 50 m^2.**
- **L/B ratio – 1.25 to 1.35.**
- **Designed filtration rate are 3000 to 6000 lit/m^2/hr**

Filter media

- Should be free from dirt, organic matter and other SS.
- It should be hard and resistant.
- Depth of sand media – **0.6 to 0.9 m**
- Effective size – **0.35 to 0.6 mm** (Common value 0.45)
- Uniformity coefficient – **1.2 to 1.7** (Common value -1.5)

Base material

- Depth 45 to 60 cm

Layer	Depth	Size in mm
topmost	15 cm	3 to 6
Intermediate	15 cm	6 to 12
Intermediate	15 cm	12 to 20
Bottom	15 cm	20 to 50

Under drainage system

- Objectives of under drainage system

1. To collect filtered water uniformly over the area of gravel bed
2. It provides uniform distribution of back wash water without disturbing or upsetting gravel layer and filter media

Example 8·1 *A city has a population of 20,000 with an averag rate of demand as 150 litres per capita per day. Find the area of slow sand filters.*

Solution:

Max. daily demand $= 1\cdot5 \times 20{,}000 \times 1{:}0 = 450{,}0000$ litres.

Let us assume the average rate of filtration as 150 litres per hour per m^2 of filter area.

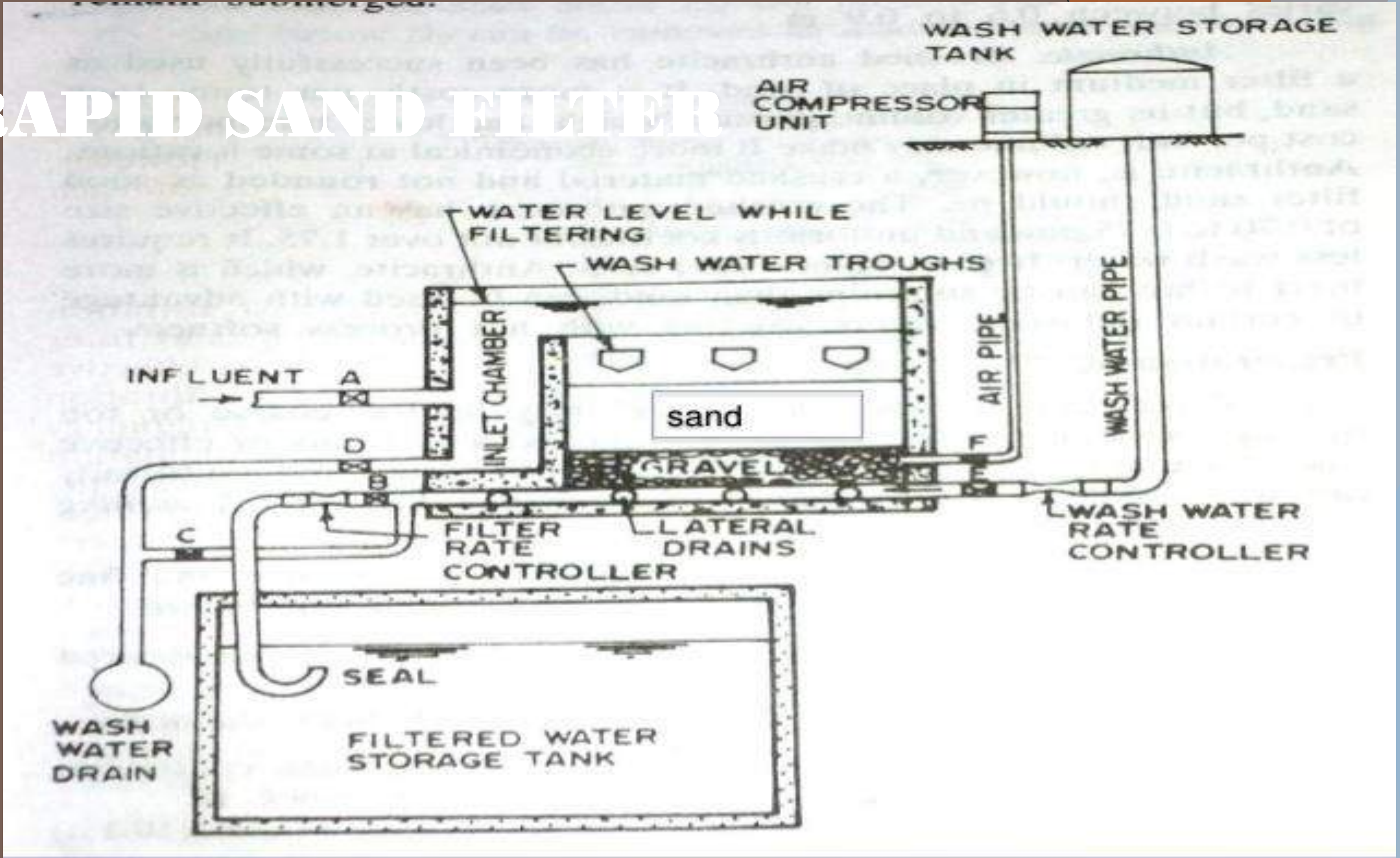
$$\therefore \quad \text{Area of filters} = \frac{450{,}0000}{150 \times 24} = 1250\ m^2.$$

Let us provide each filter unit of size $20 \times 12\cdot5$ m, giving an area of 250 m^2. Total number of filter units will be 5. Hence

provide one unit as standby, making a total of 6 units of each $20 \times 12\cdot5$ m.

SAND FILTER : GRAVITY

RAPID SAND FILTER



Working

- All valves are kept closed except valves A and B.
- Valve A is opened to permit water from clarifier
- Valve B is opened to carry filtered water to clear water sump
- **Head of 2m over sand bed is maintained**
- **Designed filtration rate are 3000 to 6000 lit/m²/hr**

- Filter run depends on quality of feed water
- Filter run may range between less than a day to several days
- **Objective of backwash is to remove accumulated particles on the surface and within the filter medium**
- Backwash is performed using wash water or air scouring.

Back washing

- Filter is back washed when head loss through it has reached the maximum permissible.
- RSF are washed by sending air and water upwards through the bed by reverse flow through the collector system.
- **2% - 4% filtered water is used for backwashing**

Steps in back washing

1. Close influent valve A
2. Close effluent valve B
3. Open air valve F, so that air blows at rate of **1 to 1.5 m^3 free air /min/m^2** of bed area for **@ 2 to 3 min**. this will break up the scum and loosen the dirt.
4. Close the air valve F and open the wash water valve E gradually to prevent the dislodgement of finer gravel.
5. Open the wastewater valve D to carry wash water to drain. Continue backwashing till wash water appears fairly clear.
6. Close the wash water valve E. Close the wastewater valve D. wait for some time till all matter in bed settles down.
7. Open valve A slightly, open valve C for carrying filtered water to drains for few minutes.
8. Close the valve C and open valve B. Open valve A completely to resume normal filtration

Example 8·2 *A city has a population of 100,000 with an average rate of demand of 160 litres per head per day. Find the area of rapid sand filters.*

Solution :

Max. daily demand $= 100{,}000 \times 160 \times 1{\cdot}5 = 240{,}00{,}000$ litres.

Let us assume an average filtration rate of 4500 litres per hour per m^2 of filter area.

$$\therefore \text{ Area of filters} = \frac{24000000}{4500 \times 24} = 222{\cdot}2 \text{ m}^2$$

Let the size of each filter unit be $9 \text{ m} \times 5 \text{ m}$.

$$\therefore \text{ No. of units required} = \frac{222{\cdot}5}{9 \times 5} \simeq 5$$

Keeping one unit as standby, provide a total of 6 units. These may be arranged in series with 3 units on either side.

Example 8·3 *A flat bottom trough is to receive the wash water*

DIFFERENCE BETWEEN SSF & RSF

Item	Slow sand Filter	Rapid sand Filter
Rate of filtration	100 to 200 lit/m^2/hr	3000 to 6000 lit/m^2/hr
Loss of head	15 cm initial to 100 cm final	30 cm initial to 3 m final
Surface area	large	small
Coagulation	Not required	required
Filter media of sand	**Effective size – 0.2 to 0.35 and C_u = 2 to 3 Depth – 105 cm**	**Effective size – 0.35 to 0.60 and C_u = 1.2 to 1.7 Depth – 75 cm**
Base material of Gravel	Size – 3 to 65 mm Depth – 30 to 75 cm	Size – 3 to 40 mm Depth – 60 to 90 cm
Under drainage system	Split tile laterals or perforated pipe laterals	Perforated of laterals with nozzles or strainer system
Method of cleaning	Scrapping of top layer 15 to 25 mm	Backwashing with compressed air and water
Amount of wash water	0.2 to 0.6 % of filtered water	2 to 4 % of filtered water

Period of cleaning	**1 to 2 months**	**2 to 3 days (24 hrs usually)**
Penetration of SS	Superficial	Deep
Further treatment needed	Chlorination	Chlorination
Efficiency	Very efficient in bacterial removal but can not remove colour and turbidity	Less bacterial removal efficiency but can remove colour and turbidity
Economy	High initial cost	Less initial cost

Flexibility	**Not flexible in meeting variations in demand**	**Quite flexible in meeting variations in demand**
Skilled supervision	Not required	Required as it involves backwashing
Depreciation cost	Relatively low	Relatively high

PRESSURE FILTER

- Pressure filters are of the same construction as gravity-type filters but the filter bed together with the filter bottom is enclosed in a watertight steel pressure vessel. The driving force for the filtration process here is the water pressure applied on the filter bed.
- Pressure filters are commercially available as complete units. They are not so easy to install, operate and maintain, particularly as it is not readily possible to inspect the condition of the media.
- For this reason they are not very well suited for application in small treatment plants in developing countries.
- **Pressure applied is 3 to 7 kg/cm^2**
- **Dia. For verticals – 2 to 2.5 m**

For horizontals – 2.5 to 8 m

- **Rate of filtration 6000 to 15000 $lit/m^2/hr$**

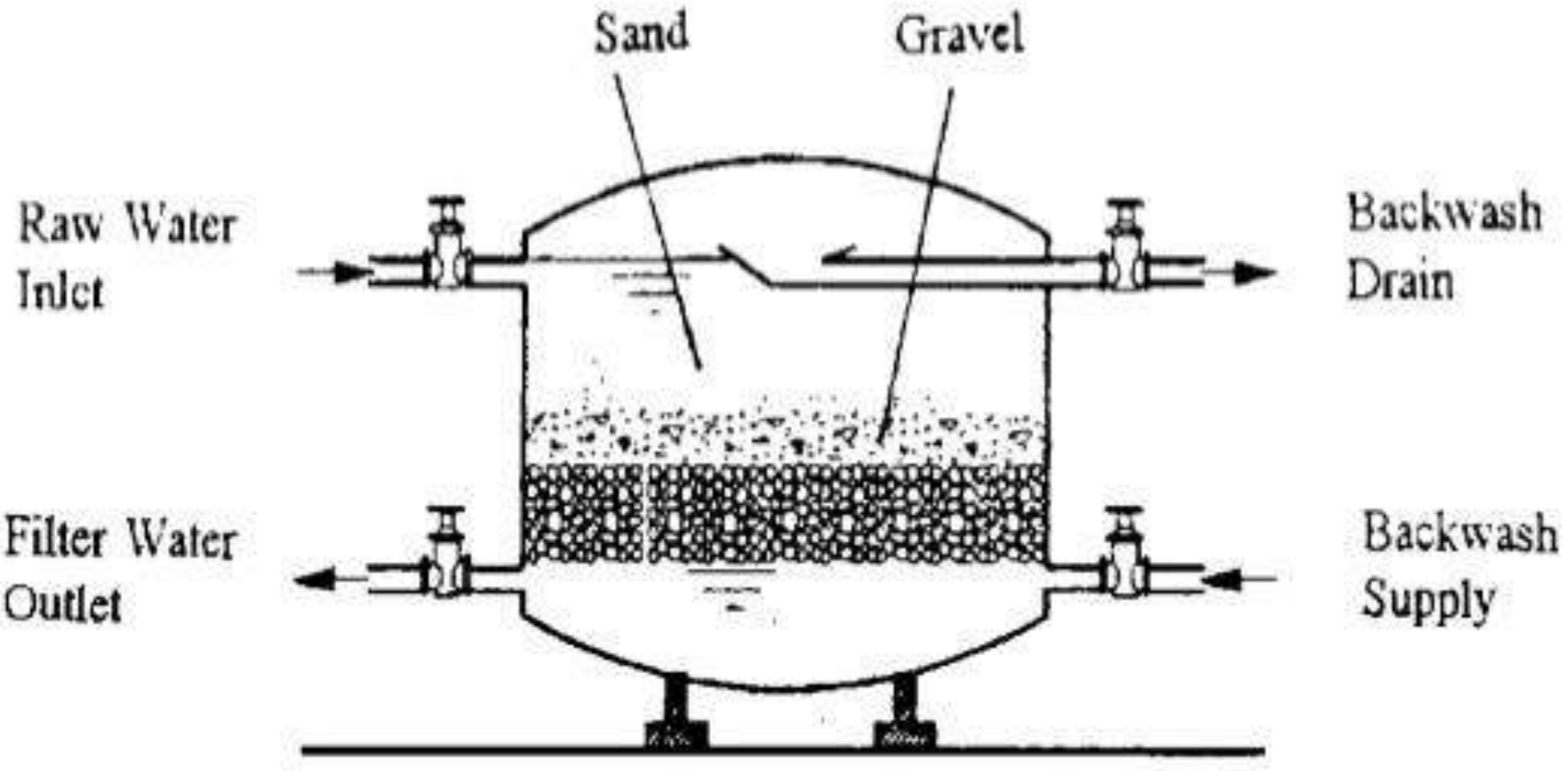


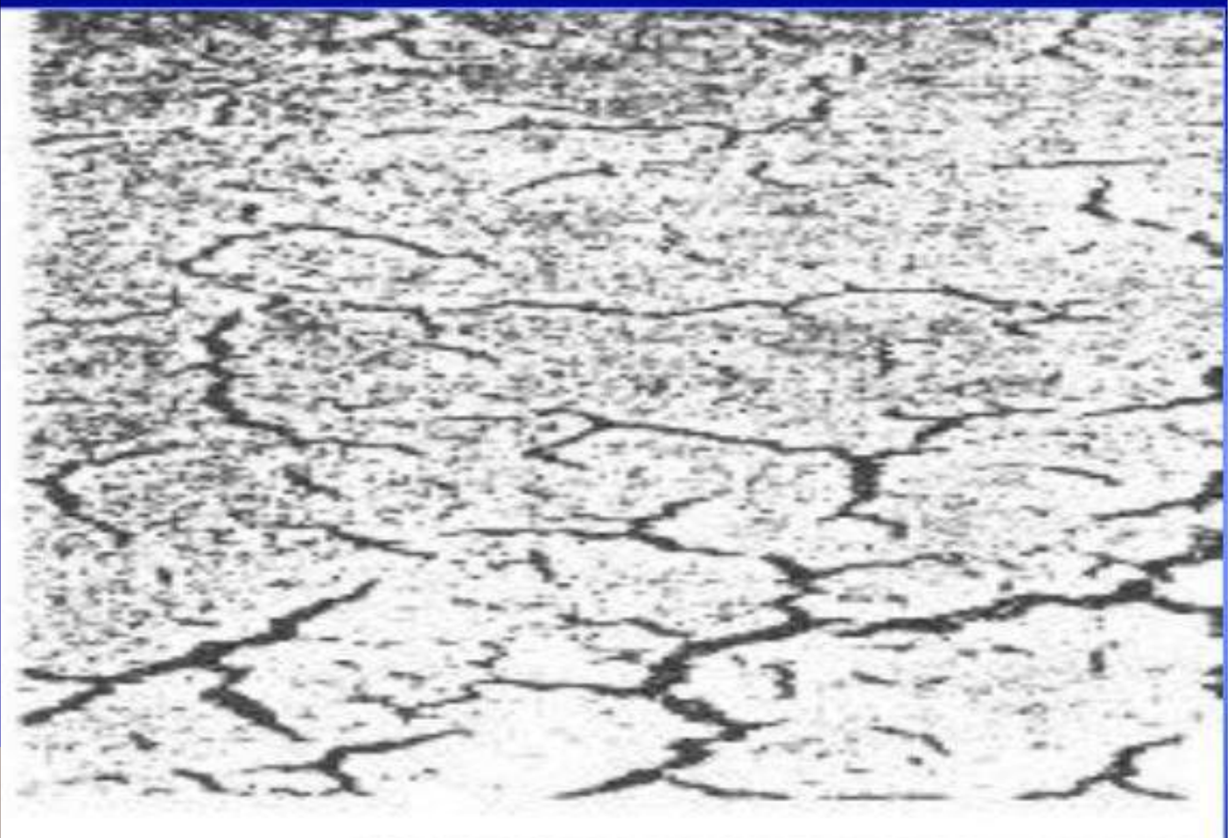
Figure 2. Pressure filter

Following filter troubles are commonly observed

1. Cracking and clogging of filter bed
2. Formation of mud balls
3. Air binding
4. Sand Incrustation
5. Jetting and Sand boils
6. Sand leakage

1. Cracking and clogging of filter bed

- Surface clogging and cracking are usually caused by rapid accumulation of solids on the top of filter media.
- Cracks are more at wall junctions.



Cracks in the Filter Bed

2. Formation of mud balls

- Mud balls are formed because of conglomeration of turbidity, floc, sand and other binders.
- Formed because of insufficient washing of sand grains.
- Size may be pea size to 2 to 5 cm or more in dia.

3.Air binding

- It is caused by release of dissolved gases and air from water to form bubbles.
- These bubbles occupy void space of the filter media sand and drainage system.
- It is caused by negative head loss, warm water and increased DO in water.
- It can be minimized by avoiding excess head loss, warming of water, control of algal growth and avoiding super saturation of water with air.

4. Sand Incrustation

It occurs due to accumulation of sticky gelatinous material or crystallization of calcium carbonate.

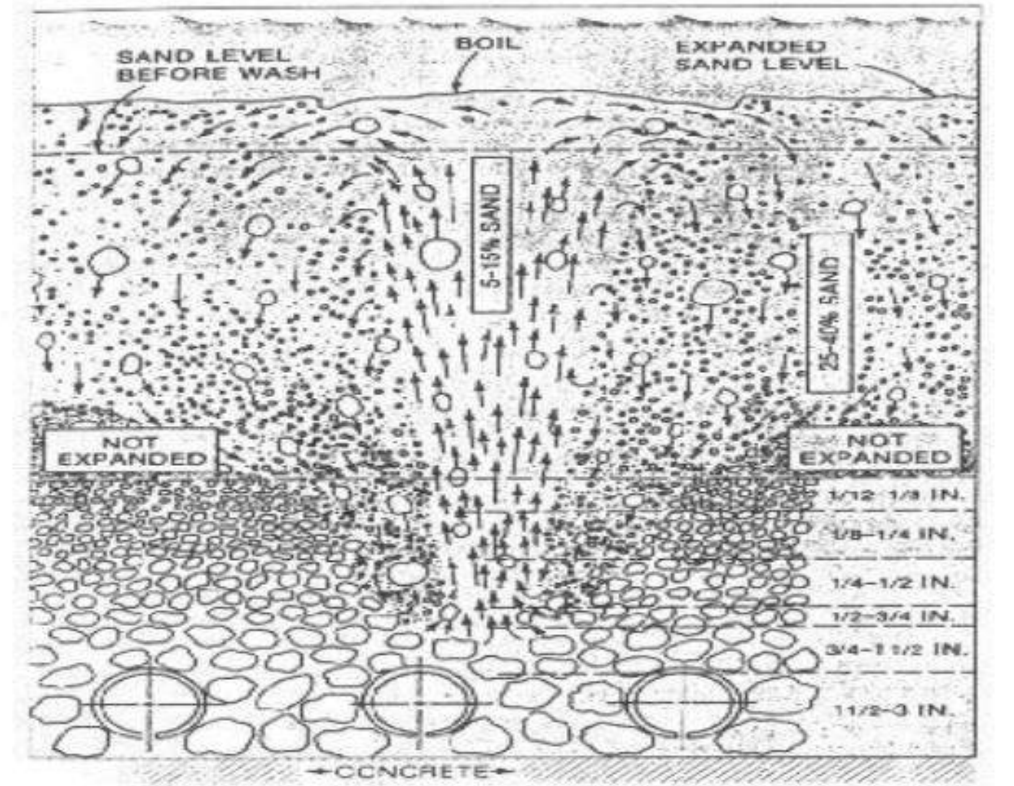
Sand grains enlarge in size and effective size changes

Carbonization of water can be done to prevent this problem.

Some times **Sodium hexa-meta Phosphate can be added to keep calcium carbonate in dissolved state**

5. Jetting and Sand boils

- These are produced when during backwashing water follows path of least resistance and break through to the scattered points due to small differences in porosity and permeability.
- Jetting can be avoided by surface wash or air scour.
- Use of 8 cm thick layer of coarse garnet is also recommended.



A Sand Boil in a Filter Bed

Hardness:

- • Water that contains dissolved Mg^{+2}, Ca^{+2} salts is called hard water.
- • Soap reacts with these ions to form soap scum.
- • Hard water also causes scale to form on the walls of water heaters, hot water pipes, and steam irons.
- • The scale reduces efficiency and plugs-up pipes.

Hardness:

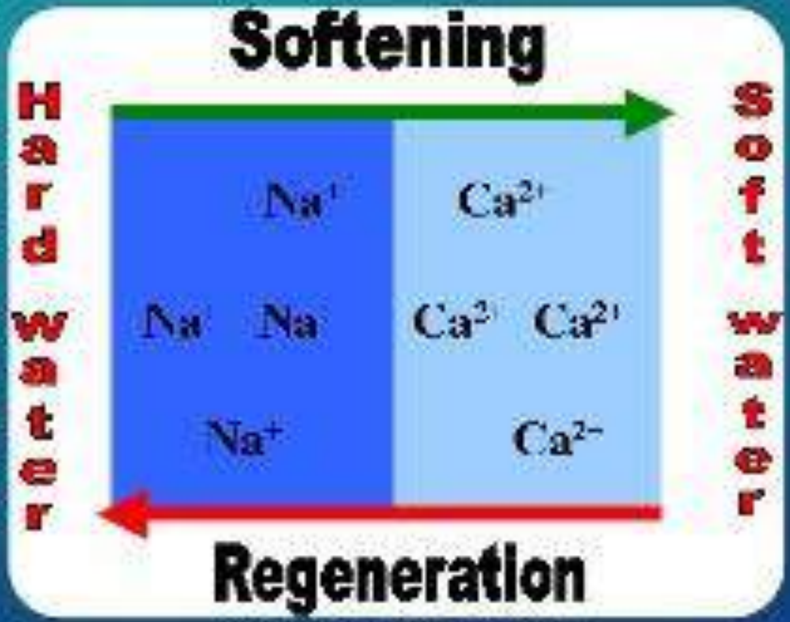
- • Water - hard when it contains relatively large amount of bicarbonates, carbonates, Sulphates and chlorides of Calcium and Magnesium.
- • These material react with soap, causing precipitation. No lather can be formed until enough soap has been dissolved to react with all these.
- • A wastage - 25ppm of soap takes place per 1 ppm of hardness of water removed by it.

Why softening?

- Reduction of soap consumption
- Lowered cost in the maintenance of plumbing
- Improved taste in foods prepared
- A must for industrial supplies

Lead contaminated site water softening basin

Courtesy: ndsu.edu



Courtesy: lenntech.com

Types of hardness:

1. Temporary hardness/carbonate hardness...

- Caused by Calcium bicarbonates[$Ca(HCO_3)_2$] or Magnesium bicarbonates [$Mg(HCO_3)_2$]
- Can be removed by boiling or lime addition.

2. Permanent hardness/non carbonate hardness....

- Caused by ..
  - ✓ Calcium sulphates[$CaSO_4$]
  - ✓ Magnesium sulphates [$MgSO_4$]
  - ✓ Calcium chlorides [$CaCl_2$]
  - ✓ Magnesium chlorides [$MgCl_2$]
- Can be removed by lime-soda process, Zeolite process or by demineralization.

REMOVAL OF TERMPORARY HARDNESS

Boiling

- $CaCO_3$ is slightly soluble in water.
- So it usually exists in water as a bicarbonate.
- Boiling will lead to the precipitation of $CaCO_3$ and release of CO_2.
- $Ca(HCO_3)_2 + \text{Heat} \longrightarrow CaCO_3 \downarrow + CO_2 \uparrow + H_2O$
- $CaCO_3$ is precipitated.
- Magnesium carbonates and bicarbonates are not satisfactorily removed.
- $MgCO_3$ is fairly soluble.

REMOVAL OF TERMPORARY HARDNESS

Addition of lime

- Hydrated lime is added to the water.
- Efficient in removal of both calcium and magnesium carbonates.

- $MgCO_3 + Ca(OH)_2 \longrightarrow Mg(OH)_2 \downarrow + CaCO_3 \downarrow$
- $Mg(HCO_3)_2 + Ca(OH)_2 \longrightarrow Ca(HCO_3)_2 + Mg(OH)_2 \downarrow$
- $Ca(HCO_3)_2 + Ca(OH)_2 \longrightarrow 2CaCO_3 \downarrow + 2H_2O$

- The precipitated compounds can be removed in the sedimentation tank.

Removal of permanent hardness

Lime soda ash – Chemical precipitation

- Both the carbonate and non carbonate hardness can be removed.
- Lime also helps in removal of free dissolved CO_2.
- $Ca(HCO_3)_2 + Ca(OH)_2 \longrightarrow 2CaCO_3 \downarrow + 2H_2O$
- $Mg(HCO_3)_2 + Ca(OH)_2 \longrightarrow Ca(HCO_3)_2 + Mg(OH)_2 \downarrow$
- $MgCO_3 + Ca(OH)_2 \longrightarrow Mg(OH)_2 \downarrow + CaCO_3 \downarrow$
- $MgCl_2 + Ca(OH)_2 \longrightarrow Mg(OH)_2 \downarrow + CaCl_2$
- Large quantity of sludges are encountered.

Lime soda ash – Chemical precipitation (Contd.)

$$MgSO_4 + Ca(OH)_2 \longrightarrow Mg(OH)_2 \downarrow + CaSO_4$$

$$CO_2 + Ca(OH)_2 \longrightarrow CaCO_3 \downarrow + H_2O$$

$$CaCl_2 + Na_2CO_3 \longrightarrow CaCO_3 \downarrow + 2NaCl$$

$$CaSO_4 + Na_2CO_3 \longrightarrow CaCO_3 \downarrow + Na_2SO_4$$

- Incrustation of filter media due to the precipitates.
- Recarbonation by passing CO_2 to again form the soluble bicarbonates.(Stabilization)
- $\underbrace{CaCO_3 \downarrow}_{\text{(Insoluble)}} + CO_2 + H_2O \longrightarrow \underbrace{Ca(HCO_3)_2}_{\text{(Soluble)}}$

Slide 22

I) Lime-Soda Method:

Advantages

- Economical
- Augmentation is easy
- Pre Addition: Less coagulant
- Increases pH – Less corrosion
- Kills pathogen upon prolonged exposure
- Reduces mineral content

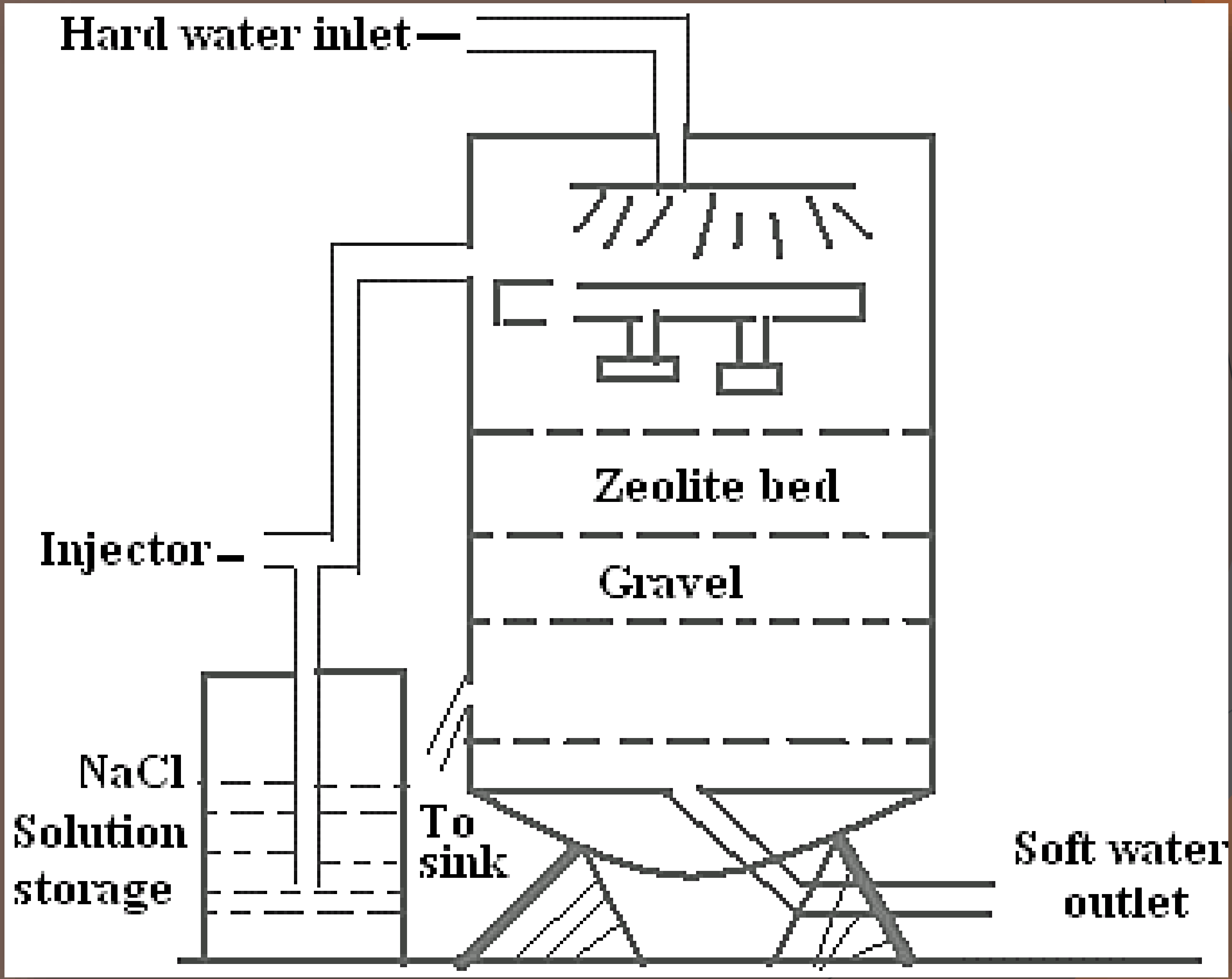
Disadvantages

- High sludge quantity
- Skilled workmanship
- Recarbonation is necessary to avoid encrustation
- Zero hardness not possible

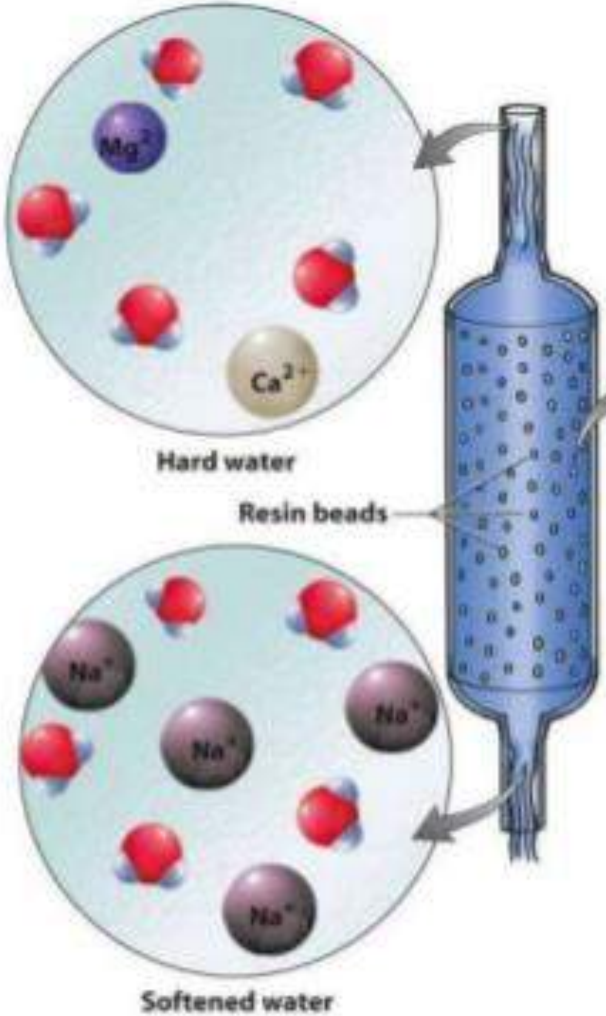
Zeolite process

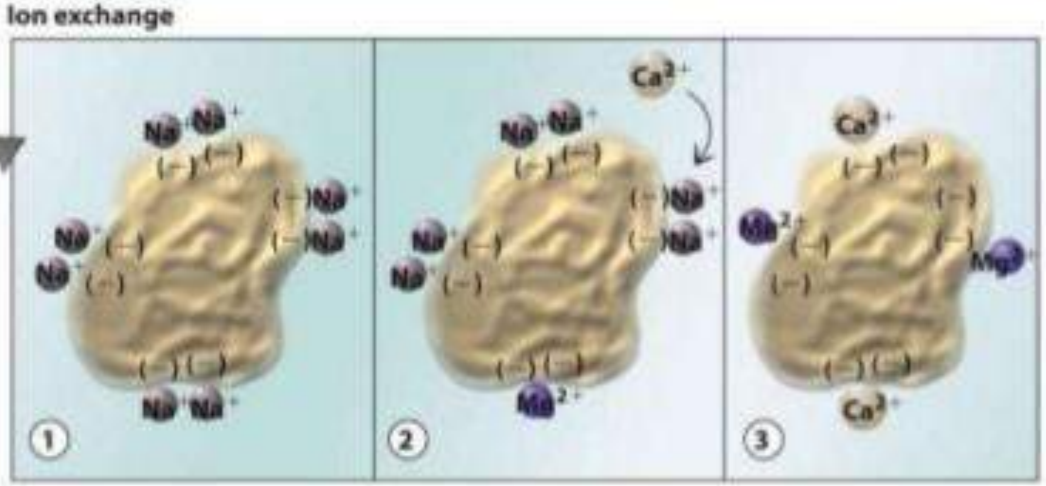
- Also known as base exchange or cation exchange process.
- Zeolites are complex compound of aluminium, silica and soda either natural or synthetic in nature.
- Natural zeolites are mainly processed by Green sang (glauconite)
- Has an exchange value of 6500 to 9500 gm/m^3
- Common artificial zeolite is purmutite. ($SiO_2Al_2O_3Na_2O$)
- Has a high exchange value of 35000 to 40000 gm/m^3

zeolite water softener



Zeolite





(1) Negatively charged sites are occupied by Na^+ ions.

(2) Hard water replaces Na^+ ions with Ca^{+2}, Fe^{+2}, and Mg^{+2} ions.

(3) Eventually the resin becomes saturated with the hard ions and is no longer effective.

Zeolite/Base Exchange/

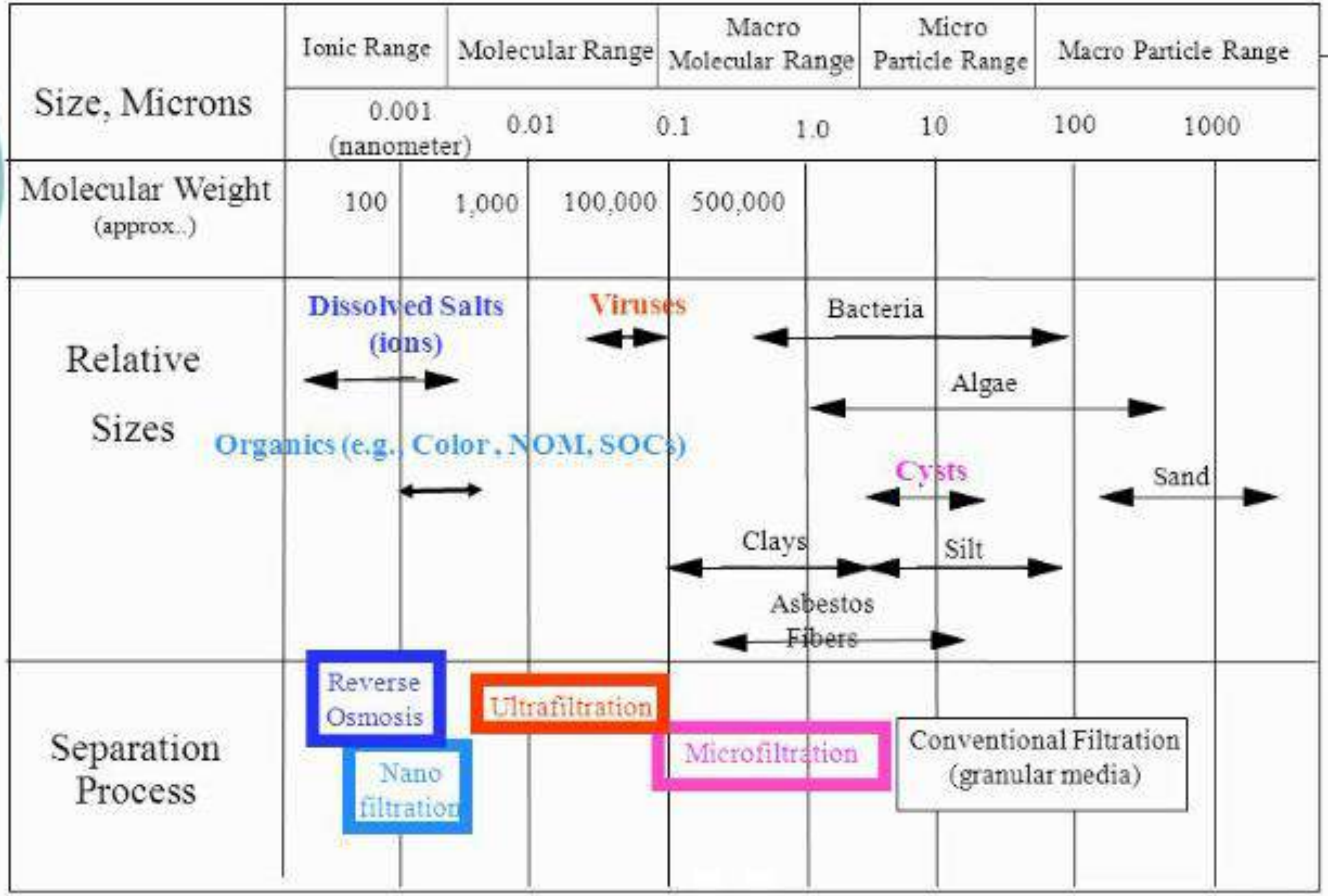
Advantage

- Zero hardness (Good for Industrial use)
- Compact, automatic, easy to operate
- No sludge
- O & M cost is less

Disadvantage

- Not suitable for turbid water
- Sodium carbonate left in water causes foaming
- Zero hardness

Membrane Separation



Membrane Filter Technology III

Filter type	Symbol	Pore Size, μm	Operating Pressure, psi	Types of Materials Removed
Microfilter	MF	1.0-0.01	<30	Clay, bacteria, large viruses, suspended solids
Ultrafilter	UF	0.01-0.001	20-100	Viruses, proteins, starches, colloids, silica, organics, dye, fat
Nanofilter	NF	0.001-0.0001	50-300	Sugar, pesticides, herbicides, divalent anions
Reverse Osmosis	RO	< 0.0001	225-1,000	Monovalent salts

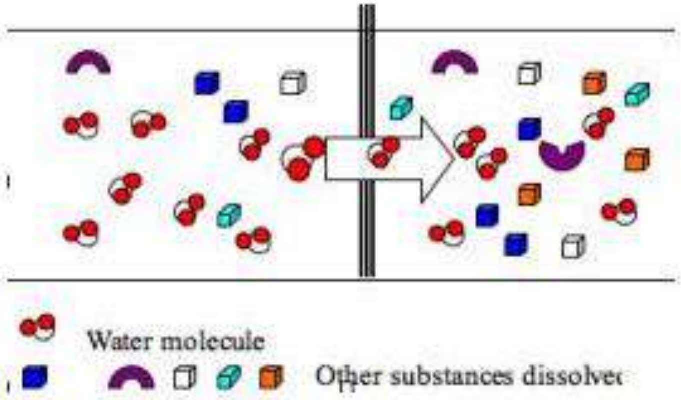
Source: http://web.evs.anl.gov/pwmis/techdesc/membrane/index.cfm

Reverse Osmosis (RO)

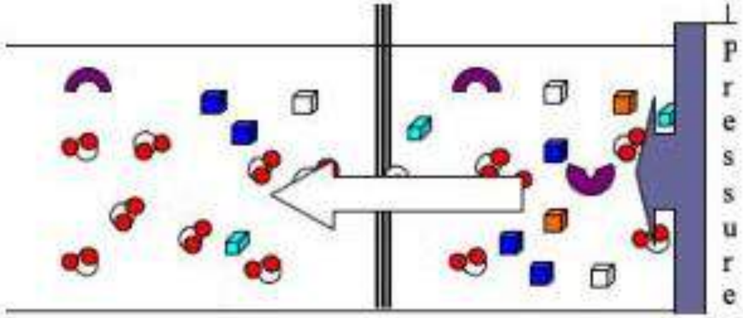
- **Typical pore size: 0.0001 micron (10^{-10}m)**
- **Very high pressure**
- **Only economically feasible large scale method to remove salt from water**
  - Salty water cannot support life
  - People can't drink it and plants can't use it to grow

How RO Works

- **Osmosis is a natural process that moves water across a semipermeable membrane, from an area of *greater concentration to an area of lesser concentration* until the concentrations are equal**
- **To move water from a *more concentrated area to a less concentrated area* requires high pressure to push the water in the opposite direction that it flows naturally**

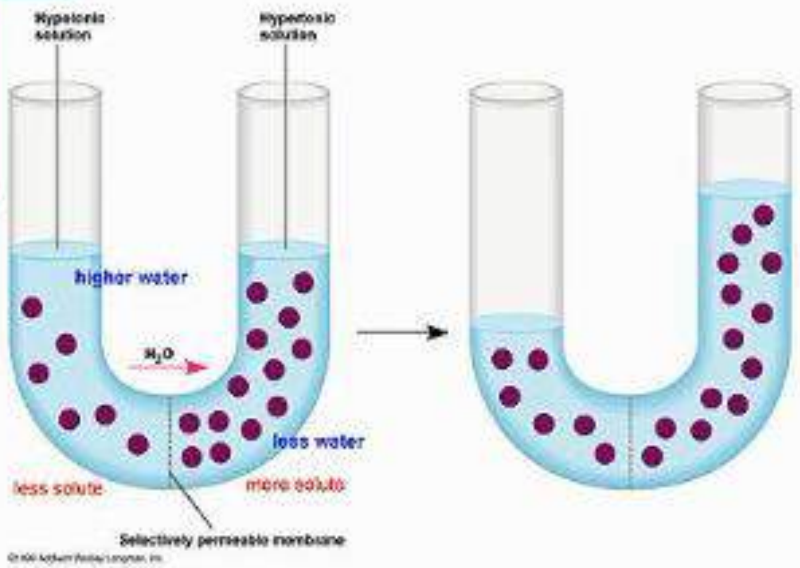


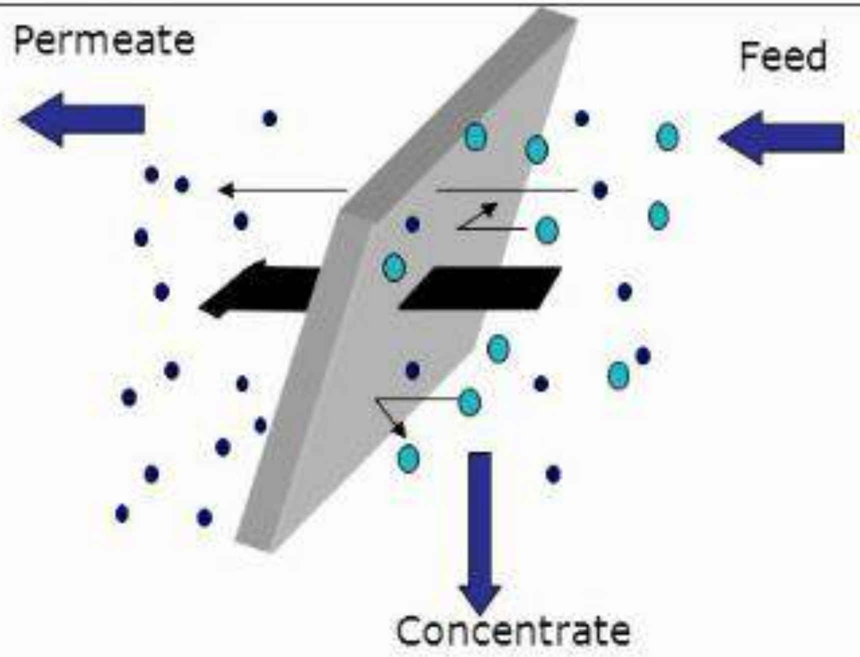
Osmosis



Reverse Osmosis

Osmosis vs. Reverse Osmosis

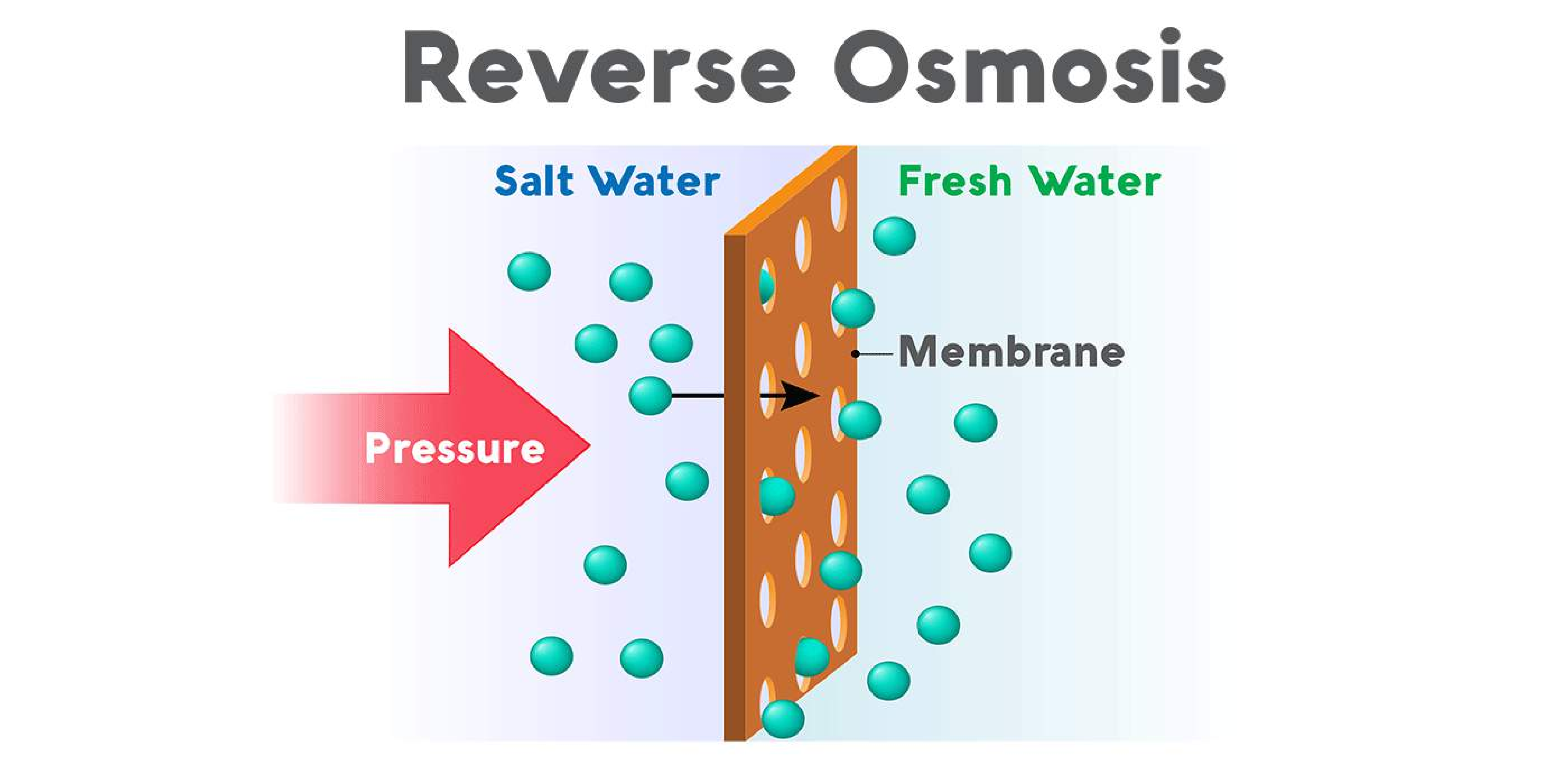




Driving Force (ΔC, ΔP)

Osmosis is the net movement (diffusion) of a solvent from a region of higher water concentration to a region of lower concentration

Reverse osmosis is the net movement (diffusion) of a solvent (water) from a region of higher salt concentration to a region of lower water concentration


Reverse Osmosis
Salt Water
Fresh Water
Membrane
Pressure

Reverse osmosis benefits

- It removes 98% of dissolved solids, which makes it healthier to drink.
- A water distiller is the only other drinking water system that also reduces TDS, but it's less efficient than an RO system.
- Harmful dissolved contaminants reduced
- Sodium reduced
- Bad tastes and odors reduced
- More environmentally friendly than bottled water
- Easy to install and maintain
- Fits under the kitchen sink

Applications of R.O. and nanofiltration:

- R.O. application mostly desalination.
- Nanofiltration first developed to remove hardness.
- Nanofltration can also be used to remove pesticides.

NEED FOR NANOFILTRATION

- Increasing demand of good quality water due to increasing population.
- Reducing the wastage and reuse of water.
- Better reliability and durability of filter membranes.
- To reduce the overall cost of operation.

Materials used in NF membranes

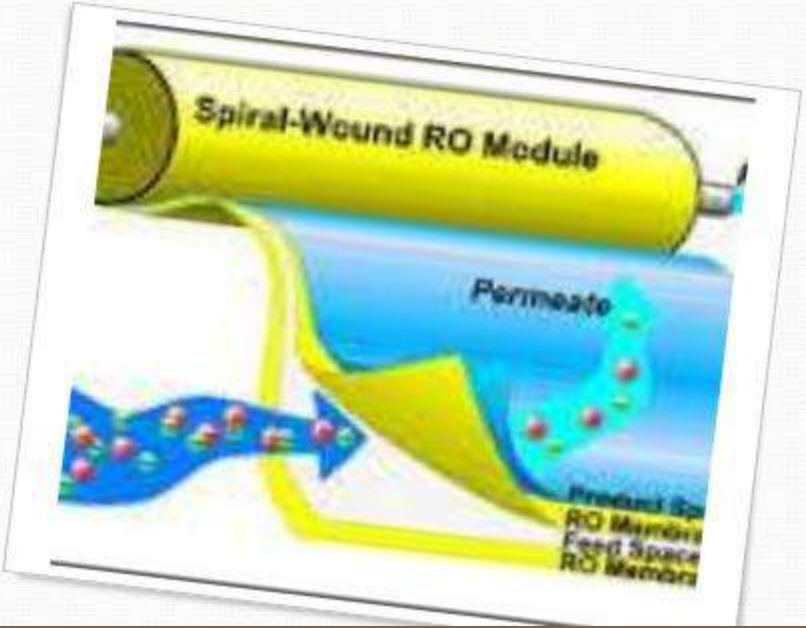
- Different polymers used are polyethersulfone, polysulfone, polyphenylsulfone, polytetrafluoroethylene, polyvinylidene fluoride, polyacrylonitriles, nylon, polypropylene, cellulose acetate (CA), regenerated cellulose, and composites.
- Ceramic and cintered metals.
- carbon nanotubes.

Two types of membranes:

i. Spiral membranes
ii. Tubular membranes

SPIRAL MEMBRANE

: Cheapest, more sensitive to pollution.



TUBULAR MEMBRANE

Not easily polluted


A cutaway of a tubular membrane module

Benefits of Nanofiltration

- Low cost of operation.
- Low energy cost.
- Lower discharge and less waste water than typical Reverse Osmosis system.
- Reduction of Heavy Metals (removes 95%).
- Reduction of water hardness.
- Reduction / Removal of viruses, bacteria and Pesticides.
- Reduction of Nitrates and Sulphides.
- Reduction of the salt content (brackish water).
- Chemical - Free filtration (No use of salt).
- pH of the water can be altered for better health.
- Ideal for municipal water supply, well water, river and rain water.
- Removes Iron, Lime and other problem causing chemicals often neglected by water softeners.

Various Applications:

1. Industrial applications:
   - Food and dairy sector.
   - edible oil processing sector.
   - Petroleum industry.
   - Drug industry.
   - Paper pulp industry
2. Water treatment.
3. Desalination of water.
4. Water softening.

Drawbacks of the process of Nanofiltration:

1. Membrane fouling.
2. Insufficient separation.
3. Treatment of concentrates.
4. Membrane lifetime and chemical resistance.
5. Insufficient rejection for individual components.

There are various ways to reduce the fouling such as:

⇧ Periodic pulsing of feed.

⇧ Periodic pulsing filtrate (backwashing).

⇧ Increasing shear by rotating membrane.

⇧ Vibrating membrane.

- **What is Disinfection & why it is done ? ?**

- **Disinfection is the process of the removal, deactivation or killing of pathogenic microorganisms.**

- Microorganisms are destroyed or deactivated, resulting in termination of growth and reproduction such that they represent no significant risk of infection. When microorganisms are not removed from drinking water, drinking water usage will cause people to fall ill.

- When water leaves the filter plant, it is till found to contain some of the impurities. These impurities can be grouped as:

  -Bacteria, Viruses, Protozoa
  -dissolved inorganic salts,
  -colour, odour and taste,
  -iron and manganese.

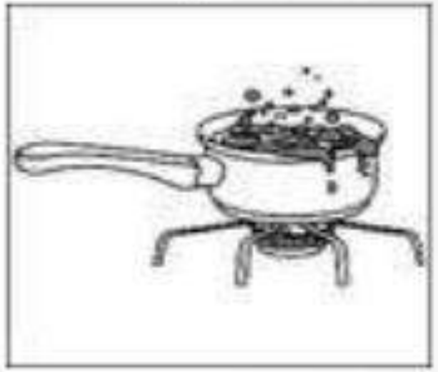
- The substances or materials which are to be used for disinfection are called the **DISINFECTANTS.**

DISINFECTION

METHODS OF DISINFECTION:

BOILING METHOD

- This is the most effective method of killing bacteria but impracticable in large scale.
- Most of bacteria are destroyed when the water has attained of about 80˚C temperature.
- Prolonged boiling is unnecessary and wasteful.



EXCESS LIME TREATMENT

- Treatment of lime is given to the water for the removal of dissolved salts.
- Excess lime added to water works as disinfecting material.
- When pH value is about 9.50, bacteria can be removed to the extent of 99.93 per cent.
- Lime is to be removed by recarbonation after disinfection.

IODINE AND BROMINE TREATMENT

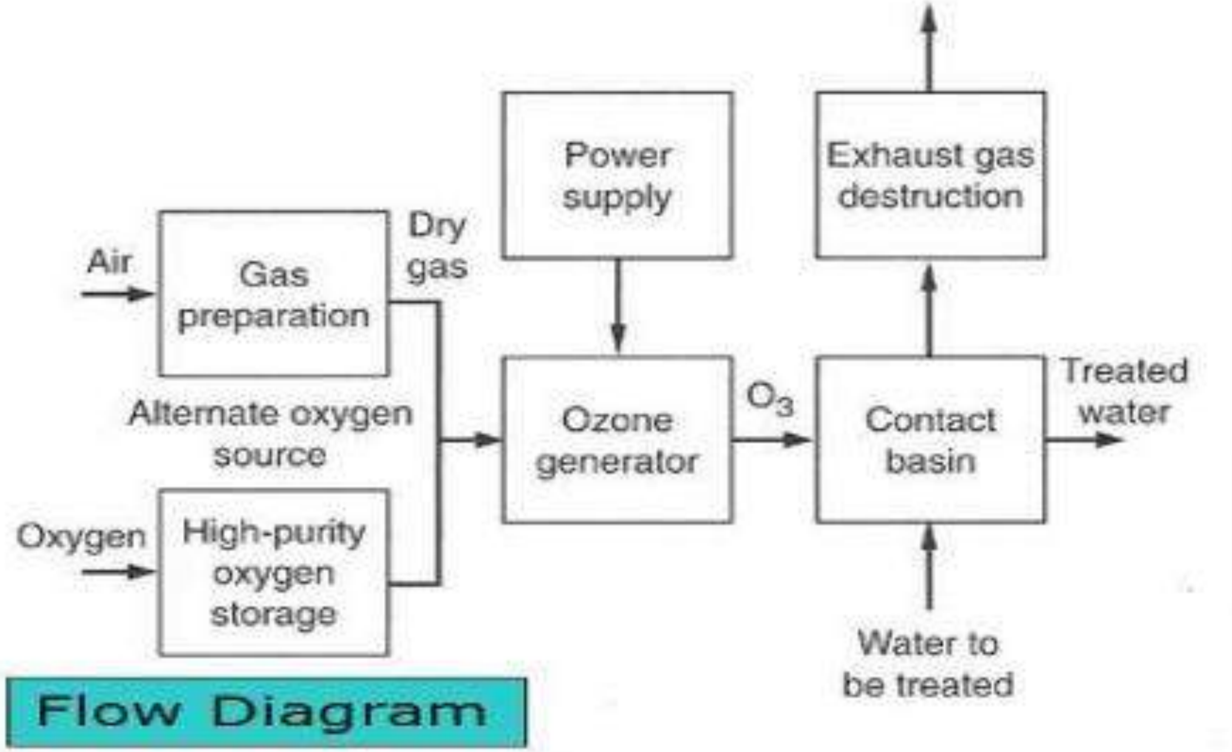
- Use of iodine or bromine is limited to small water supplies such as swimming pools, troops of army, private plants, etc.
- Dosage of iodine or bromine is about 8 p.p.m.
- Contact period with water is 5 minutes.
- Available in the form of pellets or small pills.

SILVER TREATMENT

- Colloidal silver is used to preserve the quality of water stored in jars.
- Metallic silver is placed as filter media. Water get purified while passing through theses filters.
- Dosage of silver varies from 0.05 to 1 p.p.m.
- Contact period is about 15 minutes to 3 hours.
- It is costly and limited to private individual houses only.

OZONE TREATMENT ($3O_2 = 2O_3$)

- Nascent oxygen is very powerful in killing bacteria.
- Ozone is unstable and doesnot remain in water when reaches the consumer.
- Ozoniser:



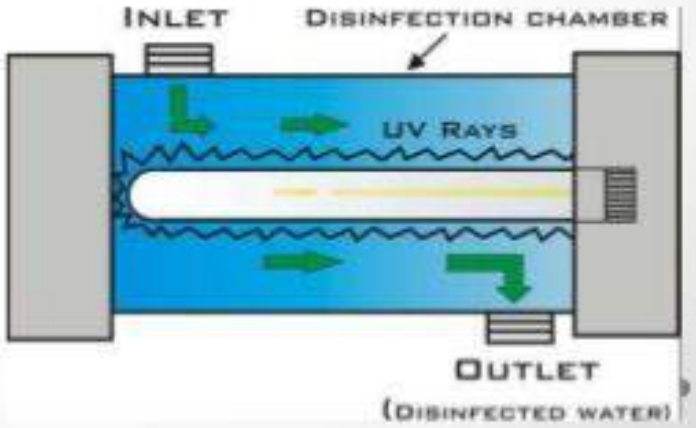
- Dosage of ozone is about 2 to 3 p.p.m. to obtain residual ozone of 0.10 p.p.m
- Contact period is about 10 minutes

POTASSIUM PERMANGANATE TREATMENT ($KMnO_4$)

- It is a powerful oxidising agent, effective in killing cholera bacteria
- Restricted to disinfection of water of village wells and ponds
- Dosage is about 2.1 p.p.m
- Contact period of 3 to 4 hours
- The treated water produces a dark brown coating on porcelain vessels and this is difficult to remove except with scratching or rubbing.

ULTRA-VIOLET RAY TREATMENT

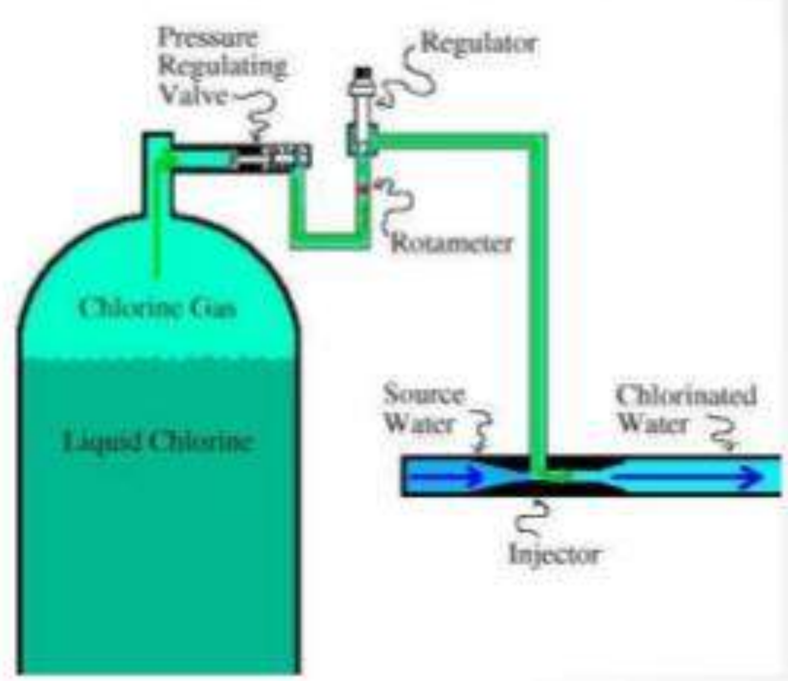
- For generating these rays, the mercury is enclosed in one or more quartz bulbs and electric current is then passed through it.
- The water should be passed round the bulbs several times .
- Depth of water over the bulbs should not exceed 10 cms.



CHLORINATION

Used as a disinfecting material as,

- It is easy to apply due to relatively high solubility of about 7000 mg per litre.
- Readily available as gas, liquid or powder.
- Very toxic to most of the micro-organisms.
- Cheap and reliable.
- Chlorine can be applied in water in one of the following ways:
  - as bleaching powder,
  - as chloramines, or
  - as free chlorine gas.



Mechanism of action:

- 1. $H_2O + Cl_2$ (at pH 7) = HCl + $HOCl$ (main disinfectant)

  $HOCl$ (at pH > 8.5) $\rightleftharpoons H^+ + OCl^-$ (minor action)

- 2. $NH_3 + HOCl = NH_2Cl/NHCl_2/NCl_3 + H_2O$
  (Mono, Di ,Tri Chloramines)

Fluoridation

- Defined as the upward adjustment of the concentration of fluoride ion in a public health water supply
- Concentration of fluoride ion in water may be consistently maintained at 1ppm by weight to prevent dental caries with minimum possibility of causing dental fluorosis.

Fluoride compounds used in water fluoridation

a) Fluorspar–

mineral containing varying amounts of CaF_2

b) Sodium fluoride–

white, odorless,free flowing material available either as a powder or a mixture of various crystals

– expensive source of fluorides

c) Silicofluoride–

obtained as by product of purification of phosphate rocks d)Sodium silicofluoride

- Most popular
- Low cost, cheapest form of fluoride
- Solutions are corrosive.

e)Hydrofluosilic acid

- More expensive

f)Ammonium silicofluoride

- Produced by neutralising fluosilic acid with either aqueous ammonia or ammonia in gaseos form

DEFLUORIDATION

- Defluoridation is defined as, „the downward adjustment of level of fluoride in drinking water to the optimal level"
- Defluoridation techniques can be broadly classified in to four categories:
- Adsorption technique
- Ion-exchange technique
- Precipitation technique

Adsorption technique

- This technique functions on the adsorption of fluoride ions onto the surface of an active agent.

i. Activated alumina

ii. Bone char

iii. Brick piece column

iv. Mud pot

Activated alumina

- aluminum sulfate have a high affinity for fluoride and can be used for removal of excess fluoride from water either alone or in combination.

  Disadvantages:

- Adsorption of fluoride is possible only at specific pH range
- Frequent activation of Alumina is needed
- Adsorption efficiency of the activated alumina diminishes with increasing number of usage-regeneration cycle.

Bone char

- Defluoridation by bone char as the ion exchange and adsorption between fluoride in the solution and carbonate of the apatite comprising bone char.
- The efficacy of the plant depends upon temperature and pH of raw water; duration for which the bone-char is in contact with raw water.
- It is a highly economic technique with a defluoridation percentage of 62 to 66

Disadvantages :

- The bone char harbors bacteria and hence unhygienic.
- nothing indicates when the material is exhausted and the fluoride uptake is ceased.

Brick pieces column

- The soil used for brick manufacturing contains Aluminium oxide. During burning operation in the kiln, it gets activated and adsorbs excess fluoride when raw water is passed through.
- In places where high alumina content soil is available, brickbat filter may be one of the options

Mud pot

- The raw pots are subjected to heat treatment as in the case of brick production. Hence the mud pot also will act as an adsorbent media.
- marginal reduction in water fluoride level from 1.8 ppm to 1.5 and 1.4 ppm at the end of 2 days and 4 days respectively
- The water pH was raised from 7.7 to 8.11 and 8.14 the end of 2 days and 4 days, which is beyond the acceptable limits of alkalinity.

Precipitation technique

- Precipitation methods are based on the addition of chemicals (coagulants and coagulant aids) and the subsequent precipitation of a sparingly soluble fluoride salt as insoluble fluorapatite
- Fluoride removal is accomplished with separation of solids from liquid.
- Aluminium salts (eg. Alum), lime, Poly Aluminium Chloride, Poly Aluminium Hydroxy sulphate and Brushite are some of the frequently used materials in defluoridation by precipitation technique

Ion-Exchange technique

- Synthetic chemicals, namely, anion and cation exchange resins have been used for fluoride removal.
- The fluoride exchange capacity of these resins depends upon the ratio of fluoride to total anions in water.

Examples:

- Polyanion (NCL) , Deacedite FF (IP), Amberllte IRA 400, Lewatit MIH - 59 ,
- defluoron 1, defluoron 2

Nalgonda technique

- National Environmental Engineering Research Institute (NEERI), Nagpur – Nalgonda technique in 1974
- Nalgonda Technique involves addition of Aluminium salts, lime and bleaching powder followed by rapid mixing, flocculation, sedimentation, filtration and disinfection.

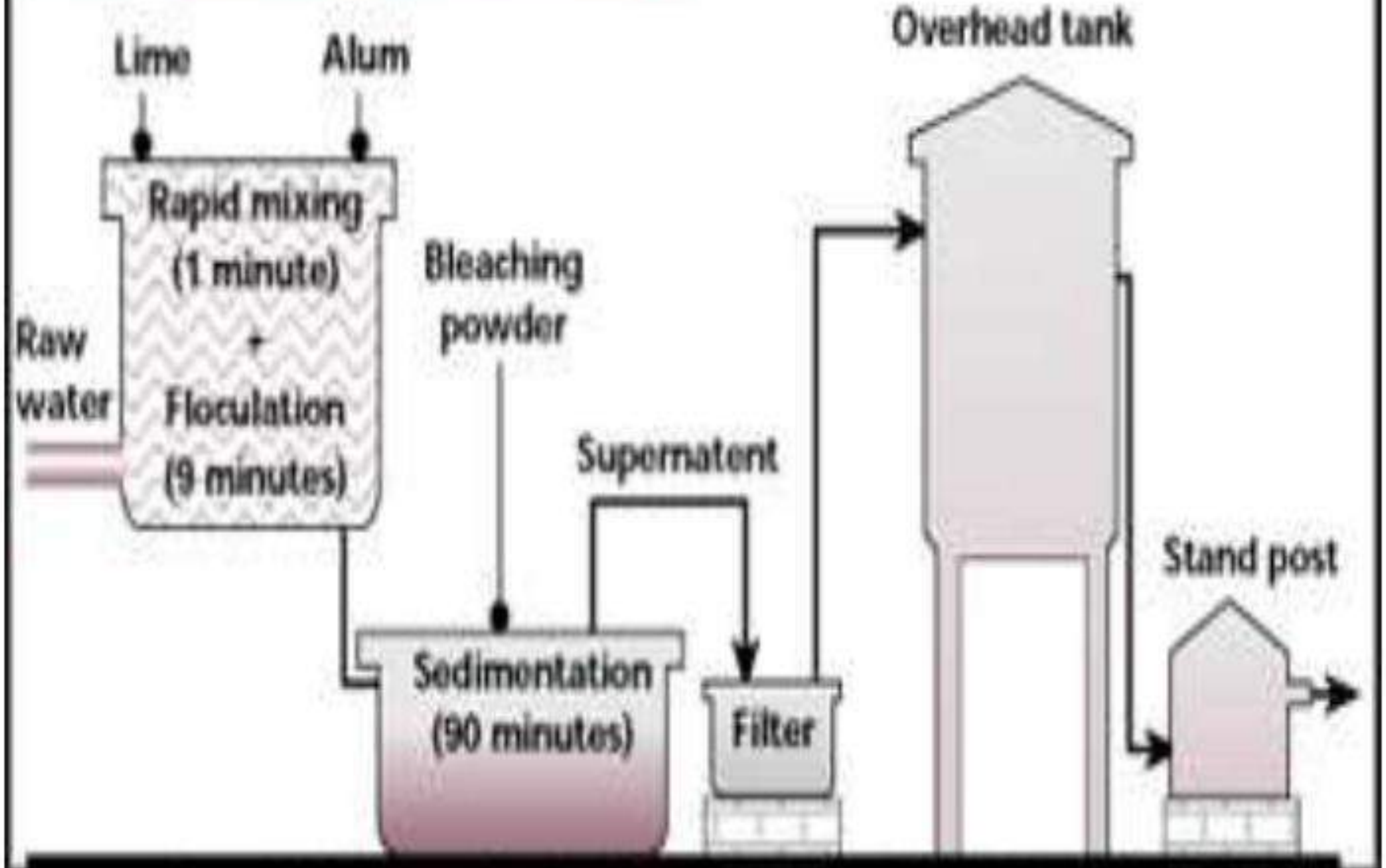
1. Aluminium salts (aluminium sulfate-alum or aluminium chloride or combination of these two) : responsible for removal of fluoride from water

2. Lime : facilitates forming dense flocks for rapid settling of insoluble fluoride salts .

Dose : empirically 1/20th of that of the dose of aluminium salt

3. Bleaching powder : disinfection-3 mg/l

Nalgonda technology



Mechanism of Nalgonda technique

–**Rapid mix:-** provides thorough mixing of chemicals

–**Flocculation:-** gentle agitation

- combination of poly hydroxy aluminum complex with fluoride & polymeric aluminum hydroxides are formed (flocs)
- turbidity, color ,odour → removed; bacterial load reduces
- Lime ensures that residual aluminum does not remain in treated water

- Sedimentation:- permits settling of flocs loaded with fluorides & other impurities
- Filtration:-rapid gravity sand filters
- Disinfection :- rechlorinated with bleaching powder before distribution

Advantages of Nalgonda technique

- Regeneration of media is not required.
- No handling of caustic acids and alkalies.
- The chemicals required are readily available and are used in conventional municipal water treatment.
- Adaptable to domestic use.
- Economical
- Can be used to treat water in large quantities for community usage.

Disadvantages of Nalgonda technique

- Desalination may be necessary when the total dissolved solids exceed 1500 mg/l.
- Generation of higher quantity of sludge
- The large amount of alum needed to remove fluoride.
- Careful pH control of treated water is required.
- High residual aluminium is reported in treated water by some authors.

METHODS OF DISTRIBUTION

By Gravitational system

By Pumping system

By Combined Gravity and Pumping system

GRAVITATIONAL SYSTEM

- water from the high level source is distributed to the consumers at low level, by the more action of gravity without any pumping
- the difference of head available must be sufficient enough, as to maintain adequate pressure
- economical and reliable
- lake or a reservoir

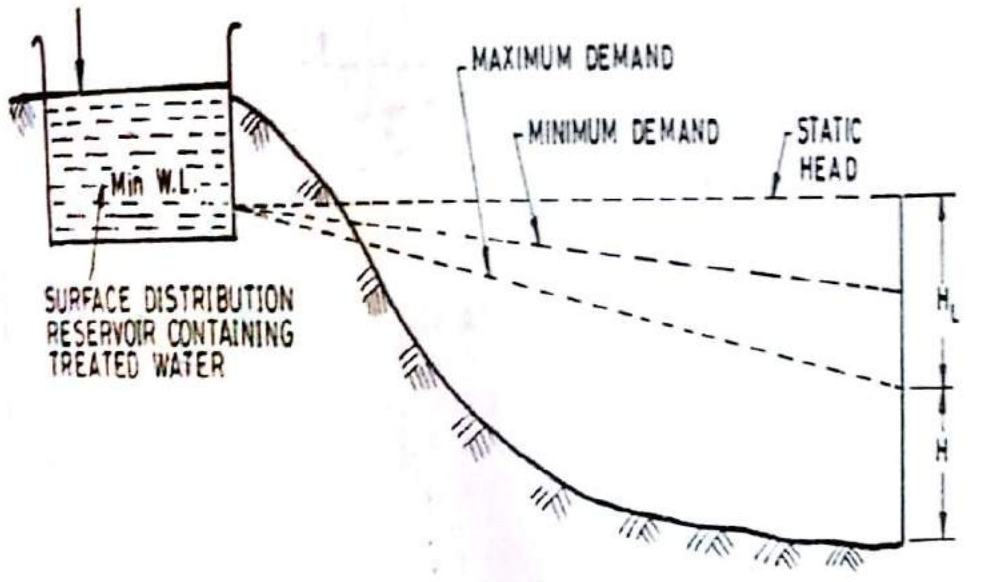


Fig. 10.6. Gravitational distribution system.

where H_L = Head loss

H = Head available to consumers.

PUMPING SYSTEM

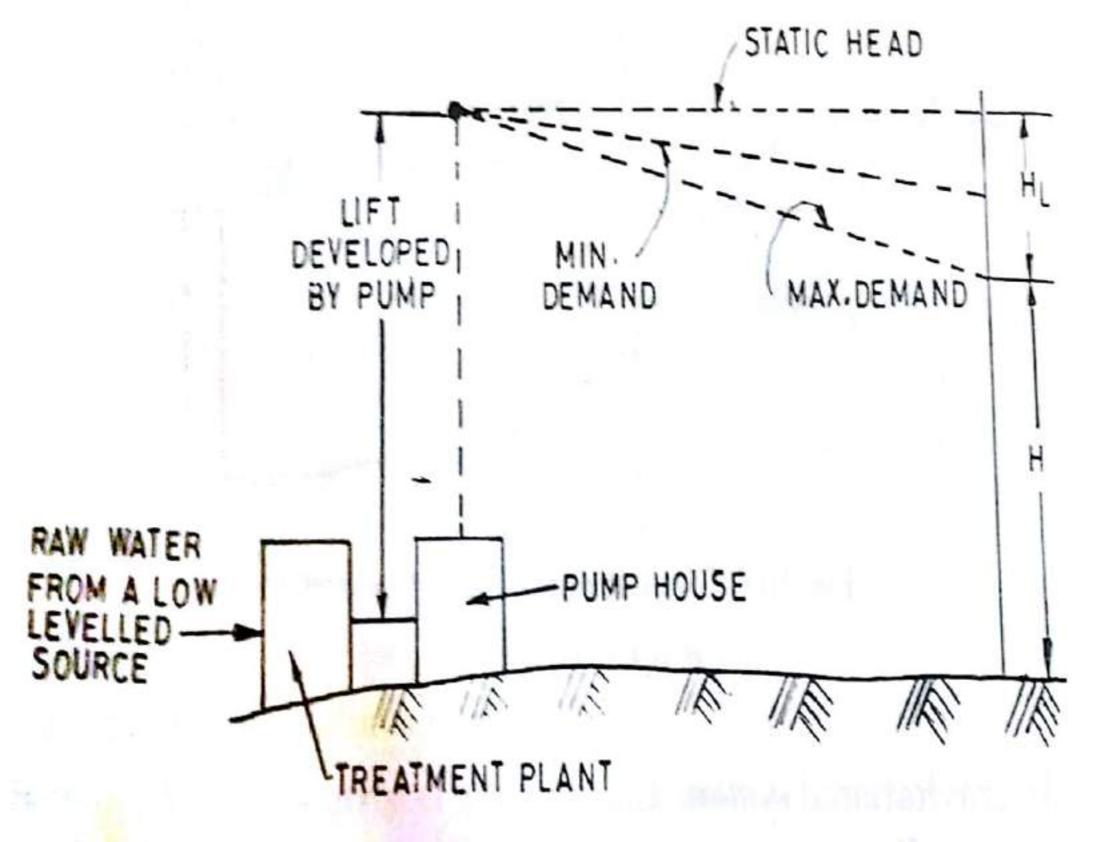
- treated water is directly pumped in to the D/S main without storing it
- High lift pumps are required

DISADVANTAGES

- Continuous attendance is needed at the pumping station,
- If power supply fails, there will be complete stoppage of w/s.

ADVANTAGES

- can force large volume of water under high pressure in the required direction, - motor pumps may be eliminated.



COMBINED GRAVITY AND PUMPING SYSTEM

- water is pumped a the constant rate and stored into an elevated D/S reservoir- it is distribution to the consumers by gravity

ADVANTAGES

- Balance reservoir of d/s reservoir can supplied places of fine & necessary pressure can be achieved by closing down the supply of localities
- worked at uniform rate
- overall cheap, efficient and reliable and hence adopted practically every where.

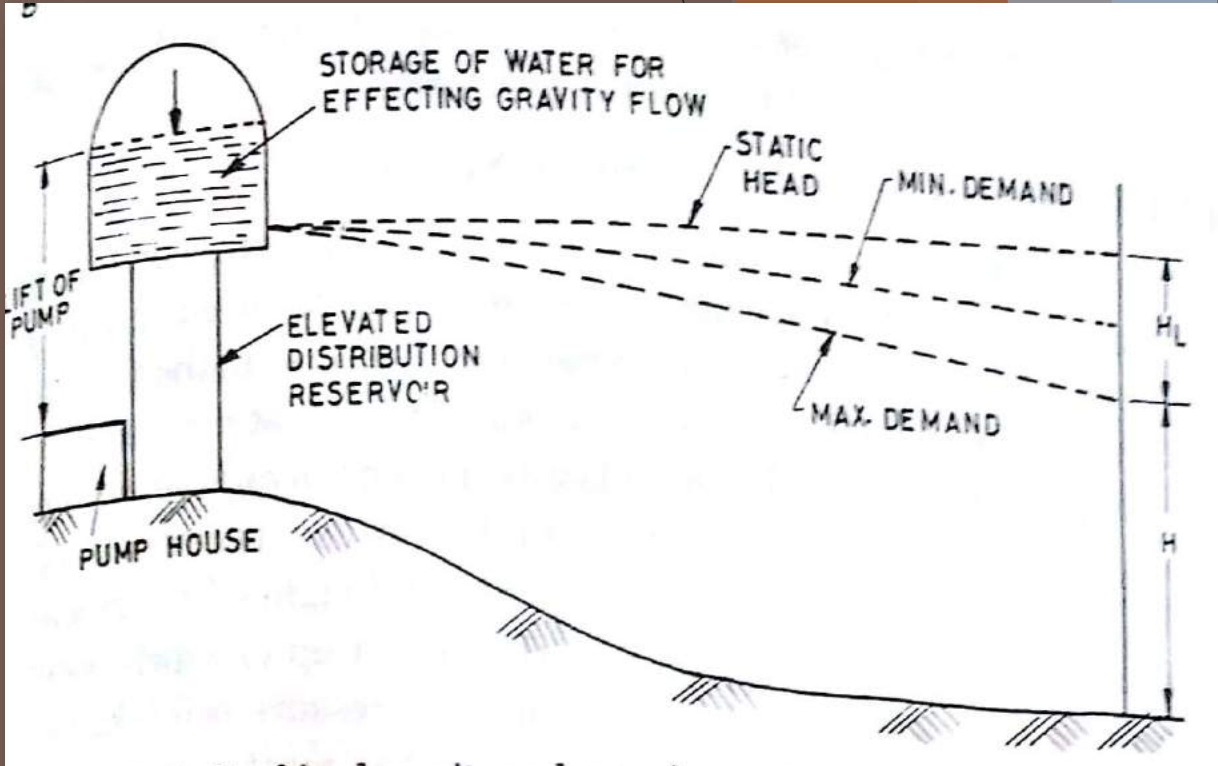


Fig. 10.8. Combined gravity and pumping system for water distribution.

MODULE 3

18CV62

SANITATION & ITS NEED

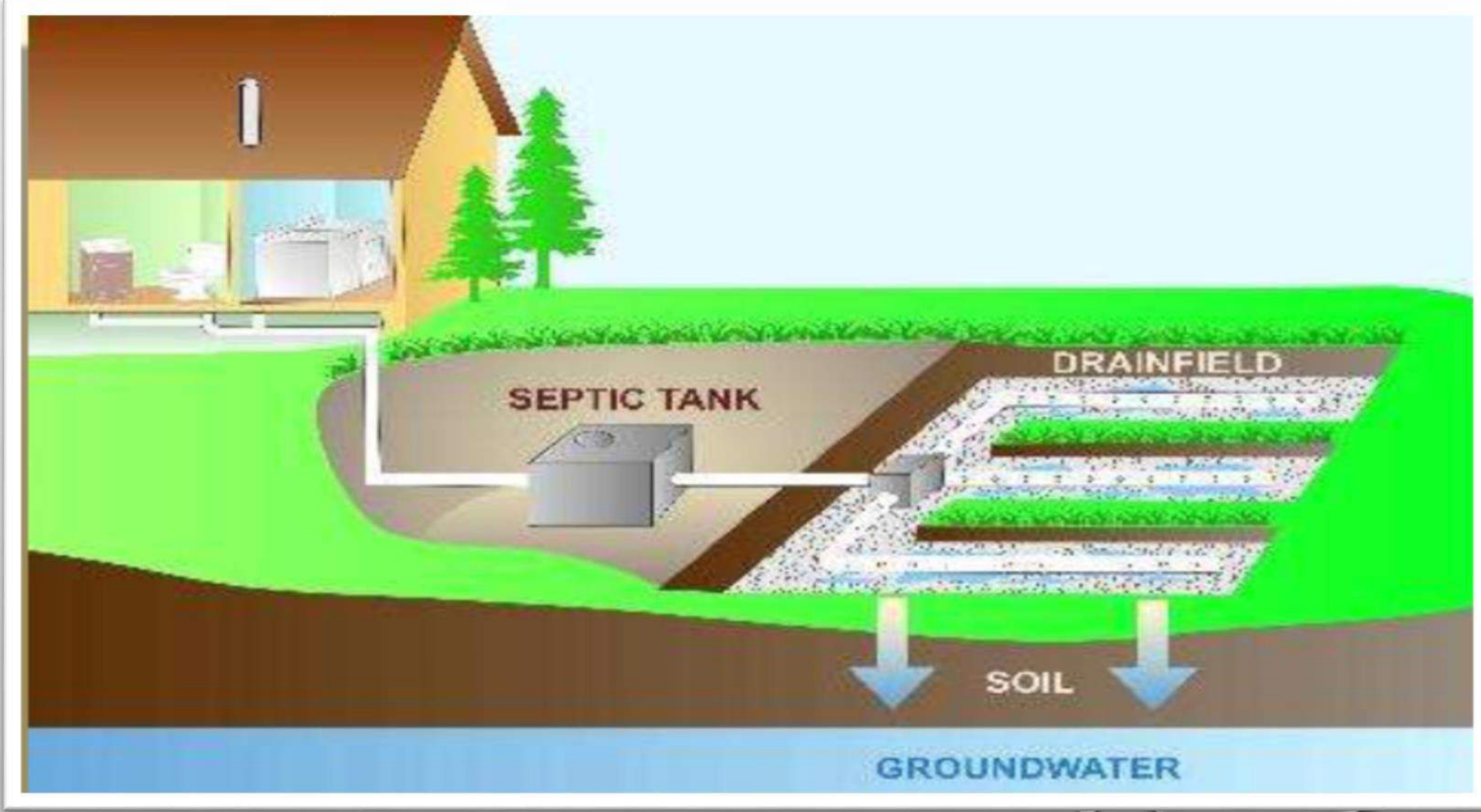
- Sanitation - public health conditions related to clean drinking water and adequate treatment and disposal of human excreta and sewage Sanitation systems aim to protect human health by providing a clean environment that will stop the transmission of disease
- Water is a basic necessity, and an important resource for sustaining life. The decline in water quality endangers the health of humans as well as the ecosystem. Clean drinking water, hygiene, and sanitation play an important part in maintaining health.
- Contaminated water causes many water-borne infections like diarrhoea, and also serves as a carrier for vectors such as mosquitoes spreading epidemics. Open defecation means no sanitation. It fouls the environment, and spreads diseases.

SANITATION & ITS NEED

- Sanitation is important for all, helping to maintain health and increase life-spans. However, it is especially important for children. Around the world, over 800 children under age five die every day from preventable diarrhea-related diseases caused by lack of access to water, sanitation and hygiene. In addition, diarrhea causes children to lose their appetites, which can lead to malnourishment. Limited access to sanitation has become such a worldwide problem that 1 in every 4 children suffer from stunted growth. This leads to "irreversible physical and cognitive damage."
- Sanitation makes a positive contribution in family literacy. According to a UNICEF study, for every 10 per cent increase in female literacy, a country's economy can grow by 0.3 per cent. Thus, sanitation contributes to social and economic development of the society. Improved sanitation also helps the environment.
- Sanitation brings numerous benefits such as reducing the burden of disease, improving quality of life, promoting the safety of women and girls, not to mention the excellent economic investment that sanitation represents.

CONSERVANCY SYSTEM

- Also called dry-System. practice from very ancient times. Actually it is out of date system even though it is prevailing in small towns, villages and undeveloped portions of the large cities.
- Various types of refuse and storm water are collected, conveyed and disposed of separately by different methods in this system, therefore, it is called conservancy system.



- Garbage or dry refuse of a town is collected in dust bins placed along the roads and streets, from where it is conveyed by trucks or covered carts once or twice in a day to the point of disposal.

- Non-combustible portions of the garbage such as sand, dust, clay ashes etc., are used for filling the low level areas to reclaim land for further development of the town.

- The combustible portion of garbage such as dry leaves, waste paper, broken furniture etc. are burnt. The decaying fruit and vegetables, grass and other things are first dried and then disposed of by burning or in the manufacturing of Manure.

- Human Excreta or Night Soil is collected separately in privies or conservancy laterins. The liquid and semi-liquid wastes are collected in separate drains of the same latrine, from where they are removed through human agency. The night soil is taken outside the town in closed animal drawn carts, trucks or tanks mounted on the trailers. The night soil is buried in trenches.

- • Sullage and Storm waters are also carried out separately in closed or open drains, upto the point of disposal, where they are allowed to mix with stream, rivers or sea without any treatment

MERITS OF CONSERVANCY SYSTEM

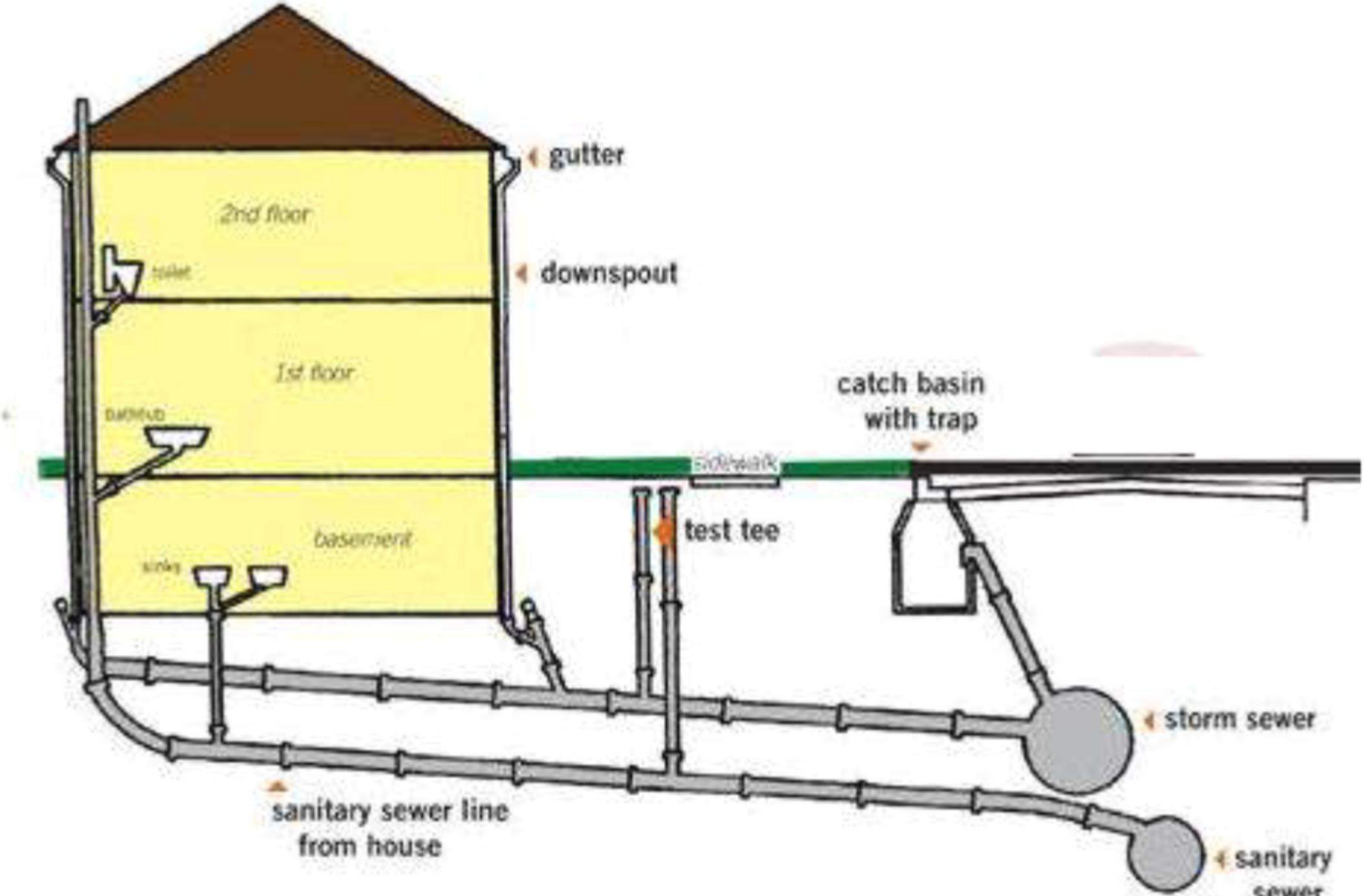
- It is cheaper in Initial cost because storm water can pass in open drains and conservancy latrines are much economical.
- The quantity of sewage reaching at the treatment plant before disposal is low.
- As the storm water goes in open drains, the sewer section will be small and will run full for the major portion of the year, due to which there will be no silting and deposits in sewer-lines.
- In floods if the water level of river rises at the out-fall, it will not be costly to pump the sewage for disposal

DEMERITS OF CONSERVANCY SYSTEM

- It is possible that storm water may go in sewer causing heavy load on treatment plants, therefore it is to be watched.
- • In crowded lanes it is very difficult to lay two sewers or construct road side drains, causing great inconvenience to the traffic.
- • Buildings cannot be designed as compact unit, because latrines are to be designed away from the living rooms due to foul smell, which are also inconvenient.
- • In the presence of conservancy system, the aesthetic appearance of the city cannot be increased.
- • Decomposition of sewage causes insanitary conditions which are dangerous to public health.
- •This system completely depends on the mercy of sweepers

WATER CARRIAGE SYSTEM

- With the development and advantages of the cities, urgent need was felt to replace conservancy system with some more improved types of system in which human agencies should not be used for the collection and conveyance of the sewage. After a large number of trials it was found that the water is the only cheapest substance, which can be easily used for collection and conveyance of sewage. Therefore it is called Water-Carriage System.
- In this system the excremental matters are mixed up in large quantity of water and are disposed off after necessary treatment in a satisfactory manner.


gutter
2nd floor
toilet
downspout
1st floor
bathtub
catch basin
with trap
sidewalk
test tee
basement
sinks
storm sewer
sanitary sewer line
from house
sanitary
sewer

MERITS

- It is hygienic method, because all the excremental matters are collected and conveyed by water only and no human agency is employed for it.
- There is no nuisance in the street of the town due to offensive matters, because all the sewage goes in closed sewers under the ground. The risk of epidemic is reduced.
- As only one sewer is laid, therefore it occupies less space in crowded lane.
- Due to more quantity of sewage, self-cleansing velocity can be obtained even at less gradients.
- Buildings can be designed as compact one unit.
- The land required for the disposal work is less as compared with conservancy system in which more area is required

- The usual water supply is sufficient and no additional water is required in water carriage system.

- • This system does not depend on the manual labours

- • Sewage after proper treatment can be used for various purposes.

DEMERITS

- • This system is very costly in initial cost.
- • The maintenance of this system is also costly.
- • During monsoon large volume of sewage is to be treated whereas very small volume is to be treated in the remaining period of the year.

SEWERAGE SYSTEMS:

CONSERVENCY SYSTEM	WATER CARRIAGE SYSTEM
Very cheap in initial cost.	It involves high initial cost.
Due to foul smells from the latrines, they are to be constructed away from living room so building cannot be constructed as compact units.	As there is no foul smell latrines remain clean and neat and hence are constructed with rooms, therefore buildings may be compact.
The aesthetic appearance of the city cannot be improved	Good aesthetic appearance of city can be obtained.
For burial of excremental matter large area is required.	Less area is required as compared to conservancy system.
Excreta is not removed immediately hence its decomposition starts before removal,	Excreta are removed immediately with water, no problem of foul smell or hygienic trouble.
This system is fully depended on human agency .In case of strike by the sweepers; there is danger of insanitary conditions in	As no human agency is involved in this system ,there is no such problem as in case of conservancy system

Collection of Sewage

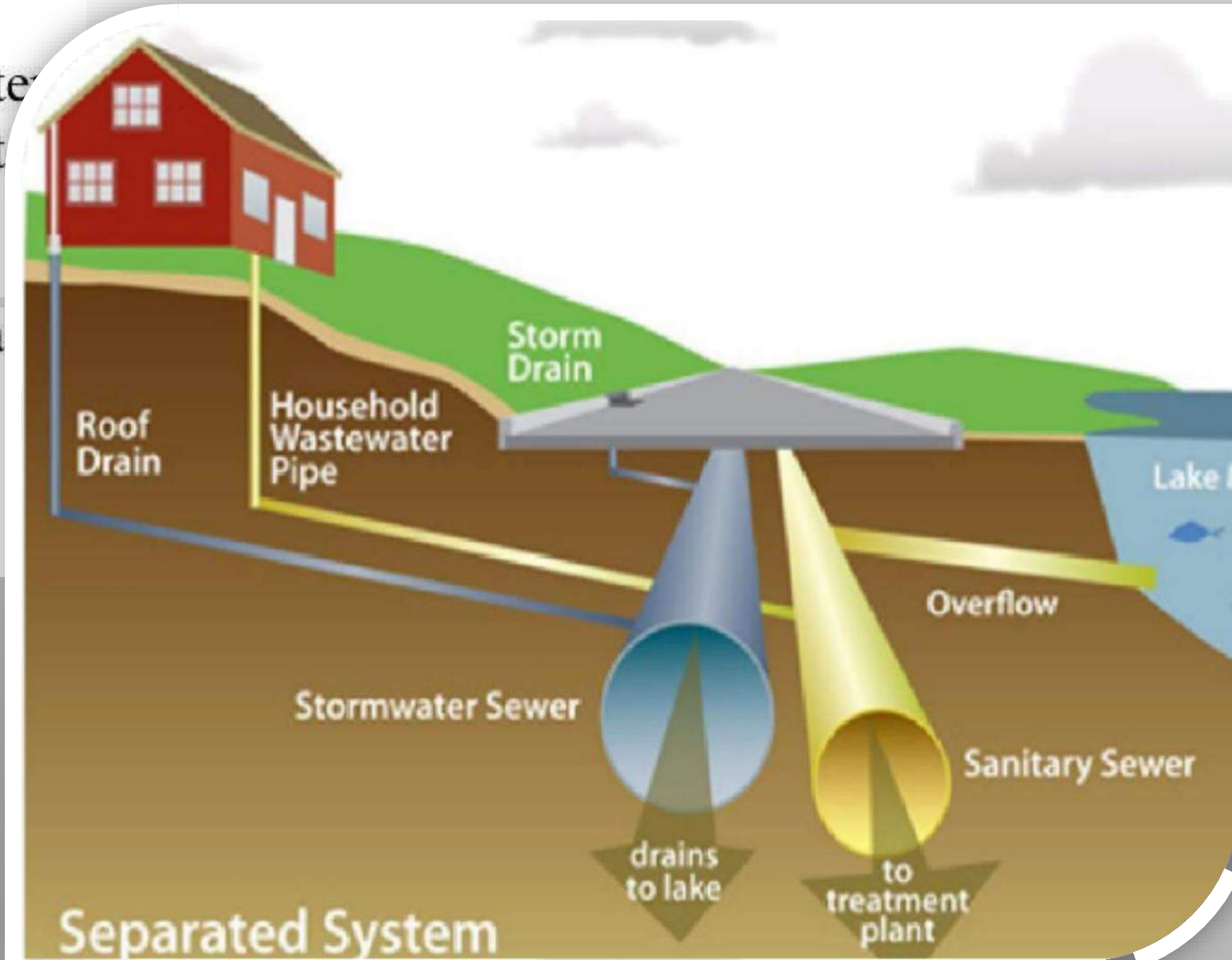
- A system of sewer pipes (sewers) collects sewage and takes it for treatment or disposal. The system of sewers is called *sewerage* or *sewerage system,*
- Where a main sewerage system has not been provided, sewage may be collected from homes by pipes into septic tanks or cesspits, where it may be treated or collected in vehicles and taken for treatment or disposal

Sewerage system, network of pipes, pumps, and force mains for the collection of wastewater, or sewage, from a community.

1. Separate Sewerage System

- In this system the sanitary sewage and storm water are carried separately in two sets of sewers.
- The sewage is conveyed to waste water treatment plant (WWTP) and the storm wat is discharges into rivers without treatment.

- The separated system is suitable when separa outlet for storm water is available and the topography is such that storm water can be disposed of in natural drains



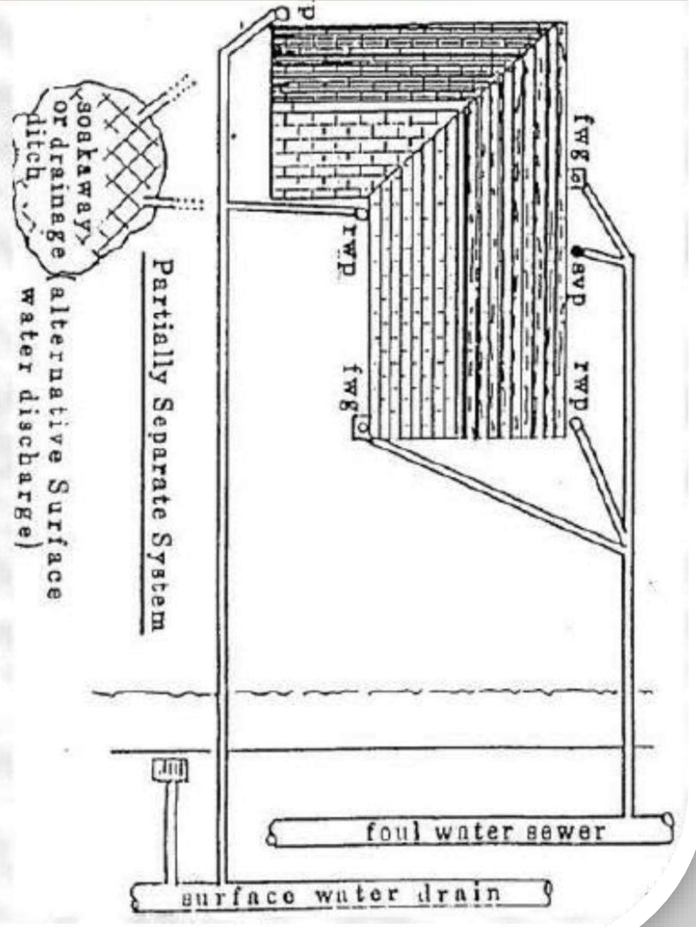
Advantages of Separate System

- The load on treatment plant is less as only sewage is carried to the plant.
- The size of sewer is small, thus economical
- When pumping is required, the system proves to be economical.
- Natural/storm water is not unnecessarily polluted by sewage.

Disadvantages of Separate System

- Cleaning of sewer is difficult due to their small size.
- The self cleansing velocity is not easily obtained.
- The storm sewers come in operation in rainy season only. They may be chocked in dry season by garbage.
- Maintenance cost is high.
- Sewage sewers are provided below storm sewer which causes greater depth and pumping at waste water treatment plant (WWTP).

Partially Separate System of Underground Drainage



2. Partially Separate Sewerage System

- This system is the compromise between separate and combine system taking the advantages of both systems.
- In this system the sewage and storm water of buildings are carried by one set of sewers while the storm water from roads, streets, pavements etc are carried by other system of sewers usually open drains.

Advantages of Partially Separate Sewerage System

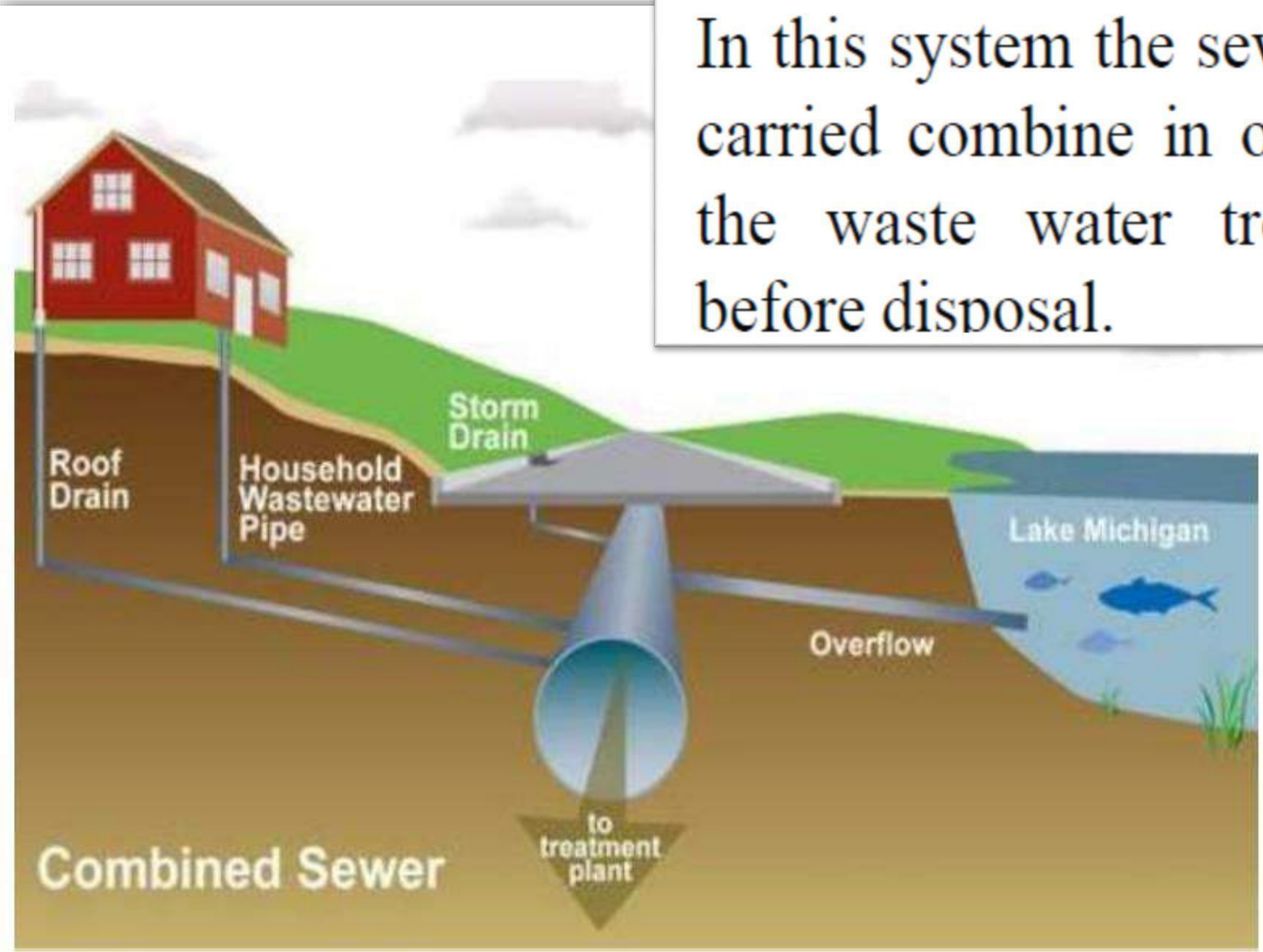
- It combines the good features of both systems.
- The silting is avoided due to entry of storm water.
- The storm water from houses is easily disposed off.
- The sewers are of reasonable size.

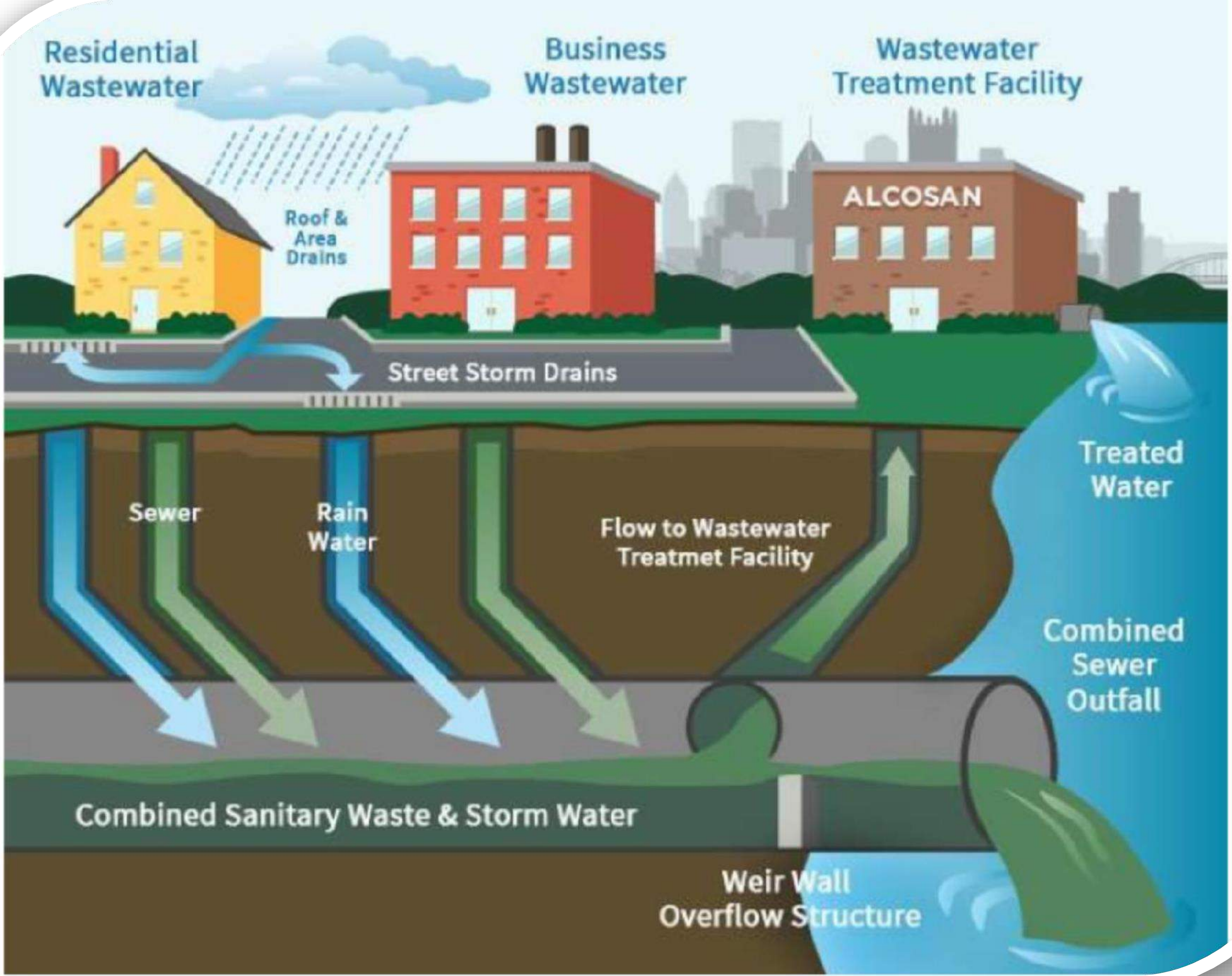
Disadvantages of Partially Separate Sewerage System

- A very small fraction of bad features of combined system are there in partially separated system.

3. Combined Sewerage System

In this system the sewage and storm water are carried combine in only one set of sewers to the waste water treatment Plant (WWTP) before disposal.




Residential Wastewater
Business Wastewater
Wastewater Treatment Facility
Roof & Area Drains
ALCOSAN
Street Storm Drains
Sewer
Rain Water
Flow to Wastewater Treatmet Facility
Treated Water
Combined Sewer Outfall
Combined Sanitary Waste & Storm Water
Weir Wall Overflow Structure

Disadvantages of Combined Sewerage System

In storm season sewer may overflow and the sewer may damage causing serious health risks

The combine sewer gets silted and becomes foul in dry days

Load on treatment plant is more because storm water is also carried there

The storm water gets polluted unnecessarily

The system becomes uneconomical when pumping is needed

Advantages of Combined Sewe System

Easy cleaning because of larger diamete

Reasonable maintenance cost.

Strength of sewage is reduced due to dilution of sewage by storm water.

This system requires only one set of sewer making it economical.

Why Estimation of Wastewater Discharge Required?

- ❖Under Estimation Would Result In Less Diameter Of Sewer Causing The Overflow Problems.
- ❖Over estimation of wastewater flow would result in a sewer of large diameter which would increase the cost of sewerage system.

Wastewater Discharge

- Dry Weather Flow
  - The Flow which always available through out the year.
  - It is the summation of domestic supply and industrial supply.
- Wet Weather Flow
  - It consist the combination of **Dry Weather Flow** And The **Storm Water flow.**
  - It is generally estimated when the combine sewerage system has adopted.

Estimation of Dry Weather Flow

- Domestic Wastewater
- Industrial Wastewater
- Ground water Infiltration in to Sewer through Joints.

Thumb Rule : Wastewater generated from a city is the 80% of the water supplied.

Factors Affect to DWF

- Rate of Water Supply
- Area Served
- Population Growth
- Infiltration as well as Exfiltration

Area Served

- Waste water generated in residential area depends upon the water supplied per capita per day.
- Waste water generated in Industrial area depends upon the type of industries.

Sr no.	Name of Industry	Unit of Production	Wastewater generation
1	Milk Production	Ton	20000
2	Steel	Ton	260000
3	Bread	Ton	2100-4200
4	Automobile	Vehicle	40000
5	Sugar	Tonne Cruched	1000 to 2000
6	Textile	100 kg	8000 to 14000

RATE OF WATER SUPPLY

- The rate of water supply to a cityltown is expressed so many litres/capita/day.
- The quantity of waste water entering the sewers would be less than the total quantity of water supplied.
- This is This is because of the fact that water is lost in domestic consumption, evaporation, lawn sprinkling. fire fighting, industrial consumption. However, private source of water supply(i e. water from domestic wells etc.) and infiltration of sub-soil water in the sewers increase the waste water flow rate.

Population Growth

- ❖ Wastewater treatment plant should also consider population forecasting for design period.
- ❖ Waste water treatment projects designed to serve for a period of 30 years.
- ❖ Design Period should neither be too long nor to short
- ❖ It should note exceed the useful life of the component structure or equipment.

Infiltration

- ❖Ground Pressure higher than pressure inside the sewer, thus Ground water entered in inside the sewer known as infiltration.
- ❖Depth of Sewer below the ground water level.
- ❖Size and length of sewer
- ❖Nature and type of soil
- ❖Workmanship during lying off sewer.

Exfiltration

- ❖Inside Sewer Pressure higher than outside Ground water, sewage shall leak out of the sewer through the faulty joints.
- ❖There is addition in sewage due to unaccounted private water supplies.
- ❖The Additional in sewage due to infiltration.
- ❖Water losses due to leakage
- ❖Some water is not entering the sewerage system e.g. gardening, garages for washing cars,etc

Wet Weather Flow (WWF)

- Storm water flow is also known as Wet Weather Flow (WWF)
- When rainfall takes place, a part of it infiltrates or percolates into the ground surface while the remaining flows over the land depending upon permeability of the ground, its surface slope and many other factors.
- The amount of water flowing over the ground surface, pavements, house roofs etc. is commonly known as 'runoff or the storm water.
- This storm water is ultimately drained through the sewers, otherwise the streets, roads etc. would be flooded.

RUNOFF/STORM WATER FLOW/WWF DEPENDS ON

(i) Catchment area

(ii) Ground slope

(iii) Permeability of ground

(iv) Extent of impervious area such as buildings, paved yards, non-absorbent road surface etc.

v) Extent of vegetation growth

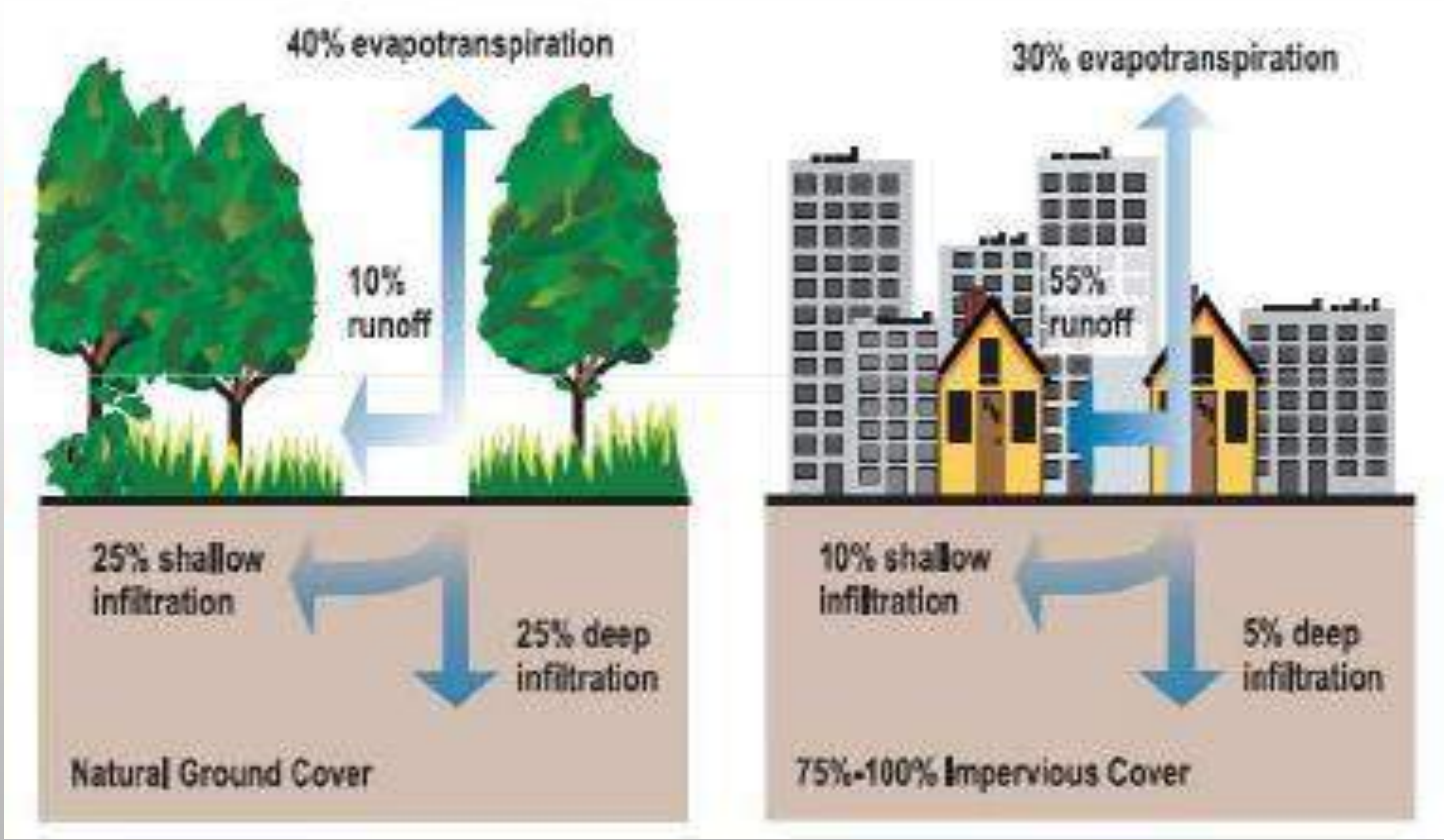
vi) Rain fall intensity

vii) Rainfall duration

viii) Condition of ground prior to the rainfall

ix) Concentration or compactness area

x) Climatic conditions such as wind, humidity, temperature etc.


40% evapotranspiration
10% runoff
25% shallow infiltration
25% deep infiltration
Natural Ground Cover
30% evapotranspiration
55% runoff
10% shallow infiltration
5% deep infiltration
75%-100% Impervious Cover

ESTIMATION OF STORM WATER FLOW 1.RATIONAL METHOD

- THE RATIONAL FORMULA IS MOST COMMONLY USED FOR DESIGN OF STORM DRAINS. IN
- TAKES INTO ACCOUNT THE FOLLOWING THREE FACTORS:

(i) CATCHMENT AREA *(A)*

(ii) IMPERMEABILITY FACTOR (I) OF THE CATCHMENT AREA.

(iii) INTENSITY OF RAINFALL *(R)*

- *THIS FORMULA CAN BE USED ONLY WHEN CATCHMENT AREA IS SMALLER THAN 400 HEC*

THE RATIONAL FORMULA

$$Q = K.A.I.R_i$$

Q = run off or storm water flow

K = constant which permits the expression of the factors A, I and R in convenient units.

Let, Q = runoff in cubic meters per second (cumec)

A = catchment area in hectares

Ai = impervious area = A x I

R_i = Intensity of rainfall in mm per hour.

I = impermeability coefficient

AVERAGE IMPERMEABILITY FACTOR

Let, $A_1, A_2, \ldots\ldots\ldots\ldots A_n$ = areas of the different surface of the catchment area.

$I_1, I_2, \ldots$ ..In= Corresponding impermeability factors surfaces for different

Impermeable area = $A_1I_1+A_2I_2+\ldots\ldots\ldots\ldots+A_nI_n=\Sigma A.I$ Hence , average impermeability factor

$$I_{avg} = (\Sigma A.I)/(\Sigma A)$$

(*a*) *General formula* :

$$R_i = \frac{25.4\,a}{t + b} \qquad \text{...(3.7)}$$

where R_i = rainfall intensity in mm/hour.

t = duration of storm in minutes ≈ time of concentration.

a, b = constants.

The United States Ministry of Health recommend the following values of constants a and b

Duration of storm	*constant a*	*constant b*
5 to 20 min.	30	10
20 to 100 min.	40	20

2. EMPIRICAL FORMULAE

If catchment area is more than 400 hec. Then following empirical formulae can be used.

i. Dicken's formula

ii. Ryve's formula

iii. Inglis's formula

iv. Talbot's formula

v. Fanning's formula

vi. Metcalf Eddy's formula

vii. McMath formula

viii. Burkli-Zeiglar formula

Time of concentration (t_c)

- Time required for the entire catchment to contribute to runoff at the point of interest for hydraulic design
  - time taken for the most hydraulically remote point of the catchment to contribute storm water to the outlet
- t_c of 10 to 300 minutes is acceptable for application in the rational method
  - for t_c <10 min., the rainfall intensity is unacceptably high
  - for t_c >300, the assumption of steady rainfall is less valid
- Factors affecting the t_c
  - Ponding, surface roughness and catchment slope
  - Fraction of impervious area and fraction of area directly connected to flow
  - Flow path length, channel slope, channel shape and flow pattern
- Urbanization decreases t_c

- DWF= % OF WASTEWATER X WATER SUPPLY X POPULATION X PEAK FACTOR

DWF = [0.8 X W/S (lpcd)X Population X 3 /(1000X 24X60X60)]m^3/S

WWF= AIR/360 m^3/S

A= AREA IN HECTARE (HA)

I= COEFFICIENT OF RAINFALL/ IMPERMEABILITY FACTOR

R= RAINFALL INTENSITY IN MM/Hr

COMBINED SYSTEM= DWF+ WWF

SEPARATE SYSTEM= DWF & WWF

PARTIALLY COMBINED SYSTEM= DWF + PORTION OF WWF

... This formula states that

$$\boxed{Q_p = \left(\frac{1}{36}\right) K \cdot p_c \cdot A} \qquad \text{...(3.1)}$$

where Q_p = Peak rate of runoff in cumecs.

K = Coefficient of runoff.

A = The catchment area contributing to runoff at the considered point, in hectares.

p_c = Critical rainfall intensity of the design frequency *i.e.* the rainfall intensity during the critical rainfall duration equal to the time of concentration, in cm/hr.

Example 3.4. *A population of 30,000 is residing in a town having an area of 60 hectares. If the average coefficient of runoff for this area is 0.60, and the time of concentration of the design rain is 30 minutes, calculate the discharge for which the sewers of a proposed combined system will be designed for the town in question. Make suitable assumptions where needed.*

Solution. Let us first assume that the town is provided with a planned water supply from the water-works at an average per capita rate equal to 120 litres/day/person*. Also assume that 80% of this water supply will be reaching the sewers as sanitary sewage.

$\therefore$ Quantity of sanitary sewage produced per day

$$= \left(\frac{80}{100}\right) 120 \times 30{,}000 \text{ litres}$$

$$= 0.8 \times 120 \times 30 \text{ cu. m} = 2880 \text{ cu. m}$$

Quantity of sanitary sewage produced per second

$$= \frac{2880}{24 \times 60 \times 60} \text{ cumecs} = 0.033 \text{ cumecs}$$

$\therefore$ Average sewage discharge = 0.033 cumecs.

Assuming the maximum sewage discharge to be three times the average, we have

Max. sewage discharge

$$= 3 \times 0.033 = \mathbf{0.1\ cumecs.}$$

The storm water discharge can be computed by using Rational formula ; *i.e.*

$$Q_p = \frac{1}{36} K \,.\, p_c \,.\, A$$

Using $$p_c = \frac{100}{T+20} \quad (\textit{i.e.}\ \text{Eq. } 3.8)$$

We have $$p_c = \frac{100}{30+20} = 2 \text{ cm/hr.}$$

$$\therefore \quad Q_p = \frac{1}{36} \times 0.60 \times 2 \times 60 \text{ cumecs} = \mathbf{2\ cumecs.}$$

Hence, the total peak discharge for which the sewers of the combined system should be designed

= Max. sewage discharge + Max. storm runoff

= 0.1 + 2.0 = **2.1 cumecs. Ans.**

Example 3.3. *The drainage area of one sector of a town is 12 hectares. The classification of the surface of this area is as follows :*

Percent of total surface area	*Type of surface*	*Coefficient of runoff*
20%	Hard pavement	0.85
20%	Roof surface	0.80
15%	Unpaved street	0.20
30%	Garden and Lawn	0.20
15%	Wooded area	0.15

If the time of concentration for the area is 30 minutes, find the maximum runoff.

Use the formula

$$R = \frac{900}{t + 60}.$$

Solution. If A represents the total area, then we have

$$K_1 A_1 = 0.85 \times 0.20\, A = 0.17\, A$$

$$K_2A_2 = 0.80 \times 0.20\,A = 0.16\,A$$
$$K_3A_3 = 0.20 \times 0.15\,A = 0.03\,A$$
$$K_4A_4 = 0.20 \times 0.30\,A = 0.06\,A$$
$$K_5A_5 = 0.15 \times 0.15\,A = 0.0225\,A$$
$$\Sigma = 0.4425\,A$$

$$K = \frac{\Sigma KA}{A} = \frac{0.4425\,A}{A}$$

or $$K = 0.4425$$

Now, in the formula of the type

$$R = \frac{900}{t + 60}$$

R is the rainfall intensity, gene concentration time in minutes.

$\therefore$ R = rainfall intensity in mm/hr

$$= \frac{900}{30 + 60} = 10 \text{ mm/hr.} = 1 \text{ cm/hr.}$$

$\therefore$ P_c to be used in our Rational formula (Eq. 3.1)

$$= 1 \text{ cm/hr.}$$

Using Rational formula, we have

$$Q_p = \frac{1}{36} K.p_c.A$$

$$= \frac{1}{36} \times 0.4425 \times 1 \times 12 \text{ cumecs.}$$

$$= 0.1475 \text{ cumecs ; say } 0.148 \text{ cumecs.}$$

$\therefore$ Maximum rate of runoff expected from the area

$$= 0.148 \text{ cumecs.} \quad \textbf{Ans.}$$

Example 3.1. *Assuming that the surface on which the rain falls in a district is classified as follows :*

20% of the area consists of roof for which the runoff ratio is 0.9, 20% of the area consists of pavements for which the runoff ratio is 0.85, 5% of the area consists of paved yards of houses for which run off ratio is 0.80, 15% of area consists of macadam roads for which run off ratio is 0.40, 35% of the area consists of lawns, gardens and vegetable plants for which the runoff ratio is 0.10, and the remaining 5% of the area is wooded for which the runoff ratio is 0.05 ; determine the coefficient of runoff for the area.

If the total area of the district is 36 hectares and the maximum rain intensity is taken as 5 cm / hr ; what is the total runoff for the district ?

$$K = \frac{K_1A_1 + K_2A_2 + K_3A_3 + \ldots + K_nA_n}{A_1 + A_2 + A_3 + \ldots + A_n}$$

$$= \frac{\Sigma KA}{\Sigma A} = \frac{\Sigma KA}{A}$$

example, we have

$$K_1A_1 = \frac{20}{100}A(0.90) = 0.18\,A$$

$$K_2A_2 = \frac{20}{100}A(0.85) = 0.17\,A$$

$$K_3A_3 = \frac{5}{100}A(0.80) = 0.04\,A$$

$$K_4A_4 = \frac{15}{100}A(0.4) = 0.06\,A$$

$$K_5A_5 = \frac{35}{100}A(0.1) = 0.035\,A$$

$$K_6A_6 = \frac{5}{100}A(0.05) = 0.0025\,A.$$

$$\therefore \quad K = \frac{K_1A_1 + K_2A_2 + K_3A_3 + K_4A_4 + K_5A_5 + K_6A_6}{A}$$

$$= \frac{0.18A + 0.17A + 0.04A + 0.06A + 0.035A + 0.0025A}{A}$$

$$= 0.4875.$$

Hence, the runoff factor for the entire area

$$= 0.4875. \quad \textbf{Ans.}$$

The peak discharge from the area may be computed by using the rational formula given by Eq. (3.1) as

$$Q_p = \frac{1}{36}\,.\,K.p_c.A$$

Here, we have

$K = 0.4875$

p_c = Critical rain intensity = 5.0 cm/hr (given)

A = 36 hectares

$$\therefore \quad Q_p = \frac{1}{36}(0.4875)(5.0)(36.0) = 2.4375 \text{ cumecs.}$$

Say 2.44 cumecs. **Ans.**

Example 3.2. *If in the above example, the density of population is 250 per hectare, and the quota of water supply per day is 225 litres ; calculate the quantity of*

(a) Sewage for which the sewers of a separate system, should be designed.

(b) Storm water for which the sewers of a partially separate system should be designed.

Solution. Area of district

$= 36$ hectares.

Population density $= 250$ persons per hectare

$\therefore$ Population $= 36 \times 250 = 9000.$

Average water supply per day

$= 225$ litres/person

$\therefore$ Average quantity of water supplied to the district per day

$= 225 \times 9000$ litres $= 20,25,000$ litres $= 2,025$ cu. m.

$\therefore$ Rate of water supplied

$$= \frac{2025}{24 \times 60 \times 60} \text{ cumecs} = 0.0234 \text{ cumecs.}$$

Assuming the sewage discharge as 0.8 times the water supplied, we have

Average rate of sewage produced

$$= 0.8 \times 0.0234 = 0.0187 \text{ cumecs.}$$

Now assuming the peak rate of sewage as three times the average,we have

The peak rate of sewage flow

$$= 3 \times 0.0187 = 0.056 \text{ cumecs.} \quad \textbf{Ans.}$$

Case. (*b*) In case of partially separate system, the storm water from roofs and paved yards of houses will be allowed to enter the sewers. Now, from the previous example, we have

$$\text{Area of roofs} = \frac{20}{100} \times 36 \text{ hectares} = 7.2 \text{ hectares.}$$

Coefficient of runoff for roofs = 0.90

$$\text{Area of pavements} = \frac{5}{100} \times 36 = 1.8 \text{ hectares}$$

Coefficient of runoff for pavements = 0.80.

$\therefore$ The discharge from roofs and pavements, as given by rational formula, using $p_c = 5$ cm/hr. is

$$= \left[\frac{1}{36} \times 0.90 \times 5.0 \times 7.2 + \frac{1}{36} \times 0.80 \times 5.0 \times 1.8\right] \text{ cumecs}$$

$$= 1.1 \text{ cumecs.}$$

Hence, the storm water which must pass through the sewers of a partially separate system = 1.1 cumecs. **Ans.**

Note. This is about 20 times the peak rate of sewage produced (*i.e.* 0.056 cumecs). Moreover, strictly speaking, the sewers of the partially separate system should be designed for carrying this storm water plus the sewage, *i.e.* for a discharge = 1.1. + 0.056 = 1.156 cumecs.

MATERIALS OF SEWER

- (I) ASBESTOS CEMENT SEWERS
- (II) BRICKS SEWERS
- (III) CAST IRON SEWERS
- (IV) CEMENT CONCRETE PLAIN OR REINFORCED
- (V) CORRUGATED IRON SEWERS
- (VI) STONEWARE SEWERS
- (VII) STEEL SEWERS
- (VIII) PLASTIC SEWERS
- (IX) WOODEN SEWERS

ASBESTOS CEMENT SEWERS

- MIXTURE OF ASBESTOS FIBERS, SILICA AND CEMENT. ASBESTOS FIBERS ACT AS REINFORCEMENT
- SIZE 10 TO 100 CM DIA AND L= 4.0 M.
- EASILY ASSEMBLED -SPECIAL COUPLING, CALLED ‗RING TIE COUPLING' OR SIMPLEX JOINT
- RESISTANT TO CORROSION , JOINTS ARE FLEXIBLE TO PERMIT 12° DEFLECTION

ADVANTAGE

- LIGHT IN WEIGHT , EASY TO CUT, INTERIOR IS SMOOTH – HIGHLY EFFICIENT

DISADVANTAGES

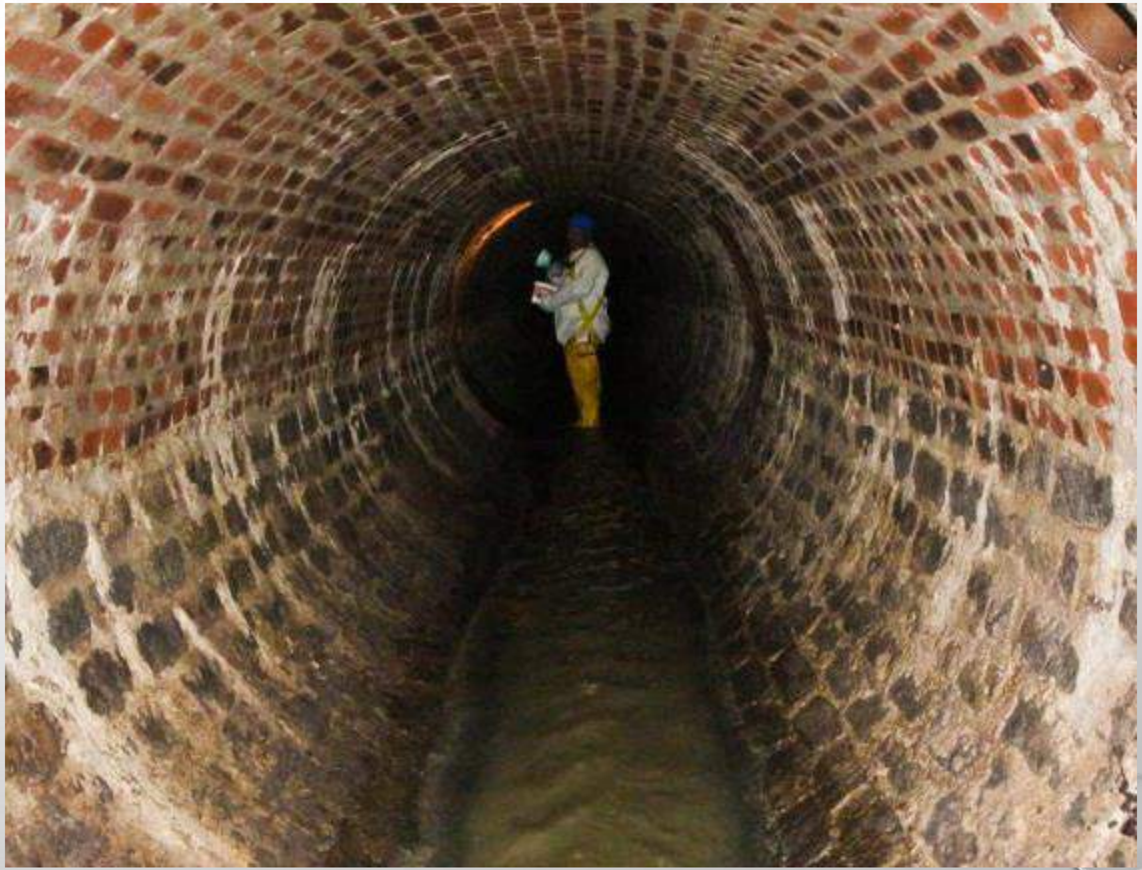
- NOT VERY STRONG, SUSCEPTIBLE TO CORROSION

Bricks sewers:

- made at site
- are replaced by concrete sewers
- large size combined sewer or particularly for storm water drains
- pipes are plastered from outside to avoid entry of tree roots and ground water
- lined from inside with stone ware or ceramic block
  - smooth and hydraulically efficient
  - make the pipe resistant to corrosion

Asbestos Cement Sewers

Bricks sewers:



CAST IRON SEWERS

- STRONGER AND CAPABLE TO WITHSTAND GREATER TENSILE, COMPRESSIVE, AS WELL AS BENDING STRESSES
- USED FOR WHERE PIPES ARE RUNNING UNDER PRESSURE OUTFALL SEWERS, RISING MAINS OF PUMPING STATIONS, AND INVERTED SIPHONS,
- HEAVY TRAFFIC LOAD, SUCH AS SEWERS BELOW RAILWAYS AND HIGHWAYS
- 100% LEAK PROOF INSIDE WITH CEMENT CONCRETE, COAL TAR PAINT, EPOXY, ETC.

ADVANTAGE

- LESS RESISTANT TO CORROSION

DISADVANTAGES

- COSTLY

Plain Cement Concrete or RCC : 1: 1.5: 3 -

- PC- 0.45 m diameter RCC -1.8m dia,
- cast in situ or precast pipes
- reinforcement 0.25% ,
- single cage - internal pressure < 0.8 m
- double cage - internal pressure >0.8 m
- **elliptical cage** - larger dia to external pressure;
- smooth and hydraulically efficient
  - make the pipe resistant to corrosion
- Advantage
  - Tension, Compression,. Easily molded, economical
- Disadvantage
  - Corrode, carrying capacity, susceptible to erosion

Plain Cement Concrete or RCC

Cast Iron Sewers

CORRUGATED IRON SEWERS:

- GALVANIZATION OR BY BITUMINOUS COATINGS
- DIA UPTO 450CM.

PLASTIC SEWERS PVC

- INTERNAL DRAINAGE WORKS IN HOUSE
- SIZES 75 TO 315 MM EXTERNAL DIAMETER
- SMOOTH INTERNAL SURFACE
- RESISTANT TO CORROSION, LIGHT WEIGHT OF PIPE, ECONOMICAL IN LAYING, JOINTING AND MAINTENANCE
- TOUGH AND RIGID

High Density Polythylene (HDPE) Pipes

- not brittle ,resistant damage
- joined by welding or with detachable joints up to 630 mm smooth internal surface
- conveyance of industrial wastewater

Corrugated iron sewers

Plastic sewers PVC

High Density Polythylene (HDPE) Pipes

STEEL SEWERS

- LIGHTNESS, IMPERVIOUSNESS AND RESISTANCE TO HIGH PRESSURE
- TRUNK OR OUTFALL SEWERS
- SITUATIONS SUCH AS PRESSURE MAIN SEWERS, UNDER WATER CROSSING, BRIDGE CROSSING, NECESSARY CONNECTIONS FOR PUMPING STATIONS, LAYING PIPES OVER SELF SUPPORTING SPANS, RAILWAY CROSSINGS, ETC.
- PRESSURE BETTER THAN CI PIPES

DISADVANTAGE

- CANNOT WITHSTAND HIGH EXTERNAL LOAD
- SUSCEPTIBLE TO CORROSION AND ARE NOT GENERALLY USED FOR PARTIALLY FLOWING SEWERS

VITRIFIED CLAY OR STONEWARE SEWERS

- USED FOR HOUSE CONNECTIONS AS WELL AS LATERAL SEWERS
- 5 CM TO 30 CM INTERNAL DIA WITH L 0.9 TO 1.2 M

ADVANTAGES

- RESISTANT TO CORROSION , INTERIOR SURFACE IS SMOOTH, STRONG IN COMPRESSION.
- DOES NOT ABSORB WATER MORE THAN 5% OF THEIR OWN WEIGHT, WHEN IMMERSED IN WATER FOR 24 H

DISADVANTAGE

- HEAVY, BULKY AND BRITTLE AND WEAK IN TENSION

Steel sewers

Vitrified Clay or Stoneware Sewers

SHAPES OF SEWERS

- GENERALLY CIRCULAR PIPES - BELOW GROUND LEVEL, SLOPPING - TOWARDS THE OUTFALL
- DESIGNED TO FLOW UNDER GRAVITY
- OTHER THAN CIRCULAR ARE ALSO USED

A. STANDARD EGG-SHAPED SEWER

B. NEW EGG-SHAPED SEWER

C. HORSE SHOE SHAPED SEWER

D. PARABOLIC SHAPED SEWER

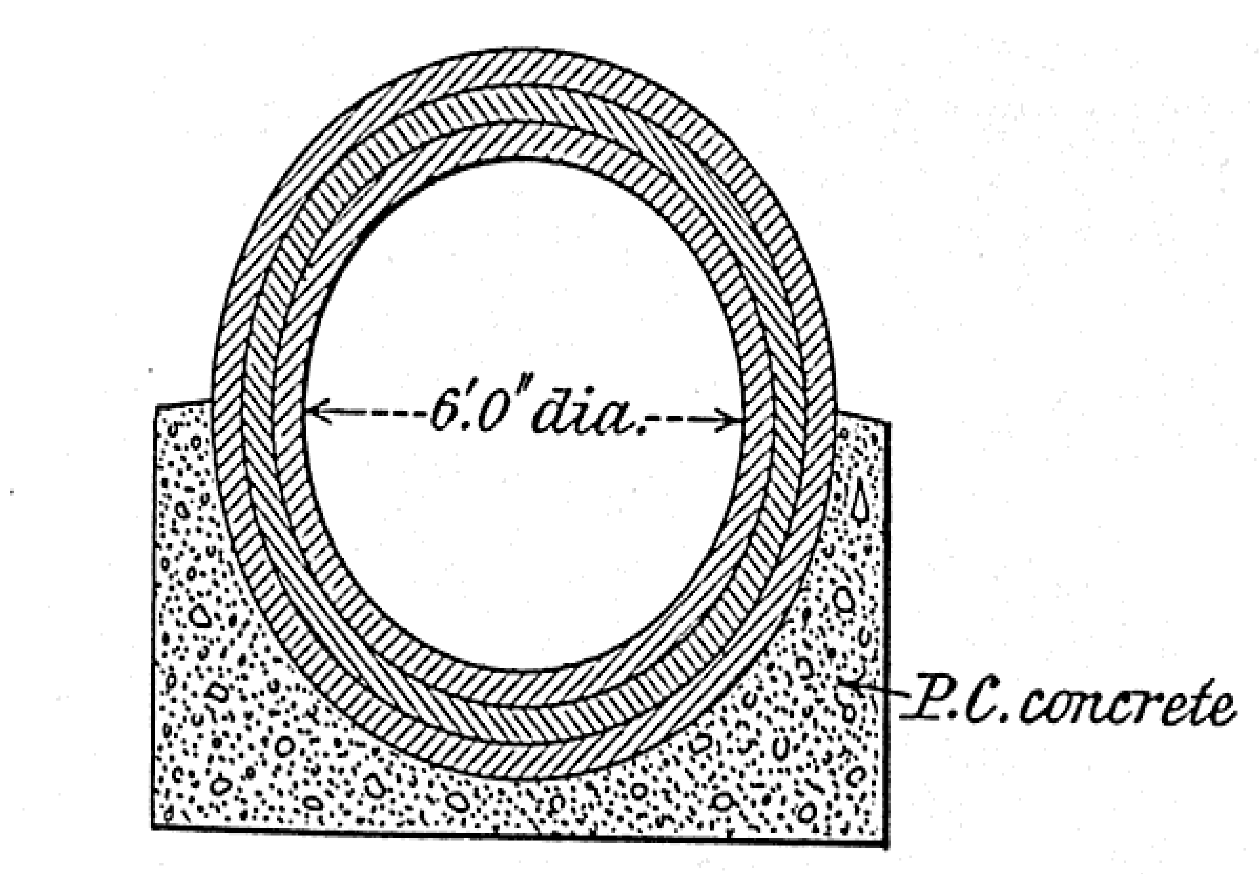
E. SEMI-ELLIPTICAL SECTION

F. RECTANGULAR SHAPE SECTION

G. U-SHAPED SECTION

H. SEMI-CIRCULAR SHAPED SEWER

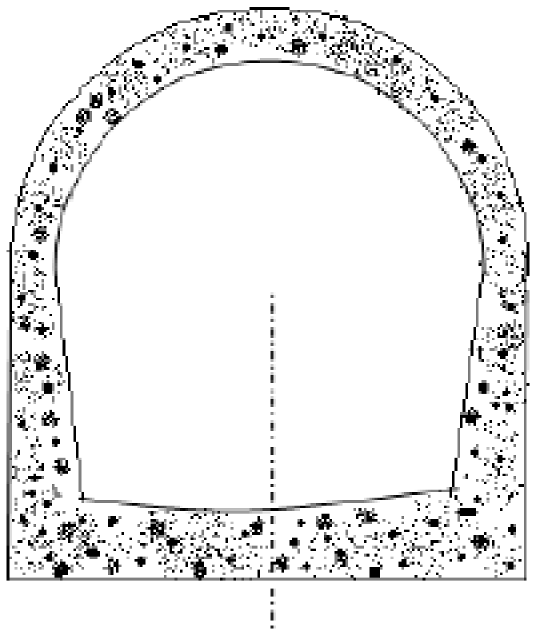
I. BASKET HANDLED SHAPE SEWER



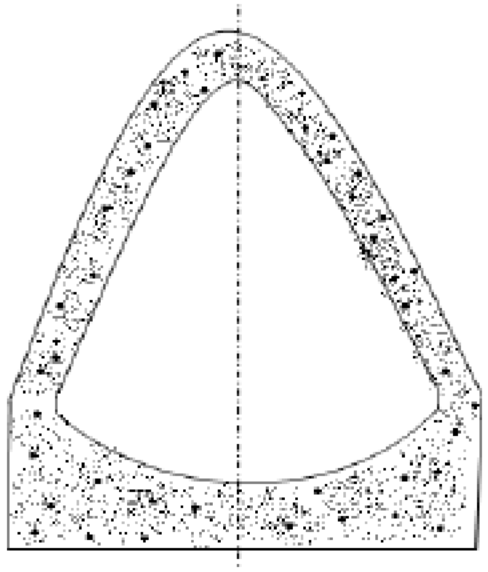
(a) Standard Egg Shaped Sewer

(b) New/ Modified Egg shaped Sewer

- got better hydraulic properties,
- it is costly
- longer perimeter more material for construction
- odd shape it is difficult to construct
- Rarely used in India
- suitable in care of combined sewers



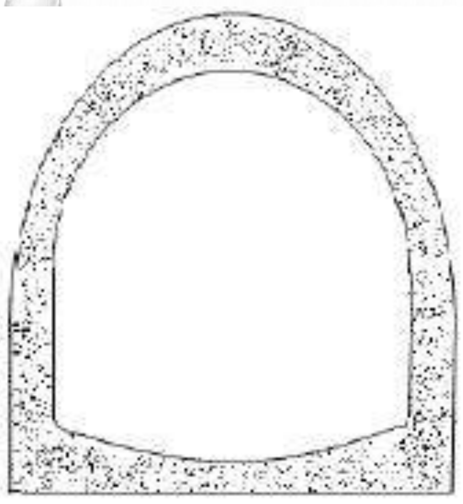
(c) Horse shoe sewer section



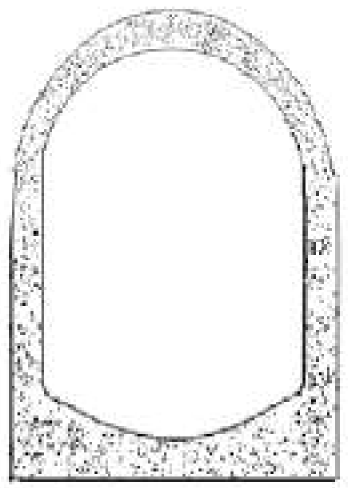
(d) Parabolic section

- semi-circular with sides inclined or vertical
- bottom may be flat, circular or paraboloid .
- large sewers with heavy discharged
- tunnels, trunk sewer

- upper arch takes the shape of parabola
- invert of the sewer may be flat, parabolic or elliptical .
- used for the disposal of relatively small quantities of sewage



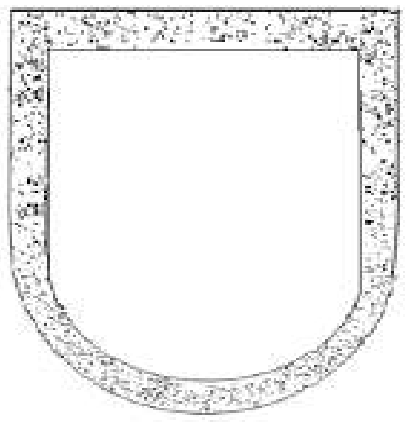
(e) Semi-elliptical section



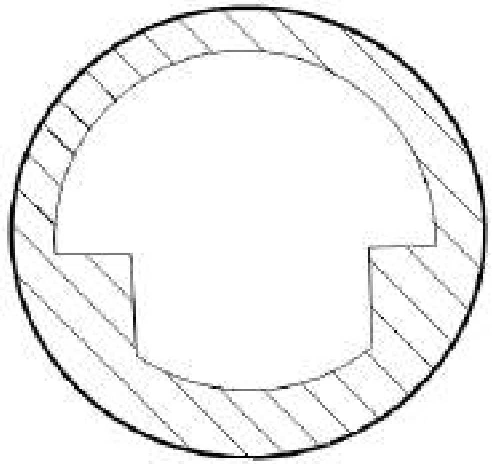
Semi-circular Section

- more suitable for soft soils as they are more stable
- not suitable for carrying low discharges
- Dia > 180 cm.

- gives a wider care at the bottom
- suitable for constructing large sewers with less available headroom
- replaced by rectangular sewers.



(g) U-shaped section



(i) Basket-Handle Section

- Trench provided at the bottom is called cunnette
- easy to construct
- invert may be flat or semi-circular
- sides are generally vertical and top may be flat or arched

- resembles the shape of a basket handle
- Small discharges flow - bottom narrower portion.
- combined sewage flows in the full section.

LAYING OF SEWERS

SURVEY LINE AND GRADE

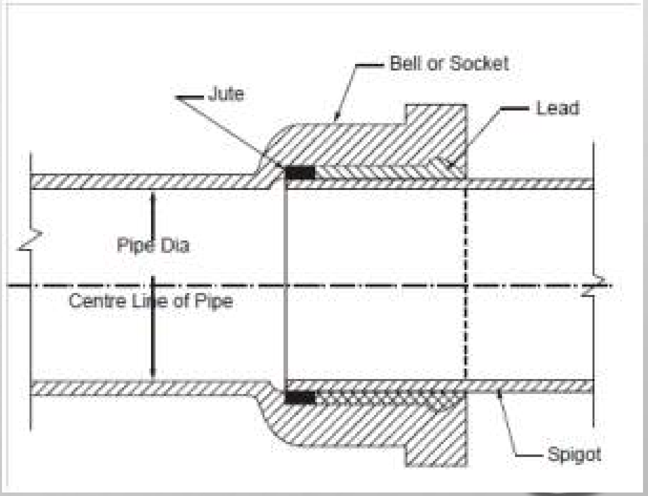
- THE CONTRACTOR SHALL SET TEMPORARY BENCH MARKS (TBM'S) AT A MAXIMUM 500 FOOT INTERVAL
- CHECK LINE AND GRADE OF THE PIPE BY LASER BEAM METHOD
- IF LINE AND GRADE DO NOT MEET SPECIFIED LIMITS, THE WORK SHALL BE STOPPED, REMEDIAL MEASURES

PIPE PREPARATION AND HANDLING

- ALL PIPE AND FITTINGS SHALL BE INSPECTED PRIOR TO LOWERING INTO TRENCH TO INSURE NO CRACKED, BROKEN
- SHALL CLEAN ENDS OF PIPE THOROUGHLY AND REMOVE FOREIGN MATTER AND DIRT FROM INSIDE OF PIPE AND KEEP CLEAN DURING AND AFTER LAYING
- PIPE SHALL BE PROPERLY LOWERED TO AVOID ANY PHYSICAL DAMAGE TO THE PIPE. NOT BE DROPPED OR DUMPED INTO TRENCHES

SEWER PIPE LAYING

- LAYING TO LINE AND GRADE IN THE TRENCH ONLY AFTER DEWATERED
- MUD, SILT, GRAVEL AND OTHER FOREIGN MATERIAL SHALL BE KEPT OUT OF THE PIPE
- ALL PIPE LAID SHALL BE RETAINED IN POSITION UNTIL SUFFICIENT BACKFILL HAS BEEN COMPLETED TO ADEQUATELY HOLD THE PIPE IN PLACE
- VARIANCE < "ONE-THIRTY SECOND OF AN INCH / INCH OF PIPE" DIAMETER AND NOT TO EXCEED ONE-HALF (½) INCH
- THE SEWER PIPE SHALL BE INSTALLED WITH THE BELL END FORWARD OR UPGRADE.
- WHEN PIPE LAYING IS NOT IN PROGRESS THE OPEN END OF THE PIPE SHALL BE
- KEPT TIGHTLY CLOSED WITH AN APPROVED TEMPORARY PLUG.



TESTING OF SEWER LINE

TYPES OF SEWER TESTS

- **SEWER TESTING FOR LEAKAGE (WATER TEST) :**
- **SMOKE TEST**
- **SEWER TESTING FOR STRAIGHTNESS OF ALIGNMENT AND OBSTRUCTION**

SEWER TESTING FOR LEAKAGE (WATER TEST)

- TO ENSURE NO LEAKAGE THROUGH THE JOINTS AFTER JOINTS TO SET IN
- TESTED FROM MANHOLE TO MANHOLE UNDER TEST PRESSURE OF 1.5M OF WATER HEAD (DEPTH OF WATER IN THE MANHOLE IS MAINTAINED AT ABOUT 1.5M.)
- LOWER END OF THE SEWER IS PLUGGED
- WATER IS THEN FILLED IN THE MANHOLE AT THE UPPER END AND ALLOWED TO FLOW THROUGH THE SEWER LINE.
- WATCHED BY MOVING ALONG THE TRENCH
- THE JOINTS WHICH LEAK OR SWEAT ARE REPAIRED, LEAKAGE PIPE WILL BE REPLACED

SEWER TESTING FOR STRAIGHTNESS OF ALIGNMENT AND OBSTRUCTION (TEST FOR STRAIGHTNESS AND OBSTRUCTION)

- TESTED BY PLACING A MIRROR AT ONE END OF THE SEWER LINE AND A LAMP AT THE OTHER END.
- IF THE PIPE LINE IS STRAIGHT, THE FULL CIRCLE OF LIGHT WILL BE OBSERVED
- IF THE PIPE LINE IS NON-STRAIGHT, THIS WOULD BE APPARENT AND THE MIRROR WILL ALSO INDICATE ANY OBSTRUCTION IN THE PIPE BARREL.
- ANY OBSTRUCTION

INSERTING AT THE UPPER END - A SMOOTH BALL OF DIA 13MM < INTERNAL DIAMETER OF SEWER PIPE

- YARN OR MORTAR THROUGH THE JOINTS ETC. THE BALL SHALL ROLL DOWN THE INVERT OF THE SEWER PIPE AND EMERGE AT THE LOWER END

SMOKE TEST

- THIS TEST IS PERFORMED FOR SOIL PIPES, VENT PIPES LAID ABOVE GROUND.
- CONDUCTED UNDER A PRESSURE OF 2.5 M OF WATER AND MAINTAINED FOR 15 MINUTES AFTER ALL TRAP REAL HAVE BEEN FILLED WITH WATER.
- THE SMOKE IS PRODUCED BY BURNING OIL WASTE OR TAR PAPER IN COMBUSTION CHAMBER OF A SMOKE MACHINE.

VENTILATION OF SEWER

- ORGANIC AND INORGANIC MATTERS - DECOMPOSE AND PRODUCE GASES
- CAUSE AIR LOCKS IN SEWERS AND AFFECT THE FLOW OF SEWAGE, DANGEROUS FOR THE MAINTENANCE SQUAD WORKING IN
- <u>METHODS OF VENTILATION</u>
- LAYING SEWER LINE AT PROPER GRADIENT.
- RUNNING THE SEWER AT HALF FULL OR 2/3 DEPTH.
- PROVIDING MANHOLE WITH GRATINGS.
- PROPER HOUSE DRAINAGE.
- PROVIDING THE VENTILATING COLUMNS OR SHAFTS.

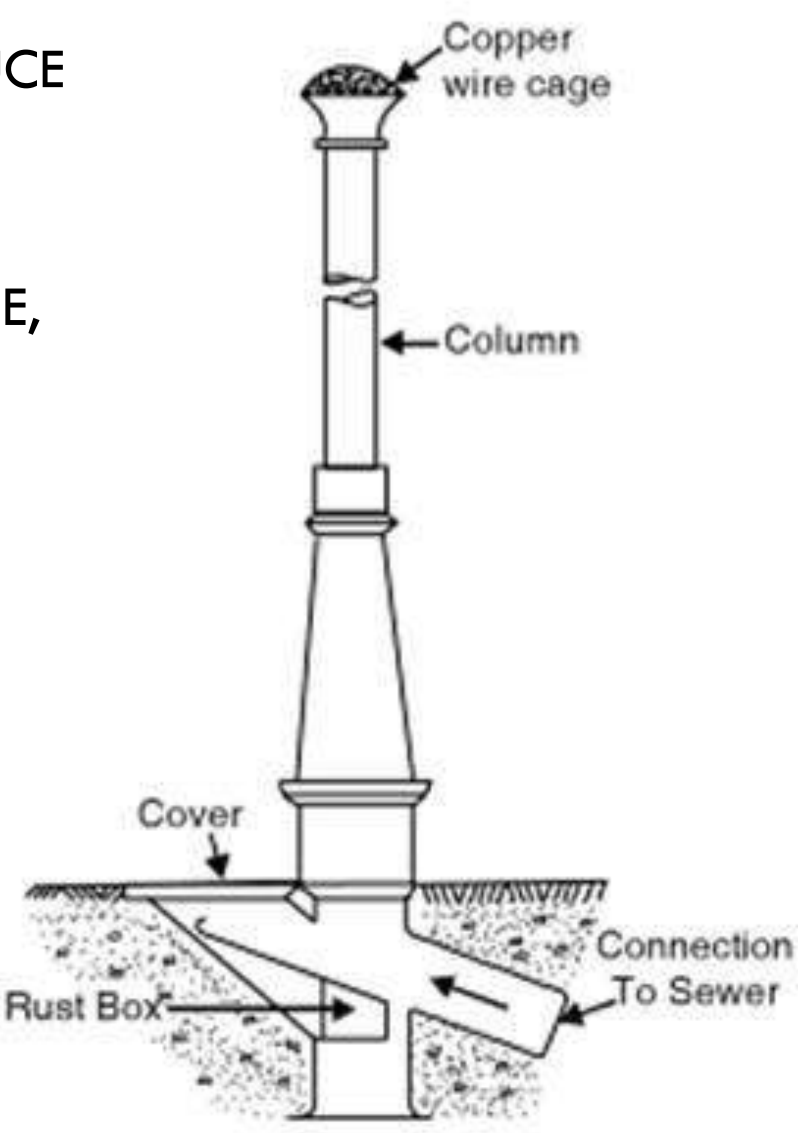
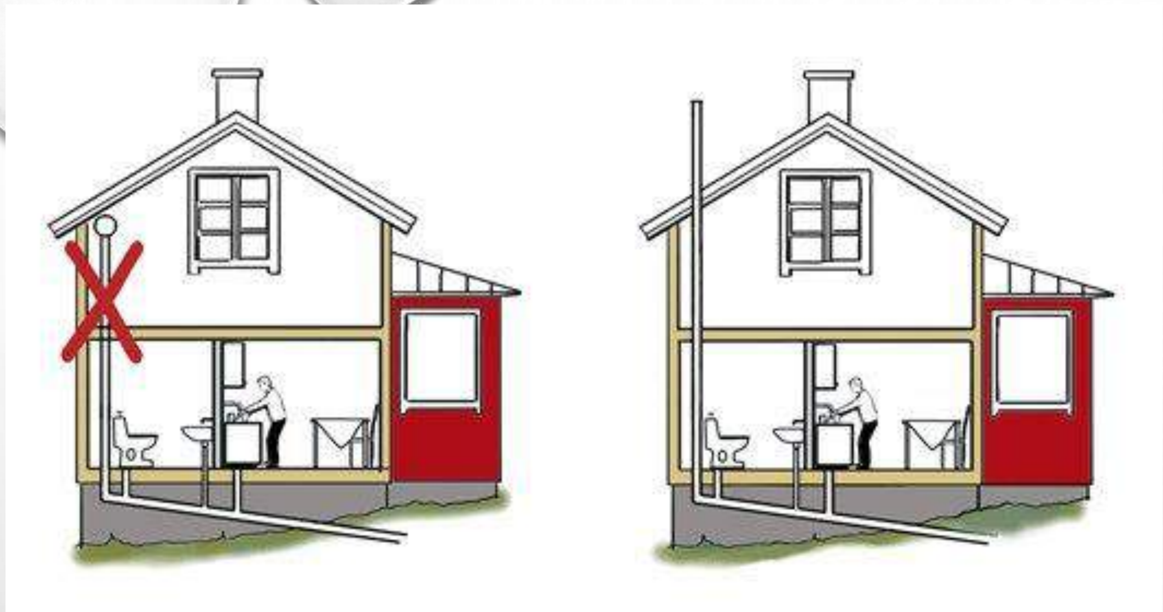
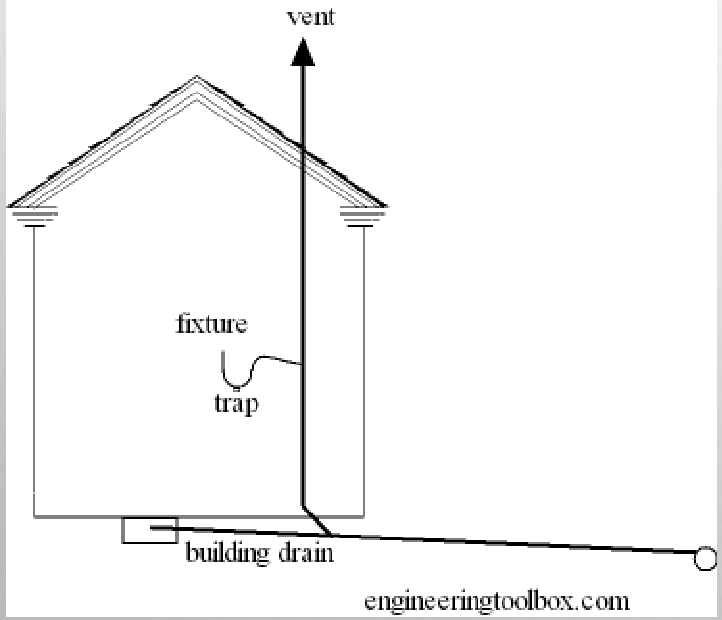


Fig. Ventilation column




vent
fixture
trap
building drain
engineeringtoolbox.com


Brighton-le-Sands
MOTORWAY 5
Port Botany
Sydney
AIRPORT
MOTORWAY 5
Liverpool
Canberra

SEWER APPURTENANCES

for proper functioning and to facilitate maintenance of the sewage system, various additional structures have to be constructed on the sewer lines. These structures are sewer appurtenances.

- MANHOLES
- INLETS
- CATCH BASINS
- FLUSHING DEVICES
- REGULATORS
- INVERTED SIPHONS
- GREASE AND OIL TRAPS
- LAMP HOLES
- LEAPING WEIRS
- JUNCTION CHAMBERS

MANHOLES

- R.C.C OR MASONRY CHAMBERS CONSTRUCTED ON THE SEWER LINE TO FACILITATE A MAN TO ENTER THE SEWER LINE AND MAKE THE NECESSARY INSPECTION AND REPAIRS
- FITTED WITH SUITABLE CAST IRON COVERS
- CHANGE IN DIRECTION, CHANGE IN PIPE SIZE, OR CONSIDERABLE CHANGE IN GRADIENT

- CLASSIFICATION - DEPTH
  - SHALLOW MANHOLE 0.75 -0.9 M
  - NORMAL MANHOLE – 0.9 – 1.5 M
  - DI

TYPES OF MANHOLE

SHALLOW MANHOLE

- AT START OF BRANCH SEWER
- INSPECTION CHAMBER

NORMAL MANHOLE

- RECTANGULAR/SQUARE
- 0.8M X 1.2 M/1M X 1M
- SECTION NOT CHANGED WITH DEPTH
- HEAVY COVER AT THE TOP

Deep Manhole

- Larger at bottom
- Reduced at top
- By providing offset
- Steps to access
- Heavy cover at the top

DEEP MANHOLE C/S

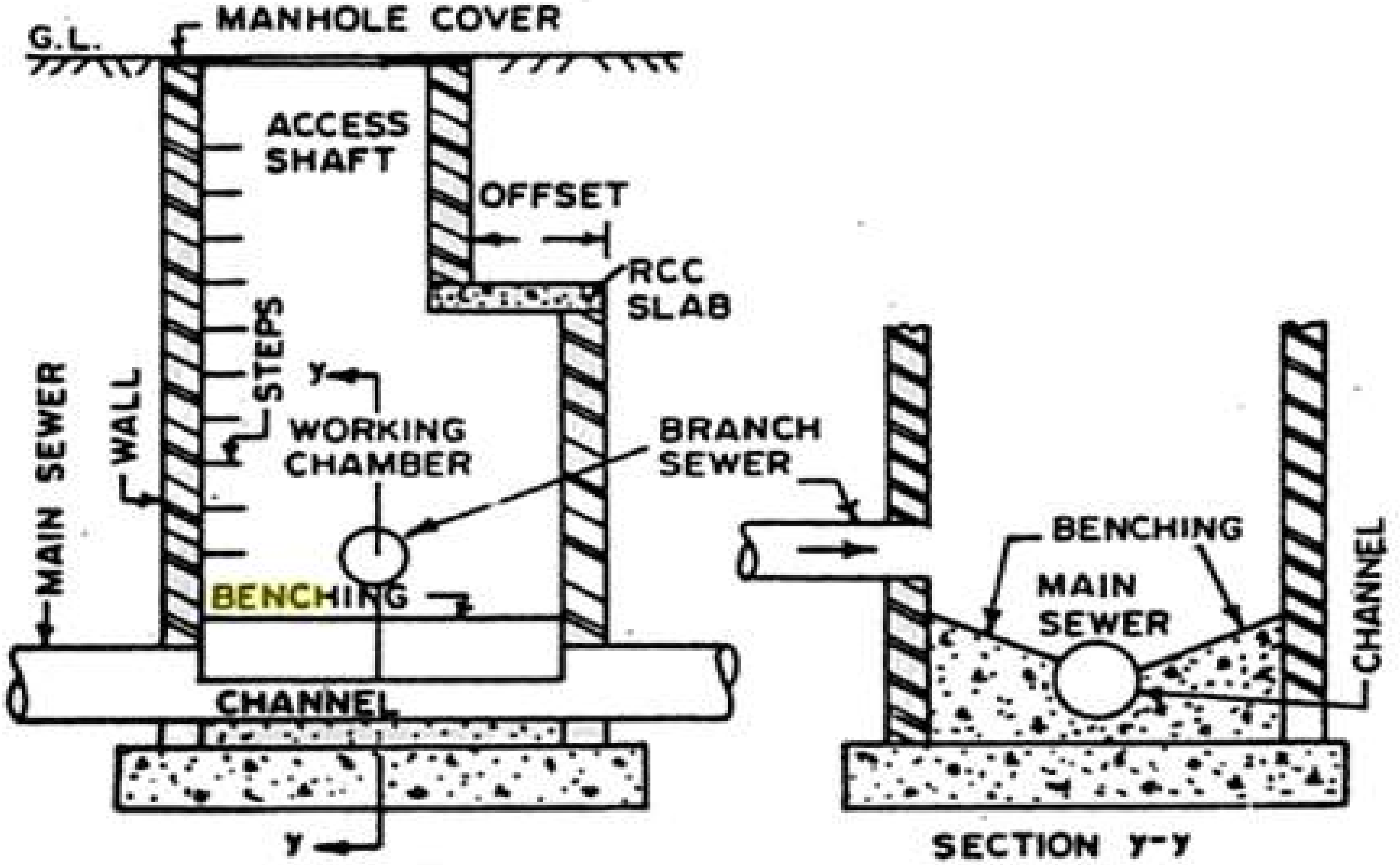
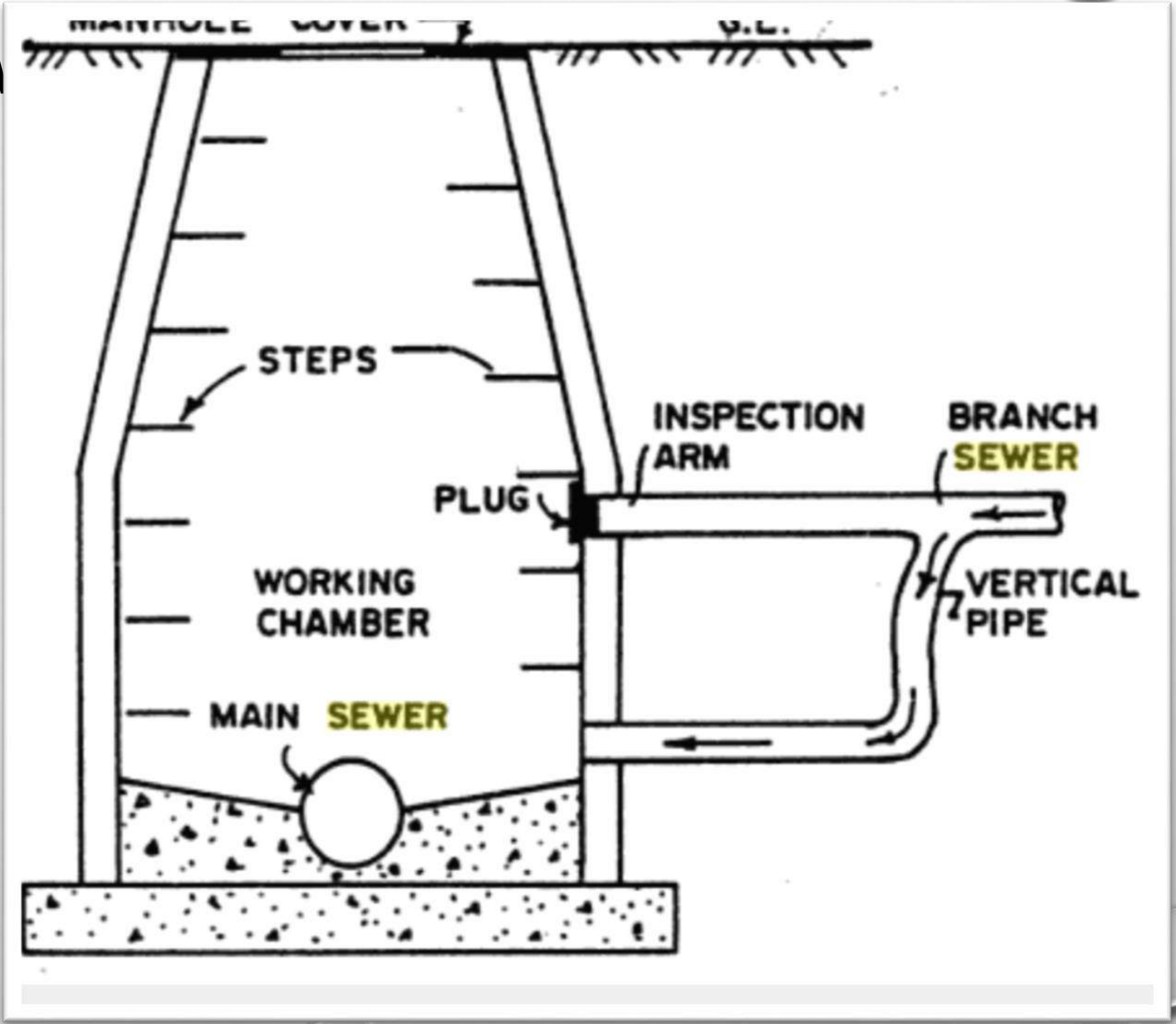


FIG. DEEP MANHOLE

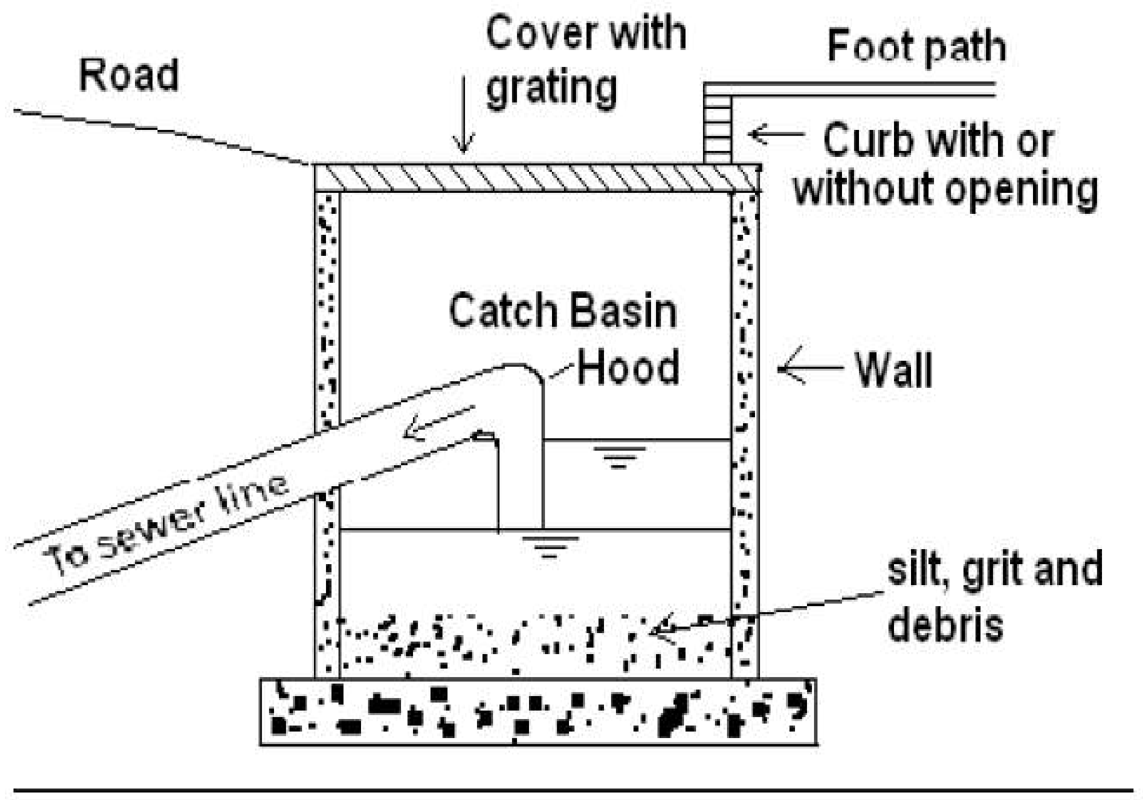
DROP M

- IT IS A MEASURE OF CONNECTING HIGH LEVEL BRANCH SEWER TO LOW LEVEL MAIN SEWER.
- THROUGH A VERTICAL PIPE.
- DIFFERENCE IN LEVEL > 60CM BETWEEN BRANCH & MAIN SEWER,
- AVOIDS INCREASING THE SEWER GRADE

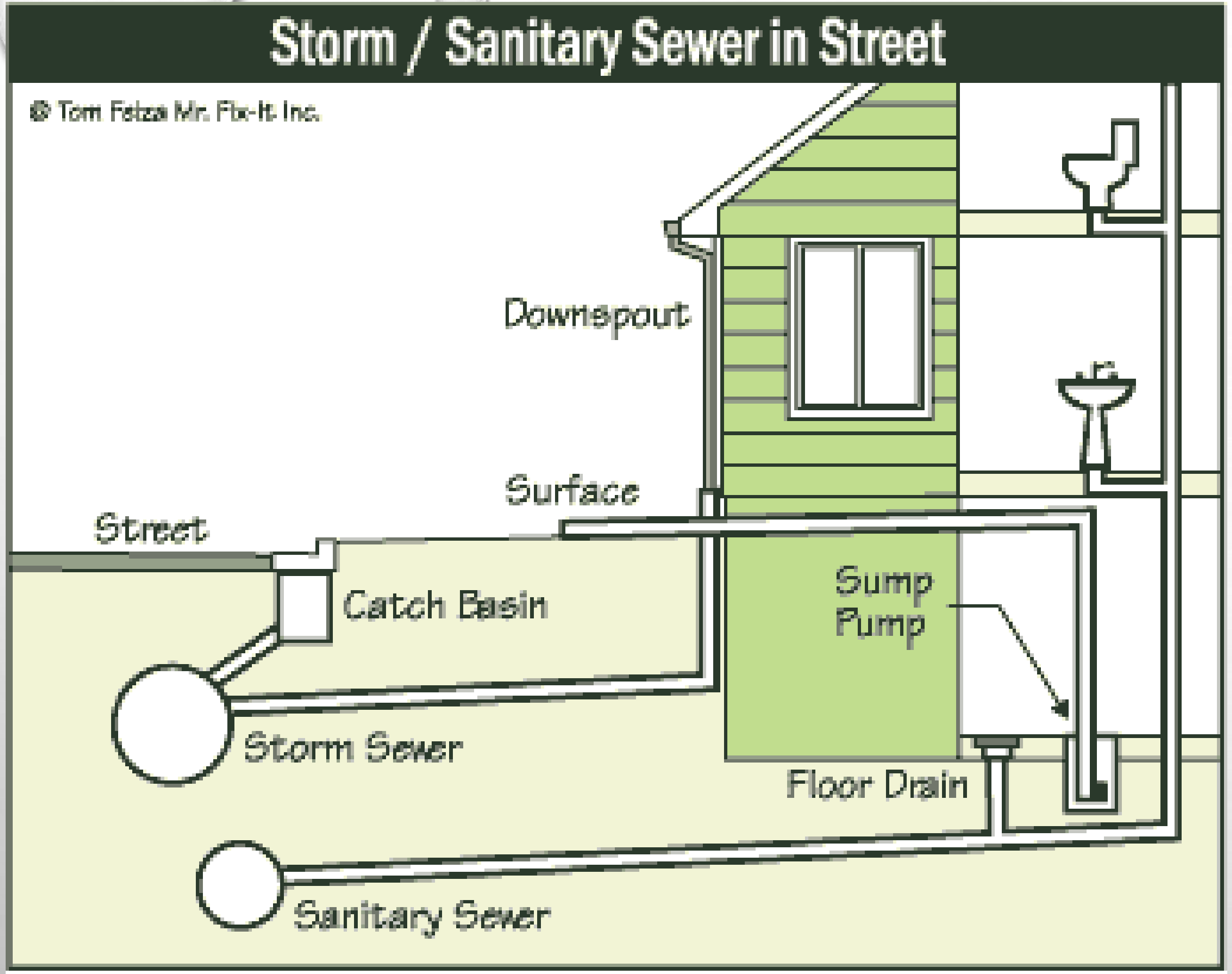


CATCH BASINS

- STRUCTURES OF PUCCA CHAMBER AND A STOUT COVER FOR THE RETENTION OF SUSPENDED GRIT, SLUDGE AND OTHER HEAVY DEBRIS AND FLOATING RUBBISH FROM RAIN WATER WHICH OTHERWISE CAUSE CHOKING PROBLEMS
- SUBMERGED OUTLET PIPE(SEWER PIPE)
- USE IS NOT RECOMMENDED
- MORE OF A NUISANCE
- SOURCE OF MOSQUITO BREEDING
- SUBSTANTIAL MAINTENANCE PROBLEMS.



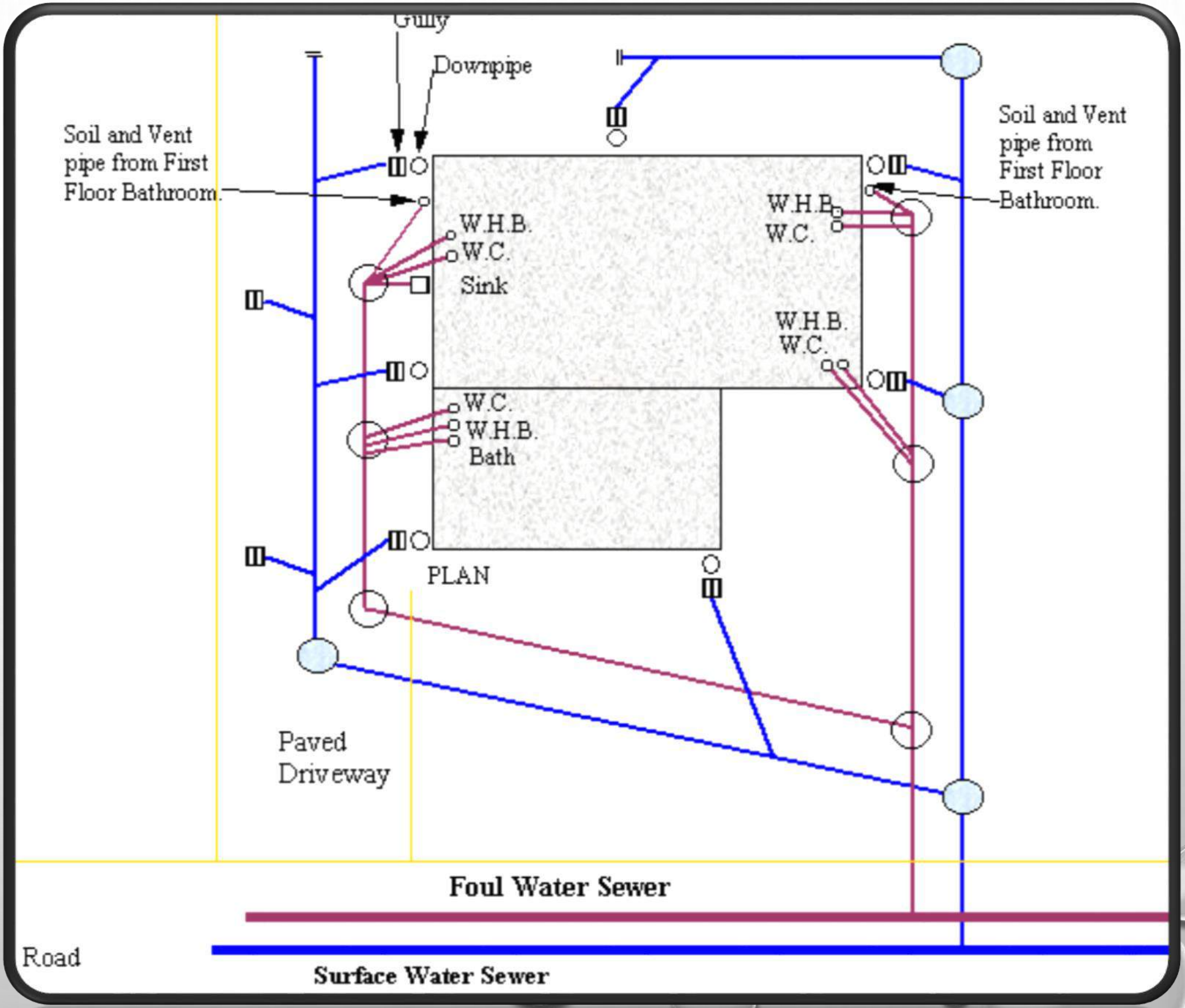
Catch Basins

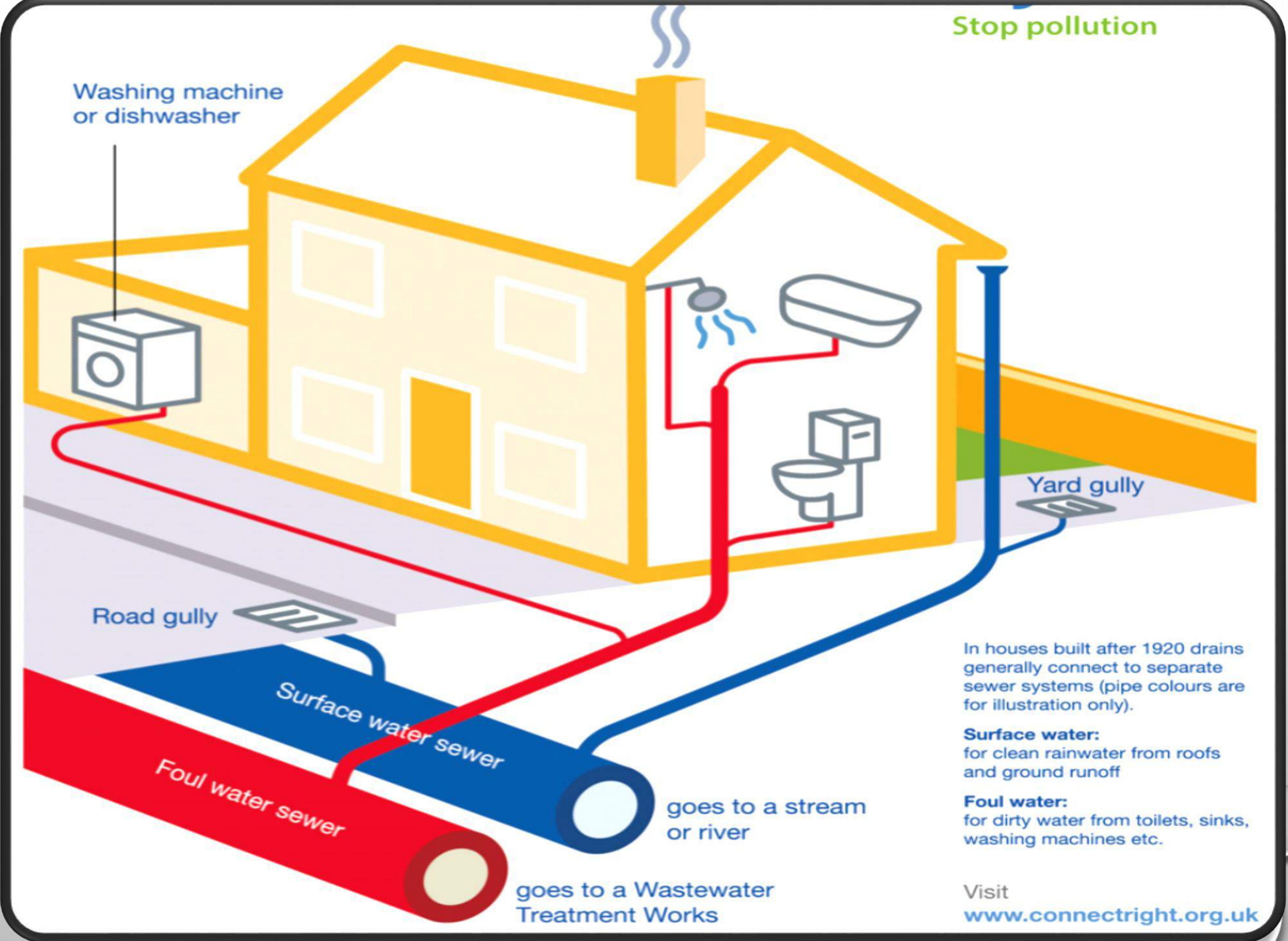

Storm / Sanitary Sewer in Street
© Tom Feiza Mr. Fix-It Inc.
Downspout
Surface
Street
Catch Basin
Sump
Pump
Storm Sewer
Floor Drain
Sanitary Sewer

BASIC PRINCIPLES OF GOOD DRAINAGE SYSTEM

- MATERIAL HAVE ADEQUATE STRENGTH AND DURABILITY.
- DIAMETER OF DRAIN TO BE AS SMALL AS POSSIBLE.
- ACCESSIBILITY- EVERY PART OF DRAIN WITHIN REACH FOR INSPECTION AND MAINTENANCE.
- LAID IN STRAIGHT RUNS AS FAR AS POSSIBLE.
- DRAINS LAID TO A GRADIENT.
- INLET BE TRAPPED. TO PREVENT ENTRY OF FOUL AIR INTO BUILDING.
- ACCESS FITTINGS - INSPECTION CHAMBER, RODDING EYE, MANHOLE - PLACED AT CHANGES OF DIRECTION AND GRADIENT.
- INSPECTION CHAMBERS PLACED AT JUNCTIONS.
- JUNCTIONS BETWEEN DRAINS ARRANGED SO THAT INCOMING DRAIN JOINTS AT OBLIQUE ANGLE IN DIRECTION OF FLOW.

TYPICAL LAYOUT PLAN SHOWING HOUSE DRAINAGE CONNECTIONS




Stop pollution
Washing machine or dishwasher
Yard gully
Road gully
Surface water sewer
Foul water sewer
goes to a stream or river
goes to a Wastewater Treatment Works
In houses built after 1920 drains generally connect to separate sewer systems (pipe colours are for illustration only).
Surface water:
for clean rainwater from roofs and ground runoff
Foul water:
for dirty water from toilets, sinks, washing machines etc.
Visit
www.connectright.org.uk

MAINTENANCE OF HOUSE DRAINAGE

FOR EFFICIENT WORKING - SHOULD BE PROPERLY MAINTAINED AND CLEANED AT REGULAR INTERVALS.

- 1. ENTRY OF UNDESIRED ELEMENTS - SHOULD TAKE EXTREME PRECAUTIONS TO AVOID ENTRY OF UNDESIRED ELEMENTS - GRIT, SAND, DECAYED FRUITS, PIECES OF CLOTHS, LEAVES, ETC.
- 2. FLUSHING - ADVISABLE TO FLUSH SYSTEM ONCE OR TWICE IN A DAY TO MAINTAIN IT IN PROPER WORKING ORDER.
- 3. INSPECTION - SHOULD BE INSPECTED AT REGULAR INTERVALS AND THE OBSTRUCTIONS IF ANY SHOULD BE REMOVED. DAMAGED PIPES SHOULD ALSO BE REPLACED.
- 4. QUALITY OF MATERIALS - BETTER QUALITY MATERIALS SHOULD BE USED

LOW COST WASTEWATER TREATMENT

SEPTIC TANK

It is a primary horizontal continuous flow type of sedimentation tank having extra provision to digestion of settled sludge

PROPERTIES.

- DETENTION TIME- 12 TO 36 HR
- IT REMOVES 60% TO 70% OF DISSOLVE MATTERS.
- CLEANING PERIOD-6 MONTH TO 3 YEARS

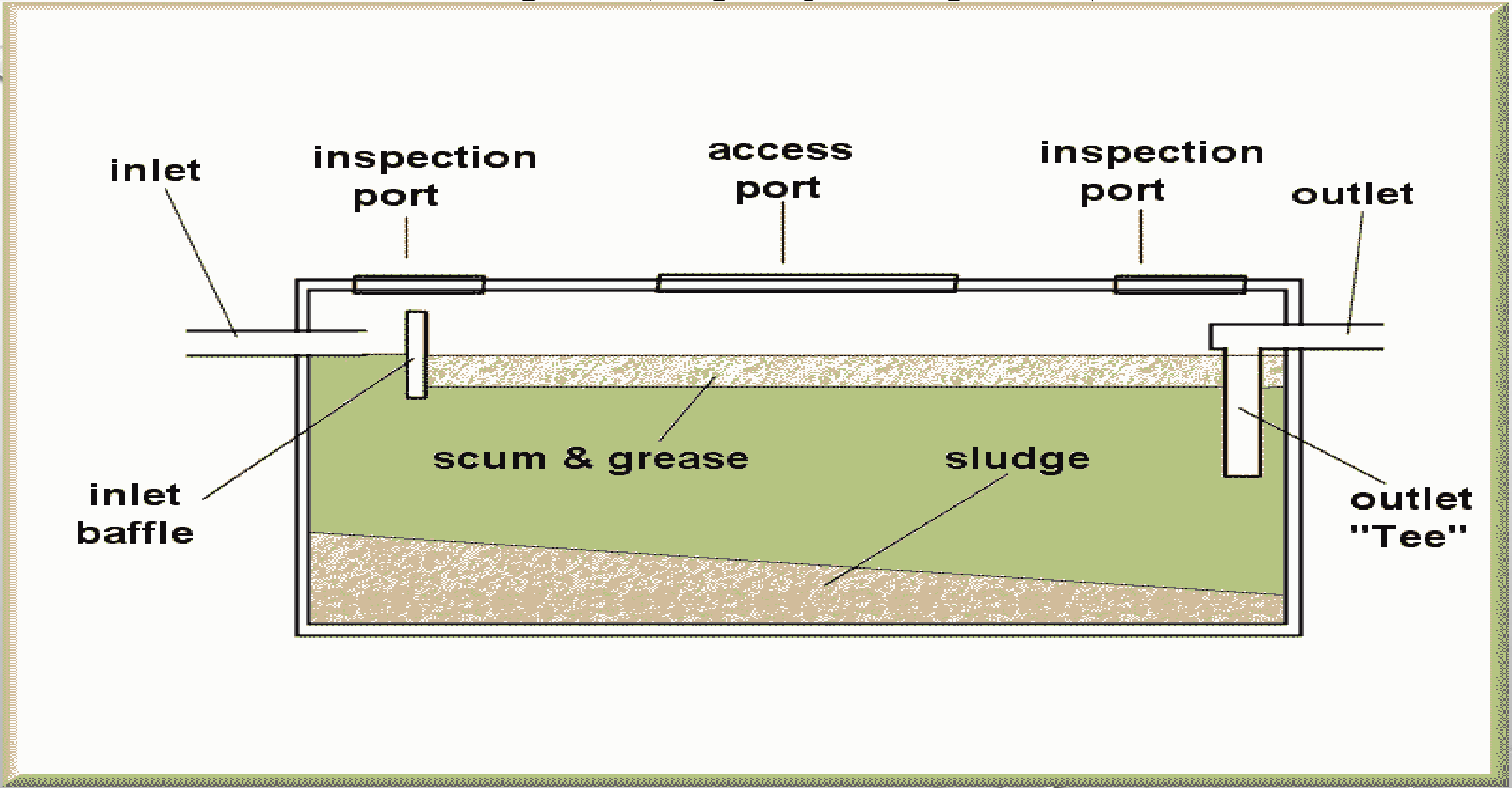
TANK FUNCTIONS

- SOLIDS REMOVAL BY SETTLING & FLOATATION
- 60-80% SOLIDS REMOVAL
- ANAEROBIC DIGESTION
- STORAGE OF SOLIDS

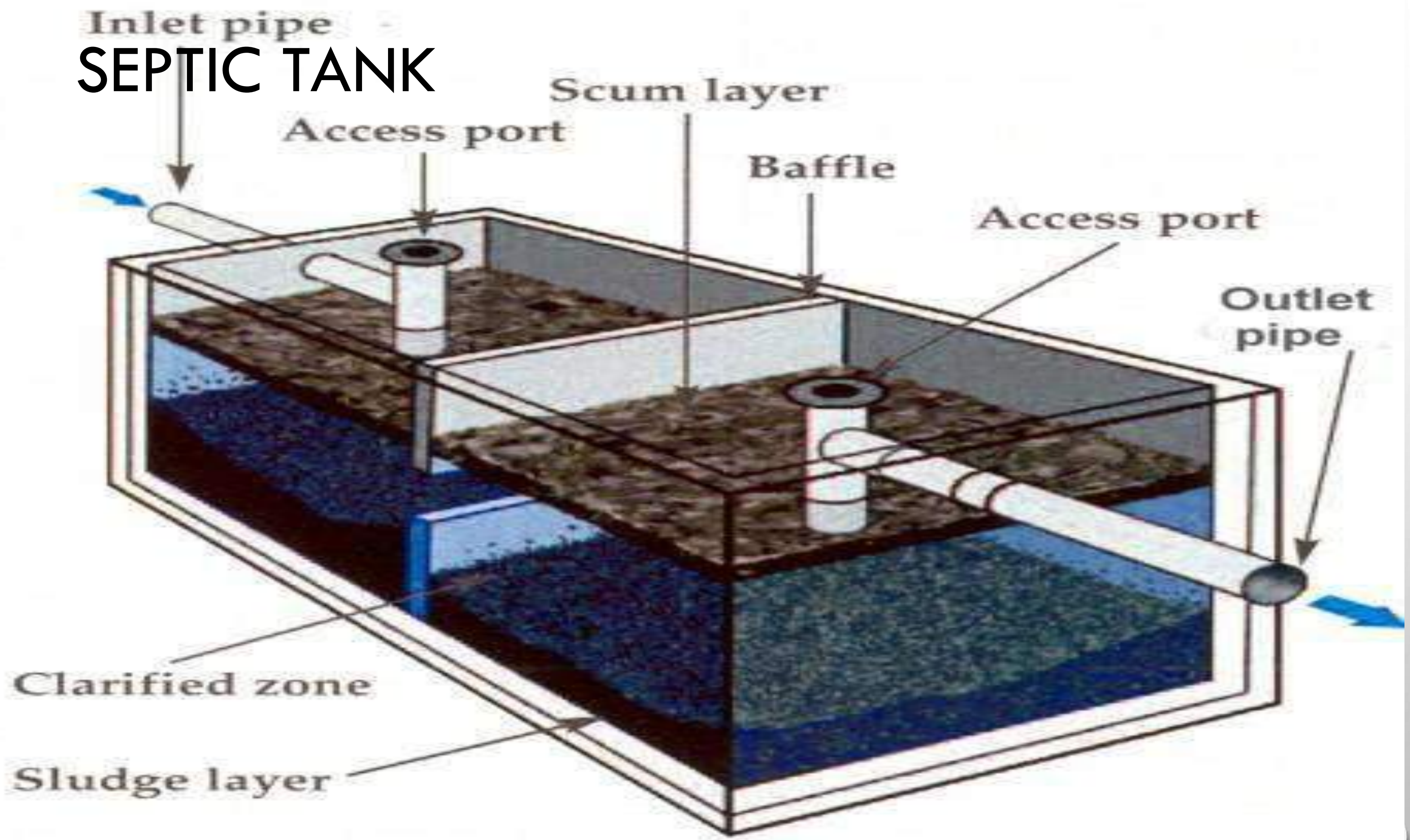
Factors that Influence Anaerobic Digestion

- pH
- Temperature
- Chemicals
- Highly variable flow patterns
- Pharmaceuticals and personal care products (PPCPs)
- Process wastewaters
- Lack of tank maintenance

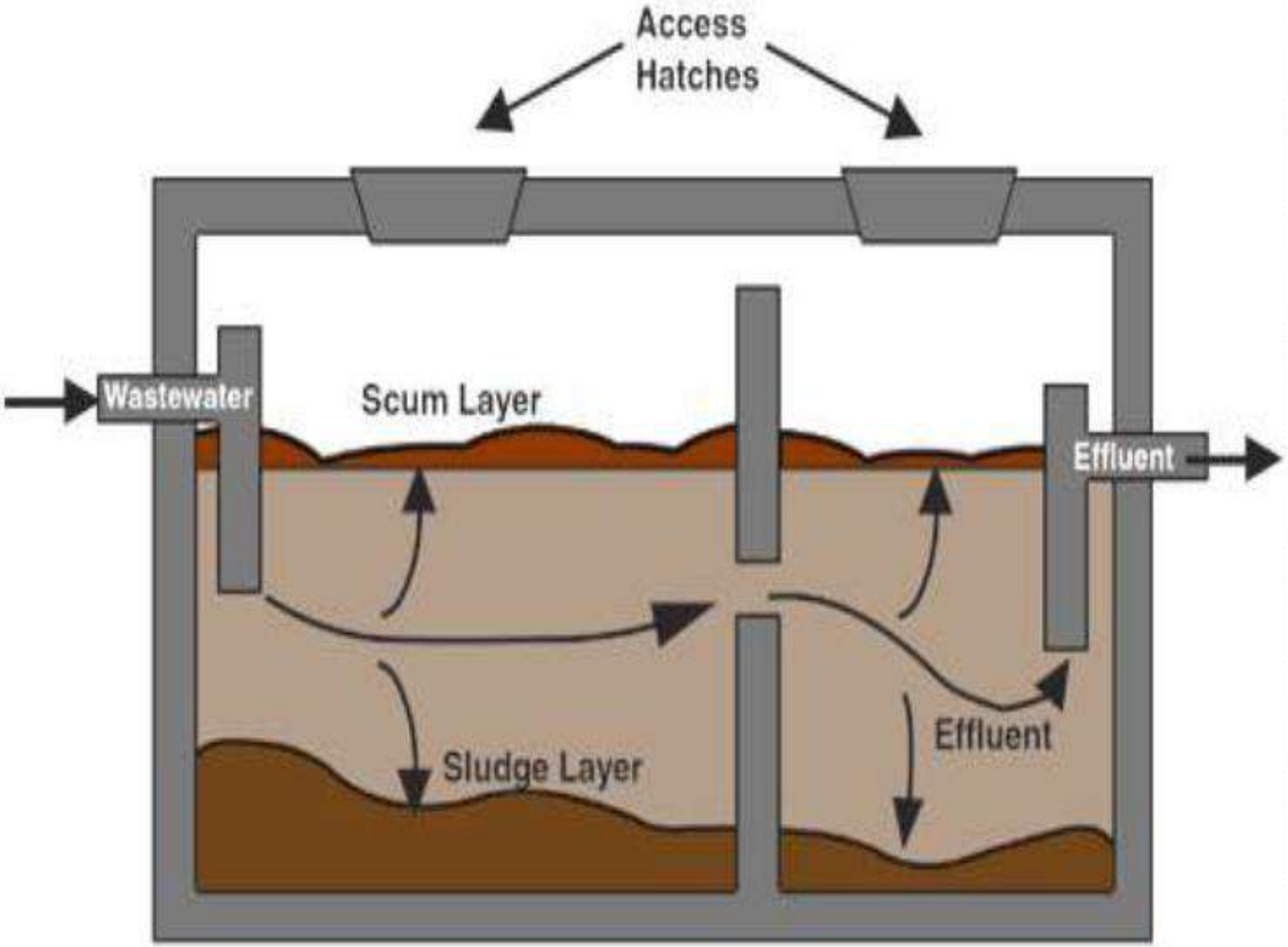
DIAGRAM OF SEPTIC TANK

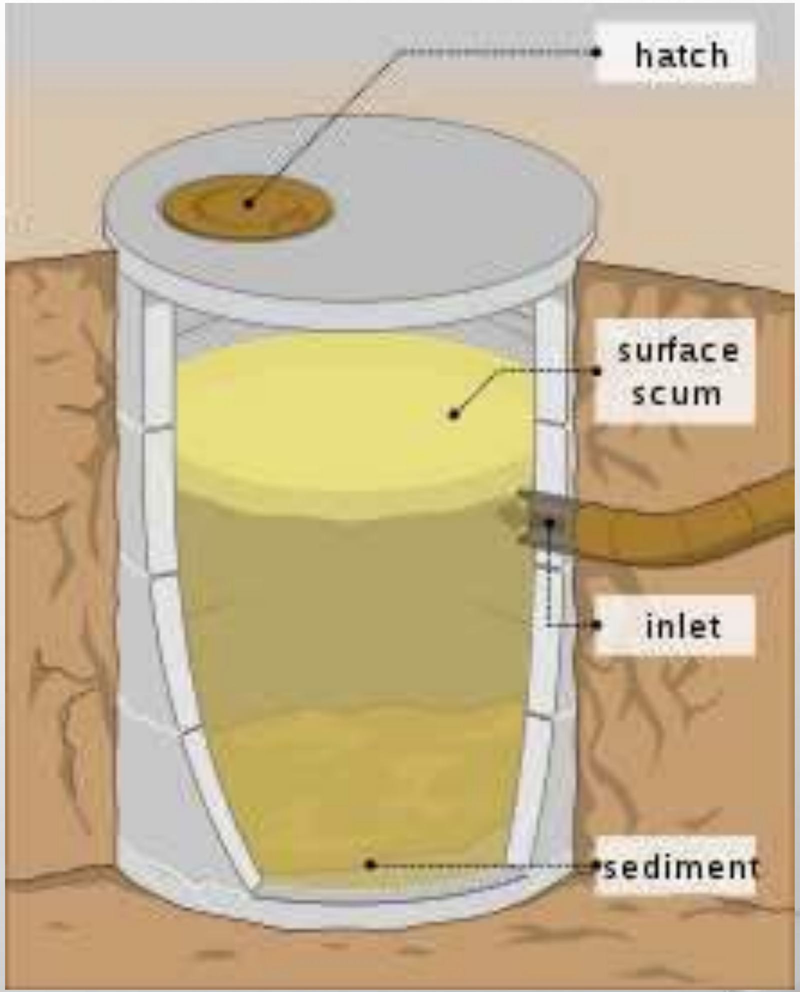


SEPTIC TANK



Element of Septic Tank





Cross Section of Tank

ANAEROBIC DIGESTION

CO_2
CH_4
H_2S
NH_3

Role of the septic tank

- Anaerobic fermentation of solids
- Reduce the load of pathogens in the effluent
- Hold the effluent for 2-3 days for improved safety
- Retain solid material to prevent blockage of further disposal system

Effluent

- Subsurface irrigation, Cess pool, Soak pit or trickling filter

MERITS AND DEMERITS OF SEPTIC TANK

Advantage

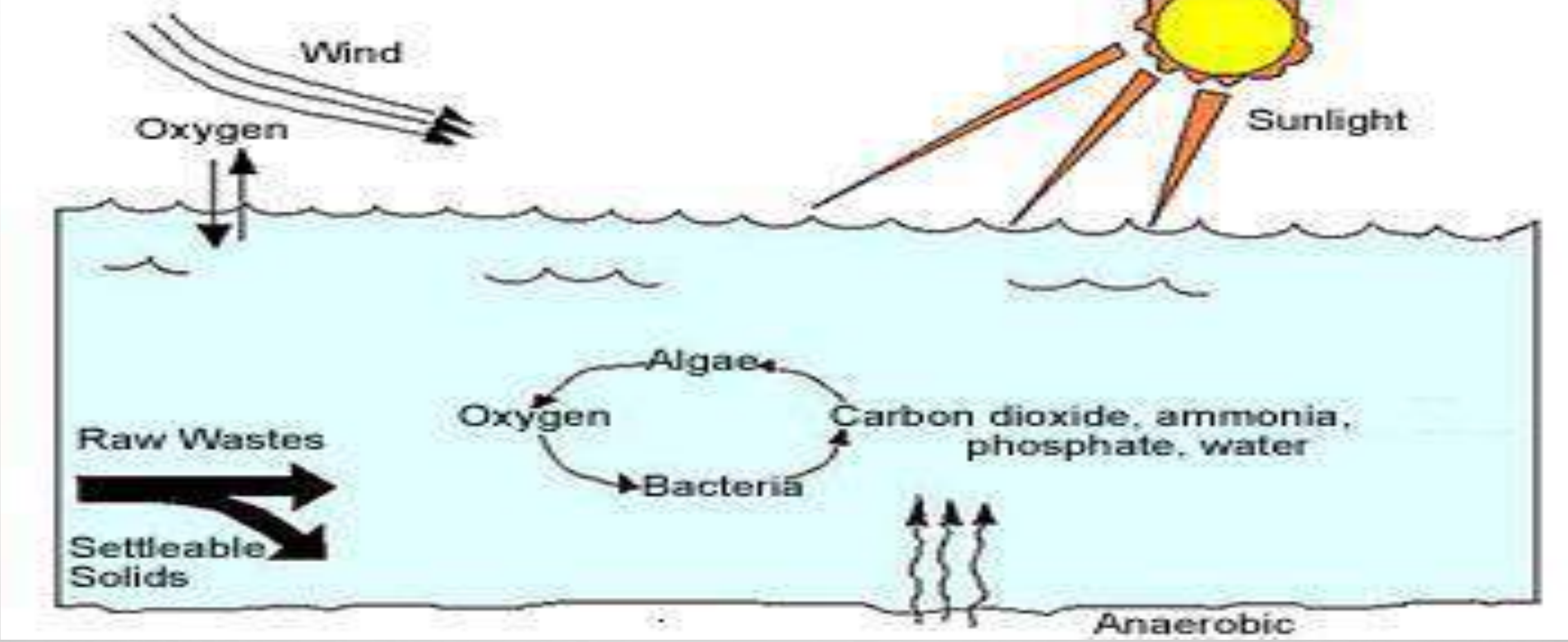
- It can be easily constructed.
- No maintenance problem.
- It excellently remove BOD.
- Very less amount of solids are produced.
- Low cost

DISADVANTAGE

- ITS SIZE SHOULD BE VERY LARGE TO SERVE MANY PEOPLE.
- SMELL PROBLEM
- IT NEEDS PERIODIC
- CLEANING.

OXIDATION POND/ WASTE STABILIZATION PONDS

- A BODY OF WASTEWATER EMPLOYED WITH THE RETENTION OF WASTEWATER OR ORGANIC WASTES UNTIL THE WASTES ARE RENDERED STABLE AND INOFFENSIVE FOR DISCHARGE INTO STREAM THROUGH PHYSICAL, CHEMICAL AND BIOLOGICAL PROCESSES INVOLVING THE ACTION OF ALGAE AND BACTERIA UNDER THE INFLUENCE OF SUNLIGHT AND AIR



1.6 mg of oxygen is released by the algae/mg of dry organic cell material synthesized

Optimum temperature 30 to 35°C

Aerobic

$$CH_2O + O_2 \xrightarrow{\text{Bacteria}} CO_2 + H_2O$$

(Oraganics)

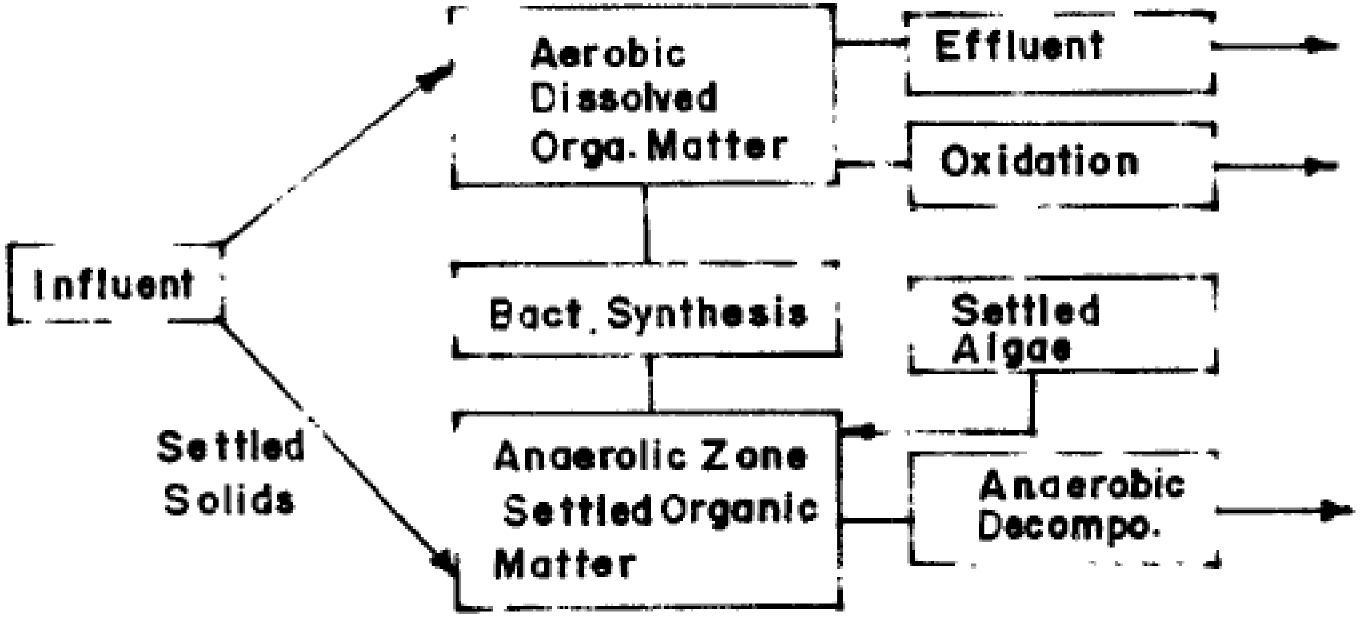
Anaerobic

$$CH_2O \xrightarrow{\text{Bacteria}} CH_3COOH \xrightarrow{\text{Bacteria}} CO_2 + CH_4$$

Algae in Turn Reuse the Corbon (CO_2) to form Algal Biomass

$$CO_2 + 2H_2O + \text{Energy} \xrightarrow{\text{Algae}} CH_2O + O_2 + H_2O$$

Fig. 2 : Process Reactions in WSP



MODULE 4

Sewer Design & Effluent Disposal:

Disposal of effluents by dilution

Design of Sewers

- The hydraulic design of sewers and drains, means finding out their sections and gradients, is generally carried out on the same lines as that of the water supply pipes.
- However, there are two major differences between characteristics of flows in sewers and water supply pipes.
- The sewage contain particles in suspension, the heavier of which may settle down at the bottom of the sewers, as and when the flow velocity reduces, resulting in the clogging of sewers
- To avoid silting of sewers, it is necessary that the sewer pipes be laid at such a gradient, as to generate self cleansing velocities at different possible discharges.
- The sewer pipes carry sewage as gravity conduits, and are therefore laid at a continuous gradient in the downward direction up to the outfall point, from where it will be lifted up, treated and disposed off.

Circular sewers running full:

Area of flow section $A = \frac{\pi}{4} D^2$

Wetted perimeter $P = \pi D$

Hydraulic mean depth $R = \frac{A}{P} = \frac{D}{4}$

Manning's formula:

Velocity of flow $V = \frac{1}{N} R^{2/3} S^{1/2}$

Discharge $Q = A.V = \frac{A}{N} R^{2/3} S^{1/2}$

$$= \frac{0.3116}{N} D^{8/3} S^{1/2}$$

ii. Circular sewers running partially full:

Central angle θ is given by $cos\frac{\theta}{2} = \left(1 - \frac{2d}{D}\right)$

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

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

2. Area $a = \frac{\pi}{4}D^2 \times \frac{\theta}{360} - \frac{D}{2}\cos\frac{\theta}{2}\frac{D}{2}\sin\frac{\theta}{2}$

$$= \frac{\pi}{4}D^2\left[\frac{\theta}{360} - \frac{\sin\theta}{2\pi}\right]$$

   Proportional area $= \frac{a}{A} = \left[\frac{\theta}{360} - \frac{\sin\theta}{2\pi}\right]$

3. Wetted perimeter p= $\pi D \frac{\theta}{360}$

Proportional perimeter= $\frac{p}{P} = \frac{\theta}{360}$

4. Hydraulic mean depth$r = \frac{a}{p} = \frac{\frac{\pi}{4}D^2\left[\frac{\theta}{360} - \frac{\sin\theta}{2\pi}\right]}{\pi D \frac{\theta}{360}}$

$$= \frac{D}{4}\left[1 - \frac{360\sin\theta}{2\pi\theta}\right]$$

Proportional hydraulic mean depth

$$= \frac{r}{R} = \left[1 - \frac{360\sin\theta}{2\pi\theta}\right]$$

5. Velocity of flow $v = \frac{1}{N} R^{2/3} S^{1/2}$

Proportional velocity $= \frac{v}{V} = \frac{N}{n}\left(\frac{r}{R}\right)^{2/3}$

Taking $\frac{N}{n} = 1$,

$$\frac{v}{V} = \left(\frac{r}{R}\right)^{2/3} = \left[1 - \frac{360 \sin\theta}{2\pi\theta}\right]^{2/3}$$

6. Discharge $q = a \times v = \frac{1}{n} r^{2/3} S^{1/2}$

Taking $\frac{N}{n} = 1$,

Proportional discharge $= \frac{q}{Q} = \left(\frac{a}{A}\right)\left(\frac{v}{V}\right)$

$$= \left(\frac{a}{A}\right)\left(\frac{r}{R}\right)^{2/3}$$

$$= \frac{\theta}{360}\left[1 - \frac{360 \sin\theta}{2\pi\theta}\right]^{5/3}$$

Design the section of combined circular sewer from the data given below:
Area to be served is 150 hectares
Population of locality is 1,00,000
Maximum permissible velocity 3.2 m/s
Time of entry is 5 minutes
Time of flow is 20 minutes
Rate of water supply is 270 lpcd
Runoff coefficient is 0.45
Assume suitable data if necessary.

Solution:

Assuming 80% of the water supplied will be reaching the sewers as sanitary sewage ,quantity of sanitary sewage produced;

Average quantity of sanitary sewage flow (DWF)

=1,00,000 x 270 x 0.8 lit/day

=(100000x270x0.80) / (1000x24x60x60) m^3/s

=0.25 m^3/s

Maximum or peak quantity of sanitary sewage

=Peak factor x DWF (Assuming peak factor = 2)

=2 x 0.25 m^3/s

=0.5 m^3/s

$T_c = T_e + T_f$

where T_c= time of concentration

T_e = time of entry

T_f = time of travel or flow

The quantity of storm water will be maximum when storm duration is equal to time of concentration.

Thus, t = t_c= 25 minutes

$$i = \frac{1020}{t + 20}$$

$i = \frac{1020}{25+20}$ = 22.67 mm/hr

The storm water runoff is given by rational formula ,as

$Q = \frac{CiA}{360}$, where C=0.45, i=22.67 mm/hr , A=150 hectare

=4.25 m^3/s

Therefore, combined discharge (Q) = 0.5+4.25

= 4.75 m^3/s

Now, $A = \frac{Q}{V} = \frac{4.75}{3.2} = 1.48$ m^2

All velocities must lie between (0.75-3.2) m/s . Hence, 3.2 m/s is assumed.

Diameter of sewer , $D = \frac{\sqrt{4A}}{\pi}$

=1.374 m

Adopting commercially available size 1.4 m,

$$A = \frac{\pi D^2}{4}$$

$$= \frac{\pi x\ (1.4)2}{4} = 1.54 \text{ m}^2$$

$V = \frac{Q}{A} = \frac{4.75}{1.54} = 3.08\ m/s$<3.2 m/s. Hence,ok.

Check for cleansing velocity during dry weather flow

$$\frac{q}{Q} = \frac{0.25}{4.75} = \frac{1}{19} = 0.0526$$

$$\frac{q}{Q} = \frac{\theta}{360}\left[1 - \frac{360 sin\theta}{2\pi\theta}\right]^{5/3}$$

$$\left[\frac{18.94}{\theta}\right]^{3/5} + \frac{360\sin\theta}{2\pi\theta} - 1 = 0$$

Solving, θ = 93°

$$\frac{\vartheta}{V} = \left[1 - \frac{360\sin\theta}{2\pi\theta}\right]^{2/3}$$

Or, v= 3.08 x 0.529 = 1.629 m/s

0.75 m/s < 1.629 m/s < 3.2 m/s Hence,ok.

Check for self cleansing velocity during minimum flow

Assume Q_{min} = 1/2 of DWF

$$\frac{q}{Q} = 0.0263$$

$$\frac{q}{Q} = \frac{\theta}{360}\left[1 - \frac{360\sin\theta}{2\pi\theta}\right]^{5/3}$$

Solving, θ = 78.077°

$$\frac{\vartheta}{V} = \left[1 - \frac{360\sin\theta}{2\pi\theta}\right]^{2/3}$$

Or, v= 3.08 x 0.43 = 1.32 m/s

0.75 m/s < 1.32 m/s < 3.2 m/s Hence , ok.

Calculate the diameter and velocity of a circular sewer at a slope of 1in 150 when it is running just full at a discharge of 1.05 m^3/s. The value of n in Manning's formula is 0.011. What will be the discharge and velocity when flowing 0.75 depth of pipe for the same slope.

Solution:

Using Manning's equation,we have

$Q = \frac{1}{n} A R^{2/3} S^{1/2}$

Q = 1.05 m^3/s , n=0.11, s=1/150

Thus, by substitution , we get

$1.05 = \left(\frac{1}{0.11}\right) x \left(\frac{\pi}{4} D2\right) x \left(\frac{D}{4}\right)^{2/3} x \left(\frac{1}{400}\right)^{1/2}$

or, D= 0.7436 m

Velocity (v) = $\frac{1}{n} R^{2/3} S^{1/2}$

$= \frac{1}{0.011} \left(\frac{0.7436}{4}\right)^{2/3} \left(\frac{1}{150}\right)^{1/2}$

= 2.42 m/s

When flowing depth of pipe = 0.75m

$$\frac{d}{D} = 0.75$$

Central angle (θ) is given by , $\cos\frac{\theta}{2} = (1 - \frac{2d}{D})$

$= (1- 2 \times 0.75)$

$\theta = 240^{o}$

$$\frac{q}{Q} = \frac{\theta}{360}\left[1 - \frac{360 sin\theta}{2\pi\theta}\right]^{5/3}$$

Discharge(Q) = 1.05 m^3/s

Therefore, q = 0.9575 m^3/s

Now,

$$\frac{\vartheta}{V} = \left[1 - \frac{360 sin\theta}{2\pi\theta}\right]^{2/3}$$

Velocity (v) = 2.42 m/s

Therefore , ϑ = 2.74 m/s

1. Calculate the velocity of flow in a sewer of circular section having diameter of **1**m, laid at gradient of **1** in **600**. Use Manning's equation taking n=**0.012**

Solution:

For sewer running half full,

$A=\frac{\pi D^2}{8}$; $P=\frac{\pi D}{2}$

$R=\frac{A}{P}=\frac{D}{4}$

Hydraulic Radius(R)=0.25m

$A=\frac{\pi D^2}{8}=0.3627m^2$

Using Manning's equation;

Velocity(V)=$\frac{1}{n}R^{\frac{2}{3}}S^{\frac{1}{2}}$=1.35 m/sec

Discharge(Q)=AV=0.3927 * 1.35 =0.053 m^3/sec

2. Design a sewer to serve a population of 120000; the daily per capita water supply allowance being 180 litres, of which 80% find its way into the sewer. The permissible sewer slope is 1 in 1000, peak factor=2 and take Manning's n=0.012

Solution :

$$\text{Average flow (DWF)} = 0.80 \times 1{,}20{,}000 \times 180 \text{ liters/day}$$

$$= \frac{120000 \times 180 \times 0.80}{1000 \times 24 \times 60 \times 60} \text{ m}^3/\text{sec}$$

$$= 0.20 \text{ m}^3/\text{sec}$$

Maximum or peak quantity of sanitary sewage

$$= \text{Peak factor} \times DWF = 2 \times 0.20 = 0.40 \text{ m}^3/\text{sec}$$

The sewer is designed for maximum discharge.

Thus using Manning's equation, we have

$$Q = \frac{A}{n} R^{2/3} S^{1/2}$$

$$n = 0.012, \quad s = \frac{1}{1000}$$

If D is the diameter of the sewer, then

$$A = \frac{\pi}{4} D^2, \qquad P = \pi D, \qquad R = \frac{A}{P} = \frac{D}{4}$$

Thus by substitution, we get

$$0.40 = \frac{1}{0.012} \times \left(\frac{\pi}{4} D^2\right) \times \left(\frac{D}{4}\right)^{2/3} \times \left(\frac{1}{1000}\right)^{1/2}$$

or, $\qquad D^{8/3} = 0.487$

or, $\qquad D = 0.763\text{m}$

Adopt commercially available size 0.8 m

Check for self cleansing velocity at maximum flow

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

$$V = 0.9 \text{ m/sec}$$

Which is 0.6 m/sec < 0.9 m/sec < 3 m/sec

Check for self cleansing velocity at dry weather flow

$$\frac{q}{Q} = \frac{0.2}{0.4} = \frac{1}{2} = 0.5$$

$$\therefore \frac{q}{Q} = \frac{\theta}{360}\left[1 - \frac{360 \sin\theta}{2\pi\theta}\right]^{5/3}$$

Or, $$\left[\left[\frac{180}{\theta}\right]^{3/5} + \frac{360 \sin\theta}{2\pi\theta}\right] - 1 = 0$$

Solving, $\theta = 180^\circ$

$$\frac{v}{V} = \left[1 - \frac{360\sin\theta}{2\pi\theta}\right]^{2/3}$$

Or, $v = 0.9 \times 1 = 0.9$ m/sec

Which is 0.6 m/sec < 0.9 m/sec < 3 m/sec

Hence OK.

Check for self cleansing velocity during minimum flow;

Assume Q_{min} = 1/2 of DWF

$$\frac{q}{Q} = 0.25$$

$$\therefore \frac{q}{Q} = \frac{\theta}{360}\left[1 - \frac{360\sin\theta}{2\pi\theta}\right]^{5/3}$$

Or, $$\left[\left[\frac{45}{\theta}\right]^{3/5} + \frac{360\sin\theta}{2\pi\theta}\right] - 1 = 0$$

Solving, $\theta = 117^\circ$

$$\frac{v}{V} = \left[1 - \frac{360\sin\theta}{2\pi\theta}\right]^{2/3}$$

Or, $v = 0.9 \times 0.682 = 0.61$ m/sec

Which is 0.6 m/sec < 0.61 m/sec < 3 m/sec

Hence OK.

3. Calculate the diameter of a sewer to serve an area of 20 square kilometer with a population density of 250 persons per hectare. The average rate of sewage flow is 350 lpcd. The maximum flow is 50% in excess of average together with the rainfall equivalent of 15 mm in 24 hrs, all of which are runoff, take the Vmax as 3m/sec.

Solution:

Total population of the area;

$$= 250 \times \left(\frac{20 \times 10^6}{10^4}\right) = 500000 \text{ persons}$$

Average sewage flow;

$$= \left(\frac{500000 \times 350 \times 10^{-3}}{24 \times 3600}\right) \text{m}^3/\text{sec} = 2.025 \text{ m}^3/\text{sec}$$

Peak or maximum flow;

$$= 1.5 \times 2.025 \text{ m}^3/\text{sec} \qquad (Q_{max.} = 1.5 \text{ times average flow})$$

$$= 3.038 \text{ m}^3/\text{sec}$$

Storm sewage;

$$Q = \frac{CiA}{360}$$

[Where, C = 1, i = 0.625 mm/hr, A = 2000 hactare]

$$= 3.472 \text{ m}^3/\text{sec}$$

Total flow for the combined sewer

$$= 3.038 + 3.472 = 6.51 \text{m}^3/\text{sec}$$

$$A = \frac{Q}{V} = \frac{6.51}{3} = 2.17 \text{ m}^2$$

Diameter of sewer, $D = \sqrt{\frac{4A}{\pi}} = 1.662 \text{ m}$

4.Calculate the diameter of combined circular sewer with the following data:

Rate of water supply=100 lpcd

Population density=100 persons/hec

Peak factor=2.7

Area=35 hectares

Rainfall intensity=15 mm/hr

Slope=1 in 750

Rugosity coefficient=0.011

Runoff Coefficient=0.4

The sewer should run 0.6 full during peak flow

Solution:

Assume 80% of supplied water converted as wastewater,

Average flow (DWF) = 0.80×3500×100 liters/day

$$= \frac{3500\times100\times0.80\times10^{-3}}{24\times3600} \text{ m}^3/\text{sec}$$

$$= 0.00324 \text{ m}^3/\text{sec}$$

Maximum or peak quantity of sanitary sewage

$$= \text{Peak factor} \times DWF = 2.7 \times 0.00324 = 0.00875 \text{ m}^3/\text{sec}$$

Quantity of storm sewage;

$$Q = \frac{CiA}{360}$$ Where, C = 0.4, i = 15 mm/hr, A = 35 hectare

$$= 0.5833 \text{ m}^3/\text{sec}$$

Therefore, combined discharge (q) = 0.592 m³/sec

$$\frac{d}{D} = 0.6$$

Central angle (θ) is given by expression $\cos\frac{\theta}{2} = \left(1 - 2\frac{d}{D}\right)$

$$= (1 - 2 \times 0.6)$$

$$\theta = 203.07^\circ$$

$$\therefore \frac{q}{Q} = \frac{\theta}{360}\left[1 - \frac{360\sin\theta}{2\pi\theta}\right]^{5/3}$$

In full flow condition (Q) = 0.88116 m³/sec

Using Manning's formula;

$$Q = \frac{A}{n} R^{2/3} S^{1/2}$$

$$n = 0.011, \; s = 1/750$$

If D is the diameter of the sewer, then

$$A = \frac{\pi}{4}D^2, \quad P = \pi D, \quad R = \frac{A}{P} = \frac{D}{4}$$

$$0.88116 = \left(\frac{1}{0.011}\right) \times \left(\frac{\pi}{4}D^2\right) \times \left(\frac{D}{4}\right)^{2/3} \times \left(\frac{1}{750}\right)^{1/2}$$

D = 1 meter

5. A city has a population of 1 lakh with a per capita water supply of 200 lpcd. Design sewer running 0.7 times full at maximum discharge. Take n=0.013, slope=1 in 600 and peak factor=2.25. Assume 80% of w/s contibutes for sewage.

Solution:

Quantity of water supplied $= \dfrac{100000 \times 200 \times 10^{-3}}{24 \times 3600} = 0.2315\ m^3/sec$

Average quantity of sanitary sewage (DWF) $= 0.8 \times 0.2315$

$= 0.1852\ m^3/sec$

Peak discharge = pf X DWF $= 0.4167\ m^3/sec$

$\dfrac{d}{D} = 0.75$

Central angle (θ) is given by expression $\cos\dfrac{\theta}{2} = \left(1 - \dfrac{2d}{D}\right)$

$= (1 - 2 \times 0.75)$

$\theta = 227.16^\circ$

$$a = \frac{\pi D^2}{4}\left[\frac{\theta}{360} - \frac{\sin\theta}{2\pi}\right] = 0.5872\ D^2$$

$$p = \pi D \frac{\theta}{360} = 1.9262\ D$$

$$r = \frac{a}{p} = 0.2962\ D$$

Using Manning's formula;

$$Q = \frac{a}{n}\ r^{2/3}\ S^{1/2}$$

$(n = 0.013,\ s = 1/600)$

$$0.4167 = \left(\frac{1}{0.013}\right) \times \left(0.5872 D^2\right) \times (0.2962\ D)^{2/3} \times \left(\frac{1}{600}\right)^{1/2}$$

D = 776 mm

Adopt commercially available size 0.8 m

Check Velocity;

$$V = \left(\frac{1}{0.013}\right) \times (0.23696)^{2/3} \times \left(\frac{1}{600}\right)^{1/2}$$

= 1.20 m/sec (0.6 to 3 m/sec OK)

Check self cleansing velocity for DWF;

$$\frac{q}{Q} = \frac{1}{2.25}$$

$$\therefore \frac{q}{Q} = \frac{\theta}{360}\left[1 - \frac{360 \sin\theta}{2\pi\theta}\right]^{5/3}$$

Or, $\left[\left[\frac{160}{\theta}\right]^{3/5} + \frac{360 \sin\theta}{2\pi\theta}\right] - 1 = 0$

Solving, $\theta = 172.42^\circ$

$$\frac{v}{V} = \left[1 - \frac{360 \sin\theta}{2\pi\theta}\right]^{2/3} = 0.97$$

Or, $v = 0.97 \times 1.20 = 1.166$ m/sec

Similarly, Velocity during minimum flow can be obtained as minimum flow is one third of average flow.

$V_{min.} = 0.845$ m/sec

As flow drops to minimum the self cleansing velocity will be maintained in the sewer. Hence, design is OK.

Hydraulic formulae

1. Chezy's formula

$$v = C\sqrt{Ri},$$

where
V= is the mean velocity [m/s],
C= is the Chézy coefficient [m½/s],
R= is the hydraulic radius (~ water depth) [m],
i= is the bottom slope[m/m].

- Constant (C) is very complex. Depends on size, shape and smoother roughness of the channel, the mean depth etc.
- C can be calculated by using Bazin's formula.

2. Bazin's formula

$$C = \frac{157.6}{[1.81 + (K/R^{1/2})]}$$

Where,
K= Bazin's constant
R= hydraulic radius

Sr. No.	Inside nature of the sewer	K values
1.	Very smooth	0.109
2.	Smooth: bricks & concrete	0.290
3.	Smooth: rubble masonry	0.833
4.	Good, earthen material	1.540
5.	Rough: bricks & concrete	0.500
6.	Rough earthen material	3.170

3. Manning's formula

$$V = \frac{k}{n} R_h^{2/3} \cdot S^{1/2}$$

V = velocity of flow (m/s)

k =conversion factor of 1.486 $(\text{ft/m})^{1/3}$

n =Manning coefficient

R_h = hydraulic radius (m)

S = slope of the water surface

* The value of "n" is calculated by kutter's formula

4. Kutter's formula

$$C = \frac{k_1 + \frac{k_2}{S} + \frac{k_3}{n}}{1 + \frac{n}{\sqrt{R}} \cdot \left(k_1 + \frac{k_2}{S}\right)}$$

Where

- C = Chézy's roughness coefficient
- S = Friction slope
- R = Hydraulic radius (m,)
- n = Kutter's roughness (unit less)
- k_1 =Constant (23.0 SI,)
- k_2=Constant (0.00155 SI,)
- k_3= Constant (1.0 SI,)

5. Hazen – William's formula

$$V = k\,C\,R^{0.63}\,S^{0.54}$$

where:

- V is velocity
- k is a conversion factor for the unit system k = 0.849 for SI units)
- C is a roughness coefficient
- R is the hydraulic radius
- S is the slope of the energy line (head loss per length of pipe)

6. Crimp and Burge's formula

$$V = 83.47\,R^{2/3}\,S^{1/2}$$

Where,

V = velocity of flow (m/s)

R = hydraulic radius (m)

S = slope of the water surface

(1) **Circular section running full :**

Here Area of C / S, $A = \frac{\pi}{4}(D^2)$, D = dia of sewer

Wetted Perimeter $P = \pi D$

$$\therefore R = \frac{A}{P} = \frac{\frac{\pi}{4}(D^2)}{\pi D} = \frac{D}{4}$$

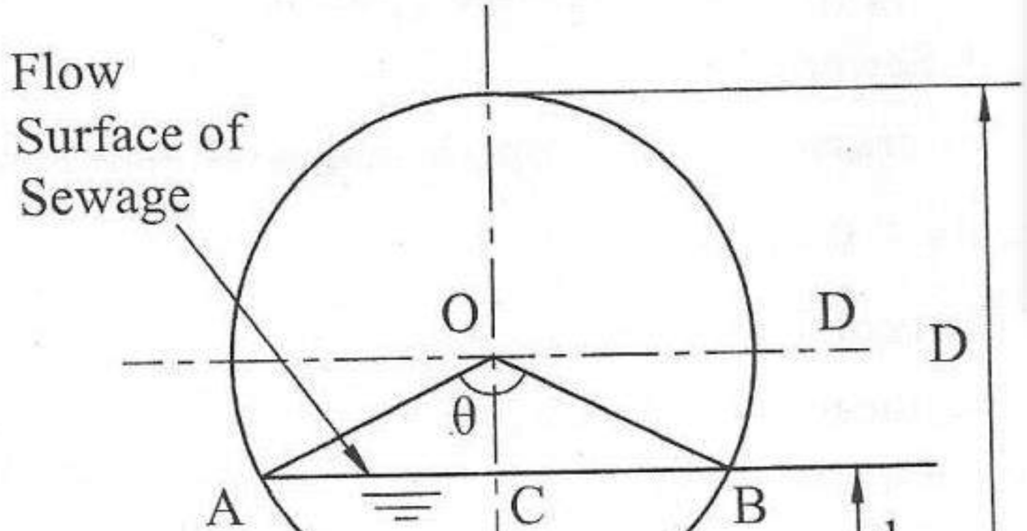
(2) **Circular section running partially full (Refer fig**

Let a = area of cross - section

b = wetted perimeter

r = H.M.D. (Hydraulic Mean Depth)

v = velocity of flow



(1) **Depth** $d = \frac{D}{2} - \frac{D}{2}\cos\frac{\theta}{2} = \frac{D}{2}(1 - \cos\theta/2)$

Proportional depth $\frac{d}{D} = \frac{1}{2}(1 - \cos\theta/2)$

(2) **Area** $a = \frac{\pi}{4}D^2 \times \frac{\theta}{360°} - \frac{D}{2}\cos\frac{\theta}{2} \cdot \frac{D}{2}\sin\frac{\theta}{2}$

$$a = \frac{\pi}{4}D^2\left[\frac{\theta}{360°} - \frac{\sin\theta}{2\pi}\right]$$

$\therefore$ Proportional Area $= \frac{a}{A}\left[\frac{\theta}{360°} - \frac{\sin\theta}{2\pi}\right]$

(3) **Wetted perimeter :**

$$P = \pi D \cdot \frac{\theta}{360°}$$

$\therefore$ Proportional wetted perimeter :

$$\frac{P}{P} = \frac{\theta}{360°}$$

(4) H.M.D. :

$$r = \frac{a}{p} = \frac{\frac{\pi D^2}{4}\left[\frac{\theta}{360^\circ} - \frac{\sin\theta}{2\pi}\right]}{\pi D \frac{\theta}{360^\circ}}$$

$$\therefore \quad r = \frac{D}{4}\left[1 - \frac{360^\circ \sin\theta}{2\pi\theta}\right]$$

$$\text{Proportionate HMD} = \frac{r}{R} = \frac{\frac{D}{4}\left[1 - \frac{360^\circ \sin\theta}{2\pi\theta}\right]}{\frac{D}{4}}$$

$$= \left[1 - \frac{360^\circ \sin\theta}{2\pi\theta}\right]$$

(5) Velocity of flow :

$$v = \frac{1}{n} r^{2/3} S^{1/2} \text{ (Manning's)}$$

where = Manning's rougosity coefficient applicable for partial fl

$$\therefore \text{ Proportional velocity, } \frac{v}{V} = \frac{N}{n}\left(\frac{r}{R}\right)^{2/3}$$

$$\text{If } \frac{N}{n} = 1, \frac{v}{V} = \left(\frac{r}{R}\right)^{2/3} = \left[1 - \frac{360^\circ \sin\theta}{2\pi\theta}\right]$$

(6) Discharge :

$$q = a \times v \text{ Taking } \frac{N}{n} = 1.0, \text{ we get}$$

$$\text{Proportional discharge } = \frac{q}{Q} = \frac{a.v}{AV} = \frac{a}{A} \times \left(\frac{r}{R}\right)^{2/3}$$

$$\therefore \frac{q}{Q} = \left[\frac{\theta}{360^\circ} - \frac{\sin\theta}{2\pi}\right]\left[1 - \frac{360^\circ \sin\theta}{2\pi\theta}\right]$$

$$= \frac{\theta}{360^\circ}\left[1 - \frac{360^\circ \sin\theta}{2\pi\theta}\right]^{-5/3}$$

$$\text{For variable value of } \frac{N}{n}, \text{ we get}$$

$$\frac{q}{Q} = \frac{N}{n}\left(\frac{a}{A}\right)\left(\frac{r}{R}\right)^{2/3}$$

With the assumption that the quantity of tractive force intensity at full flow and partial flow implies equality of cleansing, i.e., for sewers to be same self-cleansing at partial depth as full depth:

$$\tau = T$$

Therefore, $\gamma_w \cdot r \cdot s_s = \gamma_w \cdot R \cdot S$ (13)

Hence, $s_s = (R/r)\,S$

Or $$\frac{s_s}{S} = \frac{R}{r} \quad (14)$$

Therefore,

$$\frac{v_s}{V} = \frac{N}{n}\left(\frac{r}{R}\right)^{2/3}\left(\frac{s_s}{S}\right)^{1/2} \quad (15)$$

OR, by substituting $r/R = S/s_s$

$$\frac{v_s}{V} = \frac{N}{n}\left(\frac{r}{R}\right)^{1/6} \quad (16)$$

And

$$\frac{q_s}{Q} = \frac{N}{n}\frac{a}{A}\left(\frac{r}{R}\right)^{1/6} \quad (17)$$

Example: 2

A 300 mm diameter sewer is to flow at 0.3 depth on a grade ensuring a degree of self cleansing equivalent to that obtained at full depth at a velocity of 0.9 m/sec. Find the required grade and associated velocity and rate of discharge at this depth. Assume Manning's rugosity coefficient n = 0.013. The variation of n with depth may be neglected.

Solution:

Manning's formula for partial depth

$$v = \frac{1}{n} r^{2/3} s^{1/2}$$

For full depth

$$V = \frac{1}{N} R^{2/3} S^{1/2}$$

Using V = 0.90 m/sec, N = n = 0.013 and R = D/4 = 75 mm = 0.075 m

$$0.90 = \frac{1}{0.013} 0.075^{2/3} S^{1/2}$$

$S = 0.0043$

This is the gradient required for full depth.

and, $Q = A.V = \pi/4\ (0.3)^2 \times 0.90 = 0.064\ m^3/s$

At depth $d = 0.3D$, (i.e., for $d/D = 0.3$) we have $a/A = 0.252$ and $r/R = 0.684$ (neglecting variation of n)

Now for the sewer to be same self cleansing at 0.3 m depth as it will be at full depth, we have the gradient (s_s) required as $s_s = (R/r)S$

Therefore, $s_s = S / 0.684$

$= 0.0043 / 0.0684 = 0.0063$

Now, the velocity v_s generated at this gradient is given by

$$v_s = V\frac{N}{n}\left(\frac{r}{R}\right)^{1/6}$$

$= 1 \times (0.684)^{1/6} \times 0.9$

$= 0.846\ m/s$

The discharge q_s is given by

$$q_s = Q\frac{N}{n}\frac{a}{A}\left(\frac{r}{R}\right)^{1/6}$$

$q_s = 1 \times (0.258) \times (0.939) \times (0.064)$

$= 0.015\ m^3/s$

Example: 3

A combined sewer was designed to serve an area of 60 sq. km with an average population density of 185 persons/hectare. The average rate of sewage flow is 350 L/Capita/day. The maximum flow is 50% in excess of the average sewage flow. The rainfall equivalent of 12 mm in 24 h can be considered for design, all of which is contributing to surface runoff. What will be the discharge in the sewer? Find the diameter of the sewer if running full at maximum discharge.

Solution:

Total population of the area = population density x area

$= 185 \times 60 \times 10^2$

$= 1110 \times 10^3$ persons

Average sewage flow $= 350 \times 11.1 \times 10^5$ Liters/day

$= 388.5 \times 10^6$ L/day

$= 4.5\ m^3/sec$

Storm water flow $= 60 \times 10^6 \times (12/1000) \times [1/(24 \times 60 \times 60)]$

$= 8.33\ m^3/sec$

Maximum sewage flow $= 1.5 \times$ average sewage flow

$= 1.5 \times 4.5 = 6.75\ m^3/sec$

Total flow of the combined sewer = sewage flow + storm flow

$= 6.75 + 8.33 = 15.08\ m^3/sec$

Hence, the capacity of the sewer $= 15.08\ m^3/sec$

Hence, diameter of the sewer required at the velocity of 0.9 m/s can be calculated as

$$\pi/4\ (D)^2 \times 0.90 = 15.08\ m^3/s$$

Hence, D = 4.62 m

Find the minimum velocity and gradient required to transport coarse sand through a sewer of 40 cm diameter with sand particles of 1.0 mm diameter and specific gravity 2.65, and organic matter of 5 mm average size with specific gravity 1.2. The friction factor for the sewer material may be assumed 0.03 and roughness coefficient of 0.012. Consider k = 0.04 for inorganic solids and 0.06 for organic solids.

Solution

Minimum velocity i.e. self cleansing velocity

$$Vs = \sqrt{\frac{8k}{f'}(Ss-1)gd'}$$

$$Vs = \sqrt{\frac{8x0.04}{0.03}(2.65-1)x9.81x0.001}$$

$$= 0.4155 \text{ m/sec say } 0.42 \text{ m/sec}$$

Similarly, for organic solids this velocity will be 0.396 m/sec

Therefore, the minimum velocity in sewer = 0.42 m/sec

Now, Diameter of the sewer D = 0.4 m

Hydraulic Mean Depth = D/4 = 0.4/4 = 0.1 m

Using Manning's formula:

$V = 1/n\, R^{2/3}\, S^{1/2}$

$0.42 = (1/0.012) \times (0.1)^{2/3} \times S^{1/2}$

S = 1/1824.5

Example : 5

Design a sewer running 0.7 times full at maximum discharge for a town provided with the separate system, serving a population 80,000 persons. The water supplied from the water works to the town is at a rate of 190 LPCD. The manning's n = 0.013 for the pipe material and permissible slope is 1 in 600. Variation of n with depth may be neglected. Check for minimum and maximum velocity assuming minimum flow 1/3 of average flow and maximum flow as 3 times the average. (for d/D = 0.7, q/Q = 0.838, v/V = 1.12)

Solution

Average water supplied = 80000 x 190 x (1/24 x 60 x 60 x 1000) = 0.176 m^3/sec

Sewage production per day, (considering 80% of water supply) = 0.176 x 0.8 = 0.14 m^3/sec

Maximum sewage discharge = 3 x 0.14 = 0.42 m^3/sec

Now for d/D = 0.7, q/Q = 0.838, v/V = 1.12

Therefore, Q = 0.42/0.838 = 0.5 m^3/sec

Now

$$Q = \frac{1}{n}\frac{\pi D^2}{4}\left(\frac{D}{4}\right)^{2/3} S^{1/2}$$

$$Q = \frac{1}{0.013}\frac{\pi D^2}{4}\left(\frac{D}{4}\right)^{2/3}\left(\frac{1}{600}\right)^{1/2}$$

Therefore, D = 0.78 m

V = Q/A = 1.04 m/sec

Now, v/V = 1.12

Therefore v = 1.12 x 1.04 = 1.17 m/sec

This velocity is less than limiting velocity hence, OK

Check for minimum velocity

Now q_{min} = 0.14/3 = 0.047 m^3/sec

q_{min}/Q = 0.047/0.5 = 0.09

From proportional chart, for q/Q = 0.09, d/D = 0.23 and v/V = 0.65

Therefore, the velocity at minimum flow = 0.65 x 1.04 = 0.68 m/sec

This velocity is greater than self cleansing velocity, hence OK

d_{min} = 0.23 x 0.78 = 0.18 m

Questions

1. A 900 m long storm sewer collects water from a catchment area of 40 hectares, where 35% area is covered by roof (C=0.9), 20% area by pavements (C=0.8) and 45% area is covered by open plots (C=0.15). Determine the average intensity of rainfall and diameter of storm water drain. Assume the time of entry = 3 min; velocity at full flow = 1.45 m/sec; gradient of sewer = 0.001, and roughness coefficient = 0.013. The intensity of rainfall, cm/h = 75/(t + 5).
2. Explain the importance of considering minimum and maximum velocity while designing the sewers.
3. Explain 'Self-cleansing velocity'.
4. Explain important consideration while finalizing alignment and bed line of storm water drain.
5. Find the gradient required in sewer of 0.5 m diameter to maintain self cleansing velocity at flow full condition.
6. Write short notes on laying of sewer pipes. What hydraulic tests are conducted on the sewers?
7. Prepare notes on sewer maintenance.

Answers

Q. 1: Overall runoff coefficient = 0.5425; Average intensity of rainfall = 4.09 cm/h; Storm water quantity = 2.465 m^3/sec; and diameter of storm water drain = 1.556 m

DISPOSAL OF EFFLUENTS

SELF PURIFICATION

The automatic purification of natural water is known as self purification. The self purification of natural water systems is a complex process that often involves physical, chemical, and biological processes working simultaneously

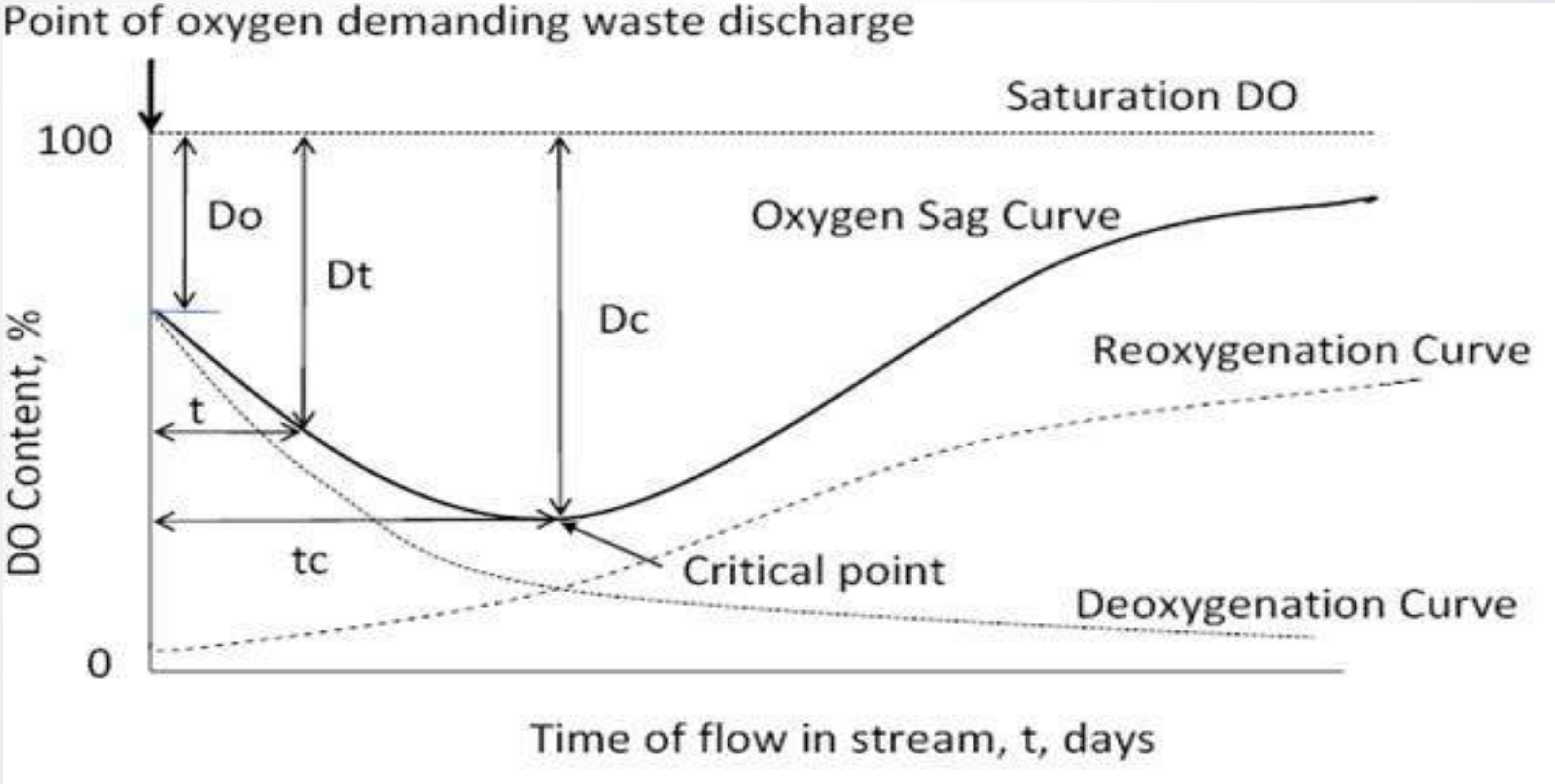
- Dilution: When sufficient dilution water is available in the receiving water body
- Current: When strong water current is available, the discharged wastewater will be thoroughly mixed with stream water preventing deposition of solids
- Temperature: The quantity of DO available in stream water is more in cold temperature than in hot temperature.
- Sunlight: sunlight helps in purification of stream by adding oxygen through photosynthesis.
- Rate of Oxidation: The rate of oxidation of organic matter depends on the chemical composition of organic matter.

CONDITIONS FAVOURABLE FOR DILUTION

- Where sewage is fresh.
- Where favorable currents exits in a stream.
- Where sewage is almost free from floating/ Settleable solids.
- Where thorough mixing is possible.
- Where diluting water has high quantities of dissolved oxygen.
- When the city is situated near river or sea.

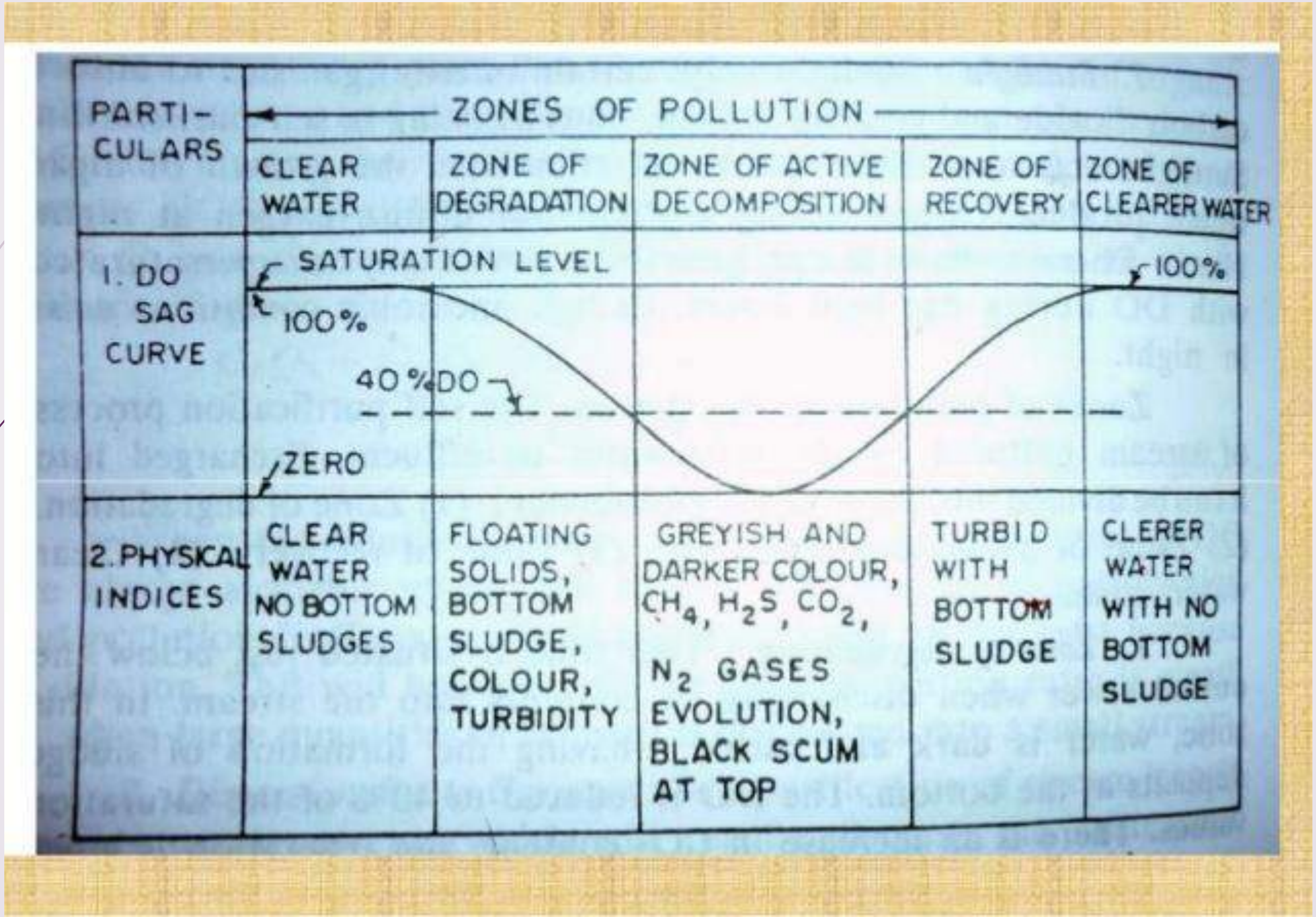
Oxygen Sag Curve

- deficit in the stream at any point of time during self purification process is the difference between the saturation DO content and actual DO content at that time.
- Oxygen deficit, D = Saturation DO – Actual DO
- de - oxygenation + re -oxygenation curves. = oxygen sag curve



Deoxygenation, reoxygenation and oxygen sag curve

Zones of purification



Mathematical analysis of Oxygen Sag Curve:
Streeter – Phelps equation

$$Dt = \frac{K' Lo}{R' - K'}\left[e^{-K't} - e^{-R't}\right] + Do.e^{-R't}$$

or

$$Dt = \frac{KLo}{R-K}\left[10^{-K.t} - 10^{-R.t}\right] + Do.10^{-R.t}$$

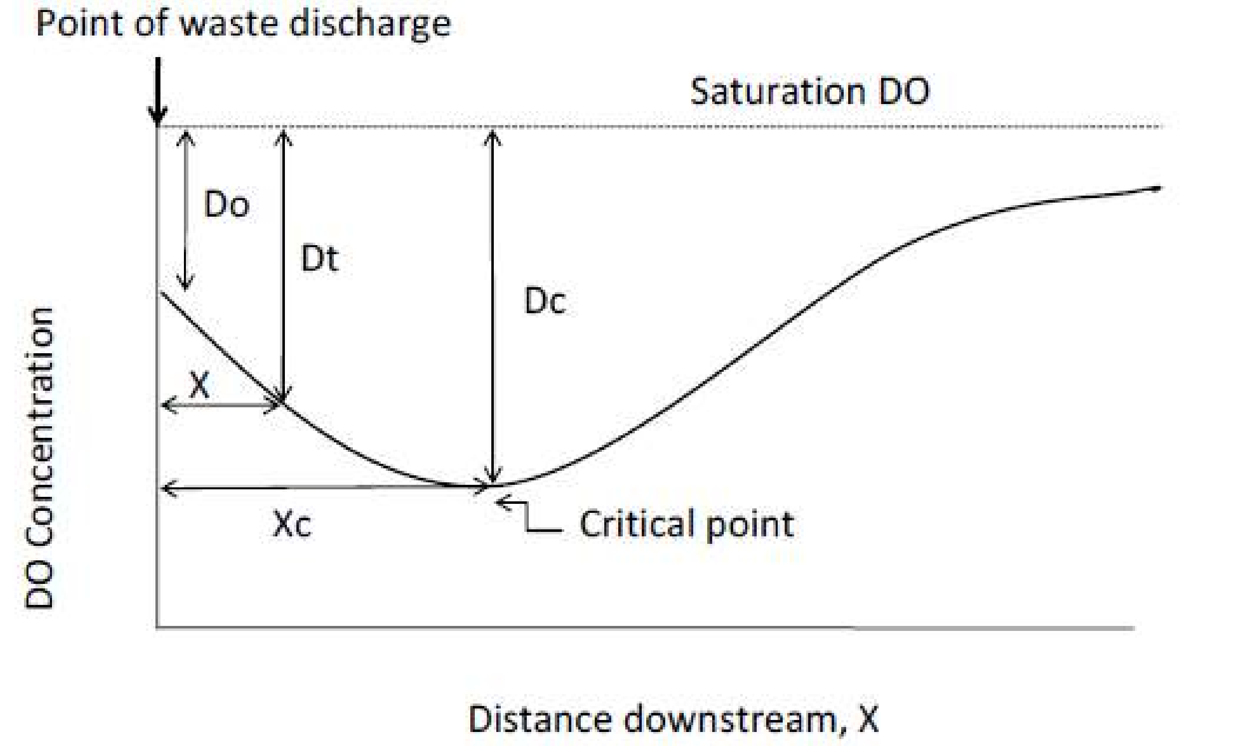


Figure 12.2 Oxygen sag curve of Streeter-Phelps equation

Where,

$K = K^1$ = BOD reaction rate constant, to the base 10

$R = R^1$ = Re Oxygenation constant to the base 10

Do = Initial oxygen deficit at the point of waste discharge at time t = o

t = time of travel in the stream from the point of discharge = x/u

x = distance along the stream

u = stream velocity

Determination of Critical DO deficit (D_C) and distance X_c

$$Dc = \frac{K'}{R'} Lo.e^{-K'tc}$$

or

$$Dc = \frac{K}{R} Lo.10^{-K.tc}$$

$$tc = \frac{1}{R'-K'} \log_e \frac{R'}{K'}\left[1 - \frac{Do(R'-K')}{K' Lo}\right]$$

or

$$tc = \frac{1}{R-K} \log_{10} \frac{R}{K}\left[1 - \frac{Do(R-K)}{K.Lo}\right]$$

The distance X_c is given by $X_c = tc \, . \, u$

u = velocity of flow in the stream

$$R_T = R_{20}(1.024)^{(T-20)}$$

$$K_T = K_{20}(\theta)^{(T-20)}$$

$K^1 = K/2.303$

$R^1 = R/2.303$

Where θ = 1.056 in general

= 1.047 for 20°C to 30°C

= 1.135 for 4°C to 20°C

$$f_s = \frac{R}{K} = \frac{R^1}{K^1} = 0.5 \text{ to } 5.0$$

SEWAGE FARMING

Process in which sewage is used for growing crops

- fertilizing elements of sewage – NPK used by the roots of crops
- nutrients of sewage make the fields fertile

APPLICATION OF SEWAGE METHODS

- Flooding method
- Surface irrigation method
- Zig zag method
- Lagooning method
- Basin method
- Sub-soil irrigation method
- RIDGE AND FERROW IRRIGATION METHOD

FLOODING METHOD –

- divided into various parts surrounded by dykes.
- sewage is filled like small ponds in between the dykes
- Depth = 3.0 - 5.0 cm

SURFACE IRRIGATION

- suited in sloping area
- parallel drains are constructed in fields
- drains are connected to a distributaries drain with regulating device
- large quantity is absorbed by the field and only excess quantity reaches another drain.

ZIG ZAG METHOD -ridges are arranged in a zag-zag method

- **LAGOONING**
- natural depression available
- Detention period = 1 month
- sludge is stabilized and dried
- shallow and must be constructed away from town

SUB SURFACE IRRIGATION

- sewage applied at the roofs of plants, through the open jointed drain-pipes
- pipes are laid at 1.0 m below the GL

BASIN METHOD:

- big trees are planted in an isolated manner,
- basins are formed around each tree and filled with sewage.
- suitable for fruit gardens

RIDGE AND FERROW IRRIGATION

Ridge

- Width=125-250 cm Length 10-30 m.

Furrow

- **Width =** 120-150 cm Depth 25-50 cm
- percolated effluent is collected in underground drains flows towards natural drainage for disposal

FAVOURABLE CONDITIONS FOR SUB-SURFACE IRRIGATION

- Subsoil water level is low.
- Land is cheap.
- Rainfall is less and irrigation water demand is heavy.
- Large areas are available.
- Where dilution water is not easily available.
- Sub-surface strata is porous, favoring infiltration.
- Climate is dry favoring drying up conditions.

SEWAGE SICKNESS

soil getting clogged and loses its capacity of receiving the sewage load when the sewage is applied continuously on a piece of land is called **sewage sickness**

PREVENTION OF SEWAGE SICKNESS:-

- Primary treatment like screening & sedimentation should be given to sewage before its application to land so that suspended solids are removed & the pores of soil will not be clogged.
- The sewage should be applied intermittently on land i.e by giving rest to the land for sometime .The land should be ploughed during non supply period of sewage so that soil gets aerated.
- Keeping some portion of land reserved in order to use the same in resting period .Enough area will be required for this purpose.
- By planting different crops on the same land by rotation system of crops .The soil will be aerated & will utilise the fertilizing elements of sewage.
- By providing sufficient under drainage system to collect the excessive sewage quantity.
- By frequent ploughing & rotation of soil.
- By not applying the sewage in excess quantity.

Streeter-Phelps equation

... Streeter-Phelps equation ; *i.e.*,

$$D_t = \frac{K_D \cdot L}{K_R - K_D}\left[(10)^{-K_D \cdot t} - (10)^{-K_R \cdot t}\right] + \left[D_0 \times (10)^{-K_R \cdot t}\right] \qquad \ldots(8.3)$$

where D_t = the D.O. deficit in mg/l after t days.

L = *Ultimate first stage B.O.D. of the mix* at the point of waste discharge in mg/l.

D_0 = *Initial oxygen deficit* of the mix at the mixing point in mg/l.

K_D = De-oxygenation coefficient for the wastewater, which can be considered as equal to the *BOD rate constant** deter-

K_R** = Re-oxygenation coefficient for the stream.

The critical time (t_c)

$$t_c = \frac{1}{K_D(f-1)} \log\left[\left\{1-(f-1)\frac{D_0}{L}\right\} f\right]$$

Self Purification Constant $\left(f = \frac{K_R}{K_D}\right)$

the critical or max. oxygen deficit

$$D_c = \frac{L}{f}\left[10\right]^{-K_D t_c}$$

$$\left(\frac{L}{D_c \cdot f}\right)^{f-1} = f\left[1-(f-1)\frac{D_0}{L}\right]$$

Example 8.1. *The sewage of a town is to be discharged into a river stream. The quantity of sewage produced per day is 8 million litres, and its B.O.D. is 250 mg/l. If the discharge in the river is 200 l/s and if its B.O.D. is 6 mg/l, find out the B.O.D. of the diluted water.*

Solution. Sewage discharge $= Q_S$

$$= \frac{8 \times 10^6}{24 \times 60 \times 60} \ l/s. = 92.59 \ l/s.$$

Discharge of the river $= Q_R = 200 \ l/s$

B.O.D. of sewage $= C_S = 250$ mg/l

B.O.D. of river $= C_R = 6$ mg/l.

Using equation (8.1), we have

B.O.D. of the diluted mixture

$$= C = \frac{C_S \cdot Q_S + C_R \cdot Q_R}{Q_S + Q_R}$$

or $$C = \frac{250 \times 92.59 + 6 \times 200}{92.59 + 200}$$

$$= 83.21 \text{ mg}/l. \quad \textbf{Ans.}$$

Example 8.2. *In the above example, what should be the river discharge, if it is desired to reduce the B.O.D. of diluted water to 20 mg/l.*

Solution. Here $C = 20$ mg/l

$$\therefore \quad 20 = \frac{250 \times 92.59 + 6 \times Q_R}{92.59 + Q_R}$$

or $$Q_R = 1521 \; l/s. \quad \textbf{Ans.}$$

Example 8.3. *A city discharges 1500 litres per second of sewage into a stream whose minimum rate of flow is 6000 litres per second. The temperature of sewage as well as water is 20°C. The 5 day B.O.D. at 20°C for sewage is 200 mg/l and that of river water is 1 mg/l. The D.O. content of sewage is zero, and that of the stream is 90% of the saturation D.O. If the minimum D.O. to be maintained in the stream is 4.5 mg/l, find out the degree of sewage treatment required. Assume the de-oxygenation coefficient as 0.1, and re-oxygenation coefficient as 0.3.*

Solution. From the table given at the end of the book in Appendix A-3, the value of saturation D.O. at 20°C is found out as 9.17 mg/*l*.

D.O. content of the stream

$$= 90\% \text{ of the saturation D.O.}$$

$$= \frac{90}{100} \times 9.17 = 8.25 \text{ mg}/l.$$

D.O. of mix at the start point (*i.e.* at $t = 0$)

$$= \frac{8.25 \times 6000 + 0 \times 1500}{6000 + 1500} \qquad (\because \text{ D.O. of sewage is zero})$$

$$= 6.6 \text{ mg}/l$$

$\therefore \qquad D_0$ = initial D.O. deficit

= [Saturation D.O. at mix. temp. – D.O. of mix.]

= 9.17 – 6.6 = 2.57 mg/l *(Assume instantaneous mixing)*

Minimum D.O. to be maintained in the stream

= 4.5 mg/*l*.

$\therefore$ Max. permissible saturation deficit (*i.e.*, critical D.O. deficit)

= D_c = 9.17 – 4.5

= 4.67 mg/*l*.

Now, using equations (8.11), the first stage B.O.D. of mixture of sewage and stream (L) is given by

$$\left[\frac{L}{D_c f}\right]^{f-1} = f\left[1-(f-1)\frac{D_0}{L}\right]$$

Substituting the values as :

$$D_0 = 2.57 \text{ mg/l} \quad \text{and} \quad D_c = 4.67 \text{ mg/l}$$

$$f = \frac{K_R}{K_D} = \frac{0.3}{0.1} = 3$$

we get $$\left[\frac{L}{4.67 \times 3}\right]^{3-1} = 3\left[1 - (3-1)\frac{2.57}{L}\right]$$

or $$\left[\frac{L}{14.01}\right]^{2} = 3\left[1 - \frac{5.14}{L}\right]$$

Solving by hit and trial, we get the value

$$L = 21.1 \text{ mg/}l$$

Now, using $Y_t = L\left[1 - 10^{-K_D \cdot t}\right]$, we have

Max. permissible 5 day B.O.D. of the mix (at 20°C)

$$Y_5 = 21.1\left[1 - 10^{-0.1 \times 5}\right] \qquad \text{(where } K_D \text{ at } 20°\text{C} = 0.1\text{)}$$

$$= 14.43 \text{ mg/}l.$$

Now, using equation (8.1), we have

$$C = \frac{C_S Q_S + C_R Q_R}{Q_S + Q_R}$$

where C stands for concentrations of B.O.D.

Substituting the values, we get

$$14.43 = \frac{C_S \times 1500 + 1 \times 6000}{1500 + 6000}$$

where C_S will represent the permissible B.O.D.$_5$ (at 20°C of course) of the discharged wastewater.

Solving, we get

$$C_S = 68.16 \text{ mg}/l.$$

∴ Degree of treatment required (per cent)

$$= \frac{\text{Original B.O.D. of sewage} - \text{Permissible B.O.D.}}{\text{Original B.O.D.}} \times 100$$

$$= \frac{200 - 68.16}{200} = \frac{131.84}{200}$$

$= 65.9\%$. **Ans.**

Example 8.4. *A city discharges 100 cumecs of sewage into a river, which is fully saturated with oxygen and flowing at the rate of 1500 cumecs during its lean days with a velocity of 0.1 m/sec. The 5-days BOD of sewage at the given temperature is 280 mg/l. Find when and where the critical D.O. deficit will occur in the downstream portion of the river, and what is its amount. Assume coefficient of purification of the stream (f) as 4.0, and coefficient of de-oxygenation (K_D) as 0.1.*

Solution. The initial D.O. of river

$$= \text{Saturation D.O. at the given temp.} = 9.2 \text{ mg/}l \text{ (say)}$$

D.O. of mix at $t = 0$ *i.e.*, at start

$$= \frac{9.2 \times 1500 + 0 \times 100}{1500 \times 100} \quad \textit{(assuming that D.O. of sewage is nil)}$$

$$= 8.62 \text{ mg/}l$$

Initial D.O. deficit of the stream

$$= D_0 = 9.2 - 8.62 = 0.58 \text{ mg/l}$$

Also, 5-day BOD of the mixture of sewage and stream is given by

$$C = \frac{C_S Q_S + C_R Q_R}{Q_S + Q_R}$$

$$= \frac{280 \times 100 + 0 \times 1500}{100 + 1500}$$

$$= \frac{280 \times 100}{1600} = 17.5 \text{ mg/l.}$$

BOD of mix at the given temp. $= Y_5 = 17.5$ mg/l

$$= \frac{280 \times 100}{1600} = 17.5 \text{ mg/l.}$$

$\therefore$ 5 day BOD of mix at the given temp. = Y_5 = 17.5 mg/l

$$Y_5 = L\left[1-(10)^{-K_D \times 5}\right] \text{ and } K_D = 0.1 \text{ (at 20°C)}$$

$\therefore$ The ultimate BOD of the mix (*i.e.* L)

$$= \frac{17.5}{0.684} = 25.58 \text{ mg/}l.$$

Now, using equation (8.11), we have

$$\left[\frac{L}{D_c \cdot f}\right]^{f-1} = f\left[1-(f-1)\frac{D_0}{L}\right]$$

$$\left[\frac{25.58}{D_c \times 4}\right]^3 = 4\left[1-\frac{3 \times 0.58}{25.58}\right]$$

or $D_c = 4.12$ mg/l. **Ans.**

Now, from equation (8.8), we have

$$t_c = \frac{1}{K_D(f-1)} \log_{10}\left[f\left\{1-(f-1)\frac{D_0}{L}\right\}\right]$$

or

$$t_c = \frac{1}{0.1(4-1)} \log_{10}\left[4\times\left\{1-\frac{3\times 0.58}{25.58}\right\}\right]$$

$$= \frac{1}{0.3}\times 0.571 = 1.905 \text{ days.}$$

Now, distance = Velocity of river × Travel time

$$= 0.1 \text{ m/sec} \times (1.905 \times 24 \times 60 \times 60 \text{ sec})$$

$$= 16{,}460 \text{ m} = 16.46 \text{ km}$$

Hence, the most critical deficit will occur after 1.905 days and at point 16.46 km downstream of the point of sewage disposal. **Ans.**

Example 8.5. *A town with a population of 30,000 has to design a sewage treatment plant to handle industrial as well as domestic wastewaters of the town. A sanitary survey revealed the following :*

Dairy wastes of 3 million litres per day with BOD of 1100 mg/l, and sugar mill waste of 2.4 million litres per day with BOD of 1500 mg/l are produced. In addition, domestic sewage is produced at the rate of 240 litres per capita per day. The per capita BOD of domestic sewage being 72 gm/day. An overall expansion factor of 10 per cent to be provided. The sewage effluents are to be discharged to a river stream with a minimum dry weather flow of 4500 litre per second and a saturation dissolved oxygen content of 9 mg/l. It is necessary to maintain a dissolved oxygen content of 4 mg/l in the stream. Determine the degree of treatment required to be given to the sewage. Assume suitable values of coefficients of de-oxygenation and re-oxygenation.

Solution. Per capita B.O.D. of the domestic sewage

$$= 72 \text{ gm/day} = 72 \times 1000 \text{ mg/day.}$$

The per capita sewage produced

$$= 240 \text{ litre/day.}$$

$\therefore$ B.O.D. per litre of the domestic sewage

$$= \frac{72 \times 1000}{240} \text{ mg/l.} = 300 \text{ mg/l.}$$

Amount of domestic wastewater produced per day

$$= 30{,}000 \times 240 \text{ litres} \doteq 7.2 \text{ million litres}$$

Net B.O.D. of all wastewaters (*i.e.* domestic + industrial)

$$= \left[\frac{7.2 \times 300 + 3 \times 1100 + 2.4 \times 1500}{7.2 + 3 + 2.4}\right] = 719 \text{ mg/l.}$$

Total wastewater discharge

$$= \frac{\text{Vol. of wastewaters entering per day}}{\text{No. of secs in 1 day}}$$

$$= \frac{3 \text{ Ml} + 2.4 \text{ Ml} + 7.2 \text{ Ml}}{1 \times 24 \times 60 \times 60 \text{ sec}}$$

$$= \frac{12.6 \times 10^6}{24 \times 3600} \; l/s = 145.8 \; l/s$$

$$= \frac{\quad}{24 \times 3600}$$

Total wastewater discharge with 10% expansion factor

$$= 1.1 \times 145.8 \ l/s = 160 \ l/s$$

Initial D.O. of saturated stream water

$= 9$ mg/l (*i.e.* saturation D.O. as given)

$\therefore$ D.O. of mixture at $t = 0$ *i.e.*, at start point

$$= \frac{\text{D.O. of river} \times Q_R + \text{D.O. of sewage} \times Q_S}{Q_R + Q_S}$$

$$= \frac{9 \times 4500 + 0 \times 160}{4500 + 160}$$

(Assuming that the D.O. of wastewaters is nil

$= 8.69$ mg/l

$\therefore$ Initial D.O. deficit

$= D_0 = 9 - 8.69$ *(assuming instantaneous mixing)*

$= 0.31$ mg/l

Also, critical D.O. deficit, *i.e.* allowable max. D.O. deficit

$= D_c = 9 - 4.0$

$= 5$ mg/l

Now, using eqn. (8.11), we have

$$\left[\frac{L}{D_c \cdot f}\right]^{f-1} = f\left[1 - (f-1)\frac{D_0}{L}\right]$$

where $D_c = 5$ mg/l,

$D_0 = 0.31$ mg/l,

$K_D = 0.1$; $K_R = 0.3$; $f = 3$

(assumed values at mix. temp.)

$$\therefore \quad \left[\frac{L}{5\times 3}\right]^2 = 3\left[1-\frac{2\times 0.31}{L}\right]$$

Solving by hit and trial,

$$L = 25.65 \text{ mg/l}$$

Max. permissible 5 day B.O.D. of mix at mix temp.

$$= Y_5 = L\left[1-(10)^{-0.1\times 5}\right]_{(K_D \text{ at mix temp. is assumed} = 0.1)}$$

$$= 0.684\, L$$

$$= 0.684 \times 25.65 = 17.54 \text{ mg/l}$$

Using eqn. (8.1) as

$$C = \frac{C_S Q_S + C_R Q_R}{Q_S + Q_R}$$

$q_S + q_R$

we get $\quad 17.54 = \dfrac{C_S + 160 + 0 \times 4500}{160 + 4500}$

where C_S = Max. permissible B.O.D.$_5$ of waste-waters

or $\quad C_S = 511$ mg/l.

$\therefore$ Permissible B.O.D. of wastewaters

$= 511$ mg/l

Initial B.O.D. of city wastewaters

$= 719$ mg/l

$\therefore$ *Degree of treatment reqd.*

$$= \frac{719 - 511}{719} = 28.93\%. \quad \text{Ans.}$$

Example 8.6. *In the previous example, determine what should be the dilution ratio if no treatment was required, and thus determine the river discharge for such a condition.*

Solution. When no treatment is required, the value of max. permissible BOD_5 of wastewaters, *i.e.* C_S should be 719. Q_R can then be determined as :

$$17.54 = \frac{719 \times 160 + 0 \times Q_R}{160 + Q_R}$$

or $$17.54\,(160 + Q_R) = 719 \times 160$$

or $$160 + Q_R = \frac{719 \times 160}{17.54} = 6559$$

or $$Q_R = 6399 \text{ l/s (say)}.$$ **Ans.**

$$\text{Dilution ratio} = \frac{6399}{160} = 39.99 \text{ ; Say 40 times.}$$

Hence when the dilution ratio is 40 and the minimum river discharge is 6400 *l/s*, no treatment will be required. **Ans.**

[**Note.** Strictly speaking, when Q_R increases, D_0 will reduce, increasing Y_5, needing repeat of calculations, to obtain precise results. But the effect will be very small and on safer side, and hence is generally ignored.

Example 8.7. *A waste water effluent of 560 l/s with a BOD = 50 mg/l, DO = 3.0 mg/l and temperature of 23°C enters a river where the flow is 28 m³/sec, and BOD = 4.0 mg/l, DO = 8.2 mg/l, and temperature of 17°C. k_1 of the waste is 0.10 per day at 20°C. The vel. of water in the river downstream is 0.18 m/s and depth of 1.2 m. Determine the following after mixing of waste water with the river water :*

(i) Combined discharge ;

(ii) BOD ;

(iii) DO ; and

(iv) Temperature. (*Civil Services, 1981*)

Solution.

Particulars of Sewage thrown	*Particulars of River*
Q_S = 560 l/s = 0.56 m³/sec	Q_R = 28 m³/sec
Concentrations (C_S)	Concentrations (C_R)
BOD = 50 mg/l	BOD = 4.0 mg/l
DO = 3.0 mg/l	DO = 8.2 mg/l
Temp. = 23°C	Temp. = 17°C.
k_1 at 20° = 0.1 per day	

(i) Combined discharge

$$= Q_S + Q_R$$

$$= 0.56 + 28 = 28.56 \text{ m}^3/\text{sec.} \quad \textbf{Ans.}$$

Now, using equation (8.1), for conc. of mix as

$$C = \frac{C_S \cdot Q_S + C_R \cdot Q_R}{Q_S + Q_R}, \text{ we have}$$

(ii) BOD of mix

$$= \frac{50 \times 0.56 + 4.0 \times 28}{0.56 + 28} = \frac{140}{28.56} = 4.9 \text{ mg/l.}$$

(iii) DO of mix

$$= \frac{3.0 \times 0.56 + 8.2 \times 28}{0.56 + 28} = 8.098 \text{ mg/l} \quad \textbf{Ans.}$$

(iv) Temp. of mix

$$= \frac{23 \times 0.56 + 17 \times 28}{0.56 + 28} = 17.12°\text{C.} \quad \textbf{Ans.}$$

Example 8.8. *125 cumecs of sewage of a city is discharged in a perennial river which is fully saturated with oxygen and flows at a minimum rate of 1600 cumecs with a minimum velocity of 0.12 m/sec. If the 5 day BOD of the sewage is 300 mg/l, find out where the critical DO will occur in the river. Assume :*

(i) *the coefficient of purification of the river as 4.0,*

(ii) *the ultimate BOD as 125% of the 5 day BOD of the mixture of sewage and river water.*

(iii) *Saturation DO of river = 9.2 mg/l*

Solution. *Saturation D.O. Concentration of the given river*

$= 9.2.$ (given in assumption iii)

The D.O. of the river at the mixing point after disposal of sewage (D)

$$= \frac{125 \times 0 + 1600 \times 9.2}{125 + 1600} = 8.53 \text{ mg/l}$$

Initial D.O. deficit $(D_0) = D_s - D$

$= 9.2. - 8.53 = 0.67$ mg/l.

BOD_5 of the river at the mixing point after disposal of sewage (Y_5)

$$= \frac{125 \times 300 + 1600 \times 0}{125 + 1600} = 21.74 \text{ mg/l}$$

The distance along the river, where the critical D.O. deficit will occur

$= S =$ Velocity × Time

$= 0.12$ m/sec × $(1.354 \times 24 \times 3600$ sec$)$

= 14.04 km ; Say 14 km

Hence, critical D.O. deficit will occur at 14 km downstream of the sewage disposal point. **Ans.**

Example 8.9. *A wastewater treatment plant disposes of its effluents into a stream at a point A. Characteristics of the stream at a location fairly upstream of A and of the effluent are as below:*

Item	*Units*	*Effluent*	*Stream*
Flow	m^3/s	0.20	0.50
Dissolved oxygen	mg/l	2.00	8.00
Temperature	°C	26	22
BOD_5 *at 20°C*	mg/l	40	3

Example 8.9. *A wastewater treatment plant disposes of its effluents into a stream at a point A. Characteristics of the stream at a location fairly upstream of A and of the effluent are as below:*

Item	Units	Effluent	Stream
Flow	m^3/s	0.20	0.50
Dissolved oxygen	mg/l	2.00	8.00
Temperature	°C	26	22
BOD_5 at 20°C	mg/l	40	3

Assume that the deoxygenation constant K_1 at 20°C (base e) = 0.20 d^{-1} and the re-aeration constant K_2 at 20°C (base e) = 0.40 d^{-1} for the mixture. Equilibrium concentration of dissolved oxygen C_s for the fresh water is as follows:

Temperature °C	18	20	22	23	24	25	26
C_s (mg/l)	9.54	9.17	8.99	8.83	8.53	8.38	8.22

The velocity of the stream downstream of the point A is 0.2 m/s. Determine the critical oxygen deficit and its location.

[Use temperature coefficients of 1.04 for K_1 and 1.02 for K_2]

Solution. K_1 at 20°C (base e)

$$= 0.2\ d^{-1} = 0.2 \text{ per day}$$

$\therefore$ K_D at 20°C (base 10)

$$= \frac{K_1}{2.3} = 0.434\ K_1$$

$$= 0.434 \times 0.2 \text{ per day} = \mathbf{0.087 \text{ per day.}}$$

Similarly, K_R at 20°C

$$= 0.434 \times 0.4\ d^{-1} = \mathbf{0.174 \text{ per day.}}$$

The formulas to be used in this question for converting K_D and K_R at any other temperature (T°C) will be

$$K_{D(T°)} = K_{D(20°)} \left[1.04\right]^{T° - 20°} \text{; and}$$

$$K_{R(T°)} = K_{R(20°)} \left[1.02\right]^{T° - 20°} \qquad \text{(as per the given values)}$$

(i) We will now determine DO, BOD and temperature of mixture as below:

$$\text{DO of mixture} = \frac{\text{DO of sewage} \times Q_S + \text{DO of river} \times Q_R}{Q_S + Q_R}$$

$$= \frac{2 \times 0.20 + 8 \times 0.50}{0.20 + 0.50}$$

$$= 6.29 \text{ mg/l.}$$

BOD$_5$ of mixture

(*i.e.* *5 day BOD at 20°C*)

$$= \frac{40 \times 0.20 + 3 \times 0.50}{0.20 + 0.50}$$

$$= 13.57 \text{ mg/l.}$$

Temperature of mixture

$$= \frac{26 \times 0.20 + 22 \times 0.50}{0.20 + 0.50} = 23.14°\text{C}$$

(*ii*) *Ultimate BOD of mixture (L)*

$$L = \frac{Y_5 (i.e.\ 5 \text{ day BOD of mixture at } 20°\text{C})}{1 - (10)^{-K_D \times 5}}$$

where K_D is at 20°C = 0.087 per day

$$= \frac{13.57}{1 - (10)^{-0.087 \times 5}} = \frac{13.57}{0.633} = 21.46 \text{ mg/l.}$$

(iii) Initial D.O. Deficit of mixture.

D.O. of mixture = 6.29 mg/l

Saturation D.O. at mixture temperature of 23.14°C

= 8.79 (interpolated from given values)

$\therefore \quad D_0$ = D.O. deficit

= 8.79 – 6.29 = **2.50 mg/l.**

(iv) Corrected values of K_D and K_R are :

$$K_{D(23.14°)} = K_{D(20°)} \left[1.04\right]^{T-20}$$

$$= 0.087\ [1.04]^{3.14} = 0.098$$

$$K_{R(23.14°)} = K_{R(20°)} \left[1.02\right]^{T-20}$$

$$= 0.174\ [1.02]^{3.14} = 0.185$$

(v) The time (t_c) after which critical D.O. deficit (D_c) occurs is given by eqn. (8.8) as

$$t_c = \frac{1}{K_D(f-1)} \log_{10}\left[\left\{1-(f-1)\frac{D_0}{L}\right\} f\right]$$

where $K_R = 0.185$

$K_D = 0.098$

$$\therefore \quad f = \frac{K_R}{K_D} = \frac{0.185}{0.098} = 1.888$$

$L = 21.45$ mg/l

$D_0 = 2.5$ mg/l.

$$\therefore \quad t_c = \frac{1}{0.098(1.888-1)} \log_{10}\left[\left(1-\frac{0.888 \times 2.5}{21.45}\right)1.888\right]$$

$$= \frac{1}{0.098(0.888)} \times 0.228 = \mathbf{2.625\ days.}$$

(vi) Now, *Distance* = Velocity × Travel time

= 0.2 m/s × (2.625 × 24 × 60 × 60 sec)

= **45.36 km. Ans.**

(vii) *D_c is now given by eqn. (8.11) as*

$$\left(\frac{L}{D_c \cdot f}\right)^{f-1} = f\left(1-(f-1)\frac{D_0}{L}\right)$$

or
$$\left(\frac{21.45}{D_c \times 1.888}\right)^{0.888} = 1.888\left(1-\frac{0.888 \times 2.5}{21.45}\right)$$

or
$$\frac{21.45}{1.888\,D_c} = \left(1.692\right)^{\frac{1}{0.888}} = (1.692)^{1.126} = 1.808$$

or
$$D_c = \frac{21.45}{1.888 \times 1.808} = 6.28 \text{ mg/l.}$$

Hence, the critical D.O. deficit equal to 6.28 mg/l occurs at 45.36 km downstream of A, after 2.625 days. **Ans.**

Example 8.10. *A treated waste water is discharged at the rate of 1.5 m^3/sec into a river of minimum flow 5 m^3/sec. The temperature of river flow and waste water flow may be assumed as 25°C. The BOD removal rate constant K_1 is 0.12/d (base 10). The BOD_5 at 25°C of the waste water is 200 mg/l, and that of the river water upstream of the wastewater outfall is 1 mg/l. The efficiency of waste water treatment is 80%. Evaluate the following :*

(i) BOD_5 at 25°C, if river water receives untreated waste water

(ii) BOD_5 at 25°C if river water receives treated waste water

(iii) ultimate BOD of the river water after it receives treated waste water.

(Civil Services, 1993)

Solution.

Discharge of waste water $= Q_W = 1.5$ m³/s

Discharge of river $= Q_R = 5$ m³/s

Temperature $= T = 25$°C

$K_{D(25°)} = K_1 = 0.12/d.$

C_W = Conc. of BOD_5 for untreated waste water = 200 mg/l

C_R = Conc. of BOD_5 for river water = 1 mg/l

Using eqn. (8.1),

(i) Conc. of BOD_5 of the mixture if untreated waste water is discharged into the river

$$= C = \frac{C_W \cdot Q_W + C_R \cdot Q_R}{Q_W + Q_R} = \frac{200 \times 1.5 + 1 \times 5}{1.5 + 5}$$

$= 46.92$ **mg/l. Ans.**

(ii) BOD$_5$ of the treated wastewater is given by

C_{TW} = 20% of the BOD$_5$ of untreated wastewater

($\because$ efficiency of wastewater treatment is 80%)

$= 20\% \times C_W = 20\% \times 200$ mg/l

$= 40$ mg/l.

BOD$_5$ of mixture if treated wastewater is discharged into the river

$$= C' = \frac{C_{TW} \cdot Q_W + C_R \cdot Q_R}{Q_W + Q_R}$$

$$= \frac{40 \times 1.5 + 1 \times 5}{1.5 + 5} = 10 \text{ mg/l.} \quad \textbf{Ans.}$$

(iii) BOD$_5$ of river water after it receives treated wastewater

= 10 mg/l (as computed above)

Ultimate BOD of this mixture

$= Y_u = L = ?$

Using eqn. (7.16), we have

$$Y_{t_{days}} = L\left[1 - (10)^{-K_D \cdot t}\right]$$

or $$Y_5 = L\left[1 - (10)^{-0.12 \times 5}\right]$$

or $$10 = L\left[1 - (10)^{-0.6}\right]$$

or $$L = 13.35 \text{ mg/l.} \quad \textbf{Ans.}$$

MODULE 5

SAMPLING TECHNIQUES

1. Grab or catch samples:

- A sample collected at a particular time and place → composition of the source at that times and place Source fairly constant in composition over a considerable period of time / substantial distances in all direction, In such circumstances same source may be quite well represented by single grab samples.

2. Composite samples:

- composite refers to a mixture of grab samples collected at the same sampling point at different time.

most useful for observing average concentrations - alternative to the separate analysis of a large number of samples,

3. Integrated samples:

- Mixture of grab samples collected from different points simultaneously or as nearly as possible

for river or stream that varies in composition across its width and depth

CHARACTERISTICS OF WASTEWATER

Physical Characteristics

- Colour

greyish brown or yellowish, - fresh sewage.

stale and septic sewage - black

(Zero DO - becomes septic).

- Odour

fresh sewage is considered as odorless, but as it starts to get stale, it begins to give

offensive odor.

Within 3 to 4 hours, all oxygen present in the sewage

gets exhausted and it starts emitting offensive odour by hydrogen sulphide gas which is formed due to anaerobic decomposition of sewage.

Colour

Temperature

- biological activity
- solubility of gases in sewage
- viscosity of sewage
- temperature, of sewage > temperature of the water
- average temperature of sewage in India is about 20^0 C - ideal temperature of sewage for
- biological activities

Turbidity

- Turbid-dirty dish water or wastewater from baths fecal matter, pieces of paper, cigarette ends, match sticks, greases, vegetable debris, fruit skins, soaps, etc
- depends on the quantity of solid matter present in suspension state

Turbidity

Chemical Characteristics

Helps in indicating the stage of sewage decomposition, its strength, and extent and type of treatment required for making it safe the chemical characteristics of sewage includes.

Solids

- Sewage -99.9 % water and only 0.1 %
- suspended solids, dissolved solids, colloidal solids, and Settleable solids.
- about 1000 kg of sewage contains
- 0.454 kg - TS
- 0.225 kg - solution,
- 0.112 kg - suspension and
- 0.112 kg - settle able.
- can be organic or inorganic
- 45 organic % & 55 % inorganic
- Inorganic matter - minerals & salts - sand, gravel, dissolved salts, chlorides, Sulphates, etc.

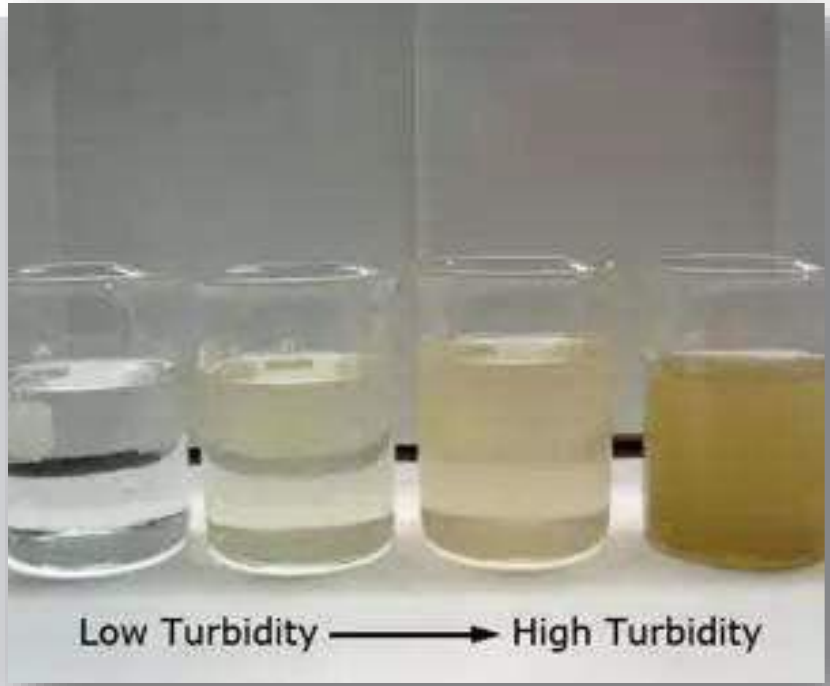
Organic matter

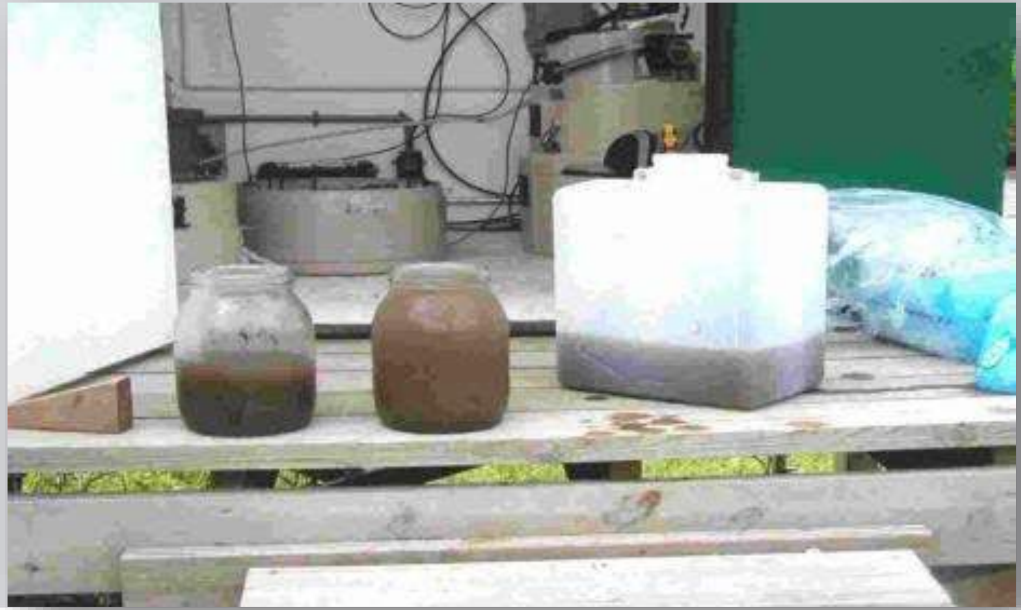
- Organic matter
- Carbohydrates like cellulose, cotton, starch, sugar, etc..
- Fats and oils
- Nitrogenous
- protein and their decomposed product, including wastes from animals,
- urea, fatty acids etc.
- inorganic solids in sewage is not harmful

Soilds

- Total Solids (S1 in mg/ lit)
- Suspended Solids (S 2)
- Dissolved Solids and colloidal (S3)
- Volatile and Fixed Suspended Solids

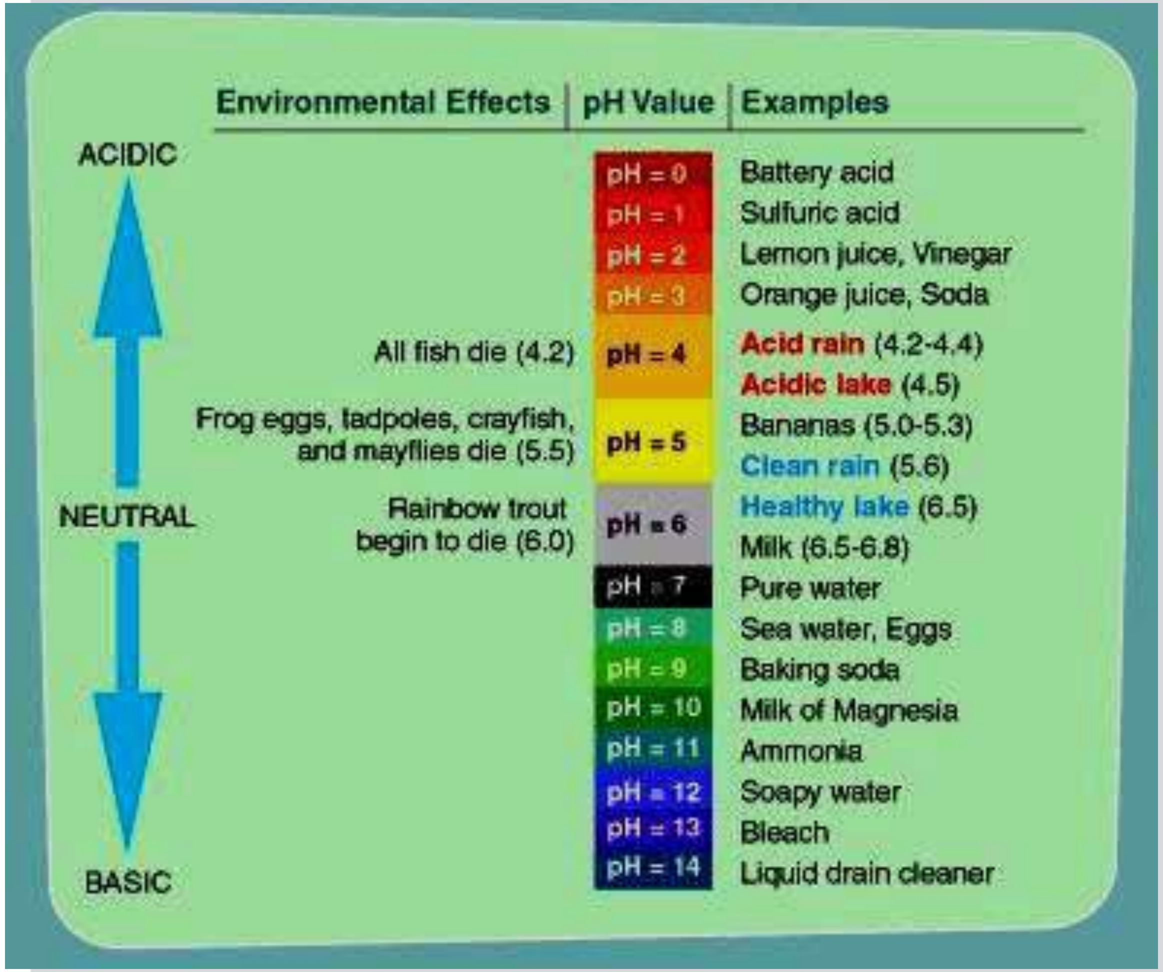
Solids

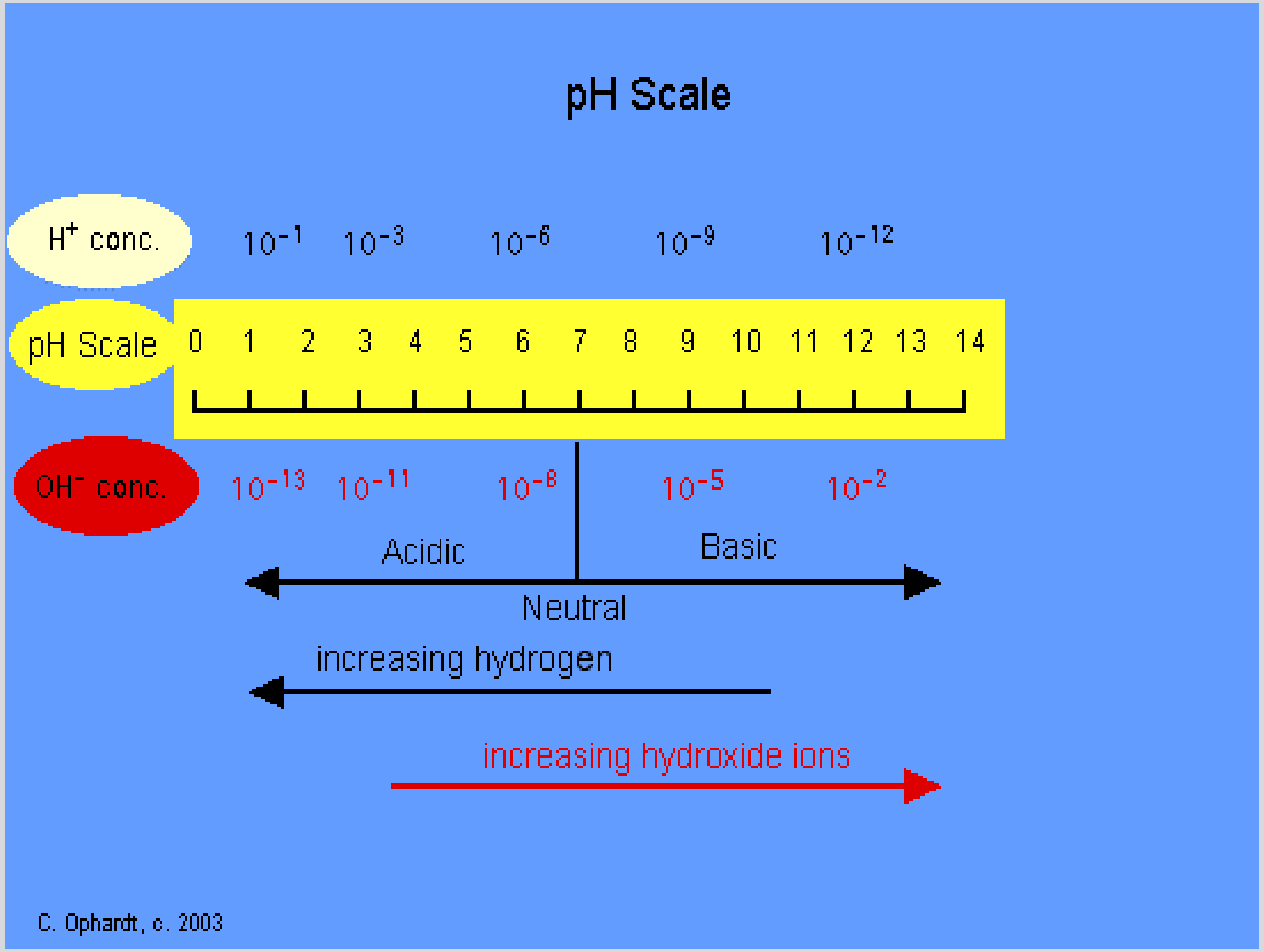




pH

- pH can be determined using pH meter (Potentiometer)
- pH tends to fall due to production of acid by bacterial action in anaerobic or nitrification processes.
- Especially the biological treatment, for better result the pH of sewage should be around 7.0 in biological treatment as microorganisms


Environmental Effects
pH Value
Examples
ACIDIC
NEUTRAL
BASIC
pH = 0
Battery acid
pH = 1
Sulfuric acid
pH = 2
Lemon juice, Vinegar
pH = 3
Orange juice, Soda
All fish die (4.2)
pH = 4
Acid rain (4.2-4.4)
Acidic lake (4.5)
Frog eggs, tadpoles, crayfish, and mayflies die (5.5)
pH = 5
Bananas (5.0-5.3)
Clean rain (5.6)
Rainbow trout begin to die (6.0)
pH = 6
Healthy lake (6.5)
Milk (6.5-6.8)
pH = 7
Pure water
pH = 8
Sea water, Eggs
pH = 9
Baking soda
pH = 10
Milk of Magnesia
pH = 11
Ammonia
pH = 12
Soapy water
pH = 13
Bleach
pH = 14
Liquid drain cleaner

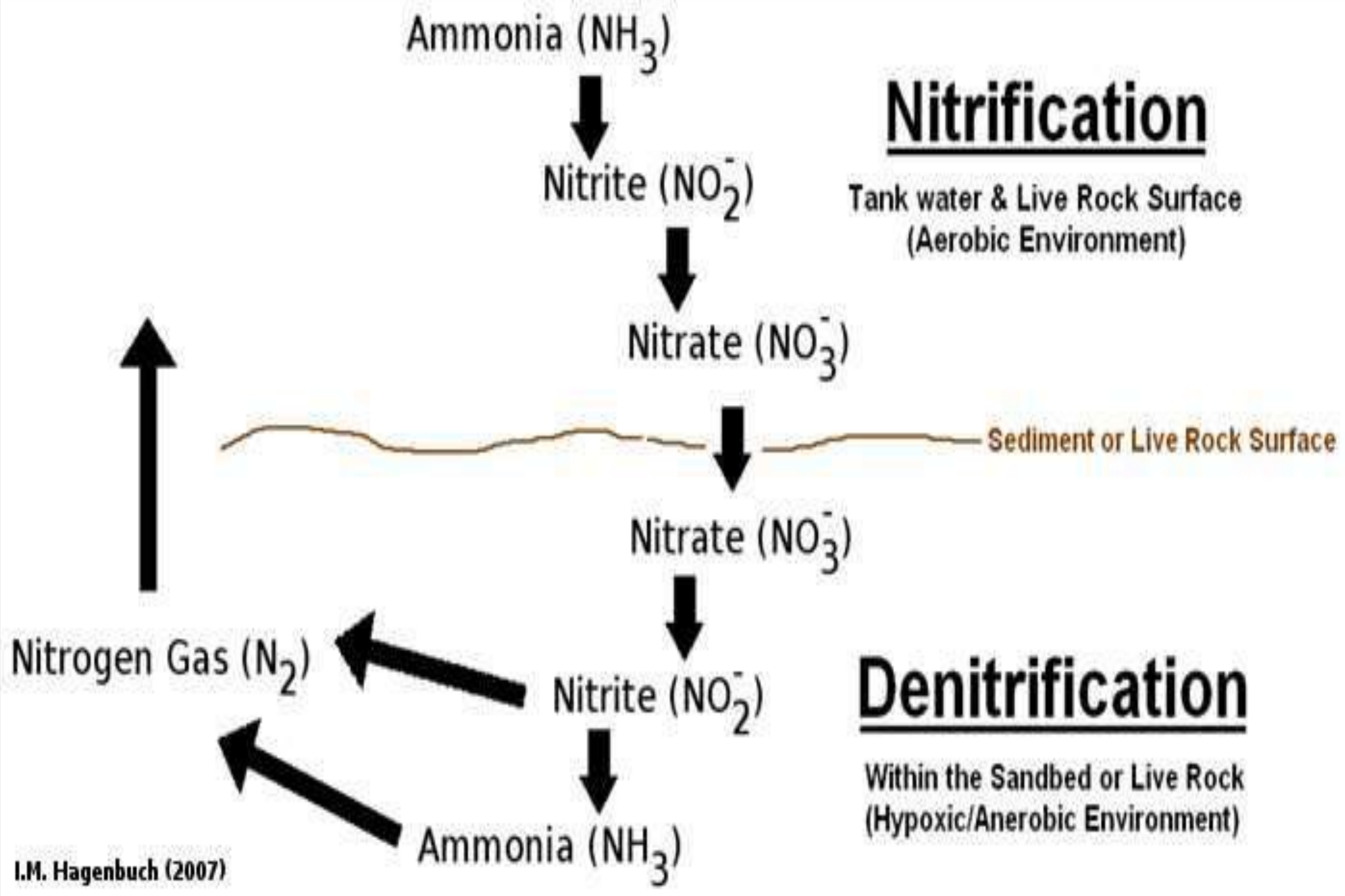

pH Scale
H+ conc.
10-1 10-3 10-6 10-9 10-12
pH Scale
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14
OH- conc.
10-13 10-11 10-8 10-5 10-2
Acidic
Basic
Neutral
increasing hydrogen
increasing hydroxide ions
C. Ophardt, c. 2003

Nitrogen Content (Nitrogen Compounds)

- The presence of nitrogen in sewage is an indication of the presence of the organic matter and may occur in one or more of the following forms:
- • Free ammonia called ammonia nitrogen
- • Albuminoid or Organic Nitrogen
- • Nitrites
- • Nitrates

- The free ammonia indicates the very first stage of decomposition of organic matter (thus indicating recent pollution);
- Albuminoidal nitrogen - the quantity of nitrogen in sewage before the decomposition of organic matter.
- Nitrates - presence of fully oxidized organic matter in sewage.
- The nitrites - intermediate stage of conversion - indicating the progress of treatment. sewage treatment is incomplete, and sewage is stale.
- nitrates - well oxidized and treated sewage.
- Nitrites are dangerous, Nitrates is not dangerous.
- Higher nitrates - adversely the health of infants - mathemoglobinemia blue baby disease
- vomit; their skin colour may become dark and may die in extreme case

Mathemoglobinemia



Chlorides Contents

- kitchen wastes, human feces and urinary discharges
- normal - 120 mg/lit, P Limit - 250 mg /lit.
- Source : ice cream plants, meat salting etc..
- Indicates : Industrial wastes or infiltration of seawater, thereby indicating strength of sewage
- standard silver nitrate solution

Fats, Oils and Greases

- from the discharge of animals and vegetable matter, or from the garages, kitchens of hotels and restaurants, etc..
- form scum on the top of the sedimentation tanks, clogs the voids of the filter media and affects the diffusion of oxygen.

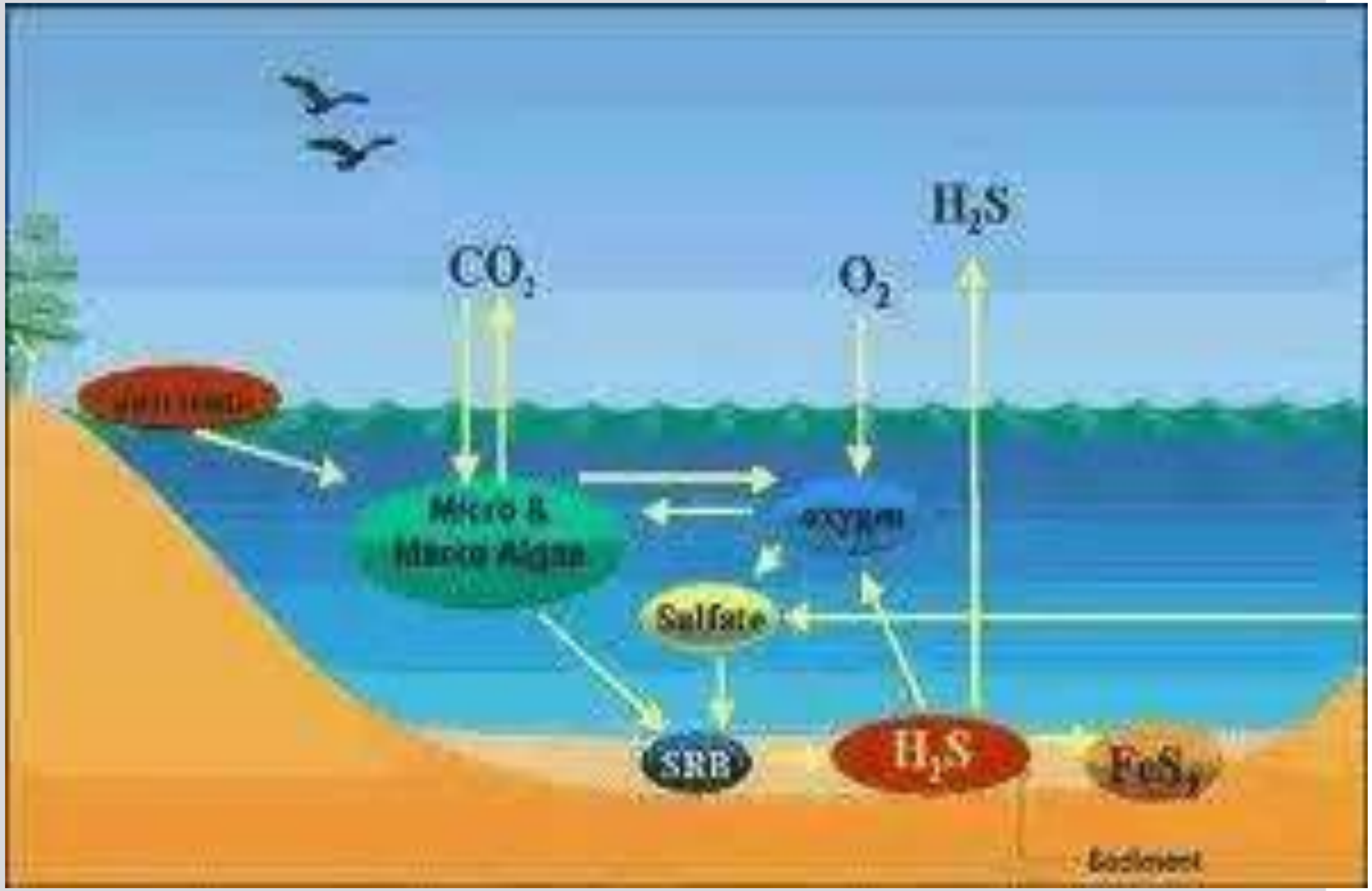
Toxic

- Copper, lead, silver, chromium, arsenic, phenols, boron, cyanides, etc.. are some of the
- Affect microorganisms resulting in malfunctioning from industrial waste.

Sulphides, Sulphates and Hydrogen Gas

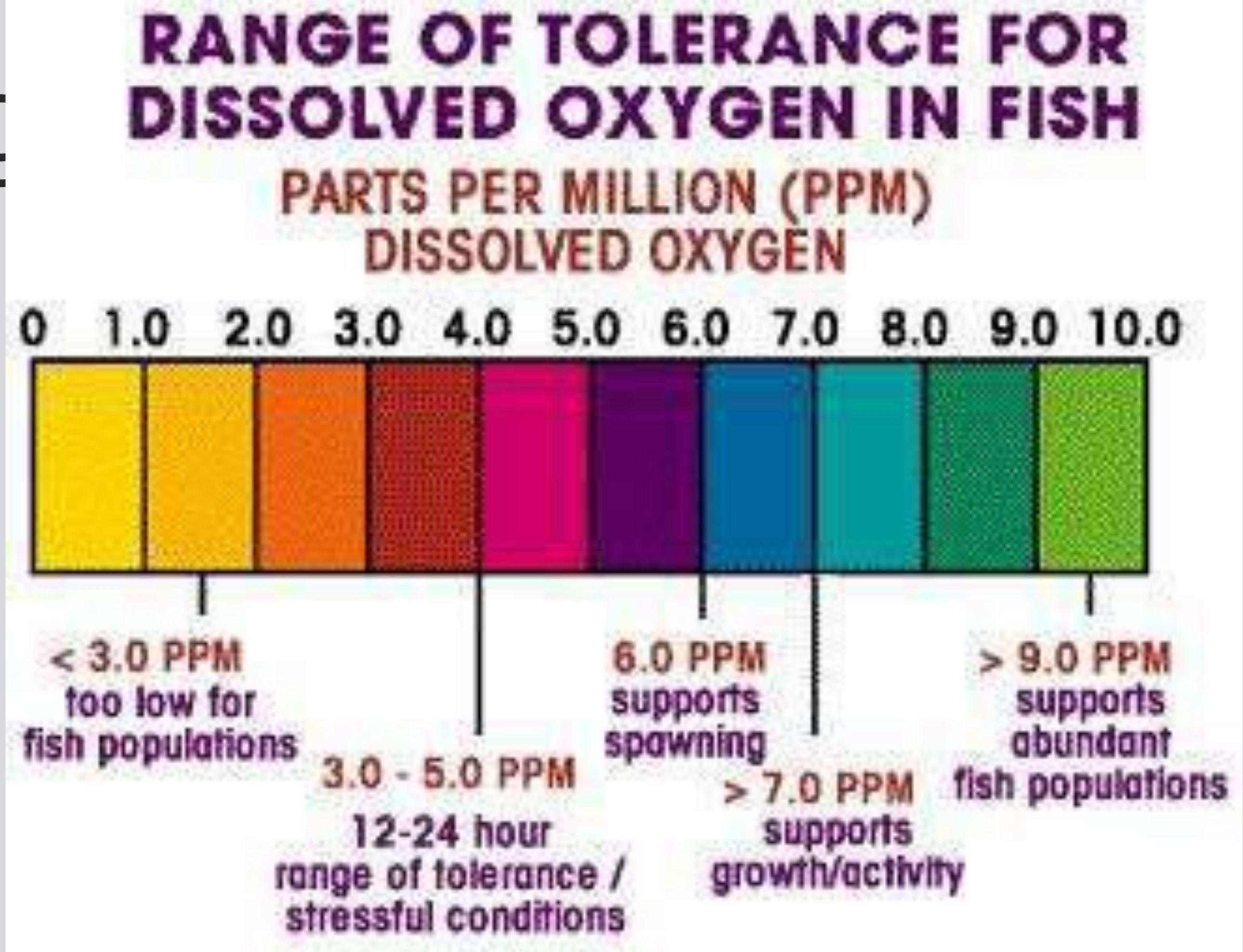
- decomposition of various sulphur containing substances in Sewage
- evolution of hydrogen sulphide gas, bad odours, - corrosion sewer pipes.
- In aerobic digestion of sewage,
  - Sulphur --→ sulphides, -→ sulphates ions In an-aerobic digestion
- In an-aerobic digestion
- sulphates --→ sulphides, -H_2S gas along with methane and carbon dioxide

Sulphides, Sulphates and Hydrogen Gas



Dissolved Oxyg

- Dissolved oxygen is the amount of oxygen in the dissolved state in the wastewater.
- at least 4 mg/l of DO is present in it
- aquatic animals like fish etc. are likely to be killed near the vicinity of disposal

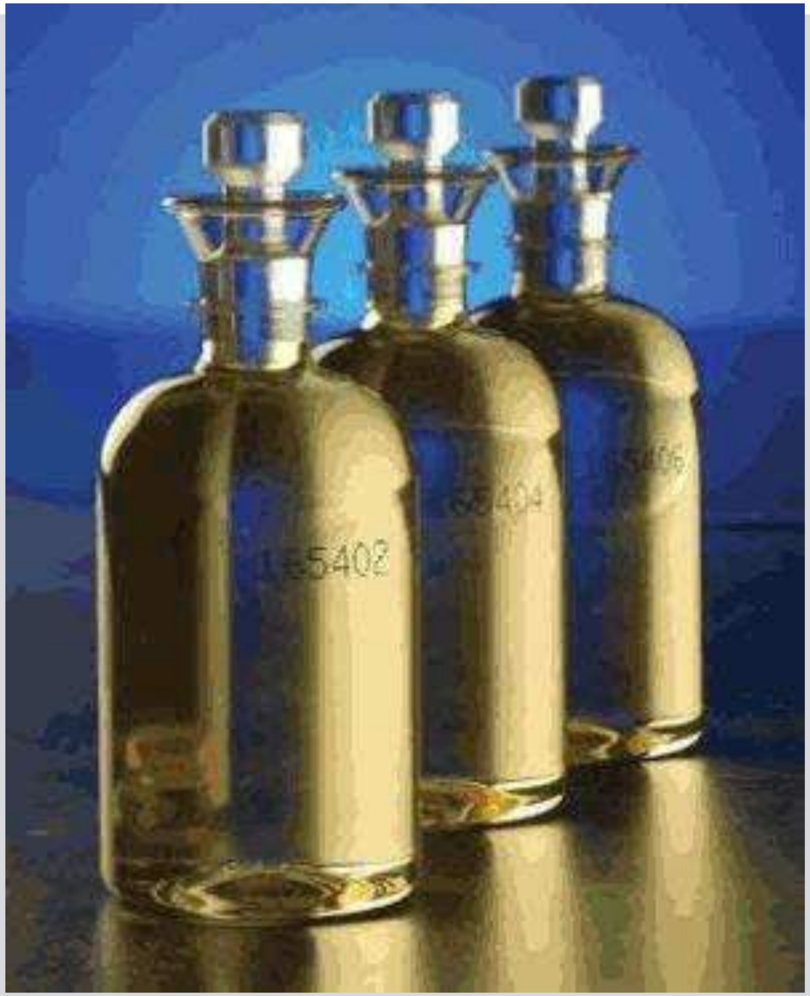


Biochemical Oxygen Demand

There are two types of organic matter

- • (i) Biodegradable or biologically active
- • (ii) Non biodegradable or biologically inactive
- • Organic matter is often assessed in terms of oxygen required to complete oxidize the organic matter to CO_2, H_2O, and other end products of Oxidation.
- • Biochemical Oxygen Demand (BOD) is defined as the amount of oxygen required by the microorganisms (mostly bacteria) to carry out decomposition of biodegradable organic matter under aerobic conditions.
- Carbonaceous matter - biodegradable carbonaceous material – Heterotrphic bacteria
- Nitrogenous matter - nitrogenous matter -autotrophic bacteria

- 5 days period is generally chosen for the standard BOD test, during which oxidation is about 60 to 70 % complete, while within 20 days period oxidation is about 95 % to 99 % complete.
- A constant temperature of 20^0 C is maintained during incubation. The BOD value of 5 Day incubation period is commonly written as BOD 5 or simply as BOD.
- Another reason to avoid interference of nitrification bacteria (starts after 6th or 7th day)

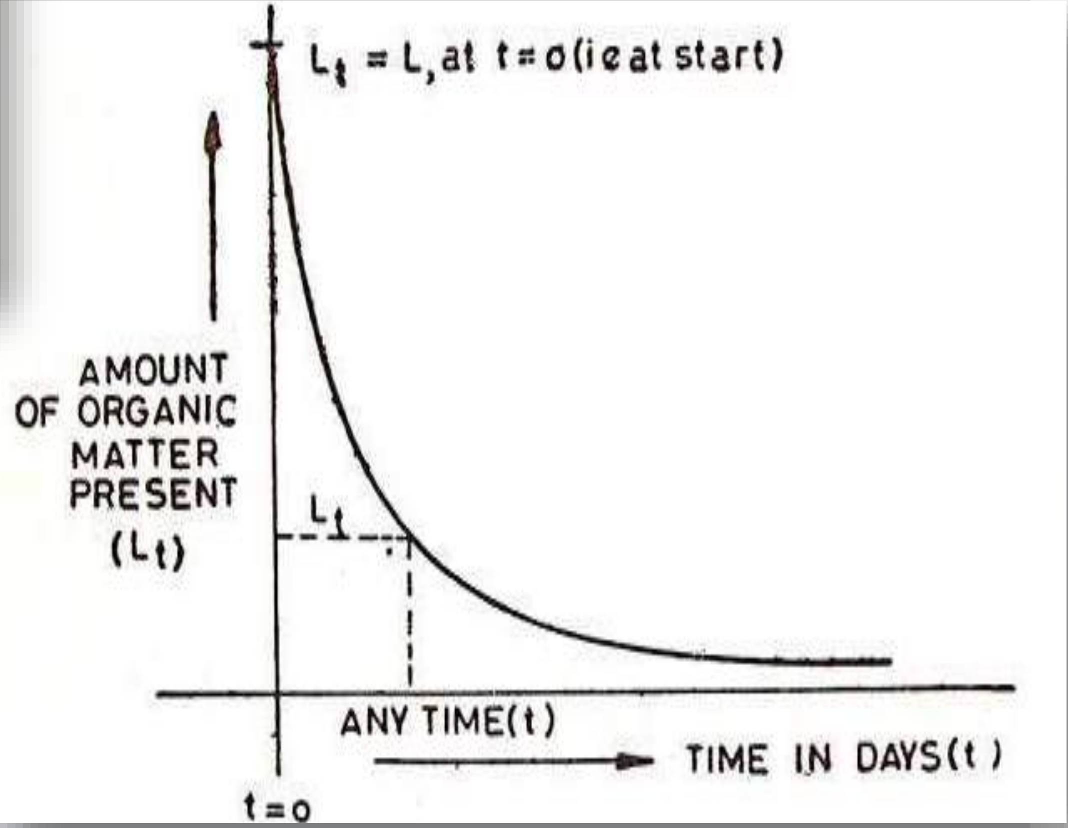


$$\text{BOD} = \text{D.O. consumed in the test by the diluted sample} \times \frac{\text{Volume of the diluted sample}}{\text{Volume of undiluted sewage sample}}$$

Thus, at a certain temperature, the rate of deoxygenation is assumed to be directly proportional to the amount of organic matter present in sewage at that time ; *i.e.*

$$\frac{dL_t}{dt} = -KL_t^* \qquad \text{...(7.12)}$$

where L_t = oxygen equivalent of carbonaceous oxidisable organic matter present in sewage after t days from the start of oxidation, in mg/l.



$$\int \frac{dL_t}{L_t} = \int -K \,.\, dt$$

or $$\log_e L_t = -K \,.\, t + C$$

where C is a constant of inte
can be evaluated
boundary conditic
start *i.e.*

when t = zero (0),

i.e. at start $L_t = L$ (say).*

Substituting in Eq. (7.13), we have

$$\therefore \quad \log_e L = K(0) + C$$

or $$C = \log_e L$$

Substituting this value of C in Eq. (7.13), we get

$$\log_e L_t = -K \,.\, t + \log_e L$$

or $$\log_e L_t - \log_e L = -K \,.\, t$$

or $$\log_e \frac{L_t}{L} = -K \,.\, t$$

We have

$$\log_{10} \frac{L_t}{L} = -K_D \cdot t$$

or
$$\frac{L_t}{L} = (10)^{-K_D \cdot t}$$

If Y_t represents the total amount of organic matter oxidised in t days (*i.e.* the BOD of t days), then we have

$$Y_t = L - L_t$$

Taking L out of bracket on R.H.S.

we have
$$Y_t = L\left[1 - \frac{L_t}{L}\right]$$

or
$$\frac{Y_t}{L} = 1 - \frac{L_t}{L}$$

or
$$\frac{L_t}{L} = 1 - \frac{Y_t}{L}$$

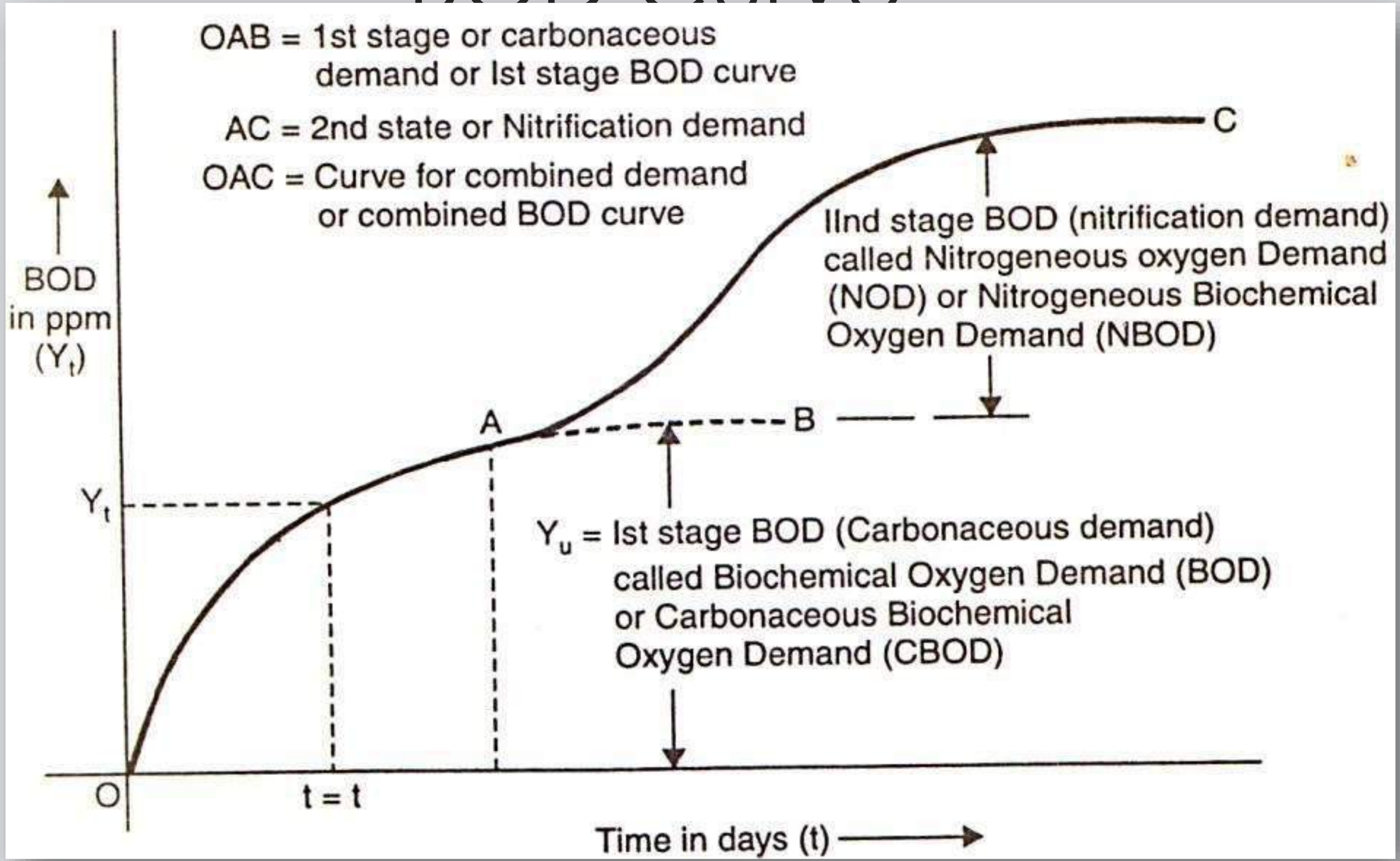
Substituting this value of $\frac{L_t}{L}$ in equation (7.15), we get

$$1 - \frac{Y_t}{L} = (10)^{-K_D \cdot t}$$

or
$$\frac{Y_t}{L} = 1 - (10)^{-K_D \cdot t}$$

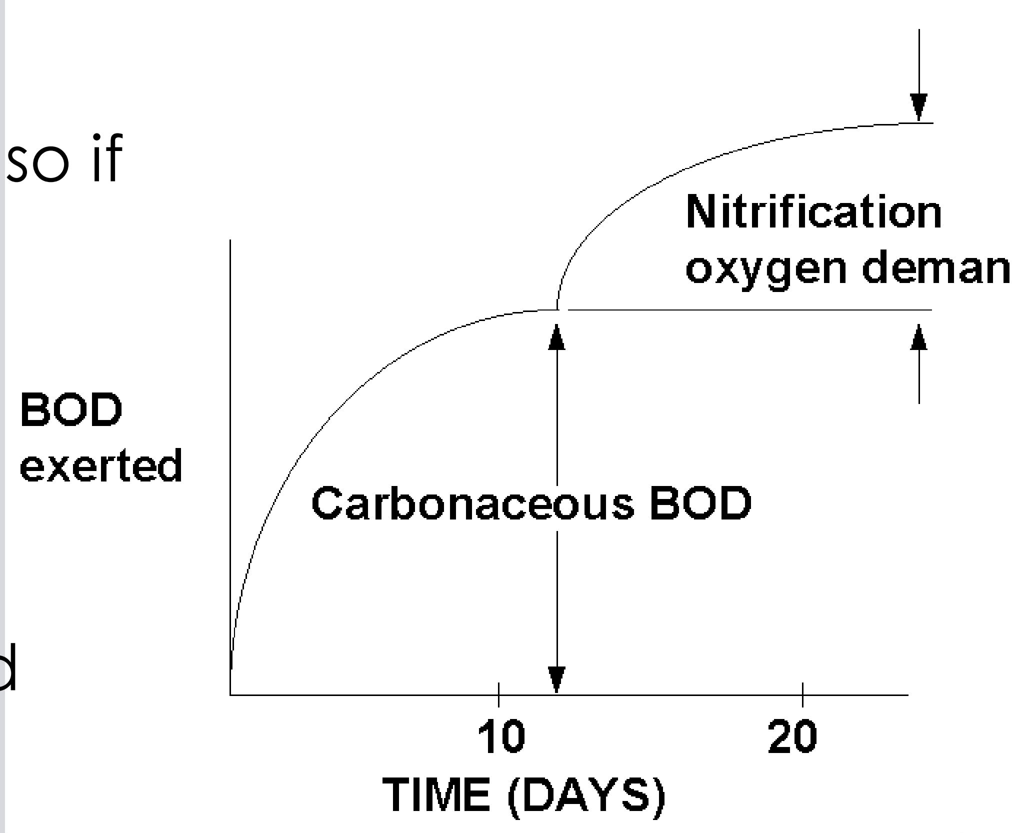
or
$$\boxed{Y_t = L\left[1 - (10)^{-K_D \cdot t}\right]} \qquad ...(7.16)$$

BOD Curve


OAB = 1st stage or carbonaceous demand or Ist stage BOD curve
AC = 2nd state or Nitrification demand
OAC = Curve for combined demand or combined BOD curve
BOD in ppm (Y_t)
C
IInd stage BOD (nitrification demand) called Nitrogeneous oxygen Demand (NOD) or Nitrogeneous Biochemical Oxygen Demand (NBOD)
A
B
Y_t
Y_u = Ist stage BOD (Carbonaceous demand) called Biochemical Oxygen Demand (BOD) or Carbonaceous Biochemical Oxygen Demand (CBOD)
O
t = t
Time in days (t)

Limitation of BOD Test

- It measures only the biodegradable organic matter.
- Time duration of the test is very long i.e. 5 days, so if quick results are needed it is not useful.
- Pretreatment is needed if the sample contains toxic waste.
- Nitrifying bacteria can cause interferences and could give higher results. To avoid them proper care must be taken.
- It is essential, to have high concentration of active



Chemical Oxygen Demand

- COD can be defined as amount of oxygen required to chemically oxidize organic matter using a strong oxidizing agent like potassium dichromate under acidic condition.
- COD test, takes 3 hours in comparison to 5 days for BOD test
- In COD test, a strong chemical oxidizing agent like potassium dichromate is used in acidic medium to oxidize the organic matter present in the waste
- Almost all type of organic matter with a few exceptions can be oxidized by the action of strong oxidizing agents under acidic conditions.

- Total Organic Carbon
- Theoretical Oxygen Demand

Biological Characteristics

- microorganisms like bacteria, algae, fungi, protozoa, etc. bacteria being the most predominant. Most of the bacteria found in the sewage are harmless non-pathogenic bacteria.
- at the time of outbreak of epidemics, certain tests may be done to find the type of pathogens

Population Equivalent

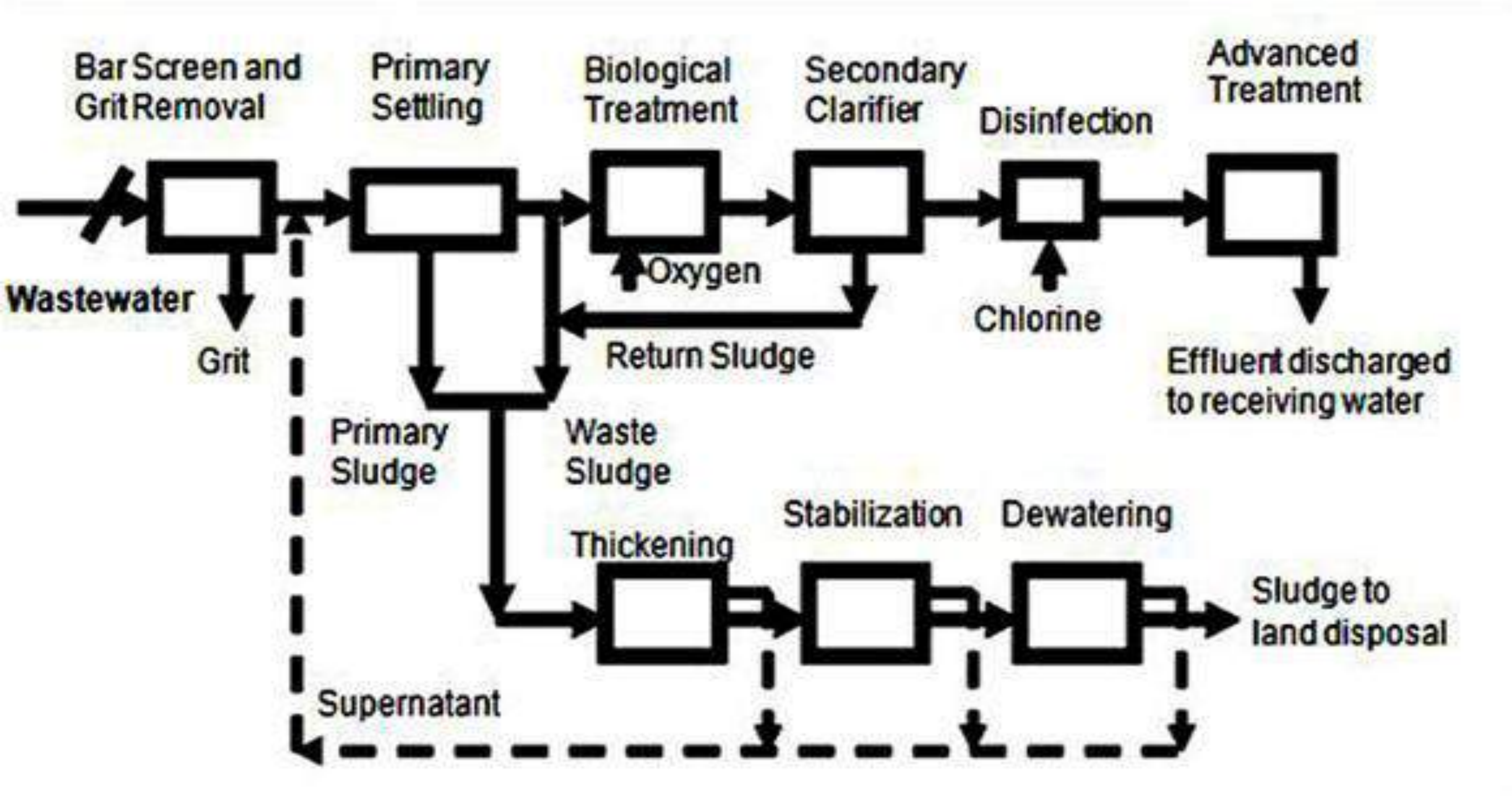
- The average standard BOD_5 of domestic sewage is worked out to be about 0.08 kg/day/ person. Hence, if the BOD_5 of the sewage coming from an industries is worked out to be 350 kg/ day, then

$$\text{Population Equivalent} = \frac{\text{Total BOD 5 of the industry in kg/day}}{0.08 \text{ kg/day/person}}$$

$$= \frac{350}{0.08}$$

$$= 4375$$

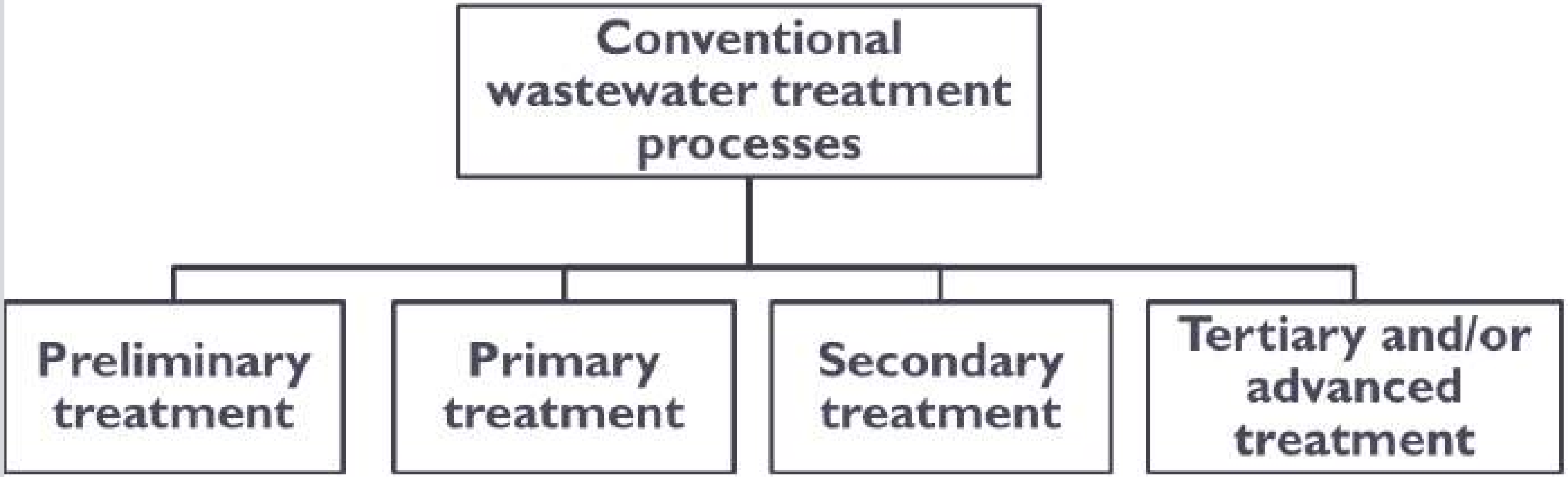
Flow diagram of Municipal



Conventional wastewater treatment processes

- consists of a combination of physical, chemical and biological processes and operations to remove solids, organic matter and sometimes, nutrients from wastewater.
- General terms used to describe different degrees of treatment, in order of increasing treatment level, are preliminary, primary, secondary, and tertiary and/or advanced wastewater treatment.
- In some countries, disinfection to remove pathogens sometimes follows the last treatment step

Conventional wastewater treatment process



Conventional wastewater treatment process

Preliminary treatment

- The objective is the removal of coarse solids and other large materials often found in raw wastewater, operations typically include coarse screening, grit removal and, in some cases, comminution of large objects

Primary treatment

- The objective is the removal of settle-able organic and inorganic solids by sedimentation, and the removal of materials that will float (scum) by skimming

Secondary treatment

- The objective of secondary treatment is the further treatment of the effluent from primary treatment to remove the residual organics and suspended solids
  - Aerobic biological treatment is performed in the presence of oxygen by aerobic microorganisms (principally bacteria) that metabolize the organic matter in the wastewater, thereby producing more microorganisms and inorganic end-products (principally CO2, NH3, and H2O)
  - Several aerobic biological processes are used for secondary treatment differing primarily in the manner in which oxygen is supplied to the microorganisms and in the rate at which organisms metabolize the organic matter

screens

- Bigger pieces of wood, plastic, metal, rubber, textile - **bar screens**
- bars - spaced about 1/2-3/4 inches apart.

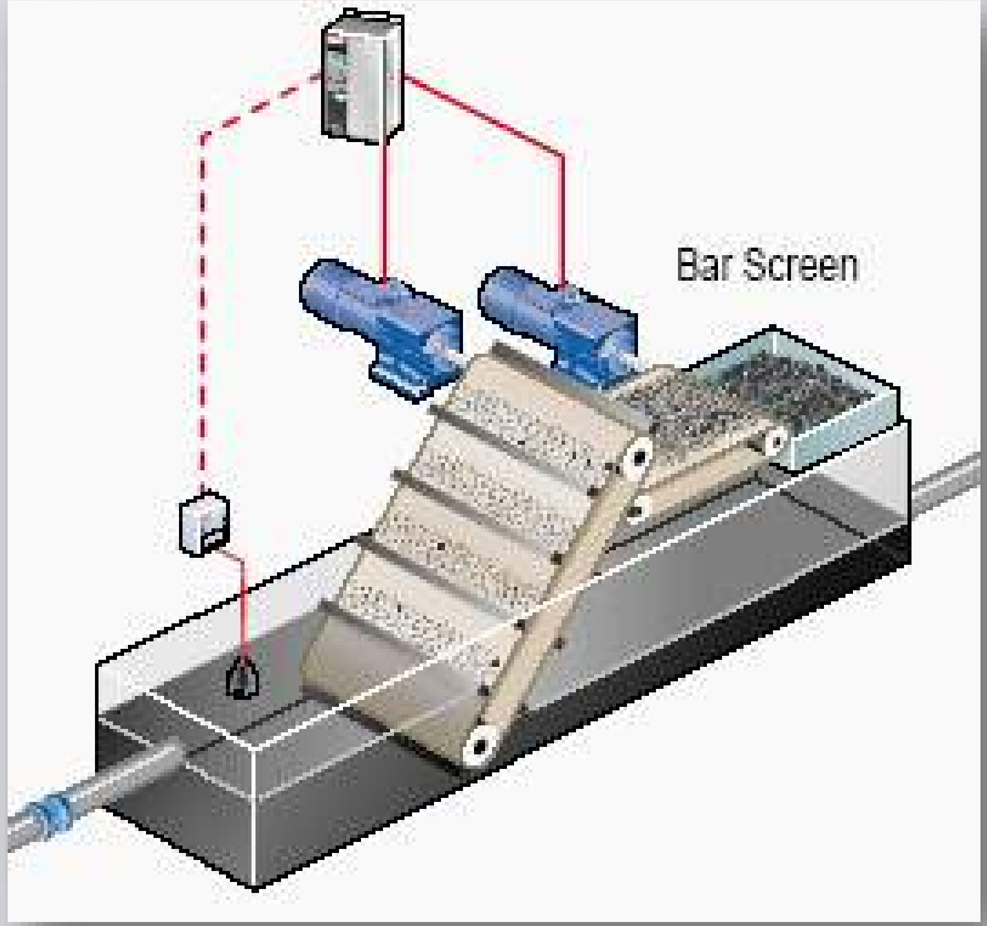
Manually cleaned bar screens

- operator must rake the collected debris from a manually cleaned bar screen.
- 45-degree angle.

Automatically cleaned bar screens

- set of rakes - chain-driven. units - operate periodically
- 60° and 90° an automatically cleaned

screens



Automatically cleaned bar screens

Manually cleaned bar screens

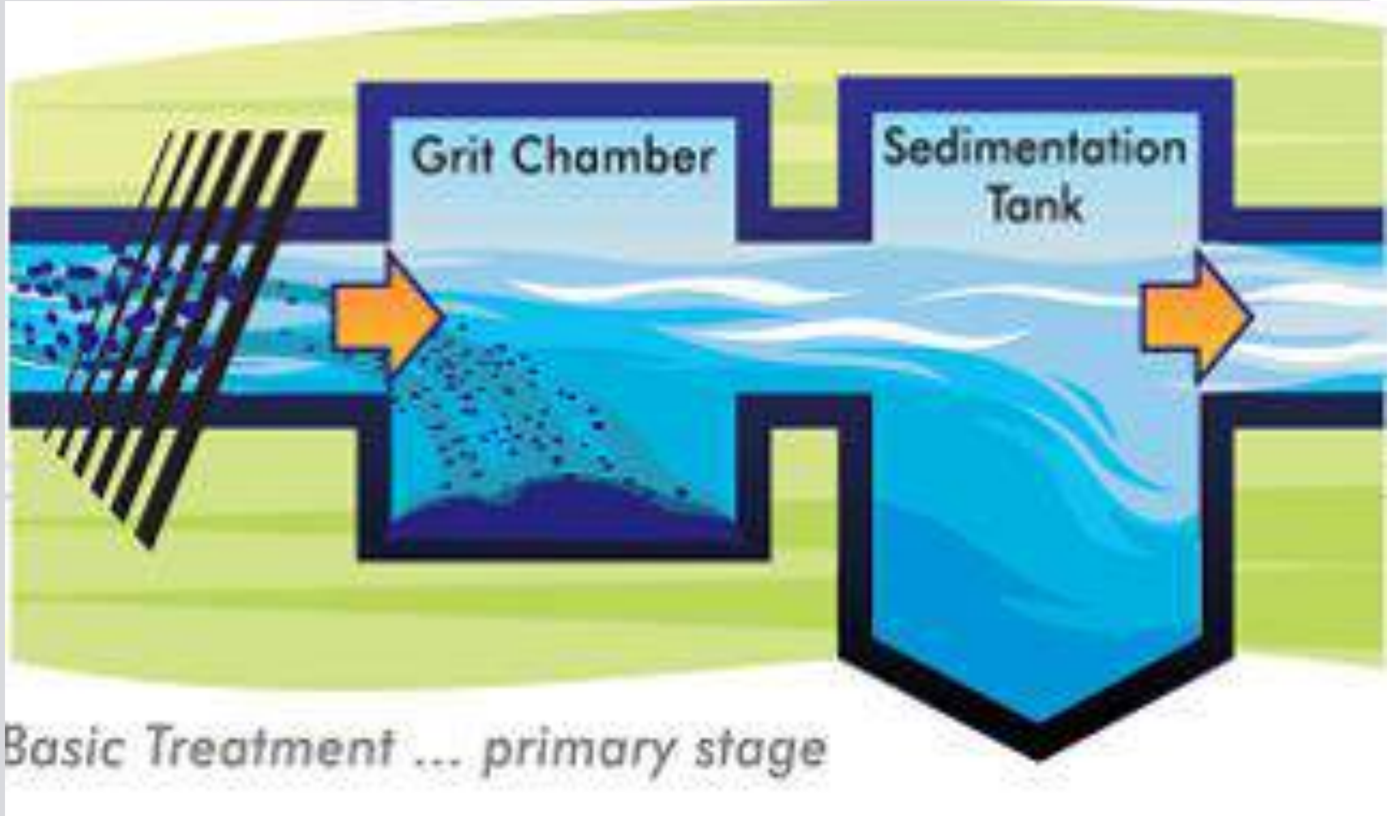
- corrosion problems with steel screens
- Hydrogen sulfide - attack the metal bars.
- Repair and replacement of the bars
- weekly inspections - rake teeth and the chain drive.

Two types of bar screens:

- ***Coarse sieves*** **-**greater than 6 mm
- ***fine sieve*** **-** smaller than 6 mm

grit chambers

Grit chambers are nothing but like sedimentation tanks, designed to separate the intended heavier inorganic materials (specific gravity about 2.65) and to pass forward the lighter organic materials



Detritus tank

- similar to a grit chamber. The difference being only in the velocity of flow and the detention period
- appreciable amount of organic matter settles down along matter by blowing the compressed air through the detritus tanks
- 2.5 to 3.5 m length and detention period 3 to 4 minutes
- V = 20cm/sec to 40cm/sec.

Oil and Grease removal

- Grease in sewage include fats, waxes, free fatty solids, calcium and magnesium soaps, mineral oils, etc.,
- restaurants, kitchens, garages, soap and candle factories, oil refineries, slaughter houses etc.,.
- foul odour may be developed at the surface, scum retards reoxygenation and thus causes anaerobic condition
- do not digest easily, in sludge digestion tanks
- promote clogging of the trickling filters.
- affect the biological activities of the organisms
- by floatation or settling as scum or sludge.

skimming tanks

- narrow rectangular tanks having at least two longitudinal baffle walls, interconnected
- Air diffusers are provided at the bottom 300 to 6000 m^3 per million litres of sewage agitates the sewage
- Air tends to change the oil and grease to a soapy mixture
- mixture is carried to the surface by the air bubbles, some of which are entrained in it and may be skimmed off.

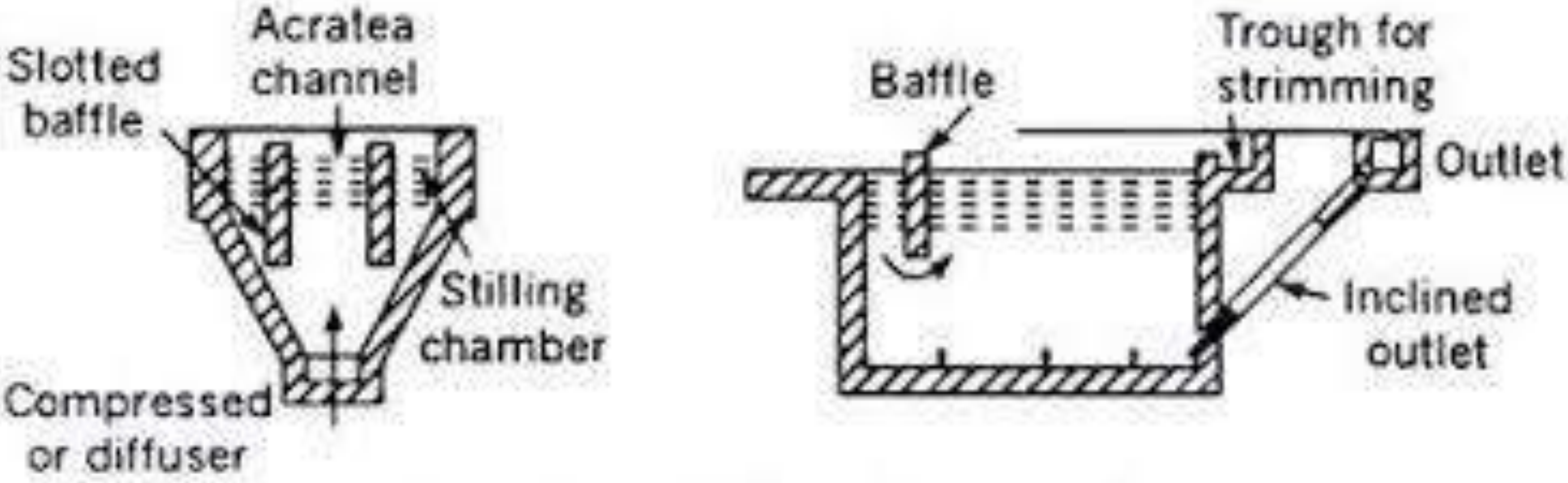


Fig. 13.5. Skimming tank.

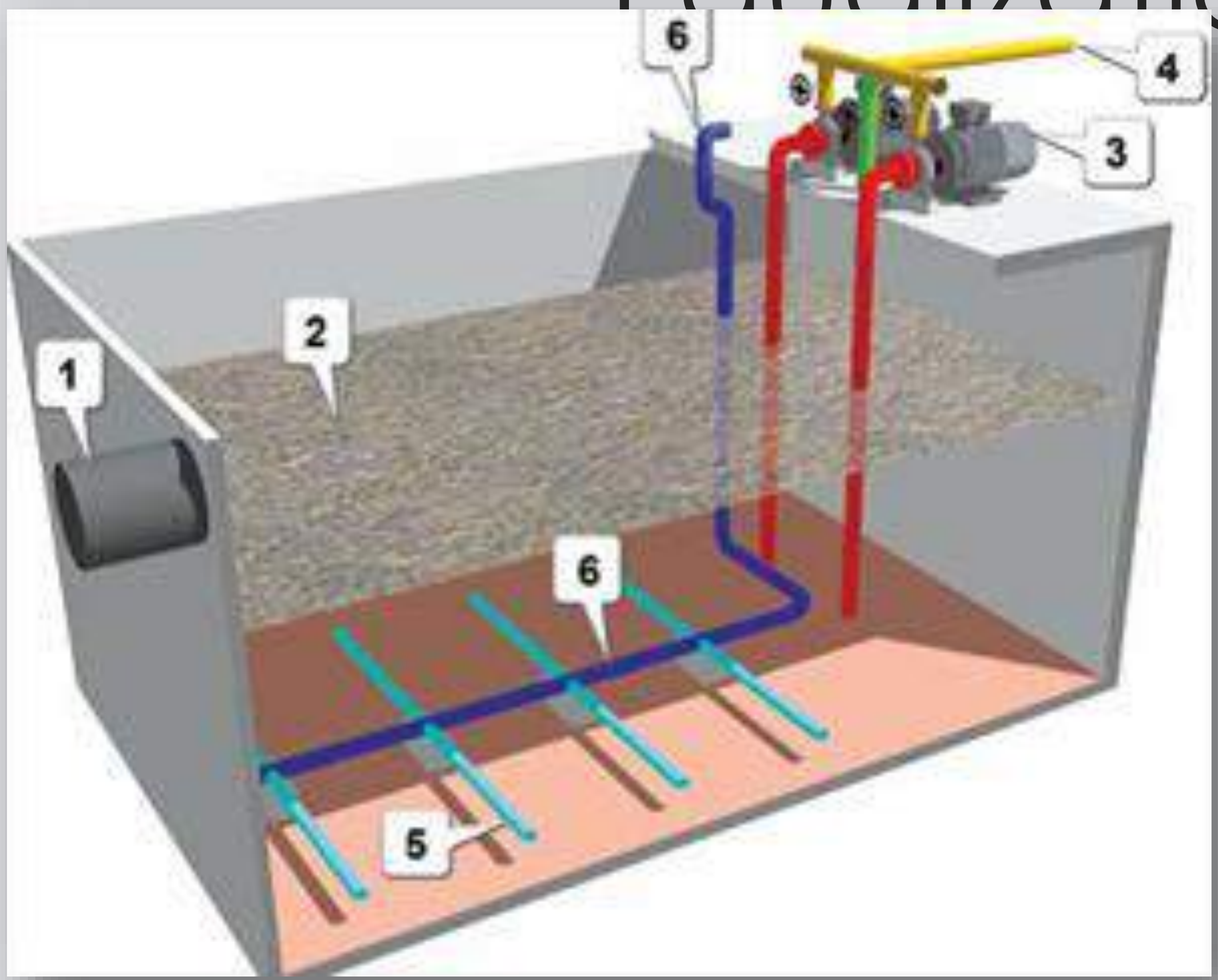
Equalization

- Basins are designed to provide consistent influent flow to downstream processes by retaining high flow fluctuations.
- Due to the additional retention time, aeration and mixing is required in to prevent the raw wastewater from becoming septic and to maintain solids in suspension.

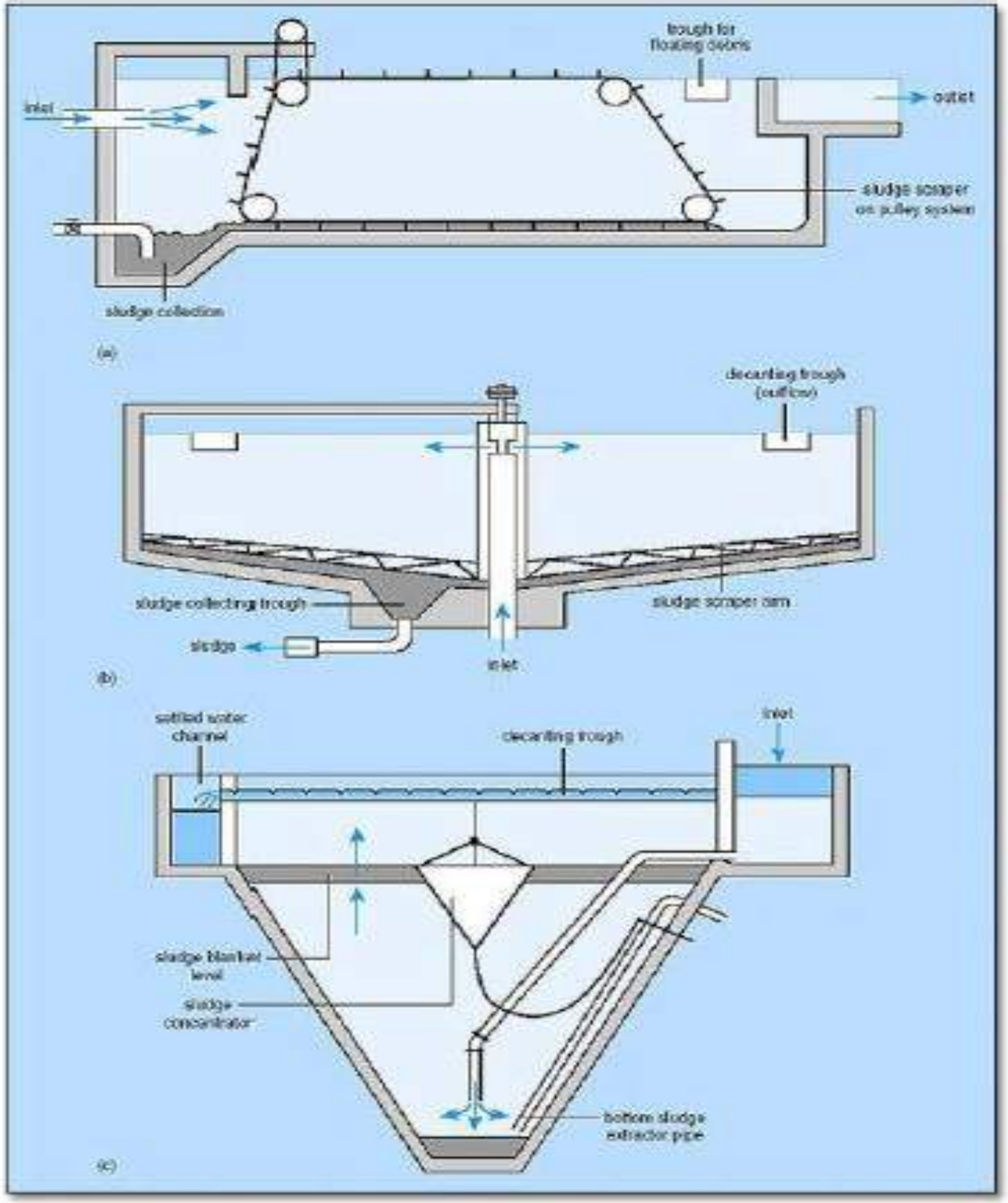
Objective –

- Decrease fluctuations in flow rate, to provide more consistent treatment
- Accomplished by storing excess wastewater during high flow periods
- Excess wastewater is released during low flow periods

Equalization Tank



Sedimentation tank and clarifiers



<u>**Typical sedimentation tanks:**</u>

(a)rectangular horizontal flow tank;

(b)circular, radial-flow tank;

(c) hopper-bottomed, upward flow tank

Secondary Treatment

Attached Growth Process

- In this process micro organisms are responsible for conversion of organic matter present in waste water to gases and cell tissues are attached to some inert medium such as rock, slag, ceramic or plastic material.

This process include:

- 1.Intermittent sand filters
- 2.Trickling filters
- 3.Rotating biological contactors
- 4.Packed bed reactors
- 5.Anaerobic lagoons.
- 6.Fixed film denitrification.

Suspended Growth Process

- In this process micro organisms are responsible for conversion of organic matter present in waste water to gases and cell tissues are maintained in suspension within the liquid in the reactor by employing either natural or mechanical mixing.

This process include:

- Activated sludge process.
- Aerated lagoons.
- Sludge digestion system.
- Suspended growth nitrification & denitrification

Combined Process

- This process includes both attached growth and suspended growth process.

This process include :

- 1.Trickling filter, activated sludge.
- 2.Activated sludge, trickling filter.
- 3.Faculative lagoons

Trickling Filters

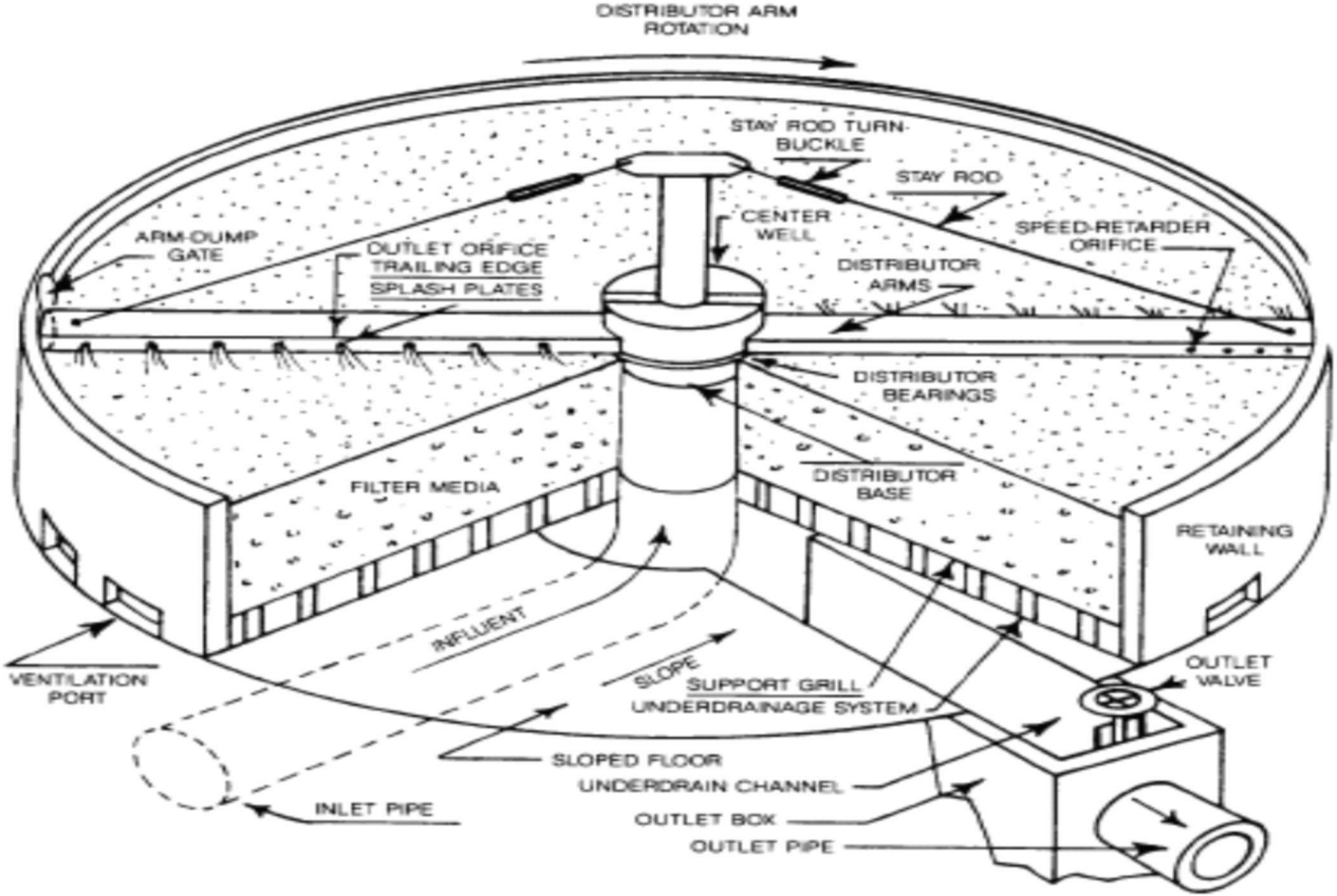
- It consists of a basin or tower filled with support media such as stones, plastic shapes, or wooden slats
- Wastewater is applied intermittently, or sometimes continuously, over the media
- Microorganisms become attached to the media and form a biological layer or fixed film
- Organic matter in the wastewater diffuses into the film, where it is metabolized

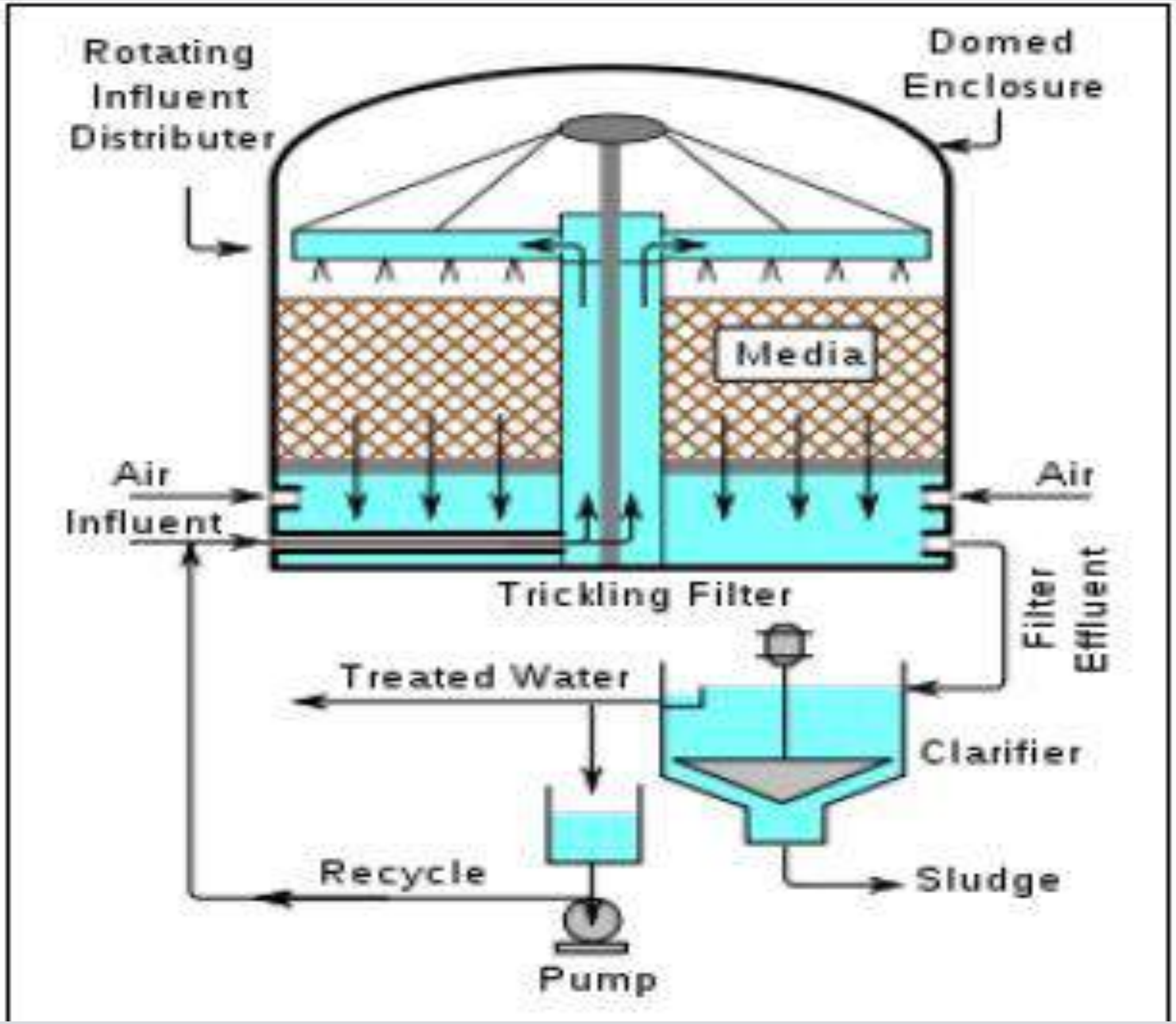
Trickling Filters

- Oxygen is normally supplied to the film by the natural flow of air either up or down through the media, depending on the relative temperatures of the wastewater and ambient air
- The thickness of the biofilm increases as new organisms grow, Periodically, portions of the film slough off the media
- The sloughed material is separated from the liquid in a secondary clarifier and discharged to sludge processing
- Clarified liquid from the secondary clarifier is the secondary effluent and a portion is often recycled to the biofilter to improve hydraulic distribution of the wastewater over the filter

Mechanism

- microorganisms - attached to an inert packing material rock, gravel, slag, sand, redwood, and a wide range of plastic and other synthetic materials.
- bed of highly permeable media on whose surface a mixed population of microorganisms is developed as a *slime layer*.
- Passage of wastewater - a gelatinous coating of bacteria, protozoa and other organisms on the media
- distribution system, media, under drains, effluent channel, secondary settling tank, and recirculation pumps and piping.


DISTRIBUTOR ARM ROTATION
STAY ROD TURN-BUCKLE
STAY ROD
CENTER WELL
SPEED-RETARDER ORIFICE
ARM-DUMP GATE
OUTLET ORIFICE
TRAILING EDGE
SPLASH PLATES
DISTRIBUTOR ARMS
DISTRIBUTOR BEARINGS
DISTRIBUTOR BASE
FILTER MEDIA
RETAINING WALL
INFLUENT
SLOPE
VENTILATION PORT
SUPPORT GRILL
UNDERDRAINAGE SYSTEM
OUTLET VALVE
SLOPED FLOOR
UNDERDRAIN CHANNEL
INLET PIPE
OUTLET BOX
OUTLET PIPE

Rotating Influent Distributer
Domed Enclosure
Media
Air
Influent
Air
Trickling Filter
Filter Effluent
Treated Water
Clarifier
Recycle
Sludge
Pump

Common Problems

Ponding

- cause odors and decrease filter efficiency.

Cause

- excessive organic loading without high R,

❖Use of small media.

❖Clogging of UGD,

❖Non-uniform media size / breaking up of media,

❖Trash or debris in filter voids

Remedies

- Spraying the surface with high pressure water hose.
- Stirring or agitating ponding area with stick, rake, etc.
- Dosing the filter with chlorine
- Flooding filter and keeping the media submerged for approximately 24 hours

- **Filter Flies**

Cause

- tiny, gnat-size flies are called psychoda.

Remedies

- Increasing recirculation
- Flushing the side walls of the filter by opening the flap valve
- Flooding the filter intermittently
- addition of Cl, - toxic to the flies and larvae.

- **Odors.**

Cause

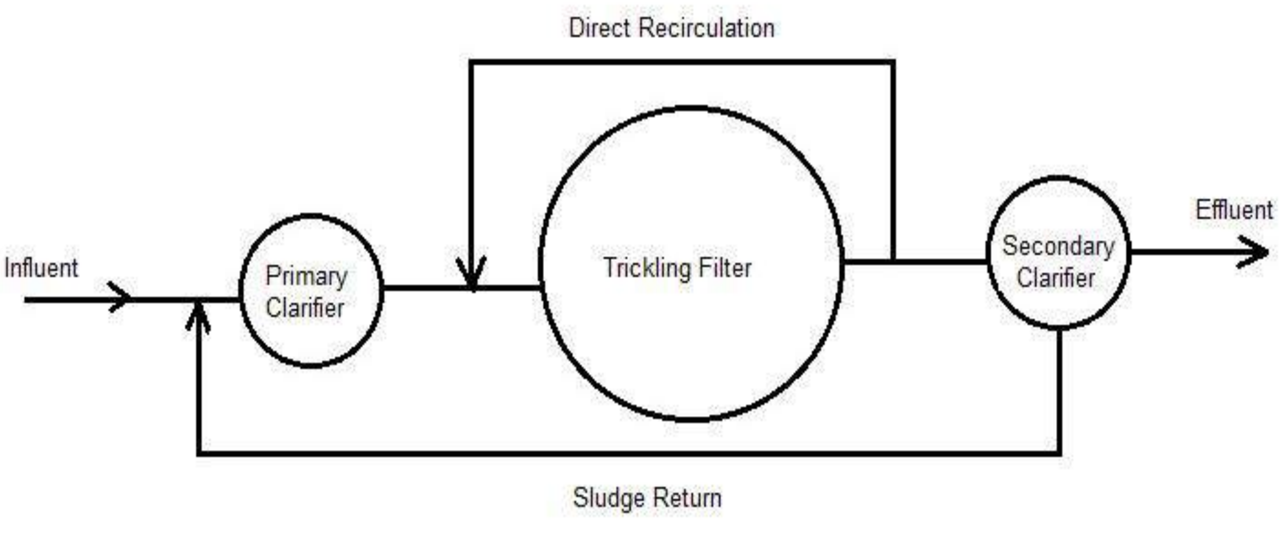
- anaerobic conditions

Remedies

- maintain aerobic condition
- Check the ventilation
- Check the UGD clogging and stoppages
- Increase R;
- odor-masking agents
- Pre-chlorination at primary tank

Conventional filters are modified high rate filters

- Better quality filtering media is used, like larger size stone media or plastic media.
- Depth of media is limited to 1.5 to 2m
- Size of UGD is increased and their slope made steeper so that the filter effluent can be collected and conveyed quickly.
- Speed of rotating arm is increased.
- Size of SST increased.



Single Stage Trickling Filter - Typical Flow Diagram

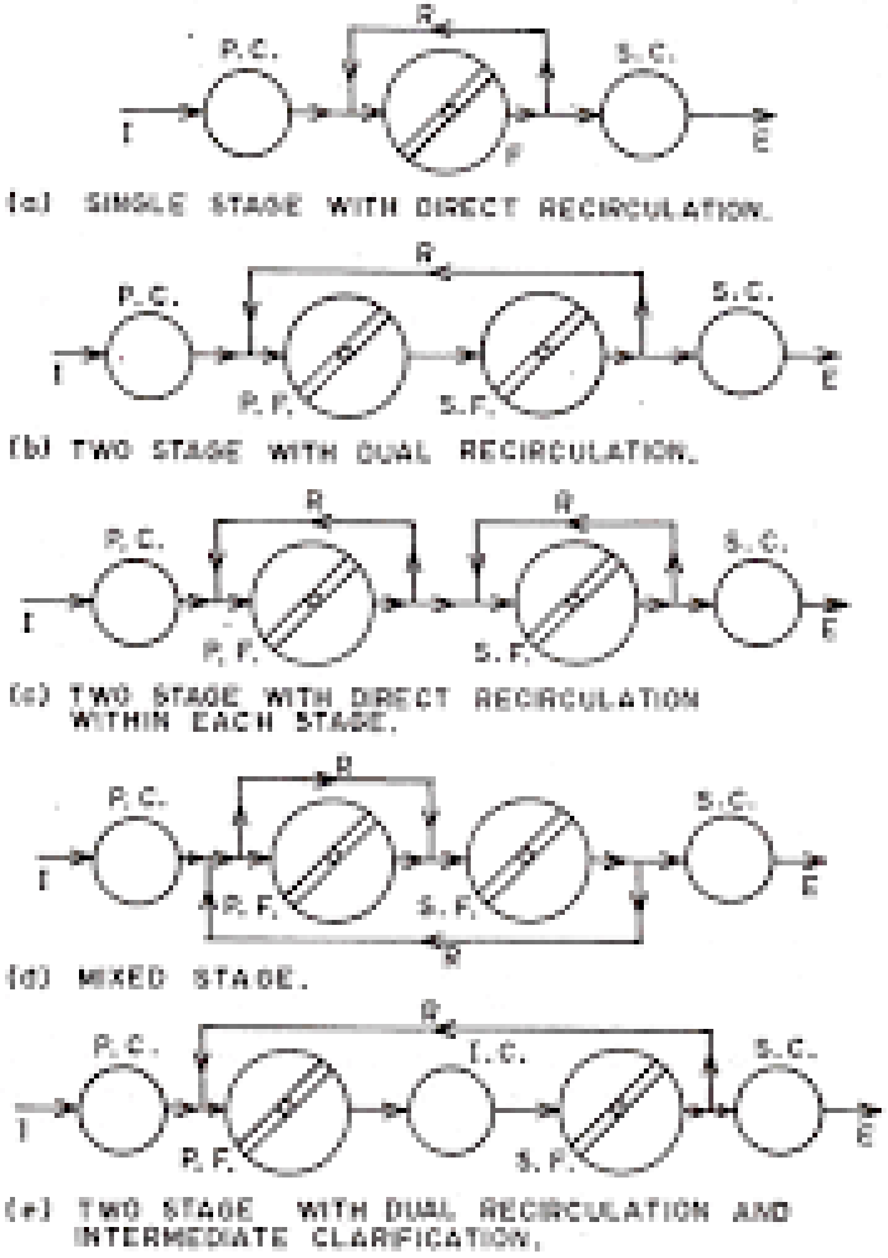


Fig. 14.[illegible] Flow diagrams of Aero-filter plants.

Comparision Between Conventional and High-Rate Trickling Filters

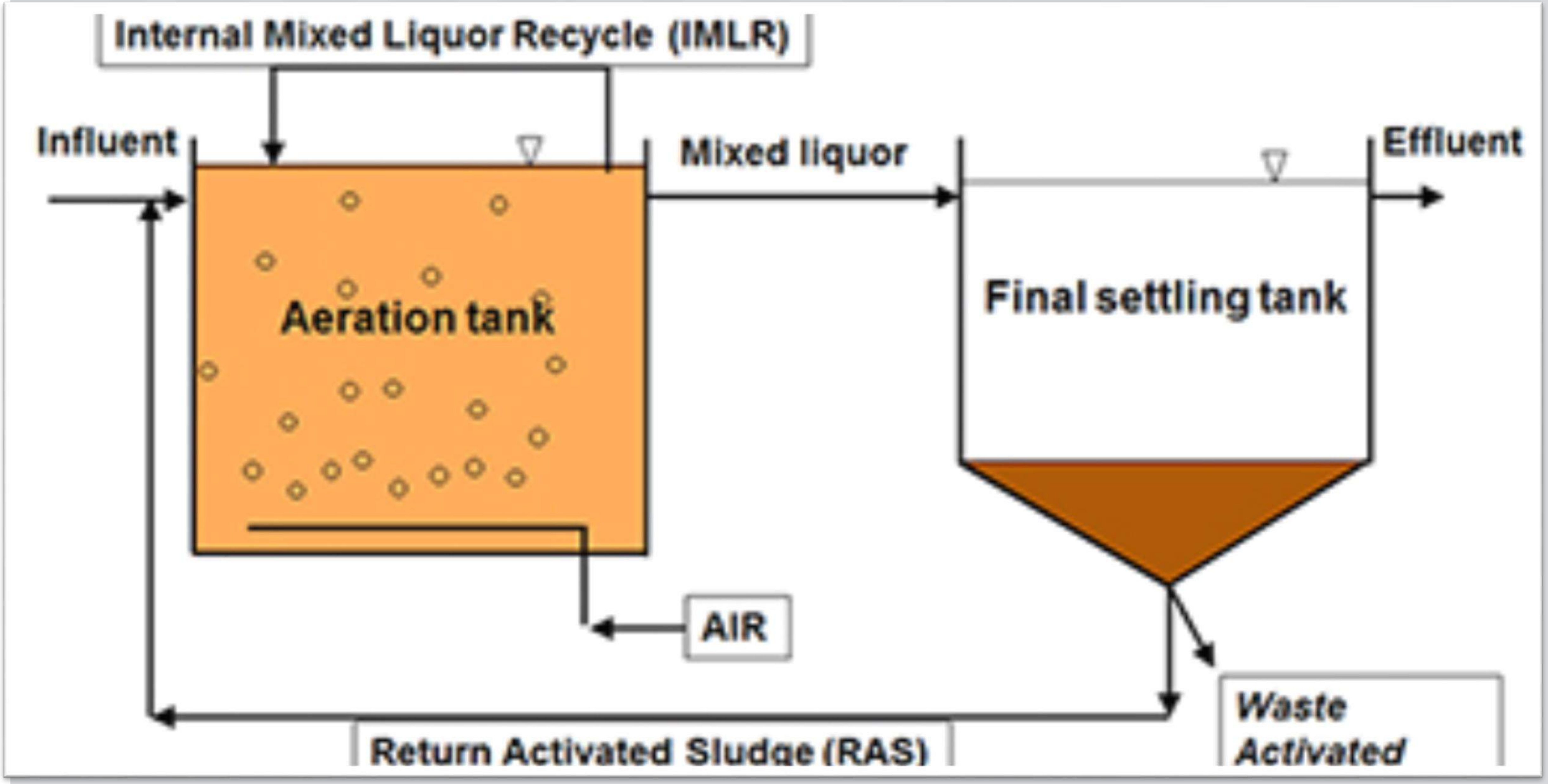
S.No.	*Characteristics*	*Conventional filter*	*High-rate filter*
1	Depth of filter media	1.8 to 3.0 m	0.9 to 2.5 m
2	Hydraulic loading ($m^3/d/m^2$)	1 to 4	10 to 40 (including recirculation)
3	Organic loading (kg $BOD_5/m^3/d$)	0.08 to 0.32	0.32 to 1.0
4	Recirculation system	Usually not provided	Always provided Recirculation ratio 0.5 to 3.0
5	Volume of filter bed	About 5 times that for a high-rate filter	About 1/5 of that for a conventional filter
6	Interval of dosing	It generally varies from 3 to 10 minutes. The sewage is usually not applied continuously but it is applied at intervals.	It is not more than 15 seconds and the sewage is thus applied continuously
7	Sloughing	Intermittent	Continuous
8	Land requirement	It requires more area of land.	It requires less area of land.
9	Cost of operation	It is more for equal performance.	It is less for equal performance.
10	Final effluent	The effluent is highly nitrified and stabilized with BOD in effluent $\leq$ 20 mg/l or so.	The effluent is nitrified up to nitrite stage only and is thus less stable, and hence it is of slightly inferior quality. BOD in effluent $\geq$ 30 mg/l or so.
11	Secondary sludge	Highly oxidized, Black colour, having light fine particles.	Not fully oxidized, Brownish black colour, having fine particles.

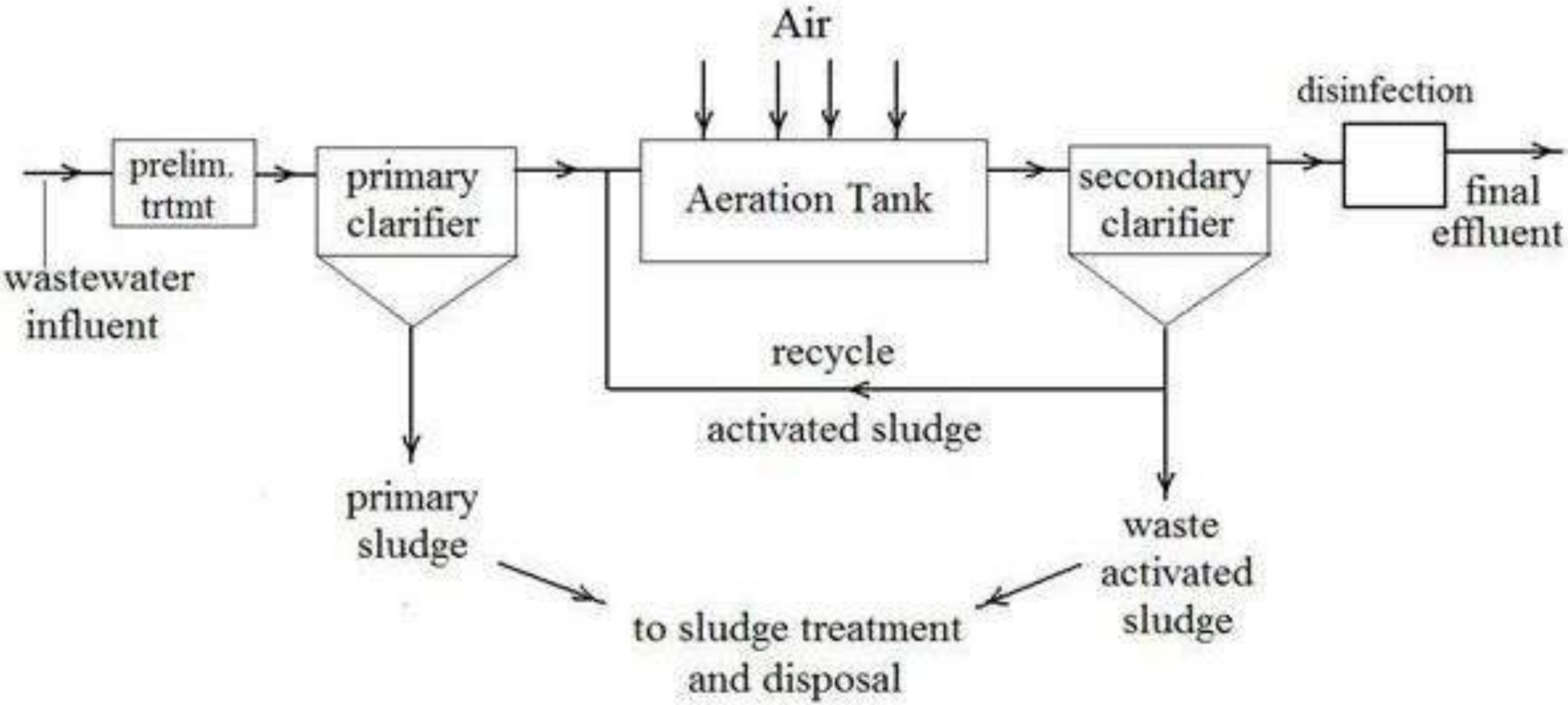
Activated Sludge Process

- In the activated sludge process, the dispersed-growth reactor is an aeration tank or basin containing a suspension of the wastewater and microorganisms, the mixed liquor
- The contents of the aeration tank are mixed vigorously by aeration devices which also supply oxygen to the biological suspension
- Aeration devices commonly used include submerged diffusers that release compressed air and mechanical surface aerators that introduce air by agitating the liquid surface
- HRT in the aeration tanks - 3 to 8 hours but can be higher with high BOD_5

Activated Sludge

- Following the aeration step, the microorganisms are separated from the liquid by sedimentation and the clarified liquid is secondary effluent
- A portion of the biological sludge is recycled to the aeration basin to maintain a high mixed-liquor suspended solids (MLSS) level
- The remainder is removed from the process and sent to sludge processing to maintain a relatively constant concentration of microorganisms in the system


Internal Mixed Liquor Recycle (IMLR)
Influent
Mixed liquor
Effluent
Aeration tank
Final settling tank
AIR
Waste
Activated
Return Activated Sludge (RAS)



Activated Sludge Wastewater Treatment Flow Diagram

Sequencing batch reactor (SBR)

- In a conventional activated sludge system, unit processes would be accomplished by using separate tanks.
- Sequencing batch reactor is a modification of activated sludge process which has been successfully used to treat municipal and industrial wastewater.
- The difference between the two technologies is that the SBR performs equalization, biological treatment, and secondary clarification in a single tank using a timed control sequence.

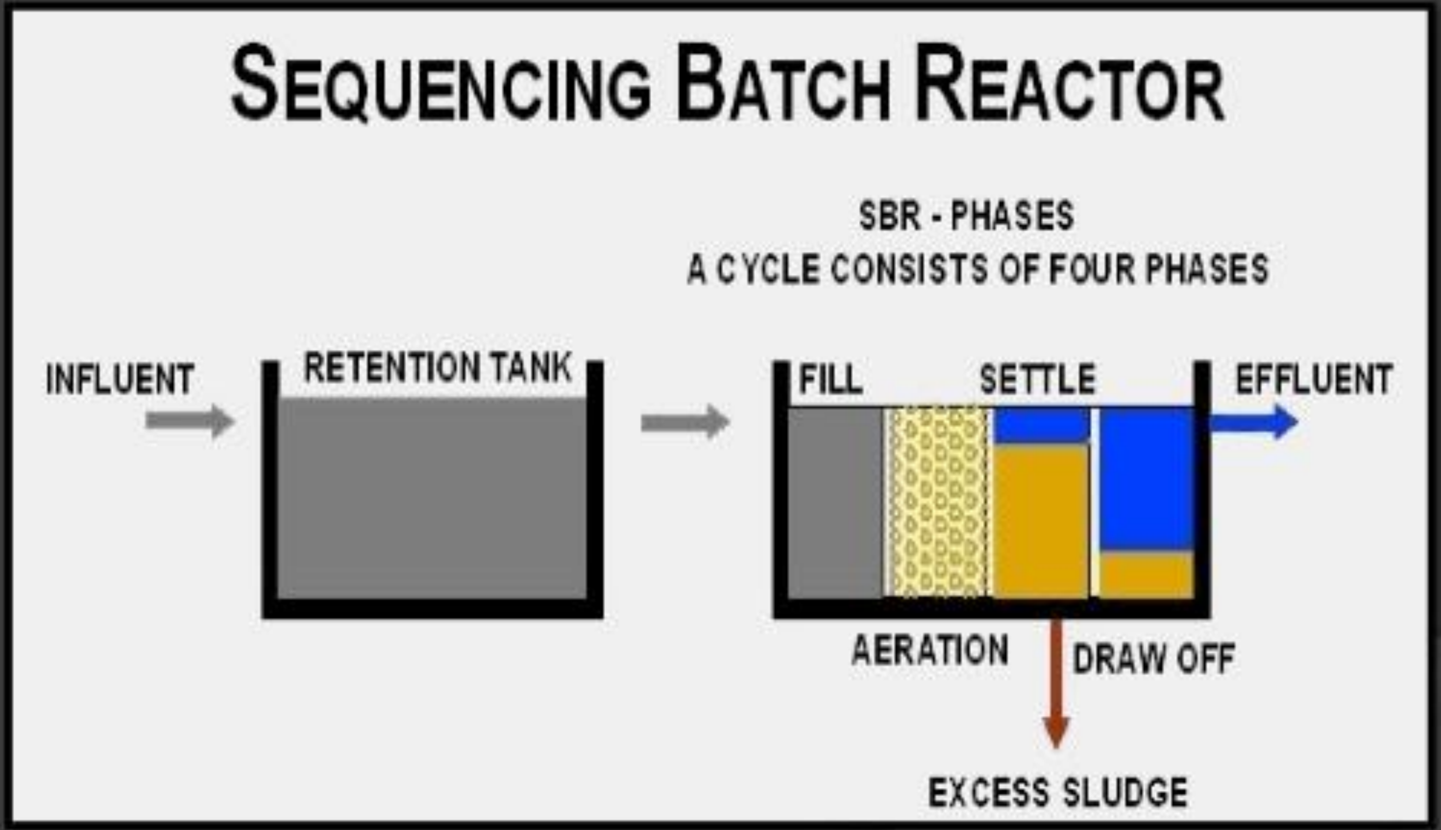
Reasons for providing SBR

- where there is a limited amount of space is available.
- Older treatment facilities can be retrofitted to an SBR (because the basins are already present).

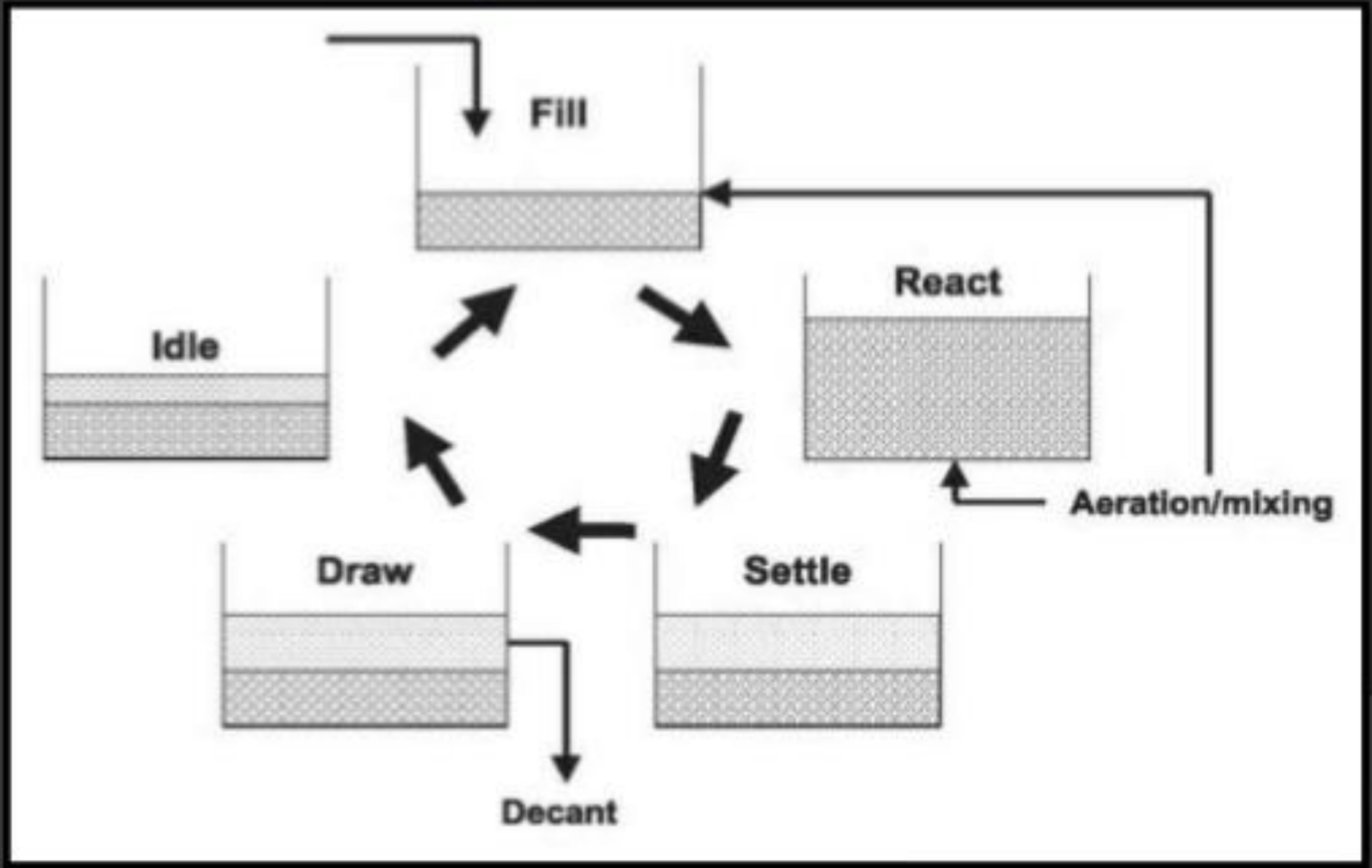
SBR Operating Principles

- SBR technology is a method of wastewater treatment in which all phases of the treatment process occur sequentially within the same tank.
- The sequencing batch reactor is a fill and draw activated sludge system. In this system, wastewater is added to a single "batch" reactor, treated to remove undesirable components, and then discharged.

Diagram showing SBR operating principles



Various phases in a typical SBR process



Fill Phase

◦ During the fill phase, the basin receives influent wastewater. The influent brings food to the microbes in the activated sludge, creating an environment for biochemical reactions to take place.

Types of fill phase

- Static fill
- Mixed fill
- Aerated fill

React Phase

◦ During this phase, no wastewater enters the basin and the mechanical mixing and aeration units are on.

Settle Phase

- During this phase, activated sludge is allowed to settle under quiescent condition . The activated sludge tends to settle as a flocculent mass.

Decant Phase

- Clarified treated effluent (supernatant) is removed from the tank. No surface foam or scum is decanted.

Idle Phase

- This step occurs between the decant and the fill phases.
- The idle period is used when the system is waiting for enough effluent to process.

Advantages of SBR

- Equalization, primary clarification, biological treatment and secondary clarification can be achieved in a single reactor vessel.
- SBR requires small space.
- SBR has controllable react time and quiescent settling.
- Minimal footprint.
- High nutrient removal capabilities.
- The BOD removal efficiency is generally 85 to 90%
- Filamentous growth elimination

Limitations of SBR

- A higher level of sophistication is required especially for larger systems, of timing units and controls.
- Higher level of maintenance associated with more sophisticated controls, automated switches, and automated valves.
- Potential plugging of aeration devices during selected operating cycles, depending on the aeration system used by the manufacturer.

MBBR (Moving bed biofilm reactor)

- Small cylindrical shaped polyethylene carrier added in aerated or non aerated basin to support biofilm growth.
- Biomass grows primarily on protected surface on the inside of the carriers.
- Air agitation or mixers are used to continuously circulate carriers. Perforated plates at the outlet of the tanks keeps biofilm carrier inside the tank.
- MBBR can be a single reactor or configured as several reactors-in-series.

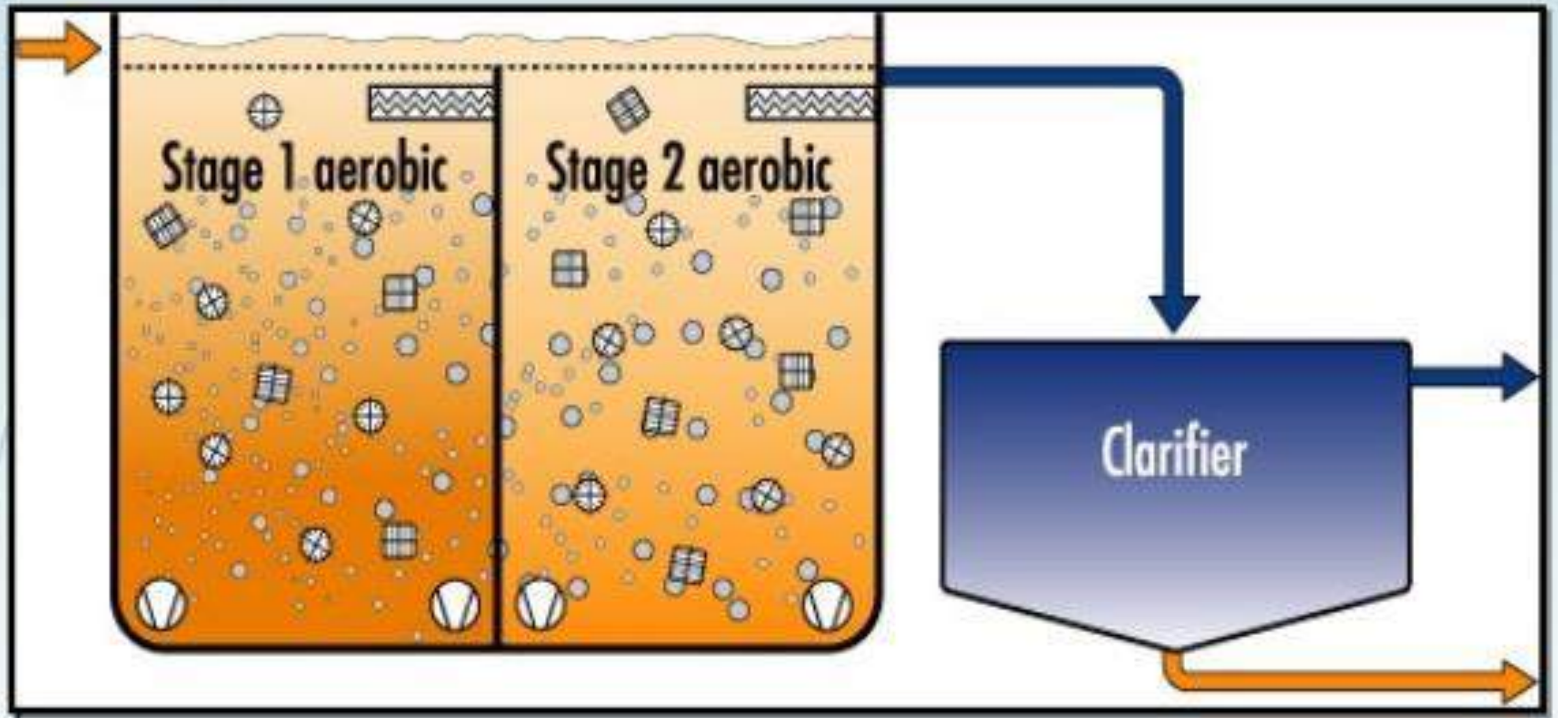
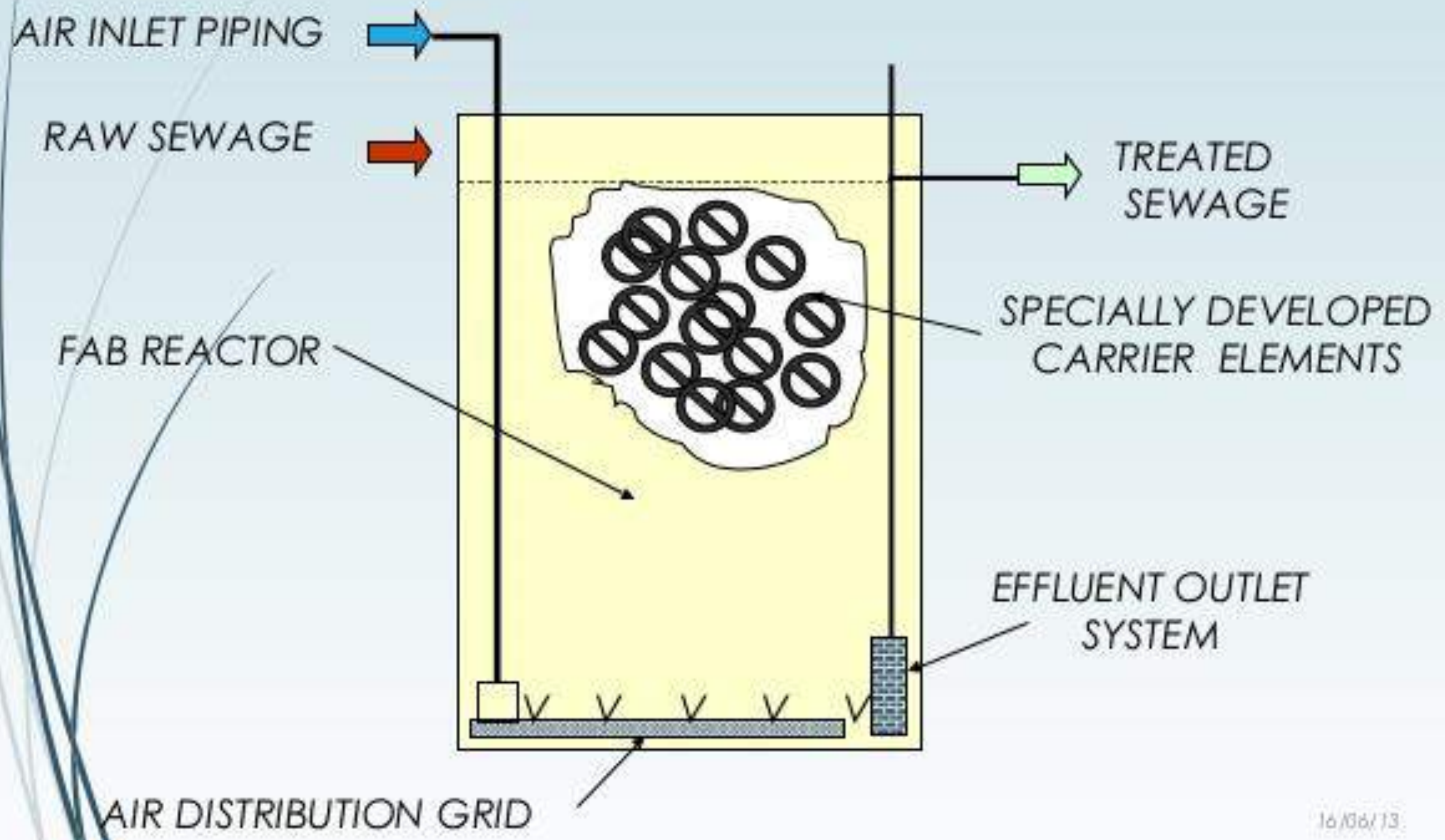
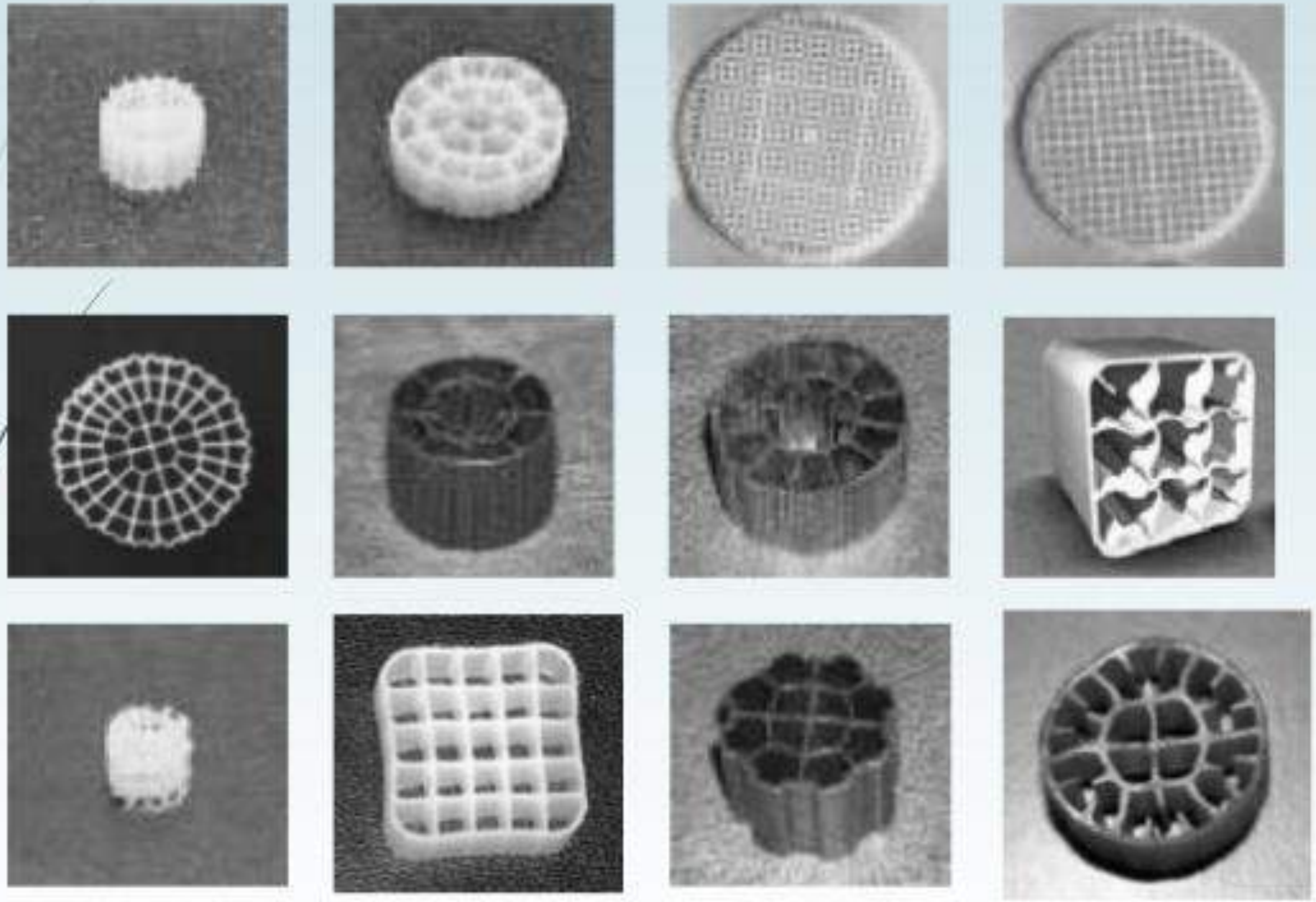


Fig: Moving Bed Biofilm Reactor (MBBR)

FAB Reactor / MBBR



Various types of bio carriers used



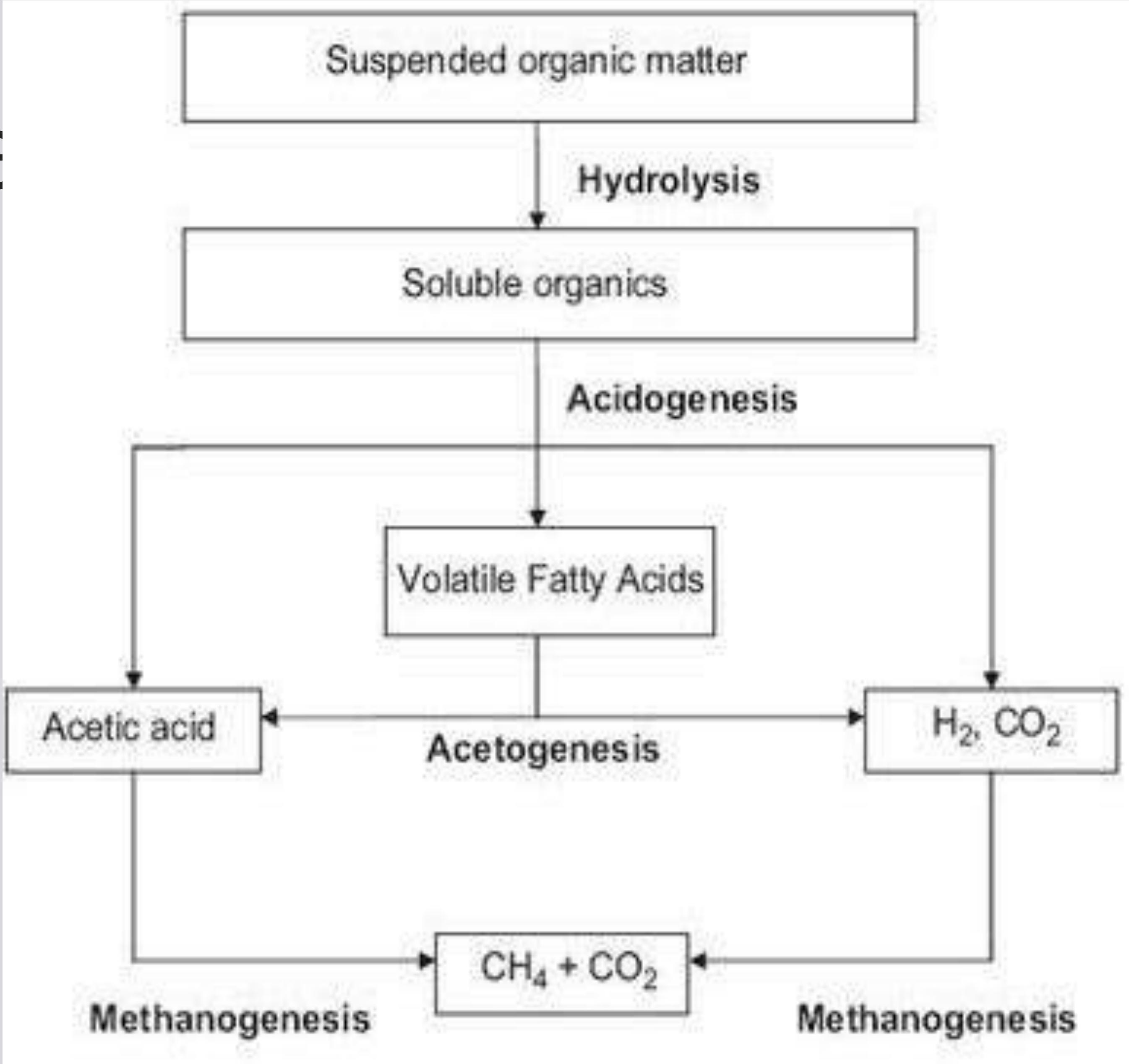
Advantages:

- High-efficiency CBOD removal and nitrification(even in low temperature environments)
- Retention of slow-growing and temperature-sensitive bacterial populations (i.e., autotrophic nitrifiers and methanol-degrading heterotrophs, respectively) low HRT
- Reduced sludge production compared with activated sludge
- No problems with odours, snails or red-worms as in air phase fixed films

Advantages:

- Smaller foot prints.
- MBBR Provides Five times the Biofilm Surface Area in less than ¼ Reactor Volume as Trickling Filter
- Can be operated at varying influent load
- Can be easily retrofitted.

Sludge



Suspended organic matter
Hydrolysis
Soluble organics
Acidogenesis
Volatile Fatty Acids
Acetic acid
Acetogenesis
H_2, CO_2
$CH_4 + CO_2$
Methanogenesis
Methanogenesis

<u>Hydrolysis</u>

- transforms suspended organic matter into soluble organics.
- polymeric compounds ---→ monomeric or dimeric compounds.
- optimal pH 6

<u>Acidogenesis</u>

- soluble organics -→ volatile fatty acids, mostly C2-C4 acids.
- volatile fatty acids lowers the pH
- pH < 4, the production of acids

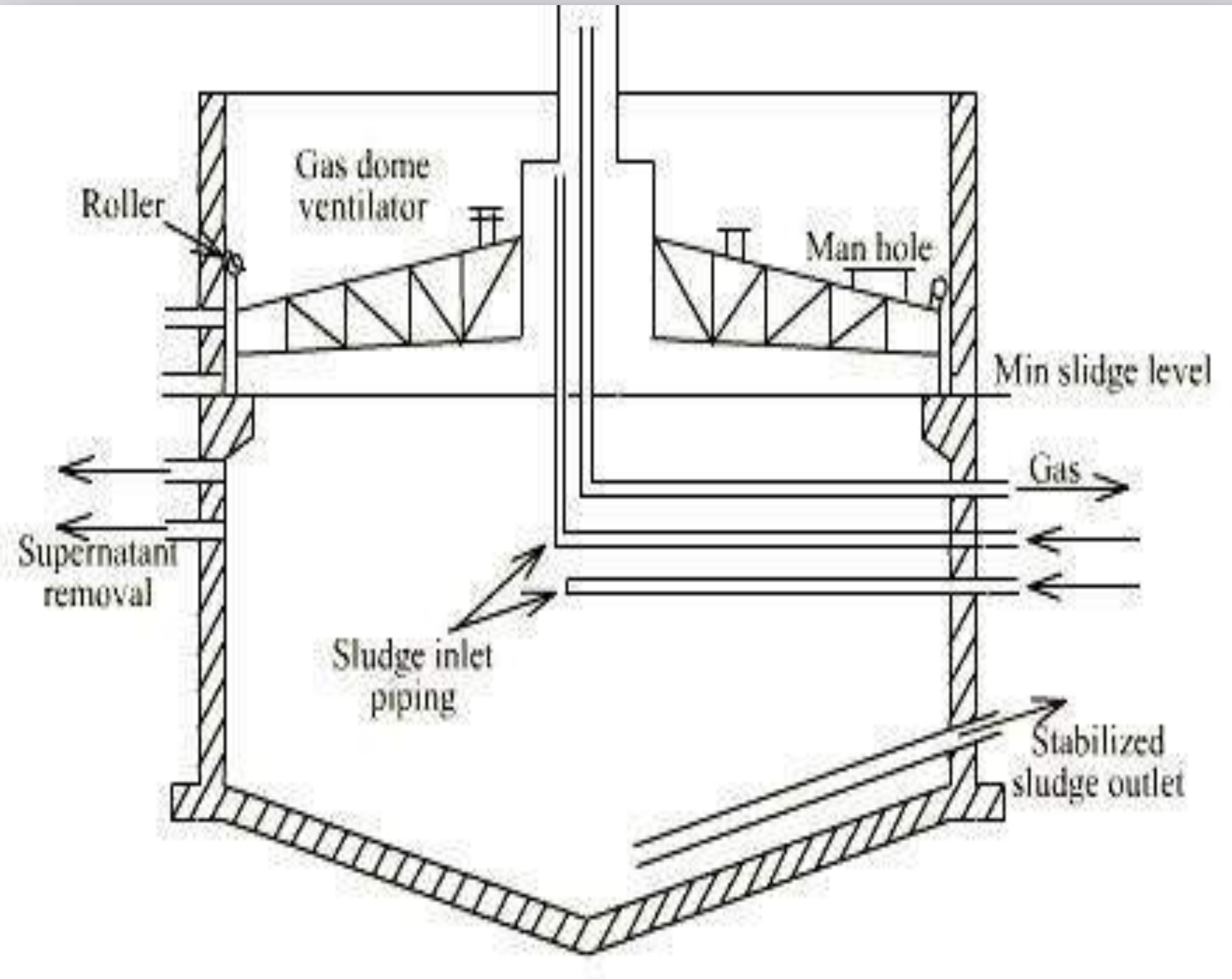
Acetogenesis

- the VFA, ethanol + Water ---→ acetic acid, CO_2 and H_2.

Methanogenesis

- 70% of acetic acid, CO_2 and H_2 --→ methane
- 30% of the biogas is produced in the hydrogenotrophic methanogenesis

RCC or steel tank of cylindrical shape with hopper bottom and is covered with fixed or floating type of roofs



Anaerobic sludge digester

Factors affecting process of Sludge Digestion

- *Temperature*
  - Thermophilic zone 40-60^{0}C and Mesophilic zone- 25 to 40^{0}C
- *pH value*
- *Seeding*
  - *Seeding with the digested sludge* encourage the growth of useful bacteria
  - *Mixing and stirring of raw sludge with digested sludge:*
    - mixed with raw sludge for optimum conditions for the growth and activities of bacteria.

Biochemical Oxygen Demand

- $L_t = L_0(1\text{- }10^{-k_1 t})$

$$\boxed{Y_t = L\left[1-(10)^{-K_D \cdot t}\right]}$$

where, L_0: Ultimate BOD, mg/L

L_t : BOD remaining at any time, mg/L

k_1: 1st order reaction rate constant,1/d

t : time, d

- Rate constant k_1 is dependent on temperature, it can be calculated as,
- $k_{1t} = k_{1\,20^{\circ}} \Theta^{T-20}$

  where, Θ=1.047
- The value of Θ is temperature dependent

Example 7.9. *The BOD_5 of a waste water is 150 mg/l at 20°C. The k value is known to be 0.23 per day. What would BOD_8 be, if the test was run at 15°?*

$K = 0.23$ (given)

$\therefore \quad K_D = 0.434\, K = 0.434 \times 0.23 = 0.0998 \cong 0.1.$

Also BOD of 5 days $= BOD_5 = 150$ mg/l (at 20°C)

Using equation (7.16), we have

$$Y_t = L\left[1-(10)^{-K_D \cdot t}\right]$$

$$\therefore \quad Y_5 = L\left[1-(10)^{-K_D \cdot 5}\right], \qquad \text{where } Y_5 = \text{BOD of 5 days}$$

or
$$150 = L\left[1-(10)^{-0.1\times 5}\right]$$

$$= L\left[1-(10)^{-0.5}\right] = L\left[1-\frac{1}{(10)^{0.5}}\right]$$

$$= L\left[1-\frac{1}{3.16}\right] = L\,[1-0.316] = 0.684\,L$$

$$\therefore \quad L = \frac{150}{0.684} = 219.4 \qquad \ldots(i)$$

or
$$L = 219.4 \text{ mg/l}.$$

Now, let us find K_D value at 15°C

Using equation (7.18), we have

$$K_{D(T^\circ)} = K_{D(20^\circ)}\left[1.047\right]^{T-20^\circ}$$

$$\therefore \quad K_{D(15^\circ)} = 0.1\left[1.047\right]^{15-20^\circ} = 0.1\left[1.047\right]^{-5}$$

$$= 0.1\left[\frac{1}{(1.047)^5}\right] = 0.1\left[\frac{1}{1.258}\right] = 0.079$$

Now, again using

$$Y_t = L\left[1-(10)^{-K_D \cdot t}\right], \text{ where } Y_t \text{ is BOD of } t \text{ days}$$

we have $$Y_8 = 219.4\left[1-(10)^{-0.079\times 8}\right]$$

$$= 219.4\left[1-\frac{1}{(10)^{0.632}}\right] = 219.4\left[1-\frac{1}{4.285}\right]$$

$$= 219.4\left[1-0.233\right] = 219.4 \times 0.766 = 168.2 \text{ mg/l}$$

Hence $BOD_8 = Y_8 = 168.2$ mg/l. **Ans.**

1. Determine 1-day BOD and ultimate first stage BOD for a w/w whose 5- day BOD is 200mg/L at 20°C. The reaction constant k (base e) = $0.23 d^{-1}$. What would have been the 5-day BOD if the test have been conducted at 25°C?

Given : k_1 (to the base e)=0.23/d

BOD_5=200mg/L at 20°C

Calculate:

L_0

BOD_1 at 20°C

BOD_5 at 25°C

Ans: Formula: $Y = L_0(1 - e^{-k_1 t})$, $L_0 = \frac{Y}{(1-e^{-k_1 t})}$, $k_{1T^0} = k_{1\,20^0} 1.047^{(T-20)}$

1) Ultimate BOD: $\therefore L_0 = \frac{Y}{(1-e^{-k_1 t})} = \frac{200}{(1-e^{-0.23*5})} = 293mg/L$

2)Determine 1-day BOD: $Y_1 = L_0\left(1 - e^{-k_1 * 1}\right)$

$$= 293(1 - e^{-0.23*1}) = 60.1mg/L$$

3)Determine 5day BOD at 25°C:

$$k_{125^0} = k_{1\,20^0} 1.047^{(25-20)} = 0.29d^{-1}$$

$$Y_5 = L^0\left(1 - e^{-k_1 * 5}\right) = 293(1 - e^{-0.29*5}) = 224mg/L$$

$$BOD, mg/L = \frac{(D_1 - D_2) - (B_1 - B_2)f}{P}$$

D1= initial DO of diluted sample

D2= Final DO of diluted sample after 5 days

B1= initial DO of dilution water

B2= Final DO of the dilution water

f = (volume of diluted sample- volume of raw sample)/volume of diluted sample

P= dilution ratio= volume of raw sample/volume of diluted sample

2. A BOD test was conducted at 20°C in which 15mL of waste sample was diluted with dilution water to 300mL.

Given:

Initial DO of diluted sample D_1 =8.8mg/L

Final DO after 5 days D_2=1.9mg/L

Initial DO of seeded dilution water B_1=9.1mg/L

Final DO of seeded dilution water B_2=7.9mg/L

Calculate

5-day BOD at 20°C

- Ans:Forrmula, $BOD, mg/L = \frac{(D_1 - D_2) - (B_1 - B_2) f}{P}$

Where $f = \frac{300-15}{300} = 0.95$

$P = \frac{15}{300} = 0.05$

$BOD, mg/L = \frac{(8.8-1.9)-(9.1-7.9)0.95}{0.05} = 115.2\text{mg/L}$

$$BOD = \text{Depletion of oxygen} \times \text{Dilution factor}$$

Depletion of Oxygen = D1-D2

D1= initial DO of diluted sample

D2= Final DO of diluted sample after 5 days

- Dilution factor

$$= \frac{100}{\text{Per cent of solution}} = \frac{100}{2} = 50$$

Example 7.2. *If 2.5 ml of raw sewage has been diluted to 250 ml and the D.O. concentration of the diluted sample at the beginning of the BOD test was 8 mg/l, and 5 mg/l after 5-day incubation at 20°C ; find the BOD of raw sewage.*

Solution. Volume of sample of sewage $= 2.5$ ml.

Volume of diluted sample $= 250$ ml.

$\therefore$ Dilution ratio $= \dfrac{250}{2.5} = 100.$

Loss of dissolved oxygen during the test

= D.O. before testing – D.O. after testing

$= 8 - 5 = 3$ mg/l.

Using equations (7.11), we have

$\therefore$ BOD of sewage = Loss of oxygen × Dilution factor

$= 3 \text{ mg/l} \times 100 = 300$ mg/l. **Ans.**

Example 7.3. *A 2% solution of a sewage sample is incubated for 5 days at 20°C. The depletion of oxygen was found to be 4 ppm. Determine the BOD of the sewage.*

Solution. Dilution factor

$$= \frac{100}{\text{Per cent of solution}} = \frac{100}{2} = 50$$

Depletion of oxygen = 4 ppm.

Using equation (7.11), we have

$$\text{BOD} = \text{Depletion of oxygen} \times \text{Dilution factor}$$
$$= 4 \text{ ppm} \times 50 = 200 \text{ ppm} \quad \textbf{Ans.}$$

Example 7.10. *The 5 day 30°C BOD of sewage sample is 110 mg/l. Calculate its 5 days 20°C BOD. Assume the deoxygenation constant at 20°C, K_{20} as 0.1.*

Solution. $K_{D(20°)} = 0.1$

Now, using equation (7.18)

$$K_{D(T°)} = K_{D(20°)} [1.047]^{T - 20°} \text{, we get}$$

$$K_{D(30°)} = 0.1 [1.047]^{30° - 20°} = 0.1 [1.047]^{10} = 0.158 \quad \text{...}(i)$$

Now, using $Y_t = L \left[1 - (10)^{-K_D \cdot t}\right]$, we get

$$Y_5 = L \left[1 - (10)^{-K_D \cdot 5}\right]$$

$$\therefore \quad Y_{5 \text{ at } 30°} = L \left[1 - (10)^{-K_D(30°) \times 5}\right]$$

or

$$110 = L \left[1 - (10)^{-0.158 \times 5}\right] = L \left[1 - (10)^{-0.79}\right]$$

$$= L\left[1-\frac{1}{(10)^{0.79}}\right] = L\,[1-0.162]$$

or $$110 = L\,(0.838) \quad \text{or } L = \frac{110}{0.838} = 131.3$$

or $$L = 131.3 \text{ mg/l.}$$

Now $$Y_{5 \text{ at } 20°C} = L\left[1-(10)^{-K_D(20°)\times 5}\right]$$

$$= 131.3\left[1-(10)^{-0.1\times 5}\right] = 131.3\left[1-\frac{1}{(10)^{0.5}}\right]$$

$$= 131.3 \times (1-0.316) = 131.3 \times 0.684$$

$$= 89.8 \text{ mg/l.} \quad \textbf{Ans.}$$

$= 89.8$ mg/l. **Ans.**

Example 7.11. *Calculate 1 day 37°C BOD of sewage sample whose 5 day 20°C BOD is 100 mg/l. Assume K_D at 20°C as 0.1.*

Solution. 5 day 20°C BOD = 110 mg/l (given)

Now using Eq. (7.16), we have

The BOD at 20°C, say after t = 5 days, is given by

$$Y_t = L\left[1-(10)^{-K_{D(20°)}.t}\right]$$

Using Y_t = 100 mg/l (given)

$$K_{D(20°)} = 0.1$$

we have $$100 = L\left[1-(10)^{-0.1\times 5}\right]$$

or $$100 = L\left[1-(10)^{-0.5}\right] = L\left[1-\frac{1}{3.16}\right]$$

$$= L\,[1-0.316] = L\,[0.684]$$

or $$L = \frac{100}{0.684} = 146.2 \text{ mg/l.}$$

Now let us work out K_D at 37°C, by using Eq. (7.18) as :

$$K_{D(T°)} = K_{D(20°)}\,[1.047]^{T-20°}$$

or $$K_{D(37^\circ)} = 0.1\,[1.047]^{37-20} = 0.1\,[1.047]^{17}$$
$$= 0.1 \times 2.4 = 0.24.$$

Now, we have to work out Y_t for one day *i.e.* Y_1 at 37°C, using

$$Y_t = L\left[1-(10)^{-K_D \cdot t}\right]$$

$$\therefore \quad Y_1 = L\left[1-(10)^{-K_D \cdot 1}\right]$$

or $$Y_1 \text{ (at 37°C)} = 146.2\left[1-(10)^{-K_{D(\text{at 37°C})} \times 1}\right]$$

$$= 146.2\left[1-(10)^{-0.24 \times 1}\right] = 146.2\left[1-\frac{1}{(10)^{0.24}}\right]$$

$$= 146.2\left[1-\frac{1}{1.738}\right] = 146.2\,[1-0.575] = 62.07.$$

Hence, Y_1 at 37°C = **62.07 mg/l. Ans.**

Hence, Y_1 at 0° C = 62.07 mg/L Ans.

Example 7.12. *The BOD of a sewage incubated for one day at 30°C has been found to be 110 mg/l. What will be the 5-day 20°C BOD ? Assume K_1 = 0.1 at 20°C.*

Solution. $Y_{1\,(\text{at }30°)} = 110$ mg/l. ; $Y_{5\,(\text{at }20°)} = ?$; $K_{D\,(20°)} = 0.1$.

First of all, let us calculate K_D at 30°C, by using Eq. (7.18) *i.e.*

$$K_{D(T)} = K_{D\,(20)}\,[1.047]^{T-20°}$$

or

$$K_{D\,(30°)} = 0.1\,[1.047]^{30-20°} = 0.1\,[1.047]^{10}$$

$$= 0.1 \times 1.583 = \mathbf{0.158.}$$

Now using Eq. (7.16), we have

$$Y_t = L\left[1-(10)^{-K_D\cdot t}\right]$$

At 30°C and for one day, we have

$$Y_{1\,(30°)} = \left[1-(10)^{-K_{D\,(30°)}\times 1}\right]L$$

or
$$110 = L\left[1-(10)^{-0.158\times 1}\right] = L\left[1-\frac{1}{1.438}\right]$$

$$= L\,[1-0.696] = L\,[0.304]$$

or
$$L = \frac{110}{0.304} = 361.8 \text{ mg/l.}$$

Now again using $Y_t = L\left[1-(10)^{-K_D\cdot t}\right]$, we have

$$Y_{5(20°)} = L\left[1-(10)^{-K_{D(20°)}\times 5}\right]$$

$$= L\left[1-(10)^{-0.1\times 5}\right] = L\left[1-\frac{1}{(10)^{0.5}}\right]$$

$$= L\,[1-0.316]$$

$$= 361.8\,[1-0.316] = 247.4 \text{ mg/l.} \quad \textbf{Ans.}$$

Example 7.13. *The BOD_5 of a waste has been measured as 600 mg/l. If k_1 = 0.23/day (base e), what is the ultimate BOD_u of the waste. What proportion of the BOD_u would remain unoxidised after 20 days.*

Solution. Use eqn. (7.16) as ;

$$Y_t = L\left[1-(10)^{-K_D \cdot t}\right]$$

Here $K = k_1 = 0.23$/day (given)

$\therefore \qquad K_D = 0.434\, K = 0.434 \times 0.23 = 0.1.$

Using $t = 5$ days, we have

$$Y_5 = \text{BOD of 5 days}$$

$$= 600 \text{ mg/l} = L\left[1-(10)^{-0.1\times 5}\right]$$

or $\qquad 600 \text{ mg/l} = L\left[1-(10)^{-0.5}\right] = L\left[1-\frac{1}{(10)^{0.5}}\right]$

$$= L\left[1-\frac{1}{3.16}\right] = L\,[1-0.316] = 0.684\,L$$

$$\therefore \quad 0.684\, L = 600 \text{ mg/l}$$

$$\therefore \quad L = \frac{600}{0.684} \text{ mg/l} = 877.5 \text{ mg/l.}$$

Hence, the ultimate BOD = 877.5 mg/l. **Ans.**

Now
$$Y_{20} = L\left[1-(10)^{-0.1\times 20}\right] = Y_u\left[1-\frac{1}{(10)^2}\right]$$

$$= Y_u\,[1-0.01] = Y_u\,[0.99]$$

$$\therefore \quad Y_{20} = 0.99\, Y_u$$

It means that 99% of BOD_u is utilised in 20 days, and hence only 1% of ultimate BOD would be left unoxidised after 20 days. **Ans.**

Example 7.14. *The following observations were made on a 3% dilution of waste water :*

Dissolved oxygen (D.O.) of aerated water used for dilution

= 3.0 mg/l

Dissolved oxygen (D.O.) of diluted sample after 5 days incubation

= 0.8 mg/l

Dissolved oxygen (D.O.) of original sample

= 0.6 mg/l.

Calculate the B.O.D. of 5 days and ultimate BOD of the sample assuming that the deoxygenation coefficient at test temp. is 0.1.

Solution. The 100% contents of the diluted sample consists of 3% wastewater and 97% of aerated water used for dilution.

Hence its D.O. = D.O. of waste water × its content

+ D.O. of dilution water × its content

$= 0.6 \times 0.03 + 3.0 \times 0.97$

$= 0.018 + 2.91 = 2.928$ mg/l.

D.O. of the incubated sample after 5 days

$= 0.8$ mg/l.

Thus, D.O. consumed in oxidising organic matter

$= 2.928 - 0.8 = 2.128$ mg/l.

$\therefore$ B.O.D. of 5 days = D.O. consumed × Dilution factor

$$= 2.128 \times \frac{100}{3} = 70.93 \text{ mg/l.}$$

Ultimate B.O.D. is given by L.

Using Eq. (7.16), we have

$$Y_t = L\left[1 - (10)^{-K_D \cdot t}\right]$$

or $$Y_5 = L\left[1 - (10)^{-K_D \times 5}\right]$$

The value of K_D at test temp. is given as 0.1. Substituting the known values in Eq. (*i*) above, we have

$$70.93 = L\left[1-(10)^{-0.1\times5}\right] = L\left[1-(10)^{-0.5}\right]$$

$$= L\left[1-\frac{1}{(10)^{0.5}}\right] = L\left[1-\frac{1}{3.16}\right]$$

$$= L\left[1-0.316\right] = L \times 0.684$$

or $$L = \frac{70.93}{0.684} = 103.7 \text{ mg/l.}$$ **Ans.**

Example 7.8. *Calculate the population equivalent of a city given (i) the average sewage from the city is 95×10^6 l/day, and (ii) the average 5 day BOD is 300 mg/l.*

Solution. Average 5 day BOD = 300 mg/l.

Average sewage flow = 95×10^6 l/day

$\therefore$ Total BOD in sewage

$$= 300 \times 95 \times 10^6 \text{ mg/day}$$

$$= 300 \times 95 \text{ kg/day} = 28500 \text{ kg/day}$$

Population equivalent

$$= \frac{\text{Total 5 day BOD in kg/day}}{0.08}$$

$$\begin{bmatrix}\text{assuming the domestic sewage} \\ \text{quantity to be 0.08 kg/person/day}\end{bmatrix}$$

$$= \frac{28500}{0.08} = 3{,}56{,}250. \text{ Ans.}$$

Example 9.1. *Estimate the screen requirement for a plant treating a peak flow of 60 million litres per day of sewage.*

Solution. Peak flow = 60 Ml/day

$$= \frac{60 \times 10^6}{1000} \text{ cu-m/day}$$

$$= \frac{60{,}000}{24 \times 60 \times 60} \text{ cu-m/sec} = 0.694 \text{ m}^3\text{/sec.}$$

Assuming that the velocity through the screens (at peak flow) is not allowed to exceed 0.8 m/sec, we have

The net area of screen openings required

$$= \frac{0.694}{0.8} \text{ m}^2 = 0.87 \text{ m}^2.$$

Using rectangular steel bars in the screen, having 1 cm width, and placed at 5 cm clear spacings, we have

The gross area of the screen required

$$= \frac{0.87 \times 6}{5} = 1.04 \text{ m}^2$$

Assuming that the screen bars are placed at 60° to the horizontal, we have

The gross area of the screen needed

$$= \frac{104}{\frac{\sqrt{3}}{2}} = \frac{104 \times 2}{\sqrt{3}} = 1.2\ \text{m}^2.$$

Hence, a coarse screen of 1.2 m^2 area is required. **Ans.**

While designing the screen, we have also to design its **cleaning frequency.** The cleaning frequency is governed by the head loss through the screen. The more the screen openings are clogged, more will be the head loss through the screen. Generally, not more than half the screen clogging is allowed. To know whether the screen has been clogged and needs cleaning, we can check or measure the head loss.

The head loss through the cleaned screen and half-cleaned screen, can be computed as follows :

Velocity through the screen = 0.8 m/sec.

Velocity above the screen

$$= \frac{0.8 \times 5}{6} \text{ m/sec} = 0.67 \text{ m/sec}$$

$$\boxed{\text{Head loss through the screen} = 0.0729\,(V^2 - v^2)} \quad \ldots(9.1)$$

$$= 0.0729\,(0.8^2 - 0.67^2) = 0.0134 \text{ ; say } \mathbf{0.013\ m.}$$

When the screen openings get half clogged, then

The velocity through the screen

$$= v = 0.8 \times 2 = 1.6 \text{ m/sec}$$

$\therefore$ Head loss = $0.0729\,(1.6^2 - 0.67^2) = 0.1538$; **say 0.15 m.**

This shows that when the screens are totally clean, the head loss is negligible *i.e.* about 1.3 cm only ; whereas, the head loss shoots up to about 15 cm at half the clogging. The screens should therefore be cleaned frequently, as to keep the head loss within the allowable range. **Ans.**

GRIT CHAMBER

Example 9.2 (a) *A rectangular grit chamber is designed to remove particles with a diameter of 0.2 mm, specific gravity 2.65. Settling velocity for these particles has been found to range from 0.016 to 0.022 m/sec, depending on their shape factor. A flow through velocity of 0.3 m/sec will be maintained by proportioning weir. Determine the channel dimensions for a maximum wastewater flow of 10,000 cu m/day.*

Solution. Let us provide a rectangular channel section, since a proportional flow weir is provided for controlling velocity of flow.

Now,

Horizontal velocity of flow = V_h = 0.3 m/sec.

Settling velocity is between 0.016 to 0.022 m/sec, and hence let it be 0.020 m/sec.

Now, Q = velocity × cross-section

or $Q = V_h \times A$

where Q = 10,000 cu m/day

$$= \frac{10000}{24 \times 60 \times 60} \text{ m}^3/\text{s} = 0.116 \text{ m}^3/\text{s}$$

$\therefore$ $0.116 = 0.3 . A$

$\therefore \quad 0.116 = 0.3 . A$

$$A = \frac{0.116}{0.3} \quad \text{or} \quad A = 0.385 \text{ m}^2.$$

Assuming a water depth (H) of 1 m above the crest of the weir, which is kept at 0.3 m above the channel bottom, we have the width (B) of the basin as

$$1 \times B = 0.385$$

or $\quad B = 0.385$ m ; say 0.4 m.

Overall depth of grit chamber (D)

= Water depth above the crest of weir + 0.3 m

+ Free board of 0.45 m

= 1.0 m + 0.3 m + 0.45 m = 1.75 m

Now, settling velocity

$$V_s = 0.02 \text{ m/sec}$$

$$\therefore \quad \text{Detention time} = \frac{\text{Water depth in the basin}}{\text{Settling velocity}} = \frac{1}{0.02} = 50 \text{ secs}$$

$\therefore$ Length of the tank = $V_h \times$ Detention time = 0.3 × 50 m = 15 m.

Example 9.3. *Design an aerated grit chamber for treating municipal waste water with average flow rate of $0.5\ m^3/s$ (43.2 MLD). Assume the peak flow rate to be 3 times the average.*

Solution. Peak flow rate = $0.5\ m^3/sec \times 3 = 1.5\ m^3/s$

Assume average liquid detention time = 3 min = 180 sec.

$\therefore$ Aerator volume = $1.5\ m^3/s \times 180\ s = 270\ m^3$

In order to drain the channel periodically for routine cleaning and maintenance, *use two chambers.*

$$\therefore \quad \text{Volume of one aerated channel} = \frac{270\ m^3}{2} = 135\ m^3$$

To determine the *dimensions of the aerated channel*, assume depth of 3 m and width-depth ratio of 2 : 1.

$\therefore$ Width of channel = Depth $\times$ 2 = 3 m $\times$ 2 = 6 m

$$\therefore \quad \text{Length of channel} = \frac{135\ \text{m}^3}{3\ \text{m} \times 6\ \text{m}} = 7.5\ \text{m}$$

Increase the length by about 20% to account for inlet and outlet conditions.

$\therefore$ Provided length = 7.5 × 1.2 = **9 m.**

Hence, use 2 chambers, each of size 9 m × 6 m × 3 m depth. **Ans.**

Air supply requirement. Assume that 0.03 m^3/min per m length may be adequate.

$$\text{Air required} = 0.03\ \frac{\text{m}^3}{\text{min . m}} \times 9\ \text{m} = \mathbf{0.27\ m^3/min.} \quad \textbf{Ans.}$$

Volume of grit produced daily. Assume that 50 m^3/M.cum of sewage of grit is produced by the incoming sewage, the daily grit volume produced

$$= \text{Peak flow rate of sewage in m}^3/\text{d} \times \text{Grit produced per m}^3 \text{ of sewage}$$

$$= 1.5\ \frac{\text{m}^3}{\text{s}} \times \left(24 \times 3600\ \frac{s}{d}\right) \times \frac{50\ \text{m}^3}{10^6\ \text{m}^3} = \mathbf{6.48\ m^3/d.} \quad \textbf{Ans.}$$

[**Note.** The grit handling facilities must be based on sustained peak flow rate. Hence, the arrangement for removal of grit @ 6.48 m^3/d must be provided.]

Example 9.4. *Design a suitable grit chamber cum Detritus tank for a sewage treatment plant getting a dry weather flow from a separate sewerage system @ 400 l/s. Assume the flow velocity through the tank as 0.2 m/sec ; and detention period of 2 minutes. The max. flow may be assumed to be three times of dry weather flow.*

Solution. The length of the tank

$$= \text{Velocity} \times \text{Detention time} = 0.2 \times (2 \times 60) = 24 \text{ m.}$$

Since the peak flow is three times the DWF, let us provide three detritus tanks, each designed for passing D.W.F.

$\therefore$ The discharge passing through each tank = 400 l/s = 0.4 m^3/sec.

$$\therefore \quad \text{Cross-sectional area required} = \frac{\text{Discharge}}{\text{Velocity}} = \frac{0.4}{0.2} = 2 \text{ m}^2.$$

Assuming the water depth in the tank to be 1.2 m, we have the width of the tank

$$= \frac{\text{Area of } X\text{-section}}{\text{Depth}} = \frac{2}{1.2} = 1.67 \text{ m ; say } 1.7 \text{ m.}$$

Hence, use a Detritus tank with 24 m × 1.7 m × 1.2 m size.

At the top, a free-board of 0.3 m may be provided ; and at the bottom, a dead space depth of 0.45 m for collection of detritus may be provided.

Thus, the overall depth of the tank

$$= 1.2 + 0.3 + 0.45 = 1.95 \text{ m}.$$

The tank will be 1.7 m wide upto 1.5 m depth, and then the sides will slope down to form an elongated trough of 24 m length and 0.8 m width at the bottom with rounded corners, as shown in Fig. 9.10.

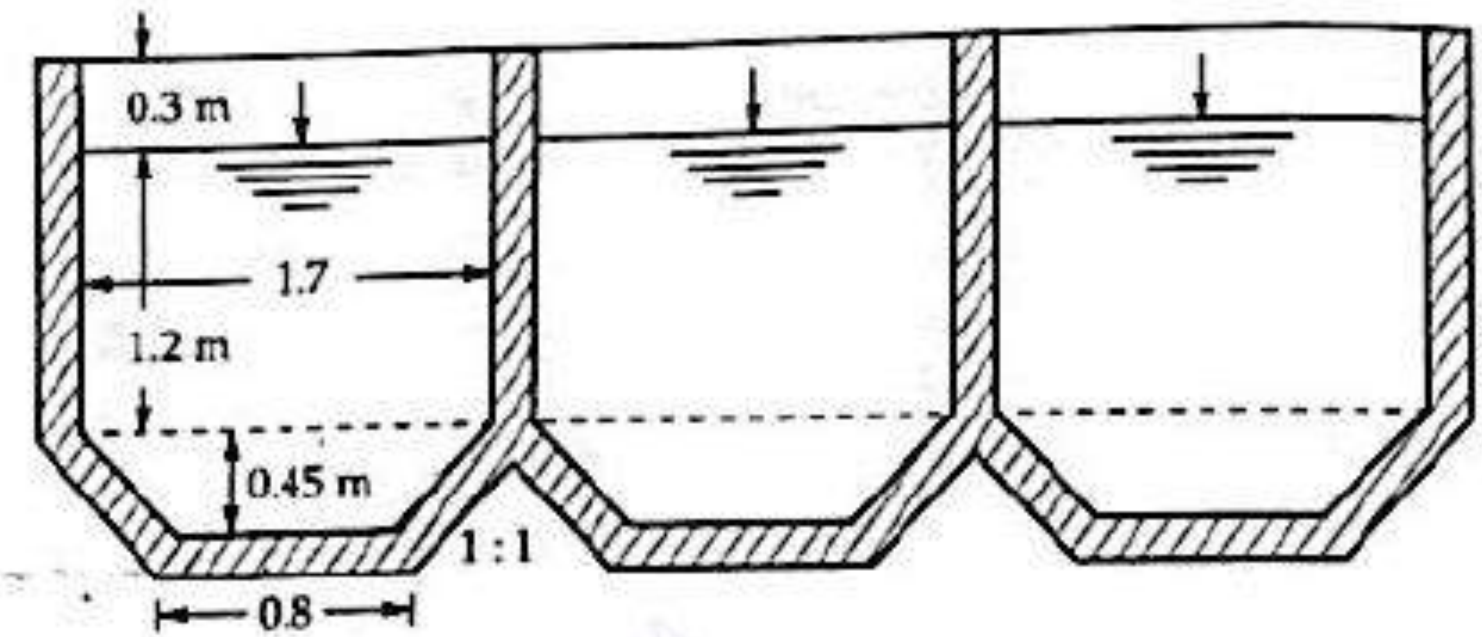


Fig. 9.10

SEDIMENTATION

called the *sludge zone.*

Example 9.5. *Design a suitable rectangular sedimentation tank (provided with mechanical cleaning equipment) for treating the sewage from a city, provided with an assured public water supply system, with a max. daily demand of 12 million litres per day. Assume suitable values of detention period and velocity of flow in the tank. Make any other assumptions, wherever needed.*

Solution. Assuming that 80% of water supplied to the city becomes sewage, we have the quantity of sewage required to be treated per day (*i.e.* max. daily).

$$= 0.8 \times 12 \text{ million litres} = 9.6 \text{ M. litres}$$

Now assuming the detention period in the sewage sedimentation tank as 2 hours, we have

The quantity of sewage to be treated in 2 hours *i.e.* the capacity of the tank required

$$Q = \frac{9.6}{24} \times 2 \text{ M. litres} = 0.8 \text{ M litres} = 800 \text{ cu. m.}$$

Now, assuming that the flow velocity through the tank is maintained at 0.3 m/minute ; we have

The length of the tank required

$$= \text{Velocity of flow} \times \text{Detention period}$$

$$= 0.3 \times (2 \times 60) \text{ m} = 36 \text{ m.}$$

Cross-sectional area of the tank required

$$= \frac{\text{Capacity of the tank}}{\text{Length of the tank}} = \frac{800}{36} \text{ m}^2 = 22.2 \text{ m}^2.$$

Assuming the water depth in the tank (*i.e.* effective depth of tank) as 3 m,

The width of the tank required

$$= \frac{\text{Area of } X\text{-section}}{\text{Depth}} = \frac{22.2}{3} = 7.4 \text{ m}.$$

Since the tank is provided with mechanical cleaning arrangement, no extra space at bottom is required for sludge zone.

Now, assuming a free-board of 0.5 m, we have

The overall depth of the tank

$$= 3 + 0.5 = 3.5 \text{ m}.$$

Hence, a rectangular sedimentation tank with an overall size of 36 m × 7.4 m × 3.5 m can be used.

[**Note.** This satisfies the requirements like : length $\not>$ 4 to 5 times the width ; and the width not more than 7.5 m or so ; the depth between 2.4 to 3.6 m, etc.]. **Ans.**

Alternatively,

Example 9.6. *Design a circular settling tank unit for a primary treatment of sewage at 12 million litres per day. Assume suitable values of detention period (presuming that trickling filters are to follow the sedimentation tank), and surface loading.*

Solution. Assuming the normal detention period for such cases as 2 hr, and surface loading as 40,000 litres/sq. m/day ; we have

The quantity of sewage to be treated per 2 hours

$$= 12 \text{ M. litres} \times \frac{2}{24} = 1 \text{ M. litres} = 1000 \text{ m}^3.$$

$\therefore$ Capacity of tank = 1000 m^3.

Now, surface loading

$$= \frac{Q}{\text{Surface area of tank}} = \frac{Q}{\frac{\pi}{4} . d^2}$$

4

or $$40{,}000 = \frac{12 \times 10^6}{\frac{\pi}{4} \cdot d^2}$$

where d is the dia. of the tank

or $$\frac{\pi}{4} \cdot d^2 = \frac{12 \times 10^6}{40{,}000}$$

or $$d = \sqrt{\frac{300 \times 4}{\pi}} = 19.55 \text{ m Say } 19.6 \text{ m.}$$

Now, effective depth of tank

$$= \frac{\text{Capacity}}{\text{Area of X-section}} = \frac{1000}{\frac{\pi}{4} \times (19.6)^2} = \frac{1000}{302}$$

$$= 3.2 \text{ m. (Say).}$$

Hence, use a settling tank with 19.6 m dia. and 3.2 m water depth (with free board of 0.3 m extra depth). **Ans.**

The efficiency of such a conventional filter plant can be expressed by the equation evolved by National Research Council of U.S.A., and given as :

$$\eta\ (\%) = \frac{100}{1 + 0.0044\sqrt{u}} \qquad \ldots(9.32)$$

where η = Efficiency of the filter and its secondary clarifier, in terms of percentage of applied BOD removed.

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

of the filter media per day. The value of organic loading for conventional filters may vary between 900 to 2200 kg of BOD_5 per ha-m. This organic loading value can be further increased to about 6000—18000 kg of BOD_5 per ha-m in high rate trickling filters.

given in Eq.

Example 9.7. (*a*) *Design suitable dimensions of a circular trickling filter units for treating 5 million litres of sewage per day. The BOD of sewage is 150 mg/l.*

(*b*) *Also design suitable dimensions for its rotary distribution system, as well as the under-drainage system.*

Solution. Total BOD present in sewage to be treated per day

$$= 5 \text{ ML} \times 150 \text{ mg/L} = 5 \times 10^6 \times 150 \text{ mg}$$

$$= 5 \times 150 \text{ kg} = 750 \text{ kg}.$$

Assuming the value of organic loading, say as, 1500 kg/hectare metre/day [*i.e.* between 900 to 2200 kg/ha-m/day], we have

The volume of filtering media required

$$= \frac{750}{1500} \text{ hectare-metre} = 0.5 \text{ ha-m} = 5000 \text{ m}^3.$$

Assuming the effective depth of filter, as, say 2 m, we have

The surface area of the filter required

$$= \frac{5000}{2} \text{ m}^2 = 2500 \text{ m}^2.$$

Using a circular trickling filter of dia 40 m, we have the number of units required

$$= \frac{\text{Total area required}}{\text{Area of one unit}} = \frac{2500}{\frac{\pi}{4}(40)^2} \approx 2 \text{ Nos.}$$

can also be worked out by assuming the value of hydraulic loading, say as, 25 million litres per hectare per day [*i.e.* between 22 to 44 ML/ha/day]

$\therefore$ Surface area required

$$= \frac{\text{Total sewage to be treated per day}}{\text{Hydraulic loading per day}}$$

$$= \frac{5 \text{ ML/day}}{25 \text{ ML/ha/day}} \frac{5}{25} \text{ hectares}$$

$$= \frac{5}{25} \times 10{,}000 \text{ m}^2 = 2000 \text{ m}^2.$$

The surface area chosen is 2500 m², which is greater than 2000 sq. m, and hence safe.

Hence, 2 units each of 40 m dia and 2 m effective depth (*i.e.* 2.6 m overall depth), can be adopted. An extra third unit as stand-by may also be constructed. **Ans.**

Example 9.9. *A town having a population of 30,000 persons is producing the following sewages :*

(*i*) *Domestic sewage @ 120 l.p.c.d. having 200 mg/l of BOD.*

(*ii*) *Industrial sewage @ 3,00,000 l.p.d. having 800 mg/l of BOD.*

Design high rate single stage trickling filters for treating the above sewage. Assuming that the primary sedimentation removes 35% of BOD. Allow an organic loading of 10,000 kg/ha-m/day (excluding recirculated sewage). The recirculation ratio is 1.0 ; and the surface loading should not exceed 170 M.l./ha/day (including recirculated sewage). Also determine the efficiency of the filter and the BOD of the effluent.

Solution. Quantity of domestic sewage produced per day

$$= 120 \times 30,000 \text{ litres/day} = 3.6 \text{ M.}l\text{/day.}$$

BOD for domestic sewage = 200 mg/l.

$\therefore$ Total BOD of domestic sewage per day

$$= 3.6 \times 200 \text{ kg/day} = 720 \text{ kg/day} \quad \ldots(i)$$

Quantity of industrial sewage produced per day

$$= 3,00,000 \text{ litres. /d}$$

BOD of industrial sewage = 800 mg/l

∴ Total BOD of industrial sewage

$$= \frac{3,00,000 \times 800}{10^6} \text{ kg/day}$$

$$= 240 \text{ kg} \qquad \text{...(ii)}$$

Total BOD of domestic as well as industrial sewage per day

$$= 720 + 240 = 960 \text{ kg/day}$$

Out of this BOD, 35% is already removed in primary clarifier.

∴ BOD to be removed by filter unit

$$= 960 \times (0.65) = \mathbf{624 \text{ kg/day.}}$$

Volume of filter media required

$$= \frac{\text{Total BOD removed}}{\text{Organic loading}} = \frac{624}{10,000} \text{ ha-m}$$

$$= \frac{624}{10,000} \times 10^4 \text{ m}^3 = \mathbf{624 \text{ m}^3}.$$

Now, the total volume of sewage flowing

$$= (3.6 \times 10^6 + 3,00,000) \text{ litres/day}$$

$$= (3.9 \times 10^6) \text{ litres/day} = 3.9 \text{ M}l\text{./day}$$

A recirculation ratio of 1 means that the volume of recirculated sewage

$$(R) = \text{Original volume} = 3.9 \text{ M.}l\text{./day} \qquad \text{...(i)}$$

Total volume (*i.e.*, original + recirculated)

$$= 2 \times 3.9 \text{ M.}l\text{/day} = 7.8 \text{ M.}l\text{./day}$$

∴ Filter area required

$$= \frac{\text{Total flow volume}}{\text{Surface loading}} = \frac{7.8 \text{ Ml/d}}{170 \text{ Ml/ha.d}}$$

$$= \frac{7.8}{170} \text{ hectares} = \frac{7.8}{170} \times 10^4 \text{ m}^2 = 458.8 \text{ m}^2.$$

∴ Dia of the filter tank required

$$= \sqrt{\frac{458.8 \times 4}{\pi}} = 24.17 \text{ m}.$$

Hence, use, say, 24 m dia tank with area as = 452.16 m^2. **Ans.**

∴ Depth of filter media required

$$= \frac{\text{Volume of filter media}}{\text{Surface area}} = \frac{624 \text{ m}^3}{452.16 \text{ m}^2} = 1.38 \text{ m.} \quad \textbf{Ans.}$$

Efficiency of this filter is given by Eq. (9.34) as

$$\eta = \frac{100}{1 + 0.0044\sqrt{\dfrac{Y}{V.F.}}}$$

where Y = Total organic load, *i.e.*, total BOD applied to filter in kg/day = 624 kg per day.

V = Volume of filter in ha-m

$$= \frac{624}{10{,}000} = 0.0624 \text{ ha-m}$$

F = Recirculation factor as given by Eq. (9.33) as :

$$= \frac{1+\frac{R}{I}}{\left[1+0.1\frac{R}{I}\right]^2} \text{ where } \frac{R}{I} = 1 \text{ (given)}$$

$$= \frac{1+1}{(1+0.1)^2} = \frac{2}{(1.1)^2} = \frac{2}{1.21} = 1.65$$

or $$\eta = \frac{100}{1+0.0044\sqrt{\frac{624}{0.0624 \times 1.65}}} = \frac{100}{1+0.0044\sqrt{\frac{10^4}{1.65}}} = 74.5\% \quad \textbf{Ans.}$$

BOD of the effluent left

$$= \frac{(100-74.5)}{100} \times 624 \text{ kg/day}$$

$$= \frac{25.5}{100} \times 624 = 159.12 \text{ kg}$$

Total volume of effluent = 3.9 M l/day

∴ BOD concentration in the effluent

$$= \frac{159.12 \times 10^6}{3.9 \times 10^6} \text{ mg}/l = 40.8 \text{ mg}/l. \quad \textbf{Ans.}$$

Example 9.10. *Determine the size of a high rate trickling filter for the following data :*

(i) *Sewage flow = 4.5 Mld ;*

(ii) *Recirculation ratio = 1.5 ;*

(iii) *BOD of raw sewage = 250 mg/l ;*

(iv) *BOD removal in primary tank = 30% ;*

(v) *Final effluent BOD desired = 30 mg/l.* (A.M.I.E. 1974)

Solution. Quantity of sewage flowing into the filter per day = 4.5 M.l/day.

BOD concentration in raw sewage = 250 mg/l.

∴ Total BOD present in raw sewage = 4.5 Ml × 250 mg/l = 1125 kg.

BOD removed in primary tank = 30%

BOD left in the sewage entering per day in the filter unit

$$= (1125)\,0.7 = 787.5 \text{ kg.}$$

BOD concentration desired in final effluent = 30 mg/l.

∴ Total BOD left in the effluent per day = 4.5 × 30 kg. = 135 kg.

∴ BOD removed by the filter = 787.5 – 135 = 652.5 kg.

∴ Efficiency of the filter

$$= \frac{\text{BOD removed}}{\text{Total BOD}} \times 100 = \frac{652.5}{787.5} \times 100 = 82.85\%.$$

Now, using equation (9.34), we have

$$\eta = \frac{100}{1 + 0.0044\sqrt{\frac{Y}{V.F.}}}$$

where $\eta = 82.85\%$

Y = Total BOD in kg = 787.5 kg.

$$F = \frac{1 + \frac{R}{I}}{\left(1 + 0.1\frac{R}{I}\right)^2} \text{ ; where } = \frac{R}{I} = 1.5 \text{ (given)}$$

$$\therefore \quad F = \frac{1 + 1.5}{[1 + 0.1 \times 1.5]^2} = \frac{2.5}{(115)^2} = \frac{2.5}{1.322} = 1.89$$

$$\therefore \qquad 82.85 = \frac{100}{1 + 0.0044\sqrt{\frac{787.5}{V \times 189}}}$$

or

$$1 + 0.0044 \,.\, \sqrt{\frac{416.6}{V}} = 1.207$$

or

$$\sqrt{\frac{416.6}{V}} = \frac{0.207}{0.0044} = 47.05$$

or $$\frac{416.6}{V} = 2213.3$$

or $$V = 0.188 \text{ hectare-m.} = 1880 \text{ m}^3$$

Assuming the depth of the filter as 1.5 m, we have

The surface area required

$$= \frac{1880}{1.5} \text{ m}^2 = 1253 \text{ m}^2$$

$\therefore$ Dia of the circular filter required

$$= \sqrt{1253 \times \frac{4}{\pi}} = 40 \text{ m.}$$

Hence, use a high rate trickling filter with 40 m dia., 1.5 m deep filter media, and with recirculation (single stage) ratio of 1.5. **Ans.**

Example 9.11. *Determine the size of a high rate trickling filter for the following data :*

Flow	= 4.5 Mld
Recirculation ratio	= 1.4
BOD of raw sewage	= 250 mg/l
BOD removed in primary clarifier	= 25%.
Final effluent BOD desired	= 50 mg/l.

Calculate also the size of the standard rate trickling filter to accomplish the above requirement. (*Calcutta University 1967*)

Solution. Total BOD present in raw sewage per day

$= 4.5 \text{ Ml} \times 250 \text{ mg/l} = 1125 \text{ kg}.$

BOD removed in the primary clarifier = 25%.

$\therefore$ BOD entering per day in the filter units

$= 0.75 \times 1125 \text{ kg} = 843.75 \text{ kg}.$

Permissible BOD concentration in the effluent = 50 mg/l.

$\therefore$ BOD allowed to go into the effluent

$= 50 \text{ mg/l} \times 4.5 \text{ Ml} = 225 \text{ kg}.$

$\therefore$ BOD removed by the filter per day = 843.75 − 225 = 618.75 kg.

$$\text{Efficiency of the filter} = \frac{\text{BOD removed}}{\text{Total BOD entering}} \times 100 = \frac{618.75}{843.75} \times 100 = 73.3\%.$$

Now, efficiency of the filter is given by Eq. (9.34) as

$$\eta = \frac{100}{1 + 0.0044\sqrt{\frac{Y}{V.F}}}$$

where Y = Total BOD applied to the filter per day in kg.

= 843.75 kg

F = Recirculation factor

V = Vol of filter in ha.m.

$$= \frac{1+\frac{R}{I}}{\left[1+0.1\frac{R}{I}\right]^2} \text{ ; where } \frac{R}{I} = 1.4 \text{ (given)}$$

$$= \frac{1+1.4}{(1+0.1\times 1.4)^2} = \frac{2.4}{(1.14)^2} = \frac{2.4}{1.3} = 1.85$$

$$\therefore \qquad 73.3 = \frac{100}{1+0.0044\sqrt{\frac{843.75}{V\times 1.85}}}$$

or $$1+0.0044\sqrt{\frac{456}{V}} = \frac{100}{73.3} = 1.364$$

or $$\sqrt{\frac{456}{V}} = \frac{0.364}{0.0044} = 82.78$$

or $$\frac{456}{V} = 6853$$

or $$V = \frac{45.6}{6853} \text{ hectare-m.} = \frac{456}{6853} \times 10^4 \text{ m}^3 = 665.4 \text{ m}^3.$$

Using 1.5 m depth of the filter, we have

Area required $$= \frac{665.4}{1.5} = 413.6 \text{ m}^2$$

$\therefore$ Dia of the filter tank required

$$= \sqrt{\frac{413.6\times 4}{\pi}} = 23.8 \text{ m.} \quad \textbf{Ans.}$$

For an equivalent standard rate filter ; $F = 1$.

$$\therefore \qquad 73.3 = \frac{100}{1 + 0.0044 . \sqrt{\dfrac{843.75}{V}}}$$

or $$1 + 0.0044 . \sqrt{\frac{843.75}{V}} = \frac{100}{73.3} = 1.364$$

or $$\sqrt{\frac{843.75}{V}} = \frac{0.364}{0.0044} = 82.73$$

or $$\frac{843.75}{V} = 6843$$

or $$V = \frac{843.75}{6843} \text{ ha-m} = 0.1233 \text{ ha-m.} = 1233 \text{ m}^3$$

$$\left[\because \text{ ha-m} = 10^4 \text{ sq. mm.} \atop = 10^4 \text{ m}^3\right]$$

Using depth of filter as 1.5 m, we have

$$\text{Surface area required} = \sqrt{\frac{1233}{1.5}} = 822 \text{ m}^2$$

$\therefore$ Dia of the filter tank required

$$= \sqrt{\frac{822 \times 4}{\pi}} = 32.4 \text{ m.} \quad \textbf{Ans.}$$

Example 9.12. *A single stage filter is to treat a flow of 3.79 M.l.d. of raw sewage with BOD of 240 mg/l. It is to be designed for a loading of 11086 kg of BOD in raw sewage per hectare metre, and the recirculation ratio is to be 1. What will be the strength of the effluent, according to the recommendations of the National Research Council of U.S.A.*

Solution. Total BOD present in raw sewage

$$= 3.79 \text{ Ml} \times 240 \text{ mg/l} = 909.6 \text{ kg}$$

Now, filter volume required

$$= \frac{\text{Total BOD in raw sewage in kg}}{\text{Given BOD loading rate of } 11{,}086 \text{ kg / ha-m}}$$

$$= \frac{909.6}{11086} \text{ ha-m} = 0.082 \text{ ha-m.}$$

Now, assuming that 35% of BOD is removed in primary clarifier, we have

The amount of BOD applied to the filter

$$= 0.65 \times 909.6 \text{ kg} = 591.24 \text{ kg.}$$

Now, using equation (9.34), we have

$$\eta = \frac{100}{1 + 0.0044\sqrt{\dfrac{Y}{V \cdot F}}}$$

where Y = Total BOD applied to the filter in kg

$= 591.24$ kg

$\therefore$ V = Vol. of the filter in ha-m. = 0.082 ha-m.

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

$$\therefore \quad F = \frac{1+1}{(1+0.1)^2} = \frac{2}{1.21} = 1.65.$$

$$\therefore \quad \eta = \frac{100}{1+0.0044\sqrt{\frac{591.24}{0.082 \times 1.65}}} = 77.47\%.$$

$\therefore$ **The amount of BOD left in the effluent**

$$= 591.24\,[1 - 0.7747] \text{ kg.} = 133.21 \text{ kg.}$$

$\therefore$ **BOD concentration in the effluent**

$$= \frac{\text{Total BOD}}{\text{Sewage volume}} = \frac{133.21 \times 10^6}{3.79 \times 10^6} \text{ mg/}l = 35.15 \text{ mg/}l. \quad \textbf{Ans.}$$

Example 9.13. *It is proposed to use a two stage plant instead of the single stage plant in example 9.12. The total volume of filter medium remains the same as was in one filter, i.e. 0.082 ha-m, and each filter is to contain half of this material, and the recirculation ratio is to be 1 for each filter. Determine the BOD of the plant effluent.*

Solution. For each filter $F = 1.65$.

For the first stage filter, the efficiency is given by

$$\eta = \frac{100}{1 + 0.0044\sqrt{\dfrac{Y}{V.F}}}$$

where Y = Total BOD applied to filter

= 591.24 kg (from previous example)

V = Volume of filter = $\dfrac{0.082}{2}$ = 0.041 ha-m

$$\therefore \qquad \eta = \frac{100}{1 + 0.0044\sqrt{\dfrac{591.24}{0.041 \times 1.65}}} = 70.85\%$$

∴ Percentage of BOD removed in first stage filter = 70.85%.

∴ Amount of BOD left in the effluent from that filter

= 591.24 [1 − 0.7085] = 172.32 kg.

For the second stage filter, the efficiency is given by equati...

For the second stage filter, the efficiency is give

The efficiency $\eta' = \dfrac{100}{1+\dfrac{0.0044}{1-\eta}\sqrt{\dfrac{Y'}{V'.F'}}}$

where $Y' = 172.32$ kg

$V' = 0.041$ ha-m.

$F' = 1.65.$

$\eta = 0.7085$

$$\eta' = \frac{100}{1+\dfrac{0.0044}{1-0.7085}\sqrt{\dfrac{172.32}{0.041\times 1.65}}} = \frac{100}{1+\dfrac{0.0044}{0.2915}\sqrt{\dfrac{172.32}{0.041\times 1.65}}}$$

$= 56.76\%$

The amount of BOD left in the effluent from the plant

$$= 171.9\left[\frac{100-56.76}{100}\right] \text{kg.} = 74.33 \text{ kg.}$$

BOD concentration in the effluent

$$= \frac{\text{Total BOD}}{\text{Sewage volume}} = \frac{74.33\times 10^6}{3.79\times 10^6} \text{ mg}/l = 19.61 \text{ mg}/l. \quad \textbf{Ans.}$$

Example 9.14. *The design flow of sewage is 3.8 million litres per day, and the BOD of the raw sewage is 300 mg/l. Design a single stage Bio filter to produce an effluent having a BOD of 45 mg/l or less.*

Solution. Total BOD present in raw sewage per day

$$= 3.8 \times 300 \text{ kg.} = 1140 \text{ kg.}$$

Assuming that 35% of this BOD is removed in the primary sedimentation tank, we have

The total daily BOD applied to the filter = 0.65×1140 kg = 741 kg.

Now, the total daily BOD present in the effluent (permissible maximum)

$$= 3.8 \times 45 \text{ kg.} = 171 \text{ kg.}$$

$\therefore$ Total daily BOD to be removed by the filter = $741 - 171 = 570$ kg.

$$\therefore \quad \text{Efficiency of the filter} = \frac{570}{741} \times 100 = 76.92\%.$$

Assuming an organic loading of say 10,000 kg/ha-m/day [*i.e.*, between 9,000 to 14,000], we have

$$\text{Volume of filter required} = \frac{\text{Total daily BOD removed}}{\text{Organic loading}}$$

$$= \frac{570}{10{,}000} \text{ ha-m.} = 0.057 \text{ ha-m.} = 570 \text{ m}^3$$

Organic loading

$$= \frac{570}{10,000} \text{ ha-m.} = 0.057 \text{ ha-m.} = 570 \text{ m}^3$$

Now, using equation (9.34), we have

$$\eta = \frac{100}{1 + 0.0044 \, . \sqrt{\dfrac{Y}{V.F}}}$$

where Y = Total daily BOD applied to filter in kg

= 570 kg.

V = Volume of filter = 0.057 ha-m.

and η = 76.92% (worked out earlier)

$$\therefore \qquad 76.92 = \frac{100}{1 + 0.0044 \, . \sqrt{\dfrac{570}{0.057\,F}}}$$

or

$$76.92 = \frac{100}{1 + 0.44 \dfrac{1}{\sqrt{F}}}$$

or $$1+\frac{0.44}{\sqrt{F}}=\frac{100}{76.92}=1.3$$

or $$\frac{0.44}{\sqrt{F}}=0.3$$

or $$F=\left(\frac{0.44}{0.3}\right)^2=2.15$$

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

or $$2.15=\frac{1+\frac{R}{I}}{\left(1+0.1\frac{R}{I}\right)^2}$$

This gives $\frac{R}{I}=1.47$ (Solving by trial)

$$\text{Detention period } (t) = \frac{\text{Volume of the tank}}{\text{Rate of sewage flow in the tank}}$$

$$= \frac{V \text{ in m}^3}{Q \text{ in m}^3/\text{day}}$$

$$= \frac{V}{Q} \text{ day} \qquad \text{...(9.41a)}$$

$$t = \frac{V}{Q} \cdot 24 \text{ hour} \qquad \text{...(9.41)}$$

where t = aeration period in hours

V = Volume of aeration tank

Q = Quantity of wastewater flow into the aeration tank, *excluding the quantity of recycled sludge.*

olumetric BOD loading or Organic loading

$$= \frac{\text{Mass of BOD applied per day to the aeration tank through influent sewage in gm}}{\text{Volume of the aeration tank in m}^3}$$

$$= \frac{Q \cdot Y_0 (\text{gm})}{V(\text{m}^3)} \qquad \text{...(9.42)}$$

where Q = Sewage flow into the aeration tank in m^3.

Y_0 = BOD_5 in mg/l (or gm/m^3) of the influent sewage.

V = Aeration tank volume in m^3

∴ F/M ratio

$$= \frac{\text{Daily BOD load applied to the Aerator System in gm}}{\text{Total Microbial mass in the system in gm}} \quad ...(9.43)$$

If Y_0 (mg/l) represents the 5 day BOD of the influent sewage flow of Q m^3/day, then eventually,

The BOD applied to the Aeration system = Y_0 mg/l or gm/m^3

∴ *BOD load applied to the aeration system* = $F = Q \,.\, Y_0$ gm/day ...(i)

$$M = \text{MLSS} \times V$$
$$= X_T \,.\, V$$

where X_T is MLSS in mg/l

ng (i) by (ii), we get

$$\text{F/M ratio} = \frac{F}{M} = \frac{Q \,.\, Y_0}{V \,.\, X_t}$$

∴ Sludge age (θ_c)

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

$$\text{Sludge age} = \theta_c = \frac{V \cdot X_T}{Q_W \cdot X_R + (Q - Q_W) X_E} \quad \ldots(9.48)$$

where X_T = Concentration of solids in the influent of the Aeration Tank, called the MLSS, *i.e. Mixed Liquor Suspended Solids*, in mg/*l*.

V = Volume of Aerator

Q_W = Volume of wasted sludge per day.

X_R = Concentration of solids in the returned sludge or in the wasted sludge (both being equal) in mg/*l*

Q = Sewage inflow per day

X_E = Concentration of solids in the effluent in mg/*l*

Example 9.29. *An average operating data for conventional activated sludge treatment plant is as follows :*

(1) *Wastewater flow* $= 35000\ m^3/d$

(2) *Volume of aeration tank* $= 10900\ m^3$

(3) *Influent BOD* $= 250\ mg/l$

(4) *Effluent BOD* $= 20\ mg/l$

(5) *Mixed liquor suspended solids (MLSS)* $= 2500\ mg/l$

(6) *Effluent suspended solids* $= 30\ mg/l$

(7) *Waste sludge suspended solids* $= 9700\ mg/l$

(8) *Quantity of waste sludge* $= 220\ m^3/d.$

Based on the information above, determine:

(a) *Aeration period (hrs)*

(b) *Food to microorganism ratio (F/M) (kg BOD per day/kg MLSS)*

(c) *Percentage efficiency of BOD removal*

(d) *Sludge age (days).*

(G.A.T.E., 1993)

(d) Sludge age (days).

Solution. Given values are symbolised as :

$Q = 35000\ m^3/d$; $V = 10900\ m^3$

$Y_0 = 250\ mg/l$; $Y_E = 20\ mg/l$

$X_T = 2500\ mg/l$; $X_E = 30\ mg/l$

$X_R = 9700\ mg/l$; $Q_W = 220\ m^3/d$

These values are now used to calculate the desired factors, as below :

(a) Aeration period (t) in hr is given by Eq. (9.41) as

$$t = \frac{V}{Q} \cdot 24 = \frac{10,900}{35,000} \times 24 = 7.47 \text{ h ; say } 7.5 \text{ h.} \quad \textbf{Ans.}$$

(b) F/M ratio F = Mass of BOD applied to aeration system

$= Q \cdot Y_0 = 35000 \times 250$ gm/day

$$= \frac{35000 \times 250}{1000} \text{ kg/day} = \mathbf{8750 \text{ kg/day}}$$

M = Mass of MLSS

$$= V.\, X_T = 10900 \text{ m}^3 \times 2500 \text{ mg/l } (i.e. \text{ gm/m}^3)$$

$$= \frac{10900 \times 2500}{1000} \text{ kg} = \mathbf{27{,}250 \text{ kg}}$$

$$\therefore \quad F/M \text{ ratio} = \frac{8750}{27{,}250}$$

$= \mathbf{0.32}$ **kg BOD per day/kg of MLSS. Ans.**

(c) *Percentage efficiency of BOD removal*

$$= \frac{\text{Incoming BOD} - \text{Outgoing BOD}}{\text{Incoming BOD}}$$

$$= \frac{250 - 20}{250} \times 100\% = \frac{230}{250} \times 100\% = 92\%. \quad \textbf{Ans.}$$

250 250

(*d*) Sludge age in days (θ_c) is given by Eq. (9.48) as

$$\theta_c = \frac{V \,.\, X_T}{Q_W \,.\, X_R + (Q - Q_W) \,.\, X_E}$$

$$= \frac{27250 \text{ kg}}{(220 \text{ m}^3/\text{d} \times 9700 \text{ mg/l}) + (35000 \text{ m}^3/\text{d} - 220 \text{ m}^3/\text{d})\, 30 \text{ mg/l}}$$

$$= \frac{27250 \text{ kg}}{\frac{220 \times 9700}{1000} \text{ kg/d} + (35000 - 220)\frac{30}{1000} \text{ kg/d}}$$

$$= \frac{27250}{2134 + 1043.4} = \frac{27250}{3177.4} = 8.58 \text{ days.} \quad \textbf{Ans.}$$

what is